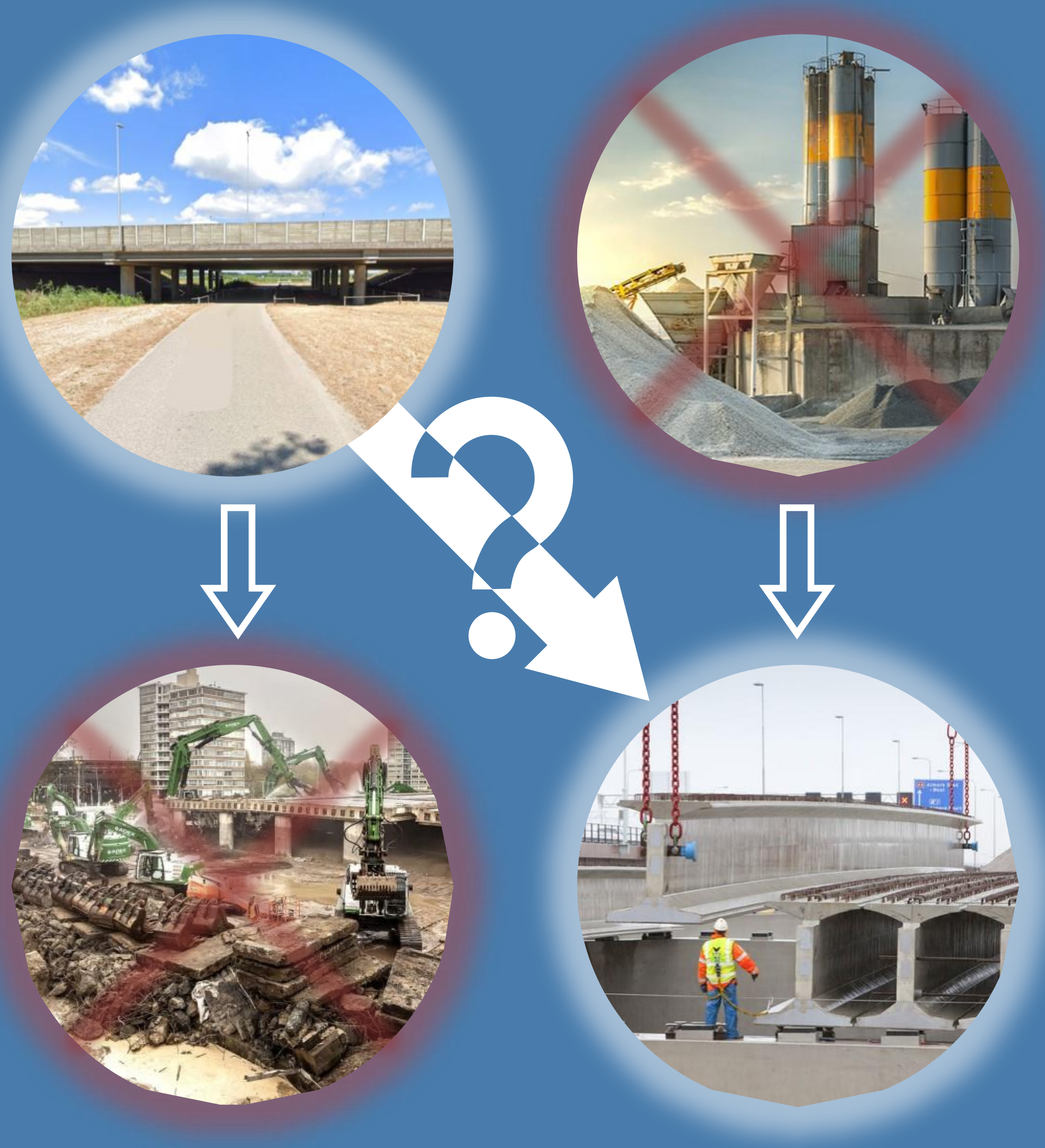
R.E. (Romy) Groeneweg

A design approach to implement reuse of existing concrete bridge girders



TU Delft Van Hattum en Blankevoort

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A design approach to implement reuse of existing concrete bridge girders

By

R.E. (Romy) Groeneweg

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Preface

This thesis is my final delivery to obtain my degree of Master of Science in Civil Engineering at the Delft University of Technology. The research is the conclusion of my two-year master program of Structural Engineering with a specialisation in Concrete Structures. I conducted this research between December 2022 and August 2023 at Van Hattum and Blankevoort as graduate.

In this research a design approach is developed that implements the reuse of existing prefabricated concrete bridge girders, especially inverted T-girders, in new bridge deck designs. Since traditional approaches do not aim at a circular economy, this research provides valuable insights in the steps needed to start with the implementation of reuse. Consequently, this research is relevant for everyone that is interested in the development to a circular construction economy and especially for engineers that have to design these bridge decks.

For me, this multidisciplinary research topic provided the unique opportunity to combine the knowledge gained from general, specialisation and elective courses of my Master program and put this knowledge into practice. I own an acknowledgement to all the people, who helped me during this research. Without their help I could not have undertaken this journey.

First, I would like to express my gratitude to my supervisor Eelco van der Weij at Van Hattum and Blankevoort. His expertise, recommendations and feedback helped me throughout the research. In addition, I would like to express my gratitude to Yuguang Yang, Marco Schuurman and Max Hendriks from my thesis committee for the feedback, discussions and guidance on the research progress.

Furthermore, I am grateful to Van Hattum and Blankevoort for giving me the opportunity to conduct my research. I would like to thank Christiaan Holtdijk for his contribution to and expertise on the financial analysis, Sebastiaan Ensink for his expertise and feedback on existing concrete girders and Felix Leenders for his involvement and suggestions. Additionally, I would like to thank Dion Baars and Jesse Maas for their input on the performed case study.

Finally, I would like to acknowledge my friends and family for their support.
I hope you enjoy reading my thesis,

Romy Groeneweg
Vianen, July 2023

Abstract

To stop the depletion of natural resources, reduce climate change and fight biodiversity loss a circular economy in 2050 is pursued. In the upcoming years a challenging opportunity arises. Many existing bridges and viaducts with bridge decks consisting of prefabricated concrete girders have to be replaced. These girders although not designed according to circularity concepts have potential to be reused in a new structure, which is in line with the highest achievable level of circularity. However, the construction market is not ready for this innovation. So, while current research still aims at the feasibility and suitability of the girders for reuse in new structures, this research aims at the next step of preparing the construction market. This is of high relevance because it speeds up the introduction process of the innovation and thereby safes girders from demolition.

In this research the focus is on the adaptions and modifications needed in the traditional design process to ensure a more frequent implementation of reuse of existing bridge girders in new designs. After a literature review into the type of bridge girders in the Netherlands, the structural feasibility, obstacles for reuse identified by the industry, the design process and environmental impacts a design approach is developed. Simultaneously to the development of this design approach a case study is performed to give a more practical view to design aspects. In this way the approach could be verified, adapted and modified.

The design approach starts at the system design, where the requirements from stakeholders, client and project are derived. These requirements are used in the girder search. In this step, in between the system and preliminary design potentially suitable girders are identified from girders available. Based on guiding principles possible span divisions are derived, which result in minimum and maximum girder length. The maximum length is based on an investigation in the environmental and structural limit for shortening in length direction. Apart from the girder length, the height, type, origin and the release date are identified as search criteria.

With potentially suitable girders design alternatives are developed, which are first structurally assessed. This assessment is based on shortening possibility, shear capacity, bending moment capacity and durability. For structurally suitable alternatives the design process continues and the environmental impact is analysed with the environmental cost indicator and an indicator for material use and origin. Together with a financial analysis these aspects form essential criteria in the multi-criteria analysis (MCA). First an MCA is performed to find the most suitable alternative with reuse of existing girders. Next this most suitable alternative is compared with an alternative with new girders in an MCA to choose the best option.

In the case study a bridge deck for a 107 [m] long bridge, divided over 5 spans with reuse of existing girders is designed. The girders that become available from the viaducts that have to be replaced due to reconstruction of a part of highway A9 between Badhoevedorp and Holendrecht are considered for the case study. Combining the project requirements and the available information on existing girders, five design alternatives seem to be possible, of which three are further investigated. One alternative consisting of HNP 750 girders has insufficient shear capacity and is identified as structural unfeasible without strengthening, which is not considered. The other two alternatives, which consist of HIP 800 girders, are structurally feasible, although the degree depends on the partially unknown stirrup layout. In the best-case scenario both girders are suitable without modifications. In the worst-case scenario both alternatives the girders have to be shortened in width direction. For one alternative this is not sufficient and additional measures are needed.

The environmental impact analysis of the bridge deck shows an average reduction of 50% on environmental cost indicator and 65% on primary resources compared to a bridge deck with new girders. The financial analysis shows that reusing existing girders is currently more expensive than using new girders. Even without considering additional demolition costs, design alternatives with existing girders are 33% to 127% more expensive. In addition, deconstruction costs can be much higher than demolition costs due to additional time needed, but this depends on the location of the existing structure.

This research provides the first setup of a design approach that can be used by project teams to shift the focus from new elements to reusing existing elements. By developing roadmaps, providing possible procedures and giving recommendations the approach gives guidance through the different steps in design. The design approach is suspectable to changes due to experiences, gained knowledge and developments in the construction industry. Therefore, it needs review over time. The design approach concentrates on inverted T-girders but can be extended and applied to other girder types as well. Other possible extensions are combining new and existing girders in a single alternative or investigating the influence of the in-situ deck. Further research can be conducted in the strengthening procedures and regulations. In addition, to support a completely circular economy, it is recommended to derive an additional quantitative environmental indicator that includes these aspects. Since this research confirms that the financial feasibility of implementing reuse is still low stimulating financial measures can be investigated. Furthermore, besides on the design process, tools and guidance are needed on other elements of the construction industry as well. For instance, procedures for planning and building contracts need to be adapted as well.

Nonetheless this research provides the foundations for a changed design approach that is needed to prepare the construction market for reusing existing girders. By guiding project teams through each step of the design, the view shifts from using new girders to reusing existing girders. This is valuable to reach the environmental objective of a circular economy in 2050.

Table of contents

1. Introduction

In this chapter the research topic is explained. First the background is provided from which the problem statement follows in the second paragraph. Nex the research objective is defined, which is delineated by the scope in the fourth paragraph. In the fifth paragraph the research questions are stated and finally the research approach and outline of the report are described.

§1.1 Background

In 2050 the Netherlands should have a completely circular economy. As intermediate goal in 2030 the use of primary resources should be reduced by 50% [5]. The construction sector can play a major role as it is responsible for 19% of the overall greenhouse gas emissions, 35% of the total waste generation and it consumes 50% of the non-renewable resources [6],[7]. The cement production for concrete is a large polluter in the industry as it contributes up to 8% of the global carbon dioxide emissions [8].

The depletion of natural resources is one of the reasons why a more circular economy is needed. In 2022 Earth overshoots day was on the 28th of July. At this day all biological resources that Earth regenerates during an entire year are used [9]. So, from this day until the end of the year natural resources are depleted and becoming more scare. Due to the increasing world population and increasing material demand every year the date is reached sooner, however the increased awareness on sustainability is slowing down the rate. Apart from reducing the depletion a circular economy also contributes to a reduction in the emission of harmful compounds and greenhouse gasses to air, water and soil, which is beneficial in the fight against biodiversity loss and climate change.

The traditional construction process is described by a linear process. Raw (primary) materials are excavated and products are produced and assembled. Next the product is used and maintained after which it is demolished and processed as waste. This process is known as cradle-to-grave and visualized in Figure 1.1 [10]. In a circular construction process primary resources are replaced by secondary resources and materials and elements are reused. Moreover, emissions of harmful compounds to the environment are significantly reduced. The carbon dioxide emissions from building materials can be reduced by 38% with a completely circular building economy [11]. The circular process, also known as cradle-to-cradle is visualized in Figure 1.2 [12].

Different levels of circularity can be distinguished. A well-known model is the 10-R model of Cramer which is an extension of the ladder of Lansink. This model is shown in Figure 1.3. A higher level of circularity involves less resources and waste [13].

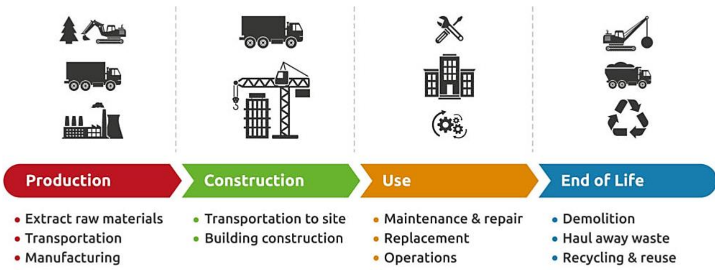


Figure 1.1: Traditional linear building process [10].

Figure 1.2: Circular building process [12].

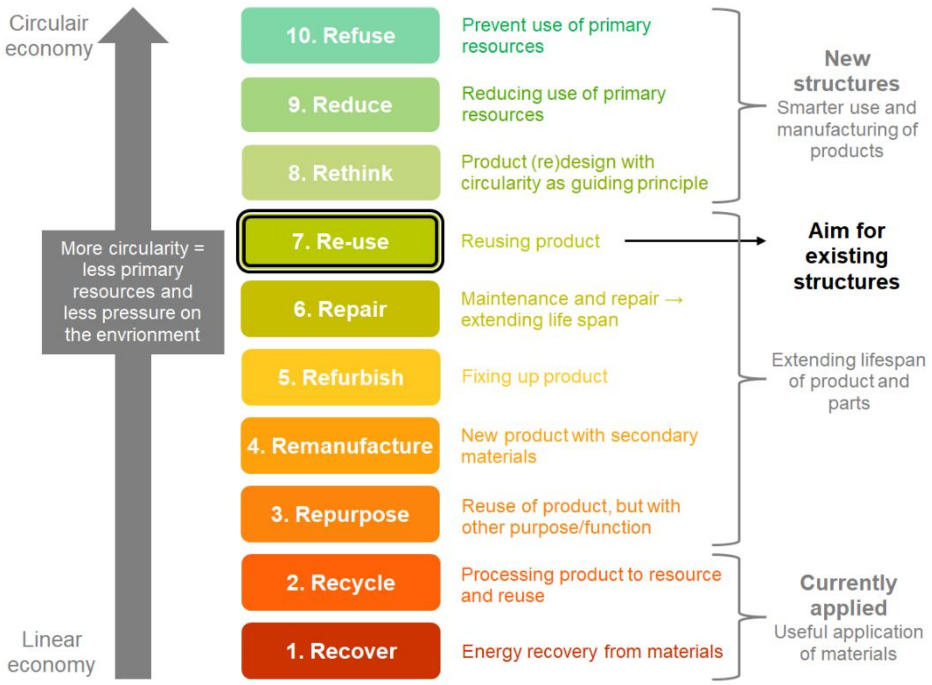


Figure 1.3: 10R-model that shows circularity levels based on [13].

§1.2 Problem statement

In the Netherlands a lot of infrastructure is constructed after World-War II. Between 1960 and 1980 more than 1700 concrete viaducts and fixed concrete bridges are built [14]. Most existing civil objects are designed with a technical life span between 60 and 120 years. So, the end of life of these objects is approaching. The objects are designed based on the standards, guidelines and traffic characteristics from that time. Over the years the traffic load as well as the traffic intensity has increased [15]. Moreover, due to progressive insights in materials and load models, standards and guidelines have changed. Furthermore, many viaducts and bridges need to be replaced, because of addition of extra lanes or reconstruction of the infrastructural network. Consequently, the functional service life is reached even sooner. As a result, a lot of infrastructural objects have to be replaced in the upcoming years.

The end-of-life of the bridge should not be confused with the end-of-life of an element. A bridge can reach the end of its technical life span, however the elements inside the bridge might still be in good condition. The same holds for bridges at the end of their functional service life.

During the construction time of this infrastructure, the awareness on the environmental impact and circularity of designs was very low. The designs are based on a linear construction process, where no attention is paid to circularity aspects. According to the design principle when the structure reaches it end-of-life its demolished and the waste is land-filled. In reality with current awareness, insights and technologies landfilling is already replaced by recover and recycle concepts. Currently more than 90% of construction waste is downcycled to filling materials [16]. In the concrete industry concrete is recycled in aggregates or downcycled to road base foundations material. For example, recycled aggregates can replace up to 40% of natural aggregate, but these concrete mixes have higher cement demand [17]. As a consequence, carbon dioxide emissions are not reduced. So, overall these low levels of circularity still do not suit with the current goals and needs for a circular economy, the reduction in primary resources and the reduction in harmful emissions.

§1.3 Objective

To reach the ultimate goal of a completely circular economy, the aim is reaching the highest level of circularity possible. The levels follow from the 10-R model. For completely new designs the focus can be on smart and optimal use, which corresponds with the first 3 levels of circularity: refuse, reduce and rethink. An example is modular design and building, which corresponds to the level of rethinking. This concept is already frequently researched and applied in several innovative projects in building and civil industry. Another example is the use of sustainable materials as geopolymer concrete, which corresponds to the level of reducing.

However, for existing structures these levels cannot be reached, as the structures are already built with certain concepts and materials. The two lowest levels on circularity, recycle and recover focus on the useful application of the materials after demolition and are commonly applied. Nevertheless, to make the economy more circular the objective is to reach higher levels on the ladder. The highest achievable level is reuse. This objective suits very well with the ambition of Van Hattum and Blankevoort to be the most sustainable civil engineering and building company by 2025 [18].

§1.4 Scope

Relating to the current challenge of the ageing infrastructure and the aim of a circular economy this research focuses on reusing existing concrete prefabricated bridge girders from viaducts and fixed bridges. Prefabricated bridge girders are chosen, because they have much more potential for reuse compared to in-situ bridges. As a result, currently innovation and small-scale projects with reusing girders are going on [19]. However, after this innovation phase a wide scale implementation is needed to reach the objective of a circular economy. This implementation starts at the design, consequently this research aims at the design process. Furthermore, since mostly inverted T-girders become shortly available this is the main focus. While the focus will only be on the Dutch civil engineering sector, the research can be of relevance for other European countries. Since, in other European countries the same challenge with the ageing infrastructure and circularity arises [20]. In short, the scope of this research will be the reuse of existing concrete prefabricated bridge girders in the design of new viaducts and fixed bridges in the Netherlands.

§1.5 Research questions

The following main research question will be answered in this thesis:

> “How should the design process of concrete bridges be adapted to make reuse of existing concrete bridge girders more common practice?”

To answer this question, the following sub-questions should be answered.

1) What are the main attention points in a process with reusing existing girders compared to using new girders?
2) What prerequisites must an existing girder meet to be eligible for reuse?
3) How can the design process of one bridge take into account the global overall implementation of reuse of existing girders?

The final outcome of this research will be a design approach in the form of a flow chart combined with a guideline. The design approach focusses on the design process of a new bridge with existing girders. By following the flow chart and steps from the guideline, the reuse of existing girders will be stimulated and implemented whenever possible.

§1.6 Research approach

To develop a design approach and answer the main research question a case study approach is used. First a literature study is conducted to provide relevant background. With this background a framework for the design approach is developed. In the framework the design process is mapped out, influencing factors are identified and road maps for calculation procedures are developed. This outline or framework will be used as basis in a case study, where a design for a concrete bridge deck consisting of reused prefabricated girders is made. Parallel to this case study the framework will be verified, modified and extended to a design approach. Finally, the case study is reviewed and the validity regarding the design approach is discussed. The diagram in Figure 1.4 gives an overview of the research approach.

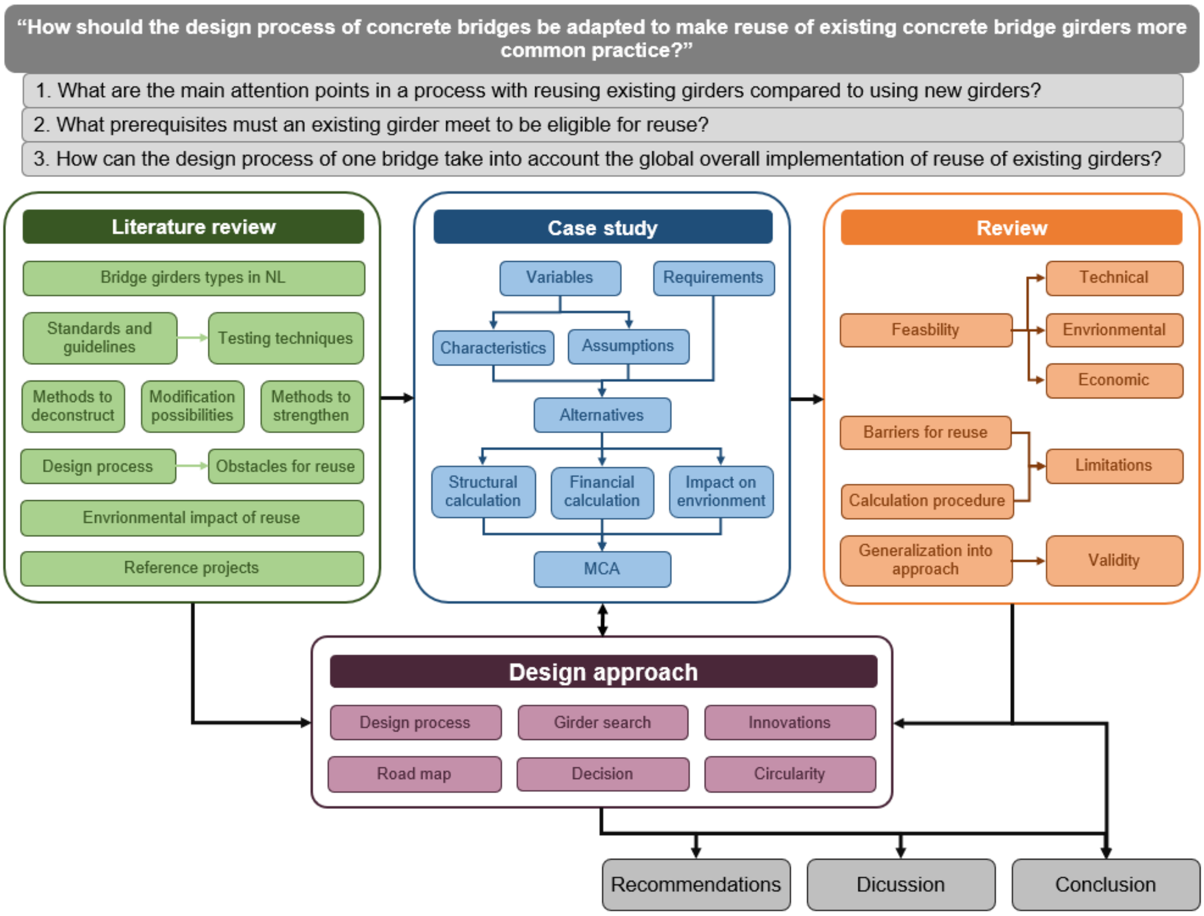


Figure 1.4: Overview of research approach.

§1.7 Outline of report

In chapter two the literature review is provided. First relevant research questions are identified after which they are answered in their own paragraph. Next in chapter three the total design approach is explained. This approach is developed in an iterative process parallel to the case study. So, this chapter already includes findings from the case study. The results from the case study are described in the fourth chapter. In this chapter many references are made to appendices that provides background to the project, calculations and more detailed results. In the fifth chapter the findings and application of the research is discussed. The final chapter provides the conclusion and recommendations for further research.

2. Literature Study

§2.1 Introduction

The first phase of research is a literature study, at the end of paragraph an answer on the following question is formulated.

§2.2 Which bridge girders become available and what are their characteristics?
§2.3 How is the structural integrity of a design with reusing existing girders assessed?
§2.4 Is reusing concrete bridge girders technically feasible?
§2.5 How does a design process with reuse differs from a traditional process?
§2.6 What are the current obstacles that limit the implementation of reuse?
§2.7 How is reuse considered in the environmental impact analysis?
§2.8 What lessons can be learned from reference projects?

In the second paragraph an inventory of girders in the Netherlands is made. The characteristics of girders in the Netherlands are important input factors during calculations and together with the availability they play a significant role in the decision-making process.

Structural integrity is based on calculation procedures, design loads and material characteristics. The third paragraph provides an overview of guidelines, which provides insight in the differences between current design standards and former ones. This helps to identify possible limitations or critical points for reusing girders. In addition, the characteristics of concrete, reinforcement and prestressing are discussed together with the testing techniques to assess these characteristics.

In paragraph four methods for deconstruction and strengthening are discussed. Moreover, possible modifications are discussed. These are important aspects for the technical feasibility of implementing reuse.

To implement reuse in design, first the traditional design process should be known. In paragraph five this process is described together with the differences between a traditional process and a process with reuse. The next paragraph provides information about the current drivers and obstacles that stimulate or limit the implementation of reuse. With this information attention points for the design approach can be derived.

Paragraph seven provides insight into measuring the environmental impact of a project. A commonly used method is a life cycle impact analysis. However, the implementation of reuse in this assessment is not straightforward. Therefore, more information is provided to answer question seven. In paragraph eight three reference projects are discussed, to see what the current state-of-art is. Finally, in the last paragraph a conclusion is given.

§2.2 Bridge girders in the Netherlands

To reuse bridge elements from existing structures information about the existing structures is required. Therefore, this paragraph presents an overview of the existing bridges in the Netherlands and their characteristics. First some numbers are presented. Followed by the characteristics of three main girder types.

§2.2.1 Numbers in the Netherlands

The Dutch infrastructure counts 85.000 bridges and viaducts [15]. 74% is owned by municipalities, 17% by water authorities and 3,4% by provinces. These bridges are small with an average area of 99 m^2, 41 m^2, 510 m^2 respectively. The remaining part of the bridges and viaducts have an average area of more than 1400 m^2 and are owned by central governmental authorities [21]. The largest part (80%) is owned by Rijkswaterstaat of which 1637 (46%) consist of prefabricated concrete bridge girders [22]. The remaining bridges are often made of solid concrete slabs or infilled concrete girders. 7% of the bridges, often movable ones, are made of steel. The concrete girder bridges can further be divided in three main categories: T-shaped girders, box girders and inverted T-shaped girders. The corresponding division of these girders is 22%, 18% and 60% [23]. These numbers are summarized in Figure 2.1. The division of span length is shown in Figure 2.2 and shows that most girders have a span length between 17,5 and 30 meters [22][21].

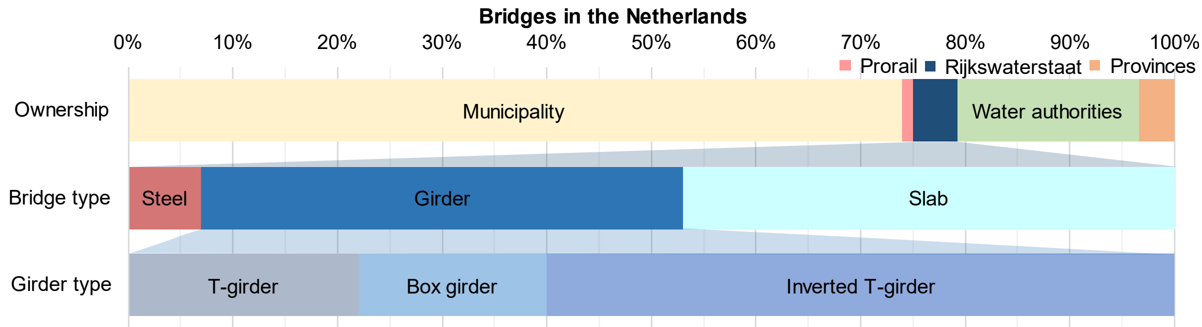


Figure 2.1: Overview of bridge girder in the Netherlands. First row represents the ownership. The second shows the division of bridge types owned by Rijkswaterstaat. The girder type is further divided in the last row.

Figure 2.3. gives an indication of the building years of girder bridges. The prefabricated and prestressed concrete construction emerged after the World War II, because of the need for cheap, efficient and fast building. With bridges this started with the construction of solid bridge deck. To safe materials T-shaped girder bridge decks were invented. From the 1960s these types give gradually way to the currently common bridge girder systems, like inverted T-girders [24]. So, during the post-war reconstruction less than 50% of the bridges is constructed with girders, while nowadays almost all fixed bridges and viaducts in the Netherlands are made of girders [21]. This explains why in Figure 2.3 the peak of the post-war reconstruction may seem small, because during the twenty centuries almost the same amount of girder bridges is being build. However, if the total amount of bridges build were displayed the post-war reconstruction peak would be much higher, while the numbers in the twenty centuries will not increase that much.

Every year 7-10 of girder viaducts are demolished and around 400 girders are released [25]. These girders have an average age of 40 years [21]. Compared to other bridge types, girder bridges are most suitable for reuse, because they consist of separate repeatable elements of high quality. This high quality is the result of prefabrication. Moreover, since only a few girder types are common in the Netherlands a lot of girder bridges are comparable.

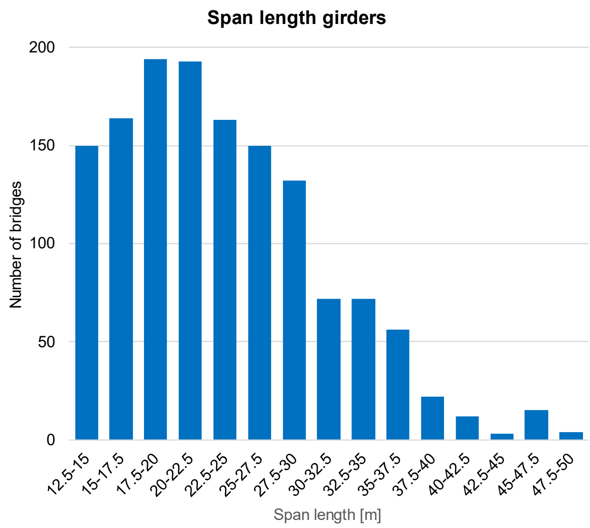


Figure 2.2: Span length girders, based on [21]..

Figure 2.3: Construction year of girder bridges, based on [21].

§2.2.2 T-shaped girder bridges

In T-shaped girder bridges prestressed T-shaped girders with a span between 20 and 40 meters are placed next to each other with room in between. This type of bridge is mostly used in static determined structures when the traffic class is relatively low and sufficient construction height is available [26]. The profile height is between 500 and 1400 [mm] and the slenderness ratio is around 21 [27]. An example is shown in Figure 2.4 [28].

The upper flanges have a width between 900 and 1500 mm and the web is approximately 200 [mm]. The girders also have a slightly wider bottom flange to accommodate the prestressing tendons. Moreover, at the ends the web is often as tick as the bottom flange [29]. In between the upper flanges of the T-girders a 0,15-0,25 [m] heigh cast-in-situ deck is casted. This layer is connected to the upper flanges of the girders by transverse prestressing. Due to the small height, the deck is vulnerable for deterioration by de-icing salts [30]. The principle is shown in Figure 2.5.

Edge beams are used to make load distribution possible and close off the edges of the bridge. These connect the girders together and fixate the bridge in transverse direction. Intermediate crossbeams can be used to increase the load distribution capacity; however, this complicates the execution. The presence of intermediate beams is also of importance during deconstruction as it complicates the process. The bottom flanges are not connected to each other. Therefore, in case of a collision one single girder must be able transfer the load to the upper part of the structure [26]. This is also one of the main reasons this girder type is not made any more since 1984 [29].

The girders are prestressed and most of the tendons have a parabolic shape [29]. So, at the supports the tendons are situated around the centroidal axis level or even above while in the middle they are located at the bottom. This is done to prevent too much prestress near the supports, which may result in tension. To create this layout sometimes tendons are anchored in the bridge deck. In most girders stirrups are not applied or only limited. Therefore, the shear capacity of this girders might be critical with current loads and regulations [30].

Brouwer investigated in his thesis a number of viaducts owned by Rijkswaterstaat that are planned to be demolished in the upcoming 15 years. From the 65 girder bridges that are investigated and will be demolished 220 T-girders with a span of 23-27 [m] will be released around 2025. So, at least this number of girders will become available [25].

Figure 2.4: Bridge consisting of T-girders with crossbeam in between [28].

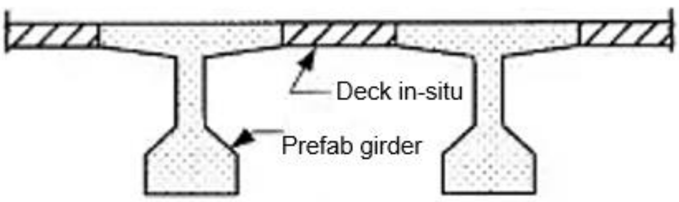


Figure 2.5: Cross-section of T-girder bridge [31], adapted.

§2.2.3 Inverted T-girder bridges

The inverted T-girders or I-girders are frequently used in the Netherlands. The typical span is 15-35 [m], however spans up to 60 [m] are possible. The slenderness ratio ranges between 20 and 28, which gives a profile height of 500–1750 [mm]. The bottom flange is 1180 [mm] wide and the minimum centre to centre distance is 1200 [mm] [32], [33]. An example is shown in Figure 2.6 [28] and a cross-section in Figure 2.7.

The principle is quite similar to T-girders. A cast in situ reinforced concrete deck with a thickness of 160-250 [mm] is applied. The bottom flanges of the girders are connected with mortar. This allows for load distribution and therefore together with edge beams this bridge type is able to resist collision loads. Intermediate crossbeams can be applied to increase the capacity further; however this is not common as it is expensive and complicates the execution. Instead, the bottom flanges and edge beams are strengthened. The application of stirrups is also limited. For reuse and assessment this might make the shear capacity critical, however due to the high concrete class applied girders in statically determined structures are still able to fulfil the requirements [30].

The concrete strength class of the prefabricated girders is much higher than the strength class of the cast-in-situ deck. For older structures the in-situ-deck has a comparable strength class between C18/22 and C25/30. Newer structures often have an in-situ deck of C30/37. The strength class of the girders starts at C42,5/52,5, but this is highly variable.

Figure 2.6: Bridge consisting of inverted T-girders [28].

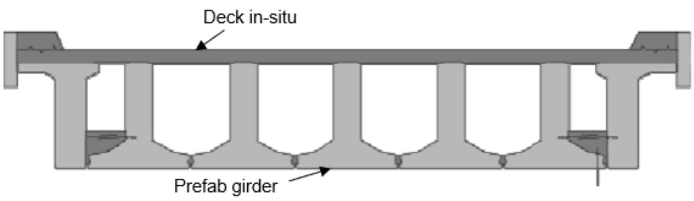


Figure 2.7: Cross-section of inverted T-girder bridge [33], adapted.

The first type of inverted T-girder is the HNP girder produced by Spanbeton from 1965 [34]. This girder does not really have stirrups [35]. Around 210 bridges owned by Rijkswaterstaat are made with this girder type [22]. In 1969 this type is succeed by the HIP-girder in which stirrups are applied [34], [35]. It is estimated that approximately 250 bridges owned by Rijkswaterstaat are made with this girder type [22]. It should be noted that some of these bridges are already demolished. This HIP-girder is succeeded by the VIP girder and later by the ZIP-girder [34]. This girder type has a thicker bottom flange and therefore better able to withstand collision loads. It was also more favourable for indeterminate structures. This ZIP-girder is produced until Spanbeton stopped with their production in the Netherlands in 2020 [36].

Betonson was another manufacturer of prefabricated beams but merged with Spanbeton in 2013 [37]. They had comparable girders, but with different names. Lambda S girders are comparable with HIP-girders. Gamma girders are comparable with VIP-girders and Lambda-Z-girders are comparable with ZIP girders [27]. The HRP-girder is another type of inverted T-girder produced by Haitsma. In 2010, Haitsma developed a HIP-girder. This is an I-shaped girder, with comparable behaviour as inverted T-girders [38]. Another inverted T-girder used in the Netherlands is the Rogir-Z-girder produced by Romein Beton. The cross-section of the several types is shown in Figure 2.8. The available sizes can be derived from Figure 2.11.

From the 65 girder bridges that Brouwer investigated 646 girders will be released around 2026 and another 141 around 2036. The span ranges from 18 [m] to more than 30 [m] [25].

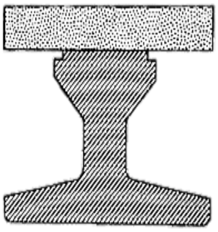


Figure 2.8 a) HNP girder from Spanbeton [35].

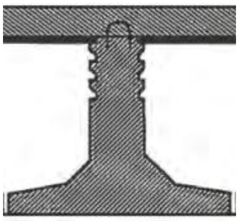


Figure 2.8 b) HIP girder from Spanbeton [35].

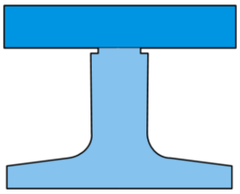


Figure 2.8 b) VIP girder from Spanbeton.

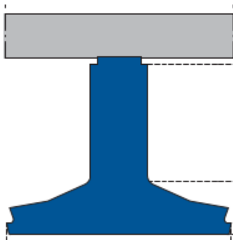


Figure 2.8 c) ZIP girder from Spanbeton [39].

Figure 2.8 e) Lambda S girder from Betonson [27].

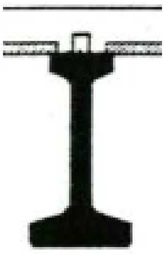


Figure 2.8 f) Gamma girder from Betonson [27].

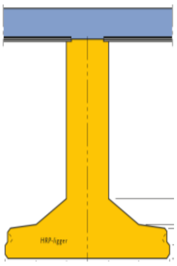


Figure 2.8 g) HRP girder from Haitsma Beton [40].

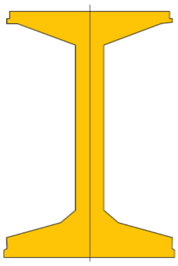


Figure 2.8 h) HIP girder from Haitsma Beton [40].

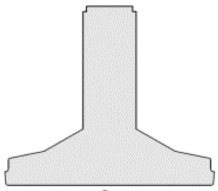


Figure 2.8 i) Rogir-Z-girder from Romein Beton [41].

Figure 2.8: Cross-sections T-girder profile.

§2.2.4 Box beam girder bridges

The slenderest type of prefabricated bridge in the Netherlands is the box girder. Therefore, it is used for larger spans. Moreover, it is suitable for statically indeterminate structures. This girder has a span of 15-68 [m] and a slenderness ratio of 28-32 [33]. The minimal concrete class is C30/37,5. Often the strength class is lower compared with T-girders and might be critical with the assessment on shear capacity [30]. However, the strength class is highly variable as in newer structures much higher strength classes are used.

Prestressed box girders are placed next to each other. In the longitudinal joints transversal post-tensioning is applied. This is shown in the cross-section in Figure 2.9. It makes the girder more expensive, however faster building is possible as no reinforcement and cast-in situ deck layer are needed. During deconstruction this also saves time and material as only the joints have to be cut. Other advantages are the high torsional stiffness and the absence of special edge beams.

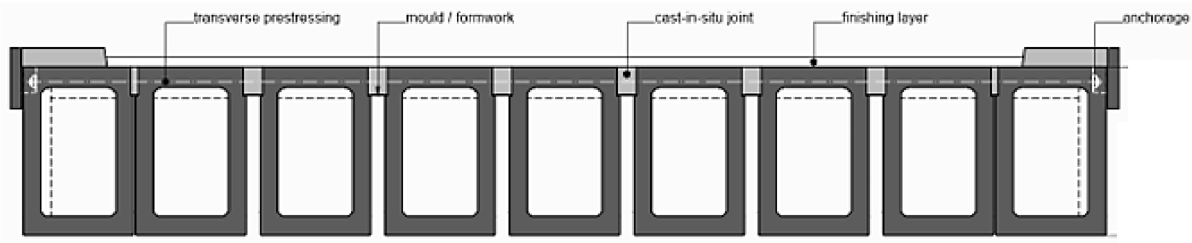


Figure 2.9: Cross-section of box beam girder bridge [42], adapted.

Again, box beams producers in the Netherlands have their own type. SDK-girders are introduced in two sizes by Spanbeton in 1975. The successor is the SKK-girder, which was made in much more sizes. Also, Betonson produced a box beam girder with the name ‘Kappa’. Haitsma Beton still produces the HKP box beam girder and Romein Beton also has two types of box beams. The Rogir-K-girder and the RHE-girder. The cross-section of the several types is shown in Figure 2.10. The available sizes can be derived from Figure 2.11.

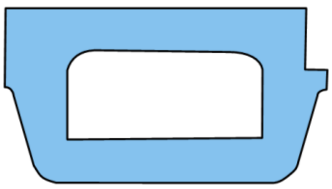


Figure 2.10 a)
SDK profile from Spanbeton.

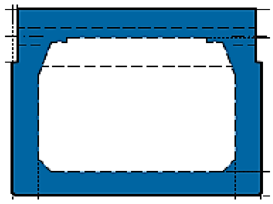


Figure 2.10 b)
SKK profile from Spanbeton [42].

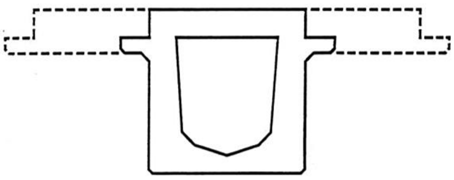


Figure 2.10 c)
Kappa profile from Betonson [27].

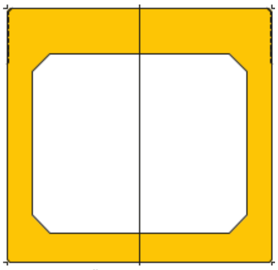


Figure 2.10 d)
HKP profile from Haitsma [43].

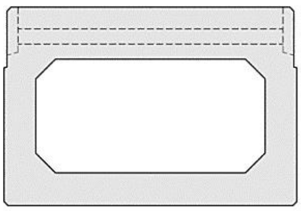


Figure 2.10 e) Rogir-K profile from Romein Beton [44].

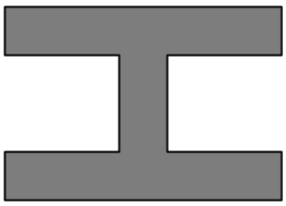


Figure 2.10 f)
RHE profile from Romein Beton.

Figure 2.10: Cross-sections box girder profiles.

According to the thesis of Brouwer 227 girders will be released around 2026 and another 444 around 2036. Most of them have a span larger than 30 [m].

§2.2.5 Which bridge girders become available and what are their characteristics?

It can be concluded that every year around 400 girders with an average age of 40 years are released. Of the prefabricated girder bridges a large part is owned by Rijkswaterstaat. Furthermore from Figure 2.1 it can be concluded that most girders used in bridges in the Netherlands are inverted T-girders. Moreover, inverted T-girders are introduced before box beam girders. Therefore, it can be concluded that in the upcoming years mostly inverted T-girders with a span between 17,5 and 30 [m] will become available. The assessment of shear capacity for these girders is most likely the determining factor for reuse. In the first years also T-girders will become available, which can only be reused in areas were collisions cannot occur. Later, also box beams will become available. Figure 2.11 gives an overview of the different types of girders available and the range of span. Only the relevant span lengths between 12,5 and 30 [m] are considered. Nonetheless many profiles exist in larger sizes as well. In blue the range for inverted T-girders is shown and in orange the range for box beam girders. With dotted lines the available sizes are shown. With arrows the start size of the different types is indicated.

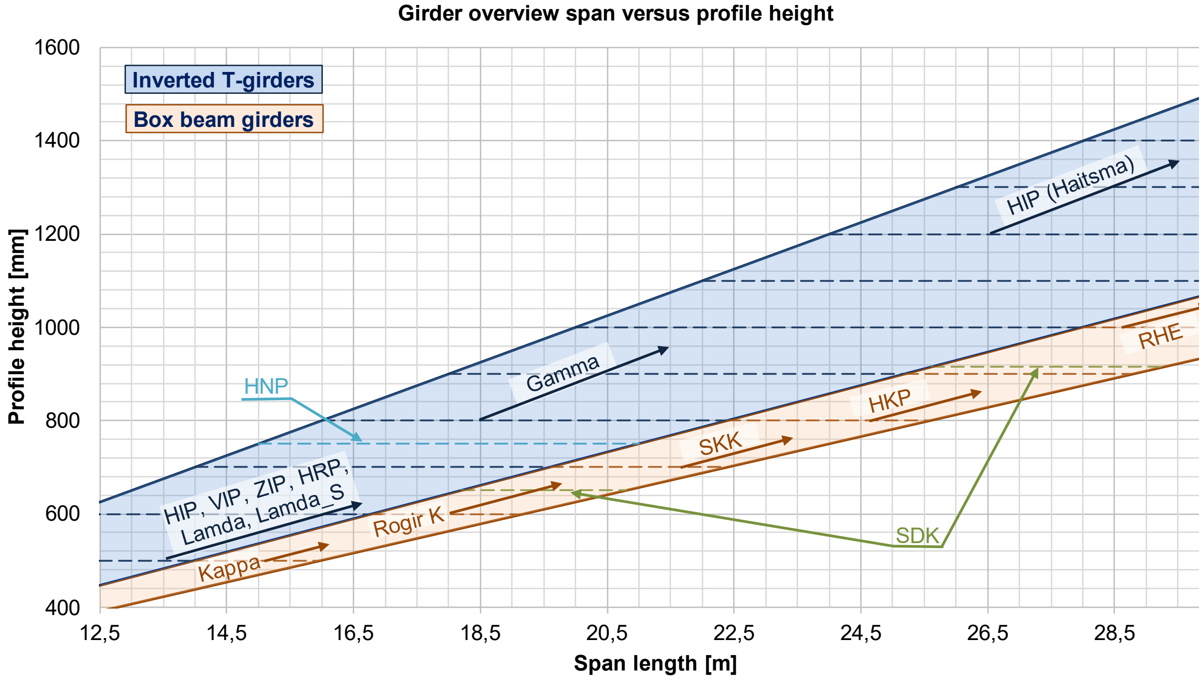


Figure 2.11: Overview of girder types and heights. Span length is shown on the horizontal axis and profile height on the vertical axes. The range for inverted T-girders is shown in blue and the range for box beam girders is shown in orange.

§2.3 Standards and materials

To assure safety and functionality constructions should comply with certain standards. Overtime these standards and regulations are revised due to changing demands from society and progressive insights in calculations methods and material behaviour. To reuse elements, it is important to know how and for which loads these elements are designed. Therefore, an overview of design standards and vehicle loading is given in the first two sections. In the third section regulations about the implementation of reused elements in new constructions is discussed.

Besides, the load and calculation procedures the material characteristics are important to guarantee the structural integrity. The three main elements in prefabricated concrete bridge girders are concrete, reinforcement and prestressing steel. For each material the common characteristics and relating testing techniques are discussed in section four to six. The final section provides a conclusion.

§2.3.1 Overview concrete design standards

Figure 2.12 shows a timeline of standards. The GVB was the first form of standards for concrete constructions. The VB and VBC are successors and nowadays the Eurocodes are available. The first national building act was published in 1992 and referred to the standards of the VBC 1990. Before this time municipalities had their own regulations in which they often referred to the standards. By the implementation of the Building Act 2012 the national guidelines for new constructions are officially replaced by the Eurocode [45].

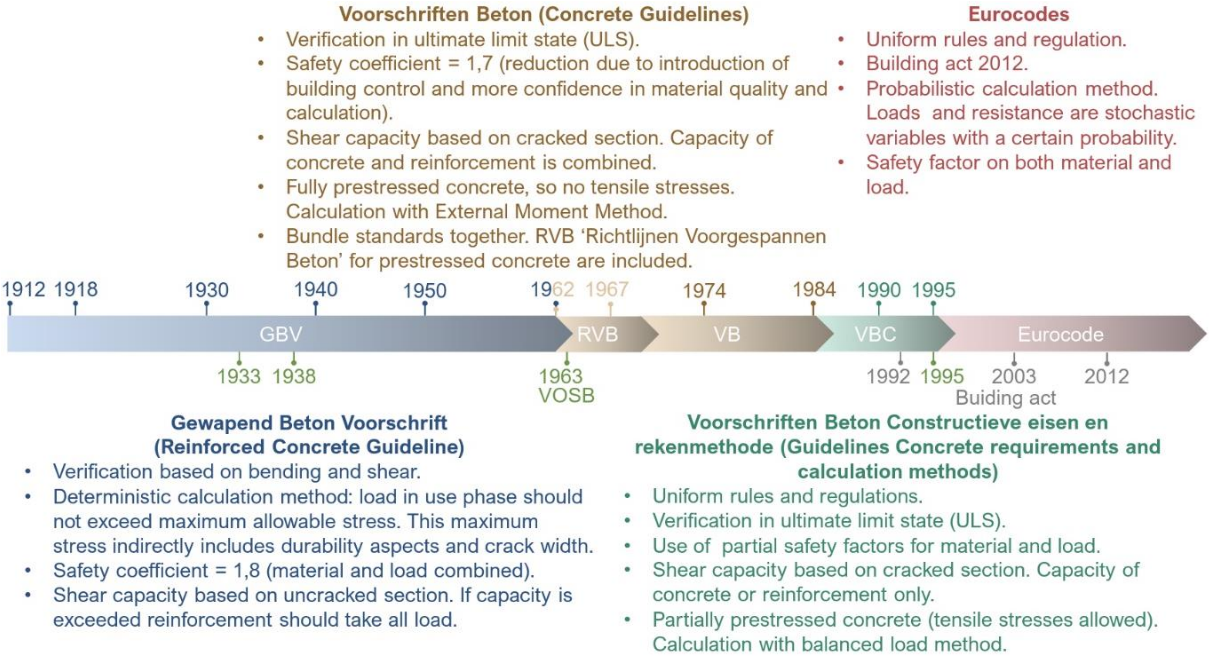


Figure 2.12: Timeline of Dutch design standards figure based on information from: [46], [47], [48], [49], [50], [51].

Over time regulations have changed, for example GBV calculations are based on maximum allowable stress. The theory behind was that stresses in steel should be limited to limit the crack widths. After more experiments, advanced knowledge and better material insights other factors as rebar diameter, distance and concrete cover are identified [46]. Eventually the maximum allowable stress method was replaced by the verification in ultimate limit state [47].

Another change is the calculation method for prestressing. Nowadays prestress is treated as applied load, however before the introduction of the VBC 1990 the external moment method ('uitwendige momenten methode' or 'parasitaire momenten') was used. The methods are comparable, but the bending moments and forces are not directly interchangeable. Therefore, design calculations based on this method cannot be used in new calculations [52]. Relating to prestress in older standards only full prestress was used. So, no tensile stresses were allowed in the concrete. By the introduction of the VBC in 1990 this changed and partial prestressing was allowed. In this way tensile stresses are allowed and can be resisted with reinforcement. The approach from the older standards is beneficial for reuse as it might give some reserve capacity to compensate for other loads, stricter requirements or a loss of prestress [49].

Also, shear standards have changed. In GVB shear capacity of concrete was based on the uncracked section. If concrete was not able to withstand the shear force the complete force should be resisted by the reinforcement. However, in many cases shear reinforcement was not necessary. In the VB, the calculation method changed to a cracked section and capacity of concrete and reinforcement could be combined [47]. As a result, shear reinforcement was needed more often. In 1976 Rijkswaterstaat started an investigation, because the large amount of shear reinforcement was questionable. This investigation led again to a reduction in reinforcement in the VBC [53], [54]. In the Eurocode, the calculation method again changed significantly. In former standards the angle of rupture was 45° but could now be lowered. In this way more stirrups could be activated. To make the calculation less complicated the contribution of concrete and reinforcement could no longer be combined.

Many existing structures will not fulfil the requirements of this new approach. As this may give problems in assessing existing structures and reusing elements the RKB (see Table 2.1) provides an alternative. For existing elements, the capacity of concrete and reinforcement can still be combined by taking the angle of rupture of 45° for reinforced concrete and 30° for prestressed concrete. Moreover, the Eurocode still provides an approach comparable to the approach from old standards for statically determinate structures in uncracked sections. In this case the shear capacity can be based on the tensile strength of concrete, hence girders after 1967 generally have sufficient shear capacity and stirrups to meet the requirements [55], [56].

§2.3.2 Overview load on bridges

The first guidelines for vehicle load on bridges originate from an order of the Ministry of Water Management in 1920. After several revisions, the first VOSB ('Voorschriften Ontwerp Stalen Bruggen) is introduced in 1933, which became an official standard in 1938. In 1963 a complete revision took place, followed by a small one in 1995. In 2001 the Eurocode came in place [51]. This is also shown in the timeline of Figure 2.12.

In the VOSB of 1938 traffic classes A to D are distinguished. Class A is the heaviest class and used for bridges in main roads. Class B is also for bridges in main roads, but exceptional traffic should be redirected over bridges belonging to class A. Class C is for bridges not used by heavy traffic and class D is meant for lightweight vehicles only. VOSB of 1963 and 1995 uses as similar classification, however class A is replaced by class 60, class B is replaced by class 45, class C by class 30 and class D no longer exist. Each lane is loaded with a distributed load and a vehicle. This vehicle has multiple axels in length (span) direction and each axle has a number of wheels over which the load is spread in width direction [57]. The loads and distances between the axles and wheels as well as the contact area of the wheel can be found in Figure 2.13 and Figure 2.14.

In VOSB 1963 a vehicle load was placed every 2,5 meters. With two vehicles the load was reduced to 90%, with three vehicles to 70% and with 4 or more to 60%. In later versions the lane width increased to three meters and a maximum of two vehicles are placed with a reduction of 20% of the load [49].

In both standards the loads are multiplied with an impact coefficient, to account for vibrations due to traffic. For longer bridges the coefficient is smaller, since the ratio between traffic load and total load is smaller. The same holds for concrete bridges, for which the ratio is smaller compared to steel ones. From 1963 external influences as temperature and wind are account for. Moreover, a load factor is introduced. For longer bridges the probability that the maximum loading occurs is smaller consequently the load can be reduced.

VOSB 1938

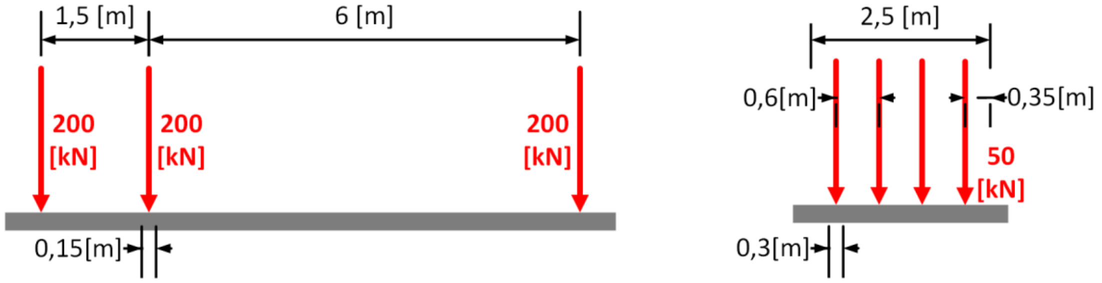


Figure 2.13 a): Distribution of load over axles in length directions (left) and wheels in width direction (right) for traffic class A.

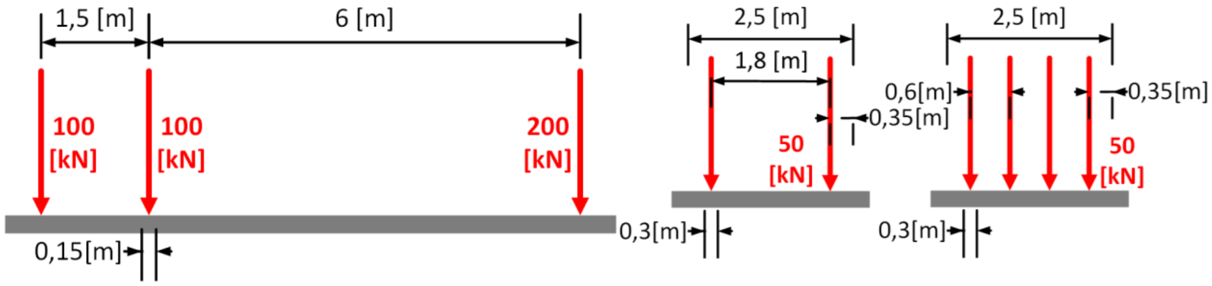


Figure 2.13 b): Distribution of load over axles in length directions (left) and wheels in width direction (right) for traffic class B. First division in width direction is the division for the axle load of 100 [kN] and the second for the axle load of 200 [kN].

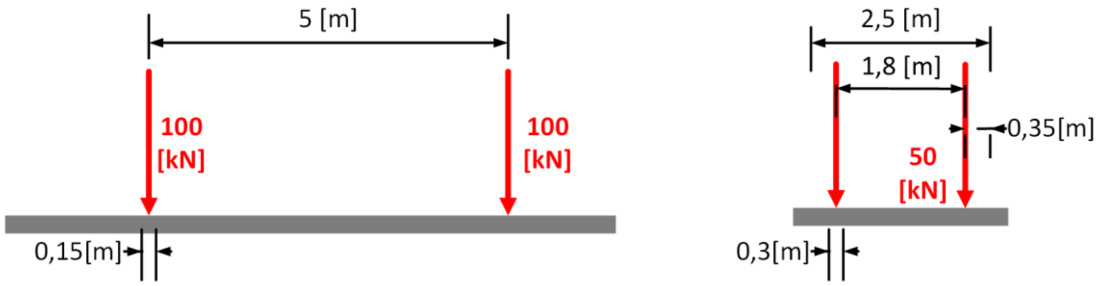


Figure 2.13 c): Distribution of load over axles in length directions (left) and wheels in width direction (right) for traffic class C.

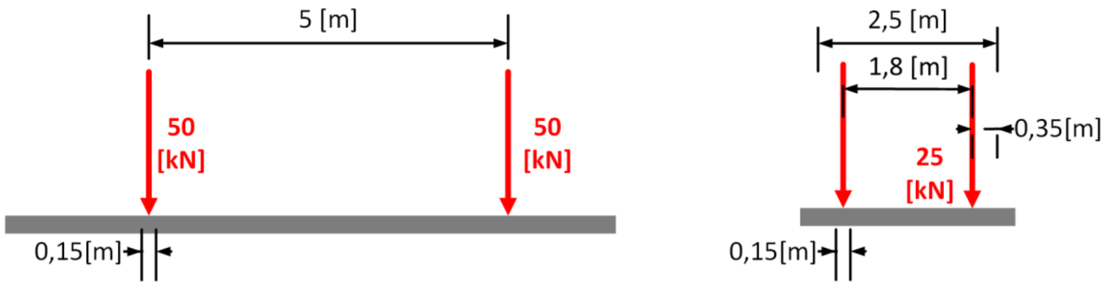


Figure 2.13 d): Distribution of load over axles in length directions (left) and wheels in width direction (right) for traffic class D.

Figure 2.13: Axle and wheel distribution VOSB 1938. On the left a cross-section in length direction, which shows the division of axle loads on a single lane. On the right a cross-section in width direction, which shows the division of the axle loads over the wheels. The distances between the axles and wheels are indicated and the size of the contact area is indicated in the bottom of the picture. Based on [57].

VOSB 1963

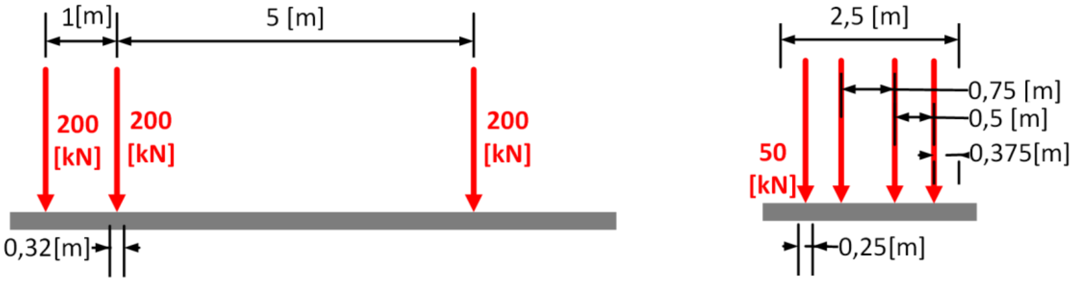


Figure 2.14 a) Distribution of load over axles in length directions (left) and wheels in width direction (right) for traffic class 60.

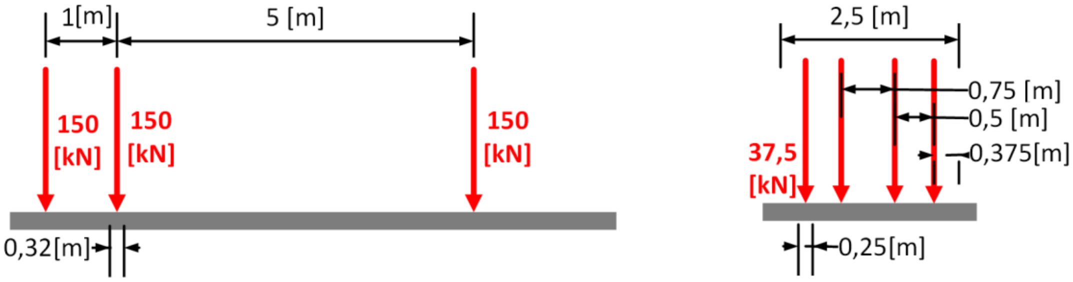


Figure 2.14 b) Distribution of load over axles in length directions (left) and wheels in width direction (right) for traffic class 45.

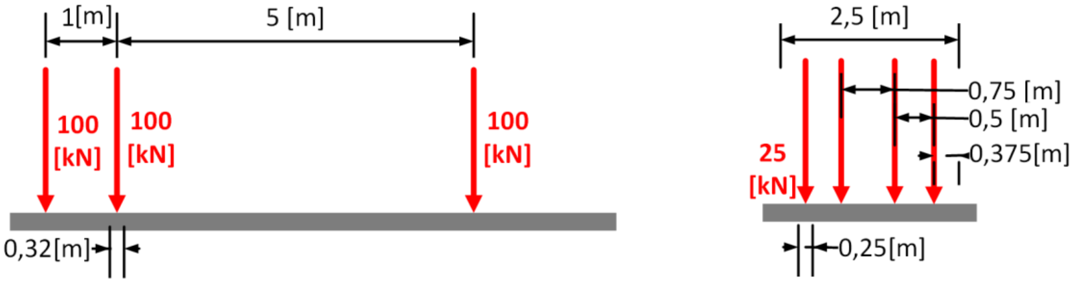


Figure 2.14 c): Distribution of load over axles in length directions (left) and wheels in width direction (right) for traffic class 30.

Figure 2.14: Axle and wheel distribution VOSB 1963. On the left a cross-section in length direction, which shows the division of axle loads on a single lane. On the right a cross-section in width direction, which shows the division of the axle loads over the wheels. The distances between the axles and wheels are indicated and the size of the contact area is indicated in the bottom of the picture. Based on [57].

Nowadays the loads are calculated according to the Eurocode. There is no distinction in traffic classes anymore. The idea is that a heavy vehicle should be able to drive everywhere, however on some roads the probability of occurrence is lower. Therefore, correction factors can be applied. There also exists a correction factor for bridges longer than 200 meters. Four load models can be considered. The first one is used to verify the ultimate limit state condition of the main load bearing structure. The lane width is three meters. On the first lane a distributed load of 9 [kN/m^2] is applied and two axle loads of 300 [kN] dived over two wheels. On the remaining lanes and area 2,5 [kN/m^2] of distributed load is applied. Moreover, on the second lane two axle loads of 200 [kN] are applied and on the third one two axle loads of 100 [kN]. The division on the first lane is shown in Figure 2.15.

The second load model consists of a single axle load with a contact area and is used to verify the local load effects. The third model deals with special vehicles and the fourth one deals with crowd loading. This last load model should only be considered if relevant or asked by client.

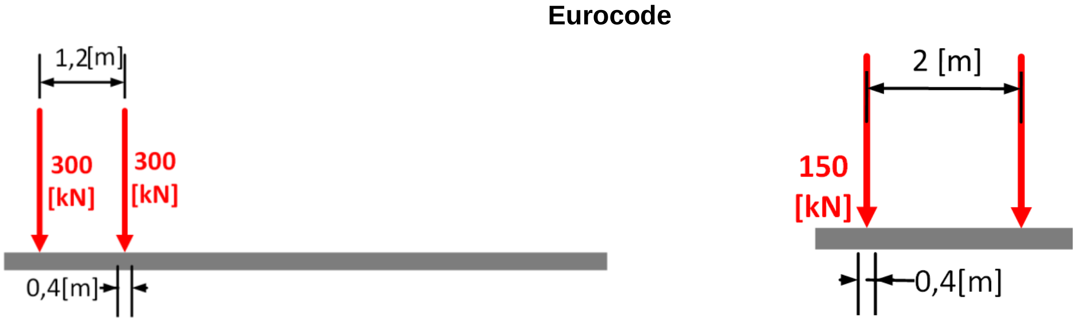


Figure 2.15: Axle and wheel distribution Eurocode. On the left a cross-section in length direction, which shows the division of axle loads on a single lane. On the right a cross-section in width direction, which shows the division of the axle loads over the wheels. The distances between the axles and wheels are indicated and the size of the contact area is indicated in the bottom of the picture.

§2.3.3 Reuse of elements

New constructions should meet the requirements from the Building Act (Bouwbesluit), which refers to the standards in the Eurocode. Moreover, for civil objects owned by Rijkswaterstaat extra guidelines, the ROK should be applied. The assessment of existing structures should be done according to the Dutch NEN 8700 series, as no Eurocode exists [45]. Again, for objects from Rijkswaterstaat an extra guideline, the RBK is available. Actually, reuse is something in between new and existing. It is a mix, where currently no official standards, nor guidelines or procedures are available for.

The process of implementing a reused element in a new construction can be divided in different steps. The first one, the inventory, takes place when the to be demolished structure is still in function. The elements that will be released are listed and a first visual inspection is performed. In a visual inspection the condition, potential damage and the demountability are assessed, which gives a first estimation about the reusability [59]. During this inspection, the element should still be in its old function. After all, the quality can be demonstrated during use [55].

In the next step the in-depth assessment and certification takes place. Preferably most of this phase also takes place before deconstruction. This increases the chances of reuse and prevents unnecessary demolition or disassembly of elements [59]. First information is gathered with archive and desktop research. Design, specification and reinforcement drawings, but also reports, calculations, design loads and standards may be found. Recent information is probably easier to find as most of it will be digital. This may relate to inspections, repairs, modifications and verification. Also, information regarding overloading or accidents is valuable. Apart from information relating to the specific structure, investigations and information about comparable structures build during the same period can be used to get a better indication of the characteristics of the elements. Another way of gathering information is field observations and testing.

Material testing is deemed necessary if archives do not provide sufficient information, if deviations are observed or material characteristic used in design are nowadays seen as too optimistic. Also, if material deterioration is suspected, for example ASR, testing is required [60]. In this case the key characteristics necessary should be determined. Followed by the appropriate test methods. For example, if the concrete compressive strength is known other characteristics like the tensile capacity can be derived. In this way the number of test techniques can be kept minimum, which saves time and money. In case of (semi)-destructive testing the constructive consequences should be kept in mind. Another attention point is the number of samples and the location of sampling. More samples give more accurate results, but also costs more time and money. For the sampling locations differences in quality and characteristics as well as the constructive consequences should be kept in mind.

Finally, the results cannot be directly used in verification calculations. The test values should be translated to calculation values, using statistical distributions [60]. In the next sections more information is provided regarding material characteristics and testing.

The assessment of elements and the determination of the remaining service life can be based on the NEN 8700 series in combination with the RBK. Moreover, for the different type of inspections and tests NEN standards and CUR recommendations are available. An overview of all relevant codes is presented in Table 2.1. This assessment is extremely important to determine the characteristics of the elements, however the assessment cannot be used to verify a new design. This is because these guidelines are only valid for existing structures with a lower residual lifetime. Nonetheless, standards for new structures do not include determining the characteristics and quality of the materials used, since new building materials are certified at the production with a CE certification. However, for second hand elements such a certification does not yet exist [59]. So, the element characteristics, material characteristics, remaining service life and capacity are determined based on standards for existing structures. Next, these characteristics are used in the calculations for a new design according to the standards and guidelines for new to build constructions [55], [61], [62].

A new design starts with the requirements from the client. What kind of bridge should it be? What kind of traffic passes the bridge? Are there any exceptional loads that should be considered? What is the span length? Etc. Requirements from standards and guidelines follow from this client's input. Usually in a design process based on these requirements the load combinations follow and suitable girder dimensions are chosen. However, with reused elements this process is more iterative. The reused elements are verified based on the load combinations that follow from the requirements. If the calculations shows that the reused girders cannot fulfil the requirements it should be determined which requirements are critical. Then the requirements can be reconsidered or a solution can be found to still meet the requirements [63]. During this iterative process additional material tests might be necessary.

To conclude an overview of the process is given in Figure 2.16. The process for the new construction is shown in purple. A new structure starts with a design that needs to be verified, before it can be constructed. The process related to the existing structure is shown in blue and green. Two routes are possible depending on the interaction and time between demolition and new design. In case the demolishment of the existing structure and the construction of the new construction directly follow up on each other the bottom route is followed. Inspection, and assessment take place and then the structure is deconstructed and directly used in the new construction. In case there is more time between demolishment and construction the top route is followed. After inspection the existing structure is deconstructed and stored until it has potential to be used in a new design. Then the assessment starts and the new construction can be built.

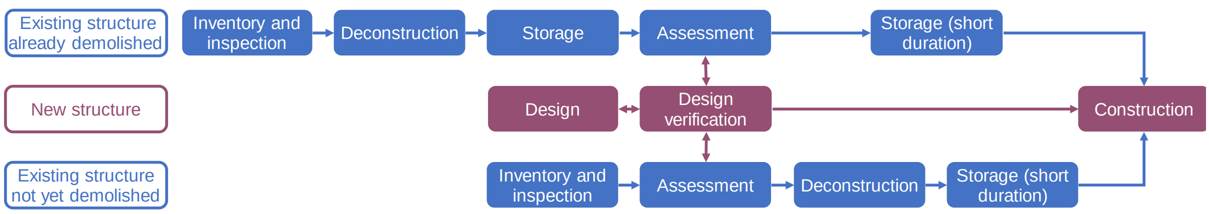


Figure 2.16: Overview process with implementing reuse

Table 2.1: Overview relevant standards and guidelines.

New structures
NEN-EN 1990: Basis of structural design
NEN-EN 1991: Action on structures - *Part 1-4: General actions – Wind actions* - *Part 1-5: General actions – Thermal actions* - *Part 1-7: General actions – Accidental actions* - *Part 2: Traffic loads on bridges*
NEN-EN 1992: Design of concrete structures - *Part 1-1: General rules and rules for buildings* - *Part 2: Concrete bridges*
ROK: Richtlijnen Ontwerp Kunstwerken from Rijkswaterstaat (guidelines design of civil objects)
Building Act 2012

Existing structures
NEN 8700: Assessment of existing structures in case of reconstruction and disapproval - *NEN 8700: Basic rules* - *NEN 8701: Actions* - *NEN 8702: Concrete structures*
NEN 2767: Assessment built environment - *Part 1: methodology* - *Part 4: Assessment infrastructure*
RBK: Richtlijnen Beoordeling Kunstwerken from Rijkswaterstaat (guidelines assessment of civil objects)
CUR-Recommendation 121: Determination of the lower limit of expected residual life of existing reinforced concrete structures

Testing
CUR-Recommendation 72: Inspection and investigation of concrete structures
CUR-Recommendation 117: Inspection and advice civil objects
NEN-EN 12504: Testing in concrete structures - *Part 1: Cored specimens - Taking, examining and testing in compression* - *Part 3: Determination of pull-out force*
NEN-EN 13791: Assessment of in-situ compressive strength in structures and precast concrete components
NEN-EN 12390: Testing hardened concrete - *Part 1: Shape, dimensions and other requirements for specimens and moulds* - *Part 2: Making and curing specimens for strength tests* - *Part 3: Compressive strength of test specimens* - *Part 4: Compressive strength – Specification for testing machines* - *Part 5: Flexural strength of test specimens* - *Part 6: Tensile splitting strength of test specimens* - *Part 10: Determination of the carbonation resistance of concrete at atmospheric levels of carbon dioxide* - *Part 11: Determination of the chloride resistance of concrete unidirectional diffusion* - *Part 12: Determination of the carbonation resistance of concrete – Accelerated carbonation method* - *Part 13: Determination of secant modulus of elasticity in compression* - *Part 18: Determination of the chloride migration coefficient* - *Part 19: Determination of resistivity*

Modification and reparation
CUR-Recommendation 118 Specialistic conservation techniques, repairing concrete
NEN-EN 1504: Products and systems for the protection and repair of concrete structures – Definitions requirements, quality control and evaluation of conformity - *Part 1: Definitions* - *Part 3: Structural and non-structural repair* - *Part 4: Structural bonding* - *Part 5: Concrete injection* - *Part 6: Anchoring of reinforcing steel bar*
CUR-Recommendation 91: Strengthening of reinforced concrete structures with carbon fibre reinforced polymer

§2.3.4 Concrete

The compressive strength of concrete is an essential material characteristic in the assessment of an element. With equations from Eurocode the compressive strength can be used to calculate for example the tensile strength and the elastic modulus. If the year of construction is known the lowest concrete strength class according to the standards valid at that time can be used. Based on research of Rijkswaterstaat concrete class C55/67 can be used for girders built before 1976. If no information is available, the currently lowest possible concrete strength class C16/20 can be used. Of course, these values are only applicable if no damage is observed [64]. If the concrete class is known from archives, this can be translated to current strength classes with the RBK.

It is recommended to perform concrete compressive tests on drilled cores to determine the compressive strength, because the above-mentioned values are conservative. With testing a higher strength will be found, which is favourable for the shear and moment capacity [46] . First of all, the strength in design is based on the 28-day strength, but since these structures are much older the strength has increased. Moreover, often higher strength concrete is used than stated in design specification, because the concrete had to be able to withstand the prestress that was applied after 16 hours [62]. If the characteristic compressive strength is based on tests the strength should be multiplied with k_t factor of 0,85 apart from the material safety factor to calculate the design compressive strength [52]. This factor is used because the design strength should be based on 28-day strength and the tests are performed after 28 days, when the strength has increased.

The Eurocode provides a formula for the strength increase overtime. There also exist a CUR-Recommendation about this topic. However, for the assessment of existing structures, it is not allowed to use this formula. For this reason, it is assumed that is not allowed for reuse either.

Testing should be done in accordance with the NEN standards, moreover Rijkswaterstaat also has guidelines regarding coring and testing. Another frequently used indirective test method for compressive strength is the Schmidt Hammer, however this method is disapproved as the correlation with strength is relative weak and influenced by other concrete characteristics [64]. Apart from the concrete compressive strength deterioration mechanisms should be investigated as well. As these mechanisms determine the residual life of the concrete element. It is expected that carbonation and chloride ingress will not be a problem for prefabricated girders, but this strongly depends on cement type and environment. Test results of girders in Poland and the Netherlands are shown in Table 2.2. As can be seen during new service life in similar environment the reinforcement will not be affected, since the concrete cover will be larger than the carbonation depth. In the GBV 1912 the minimum cover is already 15 [mm] and in newer design codes this cover has only increased. Therefore, carbonation will not reach the reinforcement and will not be a problem. Also, almost no chloride ingress was found, which is usually considered as the dominant factor for the remaining service life. This is the result of dense concrete needed for the high strength in prefabricated prestressed concrete. This dense structure also reduces the risk on alkali silica reaction (ASR) [62].

The carbonation depth can be measured with a phenolphthalein spray test. The pink colour of the spray disappears if the concrete is carbonated. This spray should be applied on a fresh fracture surface, which can be created by splitting of a core or by a local outcrop. The chloride ingress can be measured by determining the chloride concentration of concrete powder from different depths released during core drilling. If ASR or other deterioration mechanisms are suspected a microscopic investigation is needed, however this should only be done if necessary as it is expensive and time consuming.

Table 2.2: Result of tests on carbonation depth.

Location Girders	Groningen (Netherlands)	Poland
Service years	35 [62]	30-50 [65]
Carbonation depth	1 [mm] [62]	2-3 [mm] [65]
Carbonation coefficient: $K = \frac{carbonation\ depth}{\sqrt{service years}}$	K = 0,17 [mm/year]	K = 0,28-0,55 [mm/year]
Carbonation depth after service life of 100 years in new construction	**2,0 [mm]**	**3,5 – 6,2 [mm]**

§2.3.5 Reinforcement

The type of steel, yield strength and layout are the minimum characteristics of the reinforcement that should be known. The type of steel is often indicated on design drawing or can be derived from anchoring details. In case of ribbed bars, the anchoring is straight, while with smooth bars the anchoring is angled. During the GBV smooth bars are often used as secondary reinforcement and have a yield strength lower than 240 [N/mm^2]. The main reinforcement has higher strength and are always ribbed bars.

Like the concrete compressive strength if the year of construction is known the lowest yield strength according to the standards valid at that time can be used. As described in §2.3.1 during GBV the calculation method was based on allowable steel stress. This allowable stress was based on safety factors not only of the material, but also of the load. As a result, the allowable stress is much lower compared to the nowadays used yield strength divided by material factor. For higher steel strengths the difference is even larger, because requirements as crack width where also implicitly included. In the assessment of structures or new calculations the nowadays used method (f_{yd}/γ_m) can be used. In this way the capacity of the structure increases, however requirements around crack width and anchoring length should be verified. If these aspects are critical the allowable stress should be limited according to the old standards. Especially the crack width requirement can become critical since older structures often have a smaller concrete cover than nowadays required [49].

The layout of the reinforcement consists of location, cover and geometry of reinforcement. To verify the information from archives measurements can be performed with a cover meter, which is based on electromagnetic fields. If insufficient information is available or the information is questionable supplementary destructive testing can be performed in which the reinforcement is locally exposed and a small part is removed for material testing. So, destructive testing is used locally to determine the type of steel, diameter and strength characteristics. This is done at location with small bending moments and small shear forces. Non-destructive testing is used globally to determine the reinforcement configuration. Especially at location where high tensile stresses are expected [60].

The reinforcement steel can be subjected to material tests in which yield strength, tensile capacity and rupture strains are measured. However, the benefits of these tests regarding capacity are small. Moreover, many samples are needed. So, unless no information is available testing is not recommended.

§2.3.6 Prestressing

In prefabricate girders before 1965 often prestressing rods or wires are used. After this time prestress is applied with strands. The type, geometry, layout and characteristic tensile strength of prestressing should be derived from archives. Often the prestressing characteristics are available on the specification drawings only [46]. Unlike, concrete compressive strength and yield strength of reinforcement no minimum value can be used because prestressing generates load on the structure. In addition, destructive material testing is not possible, because it can be dangerous and it would damage the prestressing system and affect the bearing capacity. If no or limited information is available, characteristics may be derived from comparable structures of the same supplier with a supplementary sensitivity analysis [55].

The type of prestressing steel is related to the maximum stress allowed during stressing and the maximum initial stress. The maximum stress allowed during stressing is the stress in the steel before the occurrence of direct losses as elastic deformation and friction. The maximum initial stress is the stress directly after the occurrence of these losses. To calculate friction losses the wobble factor and friction coefficient should follow from archive information, otherwise a friction factor of 0,26 and wobble factor of 0,01 can be used.

The final stress is the stress that occurs after all time dependent losses have occurred. These time dependent losses are related to creep, shrinkage and relaxation. For structures after 1974 time-dependent losses can be taken as 12%. For structures before this time the relaxation was larger, so 17% should be used. With the reused elements, depending on the age, (almost) all losses have occurred, so this stress can be used in further calculations. An indication of the development of the losses over time is shown in Figure 2.17. The equation and values used for this figure are explained at the end of this section.

Especially between 1955 and 1965 in some structures tempered prestressing steel was used, which is vulnerable for hydrogen induced cracking. This type of corrosion causes fracture in a brittle manner when exposed to chlorides or hydrogen sulphides. Tempered prestressing steel might have been used in prestressing system of “Polensky en Zöllner” or “KA-systems.” Girders are assumed not suitable for reuse if tempered steel is used, because of the greater risk on unpredictably brittle failure [46].

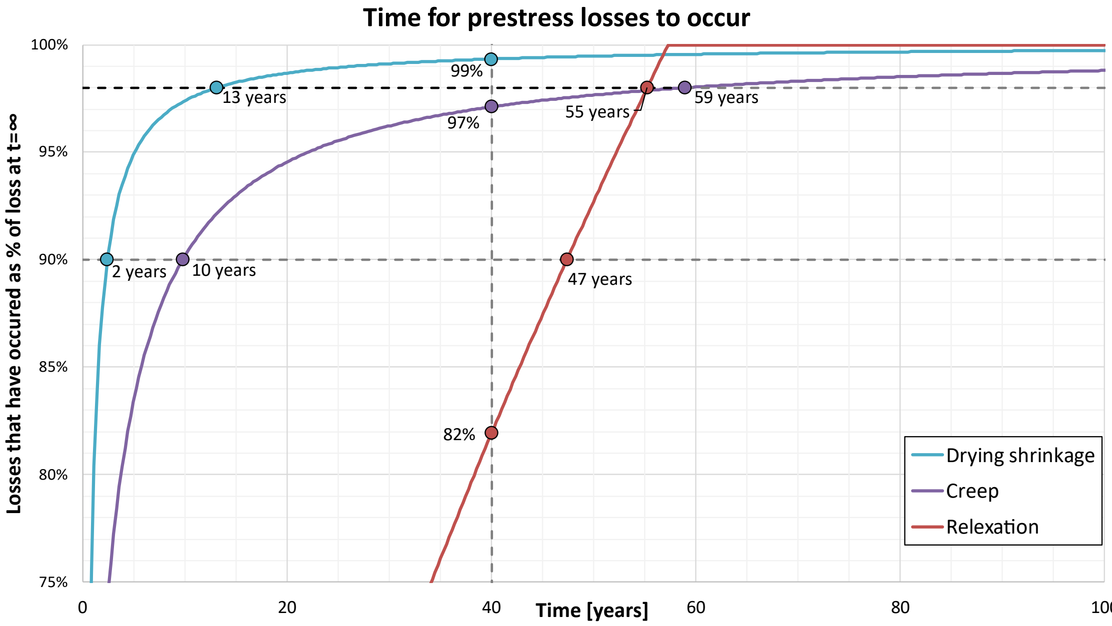


Figure 2.17: Indication of development of time-dependent losses. After 40 years (average age of girder) approximately 82% of all relaxation losses have occurred, 97% of the creep and 99% of drying shrinkage.

Explanation by Figure 2.17

- For shrinkage only drying shrinkage is considered in this figure. The other type of shrinkage is autogenous shrinkages. However, this shrinkage reaches 99% of the ultimate value in less than two years. The time dependent factor for drying shrinkage is shown in Equation 2.1. The time at the start of shrinkage is taken as zero. Normally, this will be a couple of days. However, for this consideration over many years it is not of influence. The fictive thickness of a HIP 800 girder, which is 181 [mm] is used in the calculation. For other inverted-T beams this value slightly differs, but not significantly.

Equation 2.1: Time dependent factor drying shrinkage.

$$\beta_{ds}(t,t_s) = \frac{(t-t_s)}{(t-t_s)+0{,}04\sqrt{h_0^3}}$$

t= age (days)
t_s= age (days) at start of shrinkage $t_s \approx 0$
h_0= fictive thickness of cross-section
$= \frac{2\times A_c}{u}$
A_c= cross sectional area
u= circumference subjected to drying shrinkage

- The time dependent factor for creep is shown in Equation 2.2. Similar to the shrinkage the time at load application is taken as 0. The coefficient for relative humidity depends on the relative humidity in the surrounding of the element, the theoretically thickness of the elements and the concrete strength. The maximum value however is 1500 and is used in this calculation. With smaller values creep will occur sooner.

Equation 2.2: Time dependent factor creep

$$\beta_c(t,t_0) = \left(\frac{(t-t_0)}{(t-t_0)+\beta_H}\right)^{0,3}$$

t= age (days)
t_0= age (days) at load application $t_0 \approx 0$
β_H= coefficient for relative humidity $\beta_H \leq 1500$

- Relaxation losses can be described with Equation 2.3. This is for a class 2 prestressing steel; however the time dependence does not significantly change for other classes. For the final relaxation loss the time should be taken as 500.000 hours, which is approximately 57 years. To generate the line from Figure 2.17. the relaxation values before the age of 57 years are related to the value at 57 years. After 57 years all losses are assumed to have occurred, which results in a straight line at 100%.

Equation 2.3: Relaxation loss

$$\frac{\Delta\sigma_p}{\sigma_{p,inital}}(t) = 0{,}66 \times \rho_{1000} \times e^{9,1\times\mu}\left(\frac{t}{1000}\right)^{0,75\times(1-\mu)}$$

t= age (hours)
μ= friction factor $\mu \approx 0{,}26$
$\rho_{1000} = 2{,}5$ [%] relaxation loss at 1000 hours

§2.3.7 <u>How is the structural integrity of a design with reusing existing girders assessed?</u>

In conclusion, there are no specific regulations for reuse. Therefore, in new constructions reused elements should meet the current requirements for new constructions. Existing girders are likely able to meet these requirements due to an increase in concrete compressive strength and robust design principles used. Former guidelines that are used during the construction of existing girders differ in calculation methods for prestressing and shear capacity as well as safety approach and load models. Therefore, design calculations and drawings can not be used on to one but are a valuable reference. Furthermore, the characteristics of materials have developed overtime and are reported in different ways. However, in many cases transition tables or formulas are available in guidelines and standards about assessing existing structures. Since, standards from new to build constructions do not describe methods to determine material characteristics, guidelines for existing structures might still be used for this purpose. Material characteristics of existing elements can be based on archive information together with inspections or material testing. Research on existing structures provides some minimal characteristics that can be used as well.

§2.4 Deconstruction, modification, strengthening and repair

Prior to reuse girders have to be extracted from the existing structure, which is described in the first section. In the next sections modification, repair and strengthening measures are discussed, which may be needed to make them suitable for use in the new construction. Finally, a conclusion about the technically feasibility is provided.

§2.4.1 Deconstruction of bridge deck

Before elements can be reused, they must be removed from the old structure. So, the existing structure has to be deconstructed. In the Dutch infrastructure time, traffic hindrance and noise hindrance to the surroundings are the main influencing factors in the choice for demolition process. Besides, the safety, stability and capacity of the structure during deconstruction should be guaranteed.

The process starts with removing the asphalt layers and non-structural elements as guiding rails, traffic signs and streetlights [66]. This is similar compared to demolition. Next in a traditional process the complete structure is demolished with grapples, crushers, pulverizes, shears and hammers [67]. In advance, the structure can be weakened with blasting [68]. Next the debris is pulverized and further processed, which can also be done at another location. In case of a viaduct, the underlying road is often protected with a temporary sand layer [69]. In viaducts over train tracks there are problems with falling debris, as the catenary system should not be damaged. In this case the viaduct should be cut into parts and larger parts of debris are transported and further processed at another location [66]. This is already more comparable to deconstruction. However, with deconstruction, it is important that distinctions are made between the elements. In addition, the elements should be removed carefully, without creating damage. Suitable methods are rotary sawing, wire cutting, pneumatic hammering, hydro demolishing and drilling, which are described in Table 2.3.

To remove bridge girders rotary saws can be used to cut through the cast-in-situ layer and disconnect the girders. The existence of cross and edge beams complicates and elongates the process. These beams can be detached by using wire cutting if the wire can be wrapped around the beam. Otherwise, stitch drilling is a suitable option. This total process takes approximately a week per span, however without cross and edge beams the process is quicker. When the girders are detached, they can be put on transport. The edge beams are often the decisive factor for transport [70]. Although transport can be a complicating or governing factor, it will not be a limiting factor, because the girders are prefabricated. So, they are brought to location during construction as well. However, during construction the surrounding area or circumstances as well as possible means of transport might differ.

An option is to keep the in-situ concrete layer and construct a new in-situ bridge deck on top of this layer. In this way the shear capacity of the girder increases. It is also favourable for the moment capacity and the new in-situ deck can be thinner, which saves raw materials. However, disadvantages are the increase in construction height and the often lower quality and control of cast-in-situ concrete [70]. Therefore, in most cases the layer should be removed. This can be done when the girders are still in the existing structure but can also be done at the new construction site or in a storage facility. To remove a concrete deck from an existing bridge sawing, hydro demolition and hammering are commonly used [71]. The ease of removal depends on the adhesion between girder and deck [22]. The situation for removing the deck after extracting the girders from the structure is comparable.

§2.4.2 Modification of girders

To make the bridge girders from the ‘old’ construction suitable for reuse in a new construction shortening and/or a change of skew angle might be necessary. Another possible modification is applying protection or increasing the concrete cover to meet durability requirements.

By definition, a shorter span results in smaller bending moments and shear forces. However, the bearing capacity of the girder stays the same. Therefore, in general a higher load can be carried if girders are shortened. A point of concern is splitting reinforcement. This reinforcement is applied at the ends to introduce the prestress and avoid spalling cracks. This reinforcement is lost with shortening. Due to time dependent losses the prestresses force has reduced. Moreover, the prestressing force was applied 16 hours after casting, so the tensile strength of concrete was not fully developed. So, often this reinforcement is not needed anymore, but this should be verified [70]. Finally, due to the tendon layout shortening can have a negative influence on the capacity at the support.

Shortening of prefabricated prestressed bridge girders until approximately 20% or a change of skew angle is possible with rotary or wire sawing [22]. During the execution sufficient support of the girder is required. Moreover, due to the shortening it is possible that stirrups and other reinforcement end up with too little cover. This reinforcement should be protected against corrosion [70].

Another potential modification for inverted T-girders is shortening in width direction. A part of the flanges can be cut off. By reducing the width of the girders, more girders are available to carry the loads [122]. The shear capacity is governed by the thickness of the web, so with more girders the shear capacity of the total bridge deck increases. Also, the bending moment that the girders should carry reduces, however prestressing tendons present in the flanges might be cut off. Therefore, the influence on the bending moment capacity is girder and situation depended. By cutting-off the flanges stirrup reinforcement might become exposed. This reinforcement should be protected against corrosion, with a protective coating or mortar. Another option is connection the flanges of the girders together with an in-situ layer. But in this case additional formwork is needed.

The concrete cover protects the reinforcement and/or prestressing against corrosion due to external influences and assures the transfer of anchorage forces. If the existing cover is smaller than the minimum required for durability the cover can be increased by applying an additional layer of concrete. The minimum cover for anchorage is equal to the diameter of the rebar. However according to NEN 8702 a cover until 0,5 x the diameter can also be sufficient. In this maximum case the anchorage length should be increased with a factor of 1,9 since the anchorage stress will decrease.

§2.4.3 Repair of girders

Girders might be damaged during their service life. By choosing the right methods and executing them with care the damage during deconstruction, transport and modification should be avoided as much as possible. The standards related to repair are presented in Table 2.1.
Reparations can be divided in three categories: esthetical, technical and constructional. Esthetical reparations are required for superficial damages that does not affect the durability, functionality or capacity of the element. At these places no reinforcement is present. For example, if the concrete cover is slightly damaged and is repaired with mortar. Moreover, the girders can be hosed down.

Technical reparations focus on the durability of the element. With these reparations reinforcement is present and to guarantee durability corrosion should be prevented. This type of reparation might be needed after shortening the girders and exposure of the reinforcement. Another example is reapplying or straightening connecting reinforcement.

Constructional reparations are needed if the capacity of the element is affected by the damage. However, it is not always possible to perform these reparations or they are too costly or have too much environmental impact for reused elements. For example, in case prestressing cables are damaged during testing, deconstruction or modification repair is not possible. In this case the damage can be considered in the verification calculations [22]. If this gives problems strengthening of the girder might be a suitable option [84].

§2.4.4 Strengthening of girders

Whenever existing girders do not have sufficient bearing capacity for reuse in new construction, strengthening may be possible. A distinction can be made between increasing the bending moment capacity, increasing the shear capacity and increasing the stiffness of an element. Moreover, active and passive methods exist. Active methods increase the capacity of a structure immediately after application and are suitable for all types of strengthening. Passive methods only become active when the load reaches a certain value and are suitable to increase the load bearing capacity, but only have a limited or no effect on the stiffness [79].

The feasibility of strengthening methods for existing structures is based on many factors. For example: strength increase needed, constructability, hindrance during application, aesthetics and available height [79]. But also, the economic costs, vulnerability to vandalism and maintenance are decisive factors. Reused elements are strengthened in a storage facility. So, they are not part of a structure and accessible from all sides. This makes hindrance during application and constructability insignificant. However, many other factors are even more important. For existing structures strengthening is an ultimate measure to continue the service life of the structure. Therefore, the aesthetics of the measure is not a decisive factor. Moreover, existing structures have a relative short remaining service life, so the strengthening method can have too. Regular inspections and maintenance are not an issue, because other parts of the existing structure need this as well. However, this does not apply for new structures. An aesthetically appealing structure is required because, visible measures intensify the negative associations with reused elements. Moreover, frequent maintenance and thorough inspections are not preferred. Also, the costs and environmental impact of strengthening should not lead to the exceedance of the price and impact of a new girder.

To conclude not all strengthening methods will be suitable for reusing elements. For these cases it more economical, practical or more sustainable to use new girders and find another destination for the existing ones. Table 2.4 gives an overview of strengthening methods and their suitability for reuse.

§2.4.5 Is reusing concrete bridge girders technically feasible?

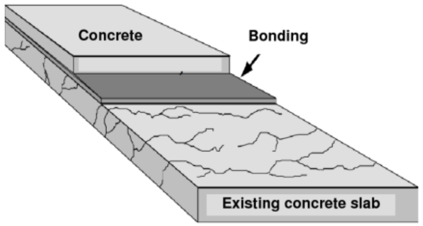
It can be concluded that reusing bridge girders is technically possible. After deconstruction modifications as shortening or change in skew angle or possible. Moreover, the cast-in-situ deck can be removed or kept based on strength, quality and construction height. Nonetheless, the deconstruction and modification process are more complicated and takes longer than the traditional process demolition and manufacturing of girders.

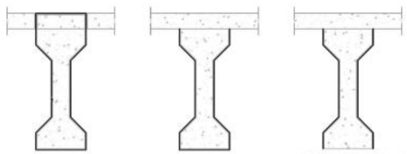
Table 2.3: Demolishing tools that can be used in deconstruction processes as well. Information from: [3], [47],[48], [49], [50], [51]

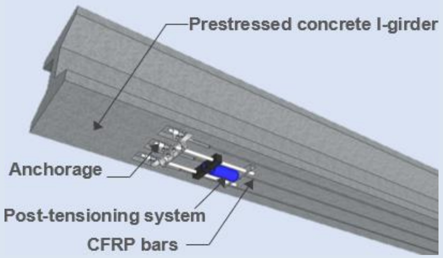
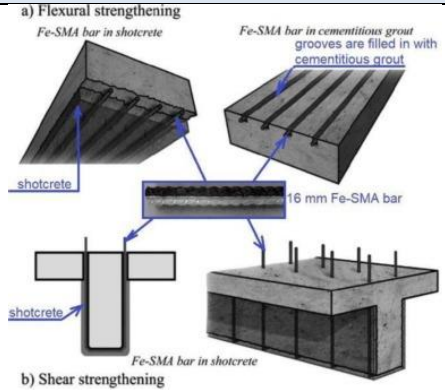
Method	Description	Picture
Rotary saw [75]	Rotary saws are used for reinforced elements on easily accessible locations. Sawing creates clean and accurate cuts. Since, the structure is cut in specific pieces there is no debris. The method is safe and easy. Moreover, limited vibrations are created. Dry sawing creates dusts; however, this is limited if wet sawing is used. In case of deconstruction and reuse wet sawing is preferred anyway as it prevents crumbling of concrete. For reuse a disadvantage may be that the cuts go through the reinforcement. So, for example connection reinforcement gets lost. Also, the smooth cut is disadvantageous for the adhesion with new concrete layers. So, surfaces may require roughening. Different cutting angles are possible; however, the depth is limited to 30-35 [cm]. This limits the use to detach for example crossbeams.	
Wire cutting [76]	With wire cutting the wire is wrapped around the element to be cut or a hole is drilled to insert the wire. This makes the method suitable for cutting members that are out of reach for other equipment. Again, it creates clean and controlled cuts and it can be performed dry or wet. Since, the remaining structure is not damaged the method is very suitable for deconstruction and reuse. Limitations are the length of the wire and the available cutting angle.	
Hammering [77]	With hammering concrete is broken into small pieces. As a result, this method creates failing materials, debris and dust. Different machines are possible. For deconstruction and reuse the power and the weight of the machine should be limited, as otherwise the risk on damaging the elements increases. As a result, the process is labour intensive and time-consuming. Hammering is often used to detach concrete elements without or with a limited amount of reinforcement. Since, the reinforcement can be saved this method is very suitable for partial demolishment and reuse of elements. It should be kept in mind that one of the elements around other elements cannot be reused and is demolished. In case of bridge decks, the girders and connection reinforcement are saved, while the in-situ concrete layer is demolished.	
Hydro demolition [78]	With hydro demolition or water jetting a high-water pressure breaks the concrete. The method is often used for partial removal of concrete. For example, to remove deteriorated concrete layers or remove the bridge deck. The method can be very precise and the concrete can be removed without damaging the surrounding concrete. Furthermore, the remaining surface is clean and has good adhesion to new concrete. With normal water the reinforcement remains intact and gets cleaned and cleared from corrosion. If the method is used for demolishing and the reinforcement should be removed as well abrasive water or adhesives can be used. A big disadvantage is the environmental impact of the method because of the water amount needed and the wastewater, slurry and debris produced.	

Drilling [70]	With drilling holes or cores are drilled. Often this is not a sole removal method, but the first step. In case of demolition holes can be used to place splitting equipment or blasting agents. For deconstruction and reuse it can be used to define the cut direction or weaken the component. For example, drill a hole to place the cutting wire. Another possibility is stitch drilling. In this case small overlapping holes around the perimeter of a specific concrete area are drilled. This method can be used to remove cross and side beams.	

Table 2.4: Methods to strengthen concrete girders. In the first column the general name of the method, followed by a description and picture in second and third. Fourth column shows for which strength deficiencies the method is suitable. The final column shows if the method is suitable for reuse.

Method	Description	Picture	Capacity increase	Reuse
Externally applied steel [79], [80], [81]	Both plates and strips can be applied at the bottom of the girder to increase the bending moment capacity. Plates can also be applied on the sides to increase the shear capacity. If applied with glue gluing shear stresses limit the maximum use of the material. For application with bolts and anchors sufficient space should be available and the cracking behaviour of concrete can be influenced by drilling holes.		Strips: bending moment Plates: bending moment and shear Stiffness increase limited	✗ due to durability aspects With cover not economical
Concrete overlay [82]	Application of extra layer of concrete on top of girders. This method is also related to the cast-in-situ layer (see §§2.4.1). Normal concrete, shotcrete or ultra-high strength concrete can be used. Moreover, fibre admixtures or reinforcement is possible. A large disadvantage is the increase in structural height and self-weight. Also, the bonding strength between exiting and new concrete is a point of attention.		Shear Stiffness Bending moment above supports in statically indeterminate structures but nullified by the extra self-weight.	✓ Depends on environmental and economic feasibility

External prestress [83]	Prestressing tendons are placed outside concrete section and prestressing force is transferred to concrete through end anchorages and deviators. With this method the strength is increased, without increasing the dead load of the element. Moreover, the method is economic and easy. However external tendons are susceptible to accidental damage and impact of vandalism. Also, regular maintenance and inspections are needed.		Bending moment Stiffness: delays moment of cracking	✗ Aesthetics and vandalism
External Carbon Fibre reinforced polymer (CFRP) [79],[81], [84], [85]	Compared to steel CFRP has low self-weight, two times higher strength and high fatigue resistance. Moreover, CFRP is not affected by de-icing salts, acids and humidity. However, material rapidly loses strength in fire. Application of CFRP might result in more brittle failure. In a passive method platers or strips are glued to the concrete surface. With an active method they are anchored and prestressed. Girders can be wrapped whit U-shape or sheets can be applied on the sides. In case full wrap is possible this is preferred, because this allows for additional mechanical bond, besides chemical one. For T-girders mechanical anchoring might also be possible.	  	Strips: Bending moment Prestressed: Bending moment and stiffness Sheets: shear capacity	✓ /✗ Potential depends on economic feasibility and aesthetic requirements

Near surface mounted strengthening (NSMS) [86]	The same principle as CFRP. It can be used with or without prestress. However, in this case the system is embedded in the concrete. So, grooves are cut into the cover of a concrete member. Next fibre reinforced polymer (FRP) bars are placed in the grooves and then the grooves are filled with epoxy adhesive or mortar grout. Compared to externally applied CFRP higher stress transfers between concrete and FRP are possible. Moreover, the system is not visible, so the structure keeps the same aesthetics and is protected from the environment, impact load and vandalism.		Bending moment Stiffness if prestressed	✓ Depends on economic and environmental feasibility
Shape Memory Alloy (SMA) [87], [88]	SMA strips are applied an anchored to the concrete. Next, they are covered with mortar and activated by heating, after heating they want return to the 'stored' shape from the production. In the production stage the strip is shaped by a combination of hold and cold rolling. Next the strip is mechanically shaped by room temperature to the delivery sizes. The wanted shortening of strips causes compressive stresses in the concrete. Instead applying an extra layer, the strips can also be embedded like NSMS. SMA materials have better fire characteristics compared to fibre reinforced products and better corrosion resistance compared to steel		Bending moment Stiffness Shear possible with U-shaped strips	✓ Depends on economics and aesthetics in case of externally applied and environmental factors in case of embedded.

§2.5 Design process

Every project is unique, is often said. However, the process that is gone through is not. In this paragraph the general approach used for projects is discussed. The first two sections are based on internal guidelines about the design approach. However, in many organisations a similar approach is used. In the first section the V-model is described, followed by a general description of the design phases in the second. In the third section the general principle of a tender is described. The fourth section highlights some differences between a regular process and a process where reused elements are implemented. Finally, a conclusion is provided.

§2.5.1 V-model

During a project the V-model shown in Figure 2.18 is used. A project starts with a contract or initiative. Next a project team is assembled and a concept is developed. Moreover, the general requirements are laid down [89]. The next step is the system design. During these first two phases there is a lot of interaction between the client and stakeholders. These steps require an integral approach [90].

After the system design more detailed design phases follow. In this phase most interaction is within the project team. When the designs are finalised the realisation of the project can start. Of course, this requires a high level of detail as well. This step is followed by the validation. During these two phases most interaction is also within the project team. The next phase of testing requires a lot of interaction with stakeholders and clients again. Finally, the products that are delivered can be used, however they should be operated and maintained [90].

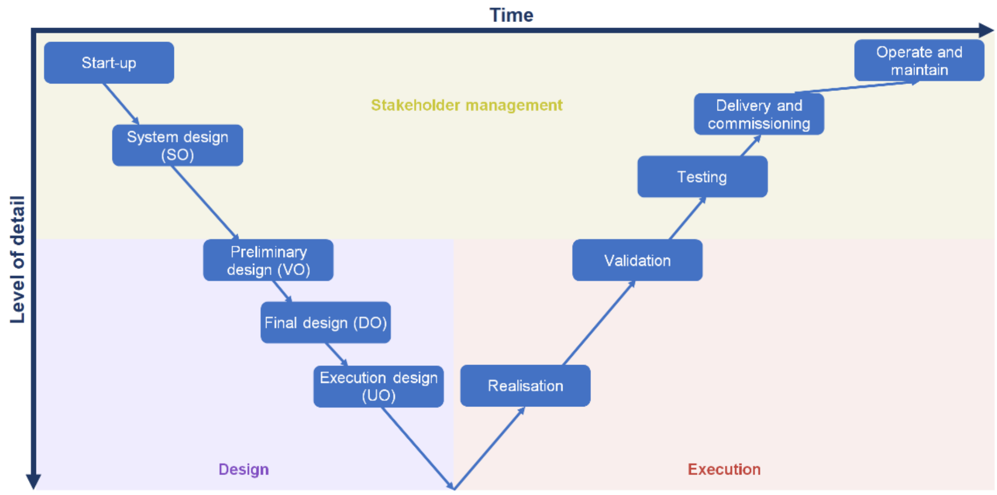


Figure 2.18: Traditional V-model for projects

The transition between phases is indicated by a ‘gate’. In this gate it is checked that the phase is really finished. This is done by verifying that all objectives are reached. These objectives are predetermined in a review plan [89]. Following this general approach helps during the complicated process of a project in which many parties are involved and have to work together. This approach supports an efficient and effective process. It contributes to one final product in which all parts work together and all requirements are met [90].

§2.5.2 Design phases

Four different design phases can be distinguished: system design, preliminary design, final design and execution design. During each design a similar approach is used, however the level of detail and final results differ. The first step is analysis. In this step the task for the current design phase is determined. So, the scope is established and approach and verifications plans are made.

In the next step: design, the task is accomplished within the predetermined boundaries. The third step is review. So, the work done in the previous step is checked. It is verified if the work is complete and has the desired quality. The verification is based on criteria set during the analysis. In the last step the task for the next design phase is specified [90].

During the system design the project is analysed and mapped out. The requirements are formulated and validated with clients and stakeholders. This is done to prevent contradictions and differences in interpretation. A clear formulation of requirements and tasks is also important for the next phases, since in these phases tasks will be divided over smaller teams. It is important that everyone has the same vision, so that in the end results can be integrated. A common way to formulate requirements is using the SMART-principle [90]. A requirement is clearly formulated if it is: specific, measurable, acceptable, realistic and time dependent [91].

During the preliminary design the design tasks and requirements are worked out in a functional and geometrical design. These designs are the basis to apply for an environmental permit. A more detailed design is made during the final design stage. In this phase different sub-systems are further developed. In the execution design, execution drawings, specifications and detailed execution plans are made [90].

§2.5.3 Tender process

Many civil engineering projects in the Netherlands are preceded by a tender procedure. This is because in most civil engineering projects a governmental or public institutions acts as client. According to procurement law, these parties are obliged to follow a tender procedure for contracts above a certain threshold value [92].

During a tender procedure a contracting authority puts out a contract and interested parties can submit a bid. Next the bidders are assessed on their suitability and capability. Then, the bids are evaluated on award criteria to find the most economically advantageous tender (MEAT). This procedure stimulates competitions between bidders and leads to the highest yield of product in terms of money and quality for contracting authorities. Moreover, it provides equal treatment and prevents nepotism as it gives everyone a fair change on the market. However, there are a lot of costs involved. Especially, for the bidders that do not win the tender [92].

In a tender procedure again, different phases can be distinguished. The first phase is the analysis in which the works are analysed and solution strategies are explored. Moreover, the basis for design alternatives is set. In the next step these alternatives are further developed. To choose a final alternative, the alternatives are compared in a trade-off matrix. In the third step the chosen alternative is worked out and finally in the last step the bid is finalised and submitted. So, during this procedure the design phases of system and preliminary design are mainly gone through. However, when a tender is awarded, these phases are gone through again to evaluate, validate and adapt where needed [93].

So, during the tender a part of the V-model is already gone through. However, the level of detail depends on the complexity of the question and requirements from the contracting authority. Moreover, the level of detail also differs between objects within the project. An object that has limited effects on planning, costs, project feasibility or clients criteria will be worked out on system level only. While an object with significant effects will be worked out on preliminary design level. For an object with that may have critical effects working out on a final design level is possible [90].

§2.5.4 Implementing reuse

In a traditional design process all phases follow each other, while in a design with reuse interaction between distinct phases is needed. The design should be suitable for reusable elements; however these elements are often not yet insight or their characteristics are (partly) unknown. Therefore, during the preliminary design assumptions are made and optimally the design is based on ranges to make it sooner applicable. During the detailed design when more information about the elements is available the assumptions are verified and corrected [17], [94].

Parallel to this design process with reuse a deconstruction process runs as well. Before a construction is deconstructed the basic characteristics of the structure and the elements should be determined. Moreover, to make an element suitable for reuse a deconstruction technique should be available and applicable [95]. Before reassembly, the characteristics of the element should be assessed, to assure that it meets the requirements. The differences between the processes are shown in Figure 2.19. So, to transform the traditional process into a design process with reuse a more integrated and holistic approach is needed in which all stakeholders cooperate together [96].

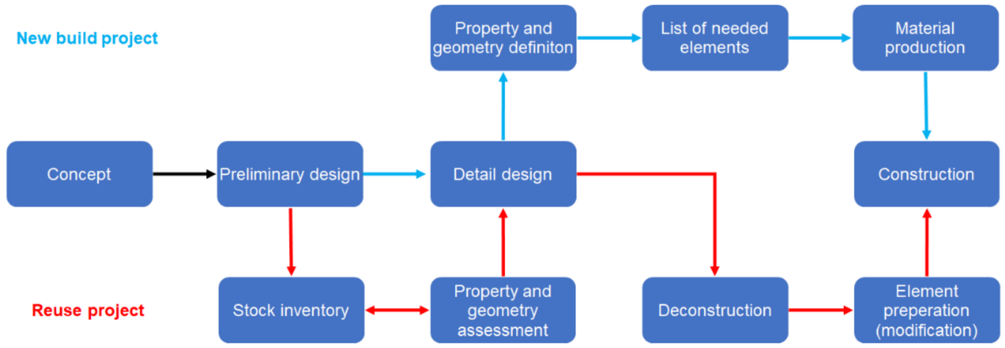


Figure 2.19: Differences between traditional design processes and design process with reuse based on [71].

The implementation of reuse might have an influence on the tender procedure as well. As stated in §2.5.3 the level of detail for an object depends on the effects it has on other aspects of the project. Compared to using new elements, using reused elements creates more uncertainty. Moreover, it can have effect on the planning and have significant effects on the costs as well. This might be a reason to work the objects that consist of reused elements out in more detail.

§2.5.5 How does a design process with reuse differs from a traditional process?

In conclusion, in a process with reuse more interaction between different design phases is required. Moreover, a demolition project and a construction project are combined. Furthermore, the type of input used in design is more variable in the beginning and the required level of detail might differ.

§2.6 Reuse in the construction sector

From the beginning of this century the environmental awareness is increasing, which is visible in the exponential increase in the amount of research papers related to circularity in the construction sector [97]. However, despite the research, pilot projects and experiments the concept of reuse is still not widely adopted by the sector. This implies that barriers for reuse exist and are difficult to overcome. The civil engineering sector and building sector are both part of the construction sector and have much in common. However, the civil engineering sector is lagging even further behind.

In this chapter factors that influence the implementation are discussed. These factors are primarily based on the building sector. First the obstacles related to construction market are discussed. This is followed by the financial aspects, technical and regulatory factors and environmental factors. Finally, a reference project in Arnhem and a brief conclusion is provided.

§2.6.1 Construction market, design process and stakeholders

Most barriers mentioned in literature are related to conservative characteristic of the construction sector which hampers innovation to circularity [98]. The construction sector is characterized by fierce price competition, which results in small profit margins and limited resources for innovation. Moreover, there are limited learning and adaptions possibilities because of the short relationships and uniqueness of projects [99]. As a result, innovations from pilot projects are not scaled up and knowledge is not sufficiently developed, shared and secured [100].

Another barrier is the lack of an established market, with supply and demand [97], [101]. This refers more or less to a chicken-egg situation [102]. Without demand for reused elements, there will be no supply and vice versa. Clients can help to overcome this obstacle by taking the lead and demanding reuse. However, currently clients are not aware of the benefits of reuse and have negative associations with reused elements. Clients are sceptic about the physical appearance as well as the structural safety [103]. A more integrated approach, pilot projects and stimulation from designers and contractors can help to motivate and educate clients [104], [105]. However, the latter is more difficult due to tendering. Moreover, currently clients are not involved, because according to standard specification contracts released materials are forfeited to the contractor. Of course, contractors have more knowledge about the demolition and new destinations for elements. However, by removing the standard rule from the contracts, clients are involved and must think about the released materials. Furthermore, they can cooperate with the contractors [55].

Another way to overcome this obstacle is a change in business model and mindset. Contractors and demolishing companies can take the lead and combine supply and demand inside their own business [102]. In this way the problem of coordination between many parties is removed. It also relates to the driving factor of a creating a green business image, which increases the competitiveness of the business [97], [104], [106].

§2.6.2 Financial aspects

The costs for deconstruction are higher compared to demolishing. Especially since deconstruction is more labour intensive and time consuming [97]. Time is already mentioned in the building sector as obstruction, however in case of viaducts and bridges this time is even more valuable, because of the accessibility of the road network. From Figure 2.19 it becomes clear that the design process is more complex, thereupon takes more time and requires more planning and interaction. As a result, it is more expensive as well. However, it is suggested that this longer design process, may shorten the construction time as it is better planned [98].

This can compensate for the longer deconstruction time, which shortens the closure of the road network and reduces the costs associated with it. By implementing reuse more frequently, gaining experience and setting protocols these factors gradually become less limiting.

Other influencing factors are storage and transport between demolition site, storage facility and new construction site. These transport costs may be higher because of the sizes of the elements; however, the distances will be shorter compared to the distance between raw material supply and factory [106]. Storage is an additional aspect regarding the traditional process. So, it is not predetermined which party is responsible for it. Moreover, a storage facility should be available and there are costs involved. However, for bridge girders the storage time can be limited, as four time more girder bridges are built than demolished [22]. Other extra financial costs are related to adaptions, testing and certification. Nevertheless, the acquisition costs are lower.

Finally, the change in design process also influences the time when money is needed. Compared to the traditional process elements are preferable bought much earlier in the design phase to cope with uncertainty about the desired characteristics and the availability of the reused items. This might result in cash flow problems as the revenues from the project follow later [97], [107]. This problem relates to the short-term financial view of the construction sector [96].

In short, the economic feasibility of implementing reuse is highly variable. It not only depends on the project characteristics, but also on the company's business model and external factors like market conditions, knowledge, storage facilities, subsidizes, taxes etc. For example, the currently increasing prices for raw materials are a financially stimulating factor [108].

§2.6.3 Technical and regulatory factors

An evident technical obstructing factor is the fact that existing structures are not build for reuse or built with hazardous or contaminated materials [94], [95]. This complicates deconstruction, because connections must be broken which are not meant to be broken and it should be done without damaging the elements. This relates to the lack of training, knowledge and experience in selective demolition [97].

Another barrier in the building industry is the long-life span of constructions. As a consequence, ownership of the building frequently changed and elements outdate or loose functionality [96],[104]. Nevertheless, objects in the civil engineering industry do not change ownership. However, because of the long-life cycle it is less likely that manufacturers or designers from the existing structure participate in demolition and reuse [106]. So, it hinders cooperation in the supply chain.

According to a survey in Norway implementing reuse is currently working around the regulations [103]. The assessment of the remaining capacity, the remaining service life and the conditions is difficult, because of limited testing possibilities and a lack of specifications and procedures. Moreover, current standards and guidelines do not support reuse and specific regulations do not exist [22], [97]. Certification and assessment of elements is further complicated by the little technical information that is available about existing structures. Since structures are built prior to the digitalisation, physical drawings need to be searched for [15].

Nevertheless, this barrier may also be misused as effortless way to not implement reuse. Since technically there are many possibilities and most elements are able to meet the requirements [22], [105], [56]. The barrier can further be removed by governmental promotion in the form of incentives, standards and guidelines or requests from clients [103],[104].

§2.6.4 Environmental factors

The ambitions and goals for a circular economy are the main drivers to implement reuse. Generating less waste and saving raw materials in the production process are the main benefits of reuse. Also, carbon dioxide and other harmful emissions released during production and transport are saved [104]. Of course, deconstruction, transportation and modifications of elements generates emissions as well. However, in general due to the smaller logistic cycles these are lower, but it depends on the project and the modifications needed. Furthermore, demolition often uses heavier machines that produces more emissions compared to machines used for deconstruction [73].

Despite the positive environmental effects of reuse, environmental aspects are frequently mentioned as obstacle for reuse. Currently there is a lack of focus, because there are so many environmental objectives. Furthermore, there are no specific strategies or policies for circularity and reuse. As a result, the focus is often too much on the reduction in greenhouse gas emissions, because this objective is clear and measurable [100]. So, to include circularity aspects the focus should shift to other environmental aspects as the depletion of natural resources and the loss of biodiversity. A change in the calculation method of the environmental impact (ECI-value) of a product could help. This impact is measured over different categories of which one is the abiotic depletion potential. Increasing the weight of this category is mentioned as stimulating measure [22], [109]. More information about the environmental impact calculation is provided in §2.7 .

In addition, the environmental advantages of implementing reuse are not translated in financial benefits, which relates back to the financial aspects. A measure, mentioned in the literature, to overcome this barrier is increasing the costs of landfilling. However, this only stimulates circularity in general and particularly the lower levels of circularity. Moreover, it is not of influence for concrete bridge girders in the Netherlands as most concrete waste is already recycled. A more suitable initiative is the obligation of an ECI-calculation in tenders. However, this brings back the barrier discussed above. Therefore, requiring a certain percentage of reused elements might be more beneficial [19]. Another option is to introduce taxes on the emissions of fossil fuels and extraction of raw materials [73].

Including circularity aspects in tenders is only possible if clients implement this or there are obligations to do so. This relates back to the ambition of the client as well as the lack of focus. Larger central governmental organisations as Rijkswaterstaat have more ambitions and financial resources to implement these aspects in tender procedures. However, smaller governmental organisations, like municipalities have not. However, these parties own a lot of infrastructural objects and can have a large market share [100].

§2.6.5 Reuse of hollow core floor slabs in Arnhem

In spring 2022 the Provincial Building of Gelderland, Prinsenhof A in Arnhem was selectively demolished and the hollow core slab floor slab elements are used in the construction of a sport hall in Arnhem. In this way seventy tons of carbon dioxide emissions are saved [110]. In addition, frames are reused in the construction of a bicycle storage at the same location and in the construction of a building in Heerde [111].

This innovation project is a valuable reference because it is a Dutch project that gives a practical view to the barriers found in literature. Besides, the project includes the complete design cycle and the original design was according to the linear design process [111]. The main stakeholders are discussed in Table 2.5. This project emphasizes the role of the client. The Province of Gelderland insisted on selective demolishing even when many parties involved were reluctant and no donor building was insight. Due to the persistence of the province to reach the highest level of circularity possible and their commitment to bring different parties from the sector together reusing many elements was possible.

In this project the barriers of the conservative construction market are observed in a linear view of parties involved and reluctance during working sessions due to competitive strategies. In this project these barriers were overcome by creating trust, taking sufficient time and keeping track of each other interest. It should be noted that the costs of this selective demolishment are higher compared to regular demolition. In this case this barrier is overcome by the provincial government [117]. Furthermore, the gap between supply and demand was observed as well. This was solved by actively searching for donor building by the province [117] and the demolition company Lagemaat BV. This company bridged the gap by combing supply and demand inside the business as it was also involved in the construction of the new building in Heerde [102].

Table 2.5: Main stakeholders in project Prinsenhof A.

Stakeholder	Role
Province of Gelderland	Initiator of the project and owner of the Provincial Building. They act as role model with the aim to get a better understanding of reuse in the construction industry and to stimulate the sector to implement reuse [111], [116].
Municipality of Arnhem	Client for the sport hall that is constructed with reused hollow core floor slabs.
Dycore	Deliverer of the hollow core floor slabs during the construction of the Provincial Building in 1987. In this project Dycore provided drawings, calculation and advice for the deconstruction, adaption and reassembly of the hollow core floor slabs [102].
Van der Horst	Building contractor that is responsible for the construction of the sport hall consisting of the reused hollow core floor slabs.
Lagemaat BV	Demolishing company that selectively demolished the provincial building. Moreover, they provided adaptions for the floor elements as well as the temporary storage of elements.
Technical University Delft	They performed test to verify the concrete strength of the floor elements, to approve them for use in the sport hall.
Barry van Waveren	Architect hired by the Province of Gelderland to explore the opportunities for reusing elements. This was needed, because at the start of the project no donor building or interested parties were insight [117].
Tielemans	Consultancy company that participated in the processes to get the floor elements from the Provincial House to the sport hall.

Apart from economic and organizational factor also technical and regulatory obstacles were faced. For example, the design of the sport hall had to be adapted to accommodate the floor elements [118]. Moreover, to meet the building regulations and requirements for assurance a protocol for reuse of hollow core floor slabs is made [111]. For this protocol, the involvement of the original deliverer of the elements as well as a testing institute were beneficial. Regarding the long life cycle the involvement of the original deliverer is considered special.

§2.6.6 What are the current obstacles that limit the implementation of reuse?

The conservative nature of the building industry is currently seen as the most important barrier to implement reuse and can only be overcome when parties involved take the initiative. Also, the economic feasibility is still seen as questionable, but might be increased by market initiatives, taxes, regulations and more common practice. Furthermore, there is no consensus about the regulations. This barrier can be further removed by the introduction of standards for reuse. Finally, the environmental objectives often stimulate only the lower levels of circularity, which can be overcome by financial measures or client requirements.

§2.7 Environmental impact analysis

The implementation of reuse is based on the need for a circular economy. The principal objective of a circular economy is a reduction of the environmental impact. To evaluate the environmental impact of buildings and civil structures an environmental impact analysis over the life cycle stages is used. This method is explained in the first section. The second section provides information about how reusing girders can be considered in this analysis. Next, ways to account for circularity are discussed. The last section provides a brief conclusion.

§2.7.1 Traditional life cycle assessment

With a life cycle assessment (LCA) the environmental performance or impact of a construction is quantified. The environmental effects of raw materials, energy and emissions are estimated over the product life stages. These life cycle stages are presented in Figure 2.20 [112]. In the Netherlands this calculation is called a “Milieuprestatie berekening” and performed according to the guideline “Bepalingsmethode Milieuprestatie” [113]. This guideline is based on Eurocode standards NEN-EN 15978 and NEN-EN 15804.

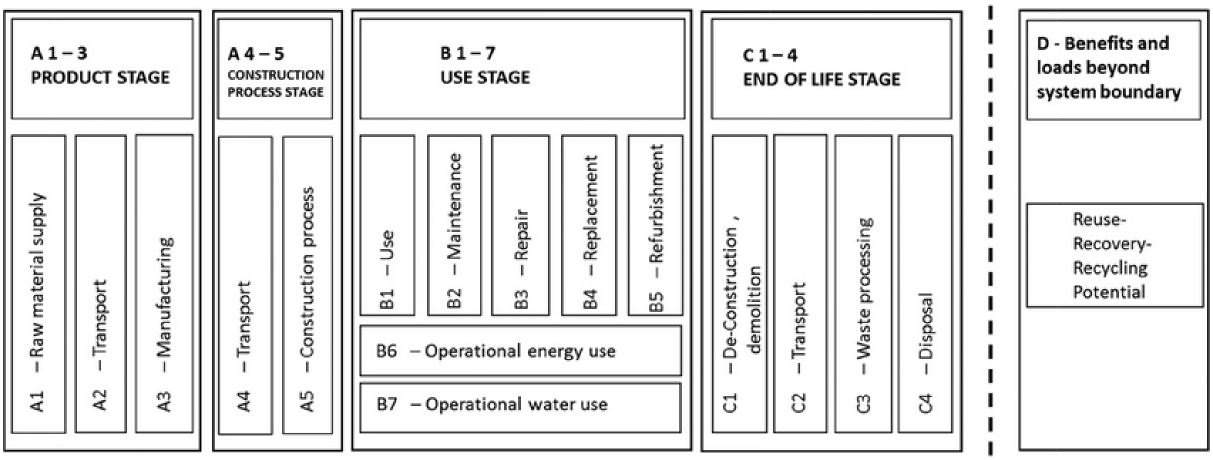


Figure 2.20: Life cycle stages [112].

The assessment can be divided into four steps. In the first step the goal and scope are defined. The goal consists of the application, reason to perform an LCA and the audience. The scope consists of:

- Functional or declared unit: this provides a reference for comparison between products.
- System boundaries: which life cycle stages are included and to what extend are involved materials and processes considered.
- Data selection: lot of data is available in the Dutch “Nationale Milieudatabase” or European EcoInvent database. Other sources of information are product manufactures/deliverers.
- Methodology: which environmental impact categories are included. NEN-EN 15804 defines 13 core environmental impact indicators. These indicators aim at climate change, ozone, acidification, eutrophication, depletion of resources and water use. There are 6 additional indicators, which aim at emissions, toxicity and soil quality. For an Environmental Cost Indicator (ECI) 11 of the 13 core indicators are mandatory.

The life cycle inventory analysis is the second step of the assessment. In this step a process tree is made to list all the relevant inputs and outputs for each module within the chosen system boundary. In the third step the environmental profile of the product is determined. This is done by linking the environmental impact of the in- and output to the environmental impact categories. Moreover, the environmental impact categories can be monetarised. In this way one value is presented that represents the shadow costs. This final value is called Environmental Cost Indicator (ECI) or ‘Milieu Kosten Indicator” (MKI). The last step is the life cycle interpretation. This phase should provide insight in which elements or life cycle stages contribute most to the environmental impact. Moreover, a sensitivity analysis can be included.

§2.7.2 Unintentional reuse

The “Bepalingsmethode Milieuprestatie” [113] provide an additional rule regarding unintentional reuse. There is unintentional reuse in case that during the initial environmental impact calculation reuse is not considered, the remaining service life is unknown or in case reuse is already fully counted in module D. So, over time unintentional reuse will slightly vanish as for new products reuse will be intended and implemented in the calculation.

A reuse factor (hergebruikfactor) H is developed to provide consistency. Moreover, it would not be fair to set the environmental impact of reusable elements on zero, because the elements have not yet amortised their complete environmental impact. This H-factor is set on 0,2. So, the environmental costs of the initial product are multiplied by this factor. The factor should only be applied on life cycle stages A1 to A3, C3, C4 and D. These are the categories that relate to the production and demolition of the product [113].

The environmental impact of the other life cycle stages should be calculated as usual, because reuse does not affect these phases. A4 and A5 are related to the construction and assembly process. Before the reused element is implemented in the new structure it will follow these life cycle stages again. The same holds for use phase B and C1 and C2 of the end-of-life stage. When the new construction reaches its end-of-life the product should again be deconstructed and transported. The environmental impact caused by adaptions, additions and modifications of the elements are also fully counted and added to all the life cycle stages. Also, environmental impacts related to additional transport are added to life cycle stage A2 [113].

During this research this factor can be applied. The bridge girders that will be reused are initially not developed for reuse. Moreover, on average bridges are demolished after approximately 40 years. This is less than the initial lifespan considered. Therefore, the environmental impact of the girders is not yet completely amortised.

§2.7.3 Circularity aspects

Including circularity aspects in this environmental cost calculation is difficult and not straight forward. To overcome these difficulties platform CB’23 is developing the ‘Kernmeetmethode’ to measure circularity [114], [115]. This method focuses on the protection of material stocks, protection of the environment and protection of existing value. Each of these objectives is linked to one or more indicators. In future versions of this method these indicators will be combined to a single value. However, currently there is no consensus and insufficient experience in defining weighing factors to combine the indicators [114], [115].

The ‘Kernmeetmethode’ is an extension to the Life Cycle Assessment. The LCA method only focuses on protection of the environment. The protection of material stock is also partly included in the environmental impact category of abiotic depletion [114]. To explain the need for a more extended method an example is provided. A modular design initially requires more materials, because of demountable connections and more robust design. If only the first life cycle is considered, a traditional design has a lower environmental impact. Consequently, this circular strategy is not stimulated. However, from a wider perceptive this circular concept is beneficial as in the future materials and environmental impact will be saved. This example shows that the LCA analysis does not consider the protection of existing value.

It should be noted that this is especially important for the top 3 levels of circularity and less for reuse of the bridge girders. It is possible that the environmental impact of the bridge with reused girders is higher than an original design. However, looking from a wider perspective there are no significant benefits. In this case transport, modifications and repair will have a significant impact and it can be concluded that reuse will not be a very circular environmentally friendly option. It will be better to recycle the concrete or apply the girders in a different function. Still, this method has added value. In case the environmental impact of a design with existing girders has a lower or comparable impact to a traditional design, it additionally shows the benefits regarding the protection of material stock.

Figure 2.21 gives a detailed overview of the indicators. Indicator 1 to 3 are related to the protection of material stocks. The first indicator classifies the input material as primary or secondary materials, physical scare or not and socio-economic scarce or not. The second indicator indicates if the output material is available for future life cycles. The material that cannot be used in future life cycles is further classified in the third indicator. The fourth indicator is about protection of the environment. In this indicator the same approach and environmental impact categories from the LCA procedure are used. This is the only indicator in which materials that are consumed and do not end up in the final product or in the waste stream are included. In all the other indicators only the materials that end up in the product or in the waste stream are included. The last indicators are about protection of existing value. The fifth one indicates the functional-technical value at the end of the life cycle, while the sixth indicates the economic value at the end of the life cycle [114], [115].

§2.7.4 How is reuse considered in the environmental impact analysis?

To calculate the environmental cost indicator (ECI-value) of an existing element that is reused in a new structure a reuse factor H of 20% is used. So, only 20% of the initial value during production, deconstruction transport and demolition (A1-A3, C3, C4 and D) is considered. The impact of modifications, additional material and additional transport are fully account for. The ECI-value focuses only on the protection of the environment. Nonetheless, reuse also aims at the protection of material stock. Therefore, the first three indicators of the Kernmeetmethode developed by platform CB’23 are of relevance as well.

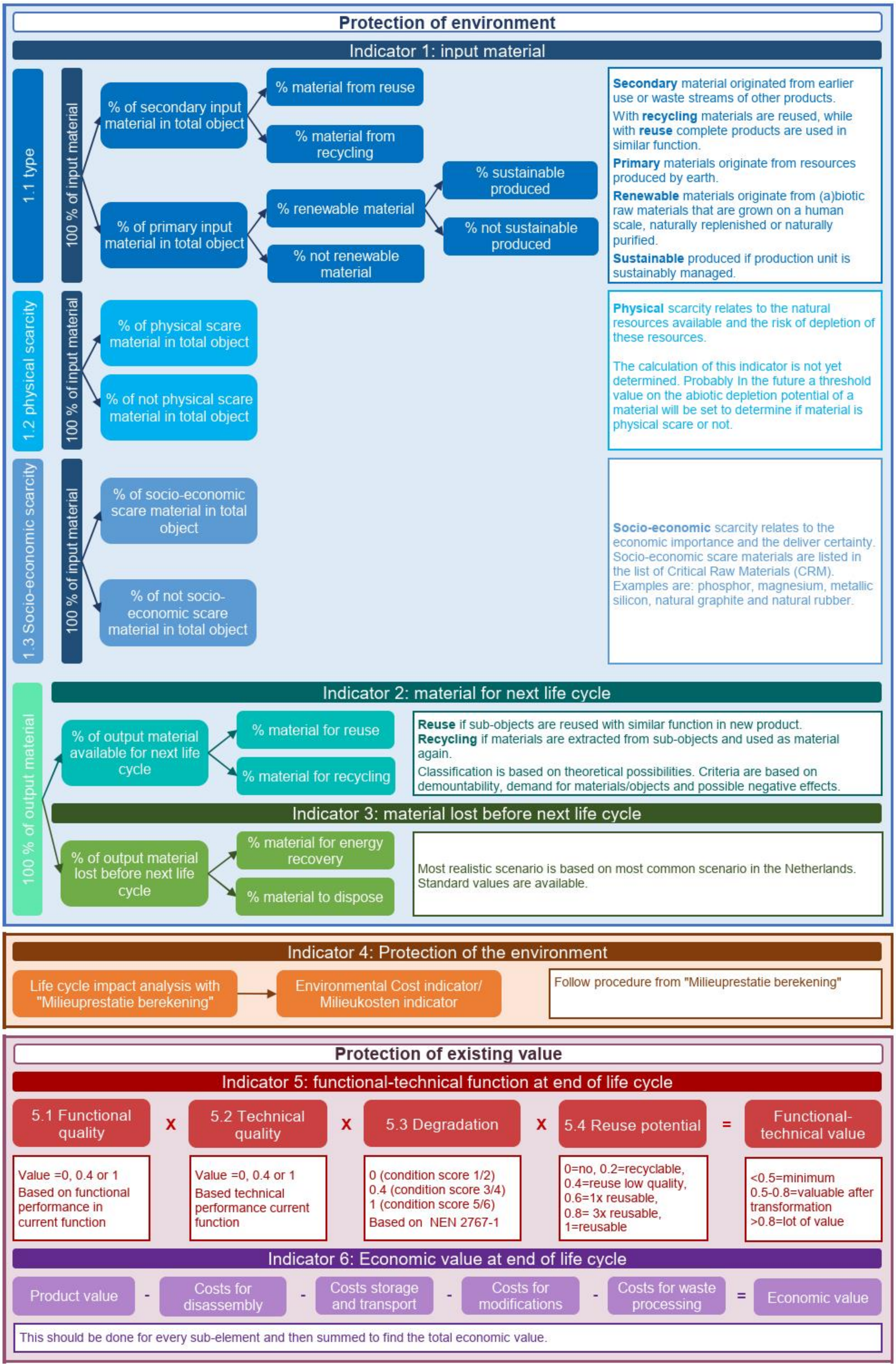


Figure 2.21: Indicators Kernmeetmethode CB'23.

§2.8 Reference projects

In this paragraph three reference projects will be discussed. The first two are Dutch projects related to a Strategic Business Innovation Research (SBIR) that focusses on circular viaducts. This SBIR was launched by Rijkswaterstaat in the beginning of 2020 [19]. It is a method that stimulates the development paths of innovations and consists of three phases. In the first phase parties with the best offer get financial resources to evaluate the feasibility of the project. In the next phase the most feasible plans are realized. So, prototypes are developed and assessed. Finally, if this phase is successful the market is made ready for the innovation in the last phase [119]. For this SBIR the second phase started in the beginning of 2021. The last project refers to the reuse of a bridge in Norway. The fourth section provides a conclusion.

§2.8.1 Combinatie liggers 2.0

'Combinatie liggers 2.0' is an initiative of Royal Haskoning DHV (engineering company), Vlasman (demolition company), SGS intron (certification company), Dura Vermeer (building company) and Haitsma (deliverer of prefabricated concrete elements) [19].

Prefabricated inverted T-girders are harvested from viaduct 'Kromwijkdreef' in highway A9 around Amsterdam and another twenty-six are harvested from viaduct 'Europaweg' in the highway of A7 around Groningen. The viaduct in the A9 is built in 1980 but is more recently widened with new girders. The girders from Groningen all date from the construction in 1985.

For deconstruction no specific provisions where taken compared to regular demolishment. The main difference was the transport in complete elements instead of parts [22]. The bridge in Groningen was statically indeterminate, which made deconstruction more difficult and time consuming. The in-situ concrete deck is removed from the girders, because it did not fulfil the current requirements and damage was observed. The total assessment of the girders showed that the girders where able to fulfil the requirements from current standards, while they were designed with old ones [62].

Seven of the girders are used in a temporary bridge in Appingedam. This destination was found by the 'Bruggenbank.' This platform from Royal Haskoning DHV is intended to bring supply and demand of elements and bridges together [19]. For this bridge no cross or edge beams where needed, which simplified the reassembly procedure. The project scores high on circularity since support blocks, sheet piles, tubular piles and impact plates are reused as well.

Another sixteen girders are used in the replacement of viaduct 'Hoog Burel' which crosses the highway A1 around Apeldoorn. For this viaduct, the girders are supported by new edge beams provided by Haitsma. For this support extra reinforcement had to be drilled in. Girders are also used for a bridge in Drenthe [55]. For the reused girders around 50% of the costs relates to the deconstruction and transportation, 20% to the modification and the remaining 30% to storage [70].

§2.8.2 Closing the loop

Project Closing the loop is a collaboration of Antea Group (design company), Nebest (investigation company), GBN (circular specialist), Strukton Civiel (contractor company). Research, investigations and information is provided by Municipality of Amsterdam, NEN, Lek Sloopwerken (demolishing company), IMd Raadgevende ingenieurs (advising company), TNO (research institute) and Madaster (circularity platform) [19].

In the first stage of the project Nebest has developed a reusability scan. With this scan the reusability of structural elements can be estimated. This is based on archive research, visual inspections and investigation with cover meter and reinforcement detector. Moreover, cores are drilled for carbonation and chloride investigations and material investigation is performed in lab. The results of this phase showed that on short term and long-term high-quality elements will become available for reuse [120].

In the next step design concepts are developed in which the reusable elements are used. By using materials that will become frequently available in the near future the concepts are broadly applicable. This is followed by the deconstruction and harvesting of elements and the construction of new structures. During the design also the circularity of the new structure is considered. So, after the service life of the new construction they can be disassembled and reused again in a third life cycle [121].

One of the prototype projects is the replacement of two viaducts in highway A76 dating from 1938 and 2004 by one new viaduct. For this new viaduct 72,5% of the elements from the old ones is reused [120]. The new viaduct has a larger span, which made the girders from the viaduct of 1938 unsuitable. To make a more circular design extra girders are harvested from a viaduct in highway A9 near Amstelveen. In the deconstruction process of the two viaducts first the in-situ deck is removed after which the elements are removed from the structure and transported. These steps are swapped compared to the project in Groningen. The girders from the A9 viaduct are shortened, which reduces the bending moment and shear forces. Moreover, more girders are placed next to each other by shortening the flanges. Together this makes the viaduct suitable for current traffic loads [122].

In total 62% can be saved on LCA-costs, 47% on CO_2-emissions and 91% on abiotic resources. Moreover, if the concept is applied in ten potential viaducts €2,8 million can be saved [120].

§2.8.3 Moving bridge in Norway

In Norway the motorway E16 between Sadvika and Wøyen is being renewed by the joint venture between Implenia and Isachens. For this new construction five bridges had to be removed. While this was going on the construction company Brødrene Rodegård AS asked if one of these bridges was suitable for repurposing [123].

The existing bridge in the centre of Sandvika crossed the E16 and consisted of thirteen prefabricated beams with cast-in-situ slabs. The bridge is deconstructed by cutting through the beams and lifting them out [124]. Next the elements are put on a transport of 135 km to Nesbyen, where the bridge becomes a part of a road to a carry and crosses the Rukkedøla river [123].

Implenia has tried to reuse bridges before but could not find a suitable destination that time. This was not due to a lack of interest, but due to a lack of resources [124].

§2.8.4 What lessons can be learned from reference projects?

Reference projects show that reusing concrete bridge girders is technically feasible and has a positive influence on the environment. Moreover, it can be cheaper. However, it also shows that a different mindset is required and that someone has to take the initiative.

§2.9 Conclusion

This literature review provides an overview of the different aspects involved with reusing elements. Girders and information about the girder should be available. Furthermore, it needs to be technically feasible to extract and adapt the girders. A design needs to be made with these girders and reusing girders has to find its place in the construction sector. Finally, reuse should contribute to sustainability as this is the primary objective. From the review it can be concluded that it is possible to reuse existing girders in new structures. By combining standards for new to build structures and standards for existing structures, the structural capacity of a bridge deck with existing girders can be verified. Furthermore, extracting girders from structures and making them suitable for reuse is possible. This is proved by small scale innovation projects.

To become a circular economy by 2050 it is important to start with reuse as soon as possible. Especially since a lot of prefabricated concrete bridge girders become available in the upcoming years and a lot of bridges and viaducts need to be build. So, a delay in the wide-scale application of reuse results in a waste of circular opportunities and a redundant appeal on natural resources. The literature indicates measures to overcome financial, technical and environmental barriers. In addition, connections in the construction industry are investigated as well as stimulating measures to raise awareness and willingness in the supply chain. Nonetheless, there is no information available about the practical steps that are needed to implement reuse in design. So, there is no specific strategy that guides project teams through the versatility of reusing existing girders. Thus, even when reuse is possible and project teams are willing to, they have no indication where to start with the design. Therefore, this research attempts to prepare the design phase for the innovation by providing a guideline on how to implement the use of existing elements in new designs.

3. Design approach

§3.1 Introduction

The design approach should stimulate the implementation of reusing existing girders during the different steps in design. First a framework or layout of the approach is developed with information from the literature study. For example, information about the design process gives an overview of steps that should be included in the approach and information about existing barriers for implementation of reuse results in attention points. Moreover, information about existing girders in the Netherlands, the standards and guidelines and the technical possibilities makes the design approach specific and provides the background for more in depth calculation procedures. The further completion and more detailed steps are included based on experiences and results from the case study that is performed. For example, road maps for calculation procedures are developed based on the calculation and results from the case study. The case study is performed parallel to the development of the approach and is discussed in chapter 4.

Before developing the outline of the framework, the general principles for implementation of reuse are considered. Moreover, the scope of the design approach is defined, based on the developments in the construction market, time limitations for the research and information availability. The principles and the scope are discussed in the second paragraph. The general layout of the framework is shown in paragraph three. In the following paragraphs the steps from the framework are discussed in detail. First the girder search, followed by roadmaps for the structural and environmental impact analysis. Next the financial analysis is discussed and finally the multi-criteria analysis (MCA).

§3.2 Guiding principles and scope

This paragraph provides the basis for the design approach. First a comparison is made with the life cycle of a product. Next the guiding principles and their interaction is described. This is followed by the scope of the design phases and finally an overview of the scope.

§3.2.1 Product life cycle

The implementation of reusing existing girders in new designs can to some extend be compared with the introduction of a new product on the market. A new product goes through a product life cycle as shown in Figure 3.1 [125]. First a product is developed. As soon as a product is ready it is introduced into the market. During this first phase, the sales are limited, there is no profit and investments are needed for promotion. As soon as customers become familiar the revenues increase. The product enters the growth phase, where the selling price is still high, but costs decrease. The maturity phase is dominated by concurrence. The growth of revenue stagnates and selling prices are lowered. During this phase competitors try to create new models to gain or keep their market share. At the end of this phase the weakest companies withdraw from the market and the revenues start to decrease. Finally, better alternatives are introduced on the market and customers start to lose interest. During this phase the product sales decrease and many parties withdraw from the market or exploit the last possibilities [126].

The development phase for reusing existing girders is the phase in which knowledge is gathered and technologies are developed to make reuse possible. During the introduction some companies will start to reuse existing girders in new designs. In this phase companies have to convince clients about the options. During the growth phase clients are widely familiar and make for example demands about reusing existing girders to contractors. During the maturity phase the concept is completely implemented and existing girders will be reused whenever possible. The decline phase will only start when new bridges are designed fully demountable and reusable and the material loop is closed. This type of circular designs can be seen as new models that make the designs with reuse of elements not meant for reuse unnecessary.

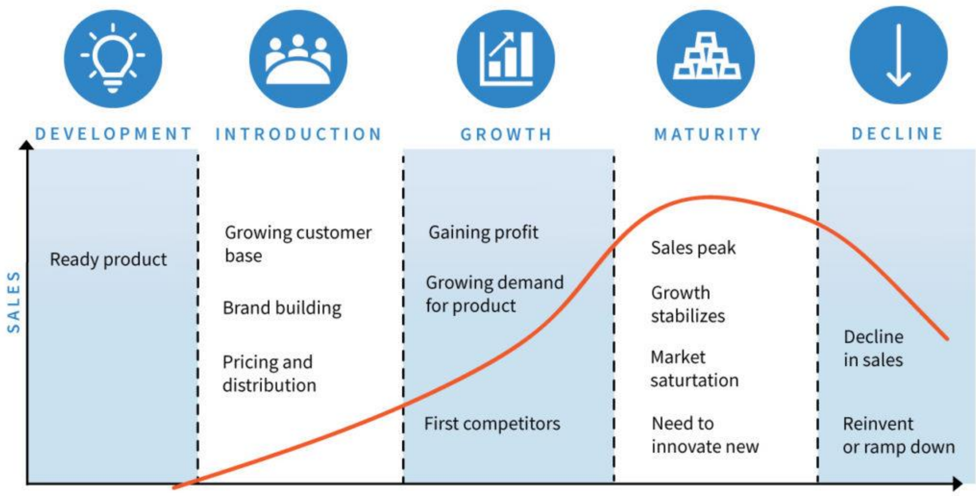


Figure 3.1: Product life cycle [125].

With the technology readiness scale, visualised in Figure 3.2 [127], the maturity of a product and the readiness for market uptake is determined [128]. A product is ready to enter the market when Technology Readiness Level (TRL) 9 is reached. Based on the SBIR project of Rijkswaterstaat the implementation of reusing existing girders is in the last phase: prototype and system development. So, somewhere between TRL 7 and 9. The two reference projects: ‘combinatie liggers 2.0’ and ‘closing the loop’ are currently constructing bridges with reused girders in the real environment, which is consistent with TRL level 7. Also, qualification and assessment of these prototype is going on, which belongs to level 8. Furthermore, Nebest is working together with Rijkswaterstaat on reusability scans and material passports to make the market ready. Also, platforms as the ‘bruggenbank’ are further developed. These developments already relate to level 9 [19], [129].

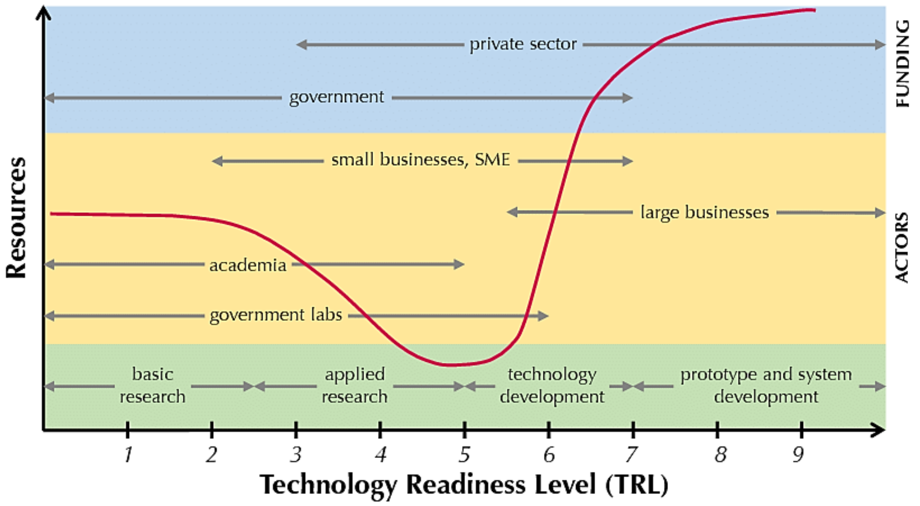


Figure 3.2: Technology readiness levels, to determine maturity and readiness for market uptake [127].

§3.2.2 Guiding principles

Before reusing existing girders is implemented in a design, several prerequisites must be met. Therefore, these conditions act a guiding principle for the framework and can be seen as the first step. The next step is to see if these prerequisites can be met more frequently. The three main prerequisites are shown in Figure 3.3. First, reuse will only be implemented if a feasible design can be developed. Moreover, girders need to be available and someone has to take the initiative to actually reuse girders. In the next sections these three guiding principles are discussed in more detail and the scope of the design approach on these principles is determined.

Figure 3.3: Guiding principles for implementation of reuse.

§3.2.3 Feasibility of design

Reusing existing bridge girders in new designs should be feasible, otherwise implementation will not be beneficial or even possible. This feasibility applies on three areas: technical, financial and environmental. A design is technically feasible if it is structurally safe and executable. The quality of the reused girders should be assured and the new design should be able to fulfil all the requirements from current standards and guidelines. The other part of the technical feasibility relates to execution. First the girders must be safely removed from the existing structure, without damage. Next, they have to be modified or strengthened if necessary and placed in the new structure. For these steps suitable techniques should be available.

A design is environmentally feasible if the environmental impact of a design with reused girder is lower compared to a traditional design. If not, implementing reuse overshoots the mark. The economic feasibility is important as well, because financials are a major driving factor for businesses. To win the competition companies are always looking for solutions which are economically distinctive. So, if using existing girders in new designs is financially attractive it will be frequently implemented. Reusing existing girders can be seen as financially feasible, if it is not significantly more expensive than using new girders or if there is a remuneration of a circular solution. Contrary to the other areas it cannot be directly determined if a design is financially feasible or not. This depends on the demands from the client. How much extra money is a more environmentally friendly design worth?

The three areas of feasibility influence each other on project level. For example, modifications are needed to make girders technically suitable. These modifications costs money and have an environmental impact. Another example are technical developments. These developments might focus on increasing the technical possibilities, so increasing the technical feasibility. However, the focus can also be on more efficient and economic technologies. This leads to lower prices and lower environmental impact. Consequently, the environmental and financial feasibility will increase.

All parts of the feasibility are part of the scope of this design approach. The technical feasibility is investigated with a structural calculation. The environmental feasibility is assessed with an environmental impact analysis. In this analysis the impact of a design with existing girders is compared with the impact of a traditional design with new girders. A similar approach is used for the financial feasibility. The costs of reusing existing girders are compared with the costs of new girders.

§3.2.4 Availability

The second prerequisite is availability, which relates to the construction market. Reuse can only be implemented if there are girders available and information is exchanged. In this way, matches between existing structures and new to build viaducts and bridges can be found. In other words, a market with supply and demand has to be established. In this market stakeholders interact and negotiate with each other.

An example to illustrate. Party A has to design a new bridge that will be constructed in two years. In the vicinity party B is going to demolish a girder bridge next year. Since B has communicated this to the construction sector well in advance, A anticipates on reuse during the first stage in design. As a result, the existing girders are reused in the new bridge. Otherwise, A would use a traditional design approach, because implementing reuse is not seen as option and would require too much time and effort.

Currently, no market with supply and demand is established and information exchange is limited. This is because reusing existing girders in new designs is not yet ready to be introduced on the market, which relates back to the Technology Readiness Level (see §3.2.1).

This factor will not be part of the scope, because the situation will change quickly. The reason is that there is a lot of process and ongoing developments in this field. Therefore, it is assumed that in the near future more information about existing girders will become available. Also, the supply of girders and exchange of information will improve. As a result, a market will evolve. This assumption is based on the innovation projects from Rijkswaterstaat, the reusability scan developed by Nebest and platforms as for example the ‘bruggenbank that are further deployed. So, for this decision framework it will be assumed that reusing existing girders is ready to enter the market. In relation to the product life cycle the scope will be the introduction and growth phase. However, the approach will also provide a basis for the maturity phase. Ideally during the maturity phase a fully developed detailed approach is available and used in daily practice.

Another part relating to the availability and the construction market is the interaction and involvement of stakeholders. This framework will not deal with the interaction in the supply chain specifically. However, stakeholder management will come back in several steps as it is connected to almost all aspects. The last factor related to the construction market is the timing and scheduling of the logistic process. This is included in the design approach to a small extend by reviewing and given recommendation based on the process during the case study.

§3.2.5 Initiative

The third and last precondition is the initiative. Reuse will only be implemented if someone actually takes the initiative and establishes it. This initiative can come from the contractor. In this case it relates to the ambition for a circular economy. Therefore, someone has to be aware of the environmental problems and the need for circularity. The initiative may also come from regulatory authorities that make demands. In this case the environmental awareness is also important, because it shows the relevance of the demands. The relevance and ambitions for a circular economy are already clear. There is a European agreement and the Dutch government has set objectives. Moreover, many companies have set goals on sustainability and stimulate their co-workers. For instance, the ambition of Van Hattum and Blankevoort to be the most sustainable civil engineering and building company by 2050 [18]. Therefore, it can be concluded that it is not of relevance to include this in the scope.

While the ambition or relevance is a driving factor for the implementation of reuse, it is not sufficient. The initiator should also have sufficient technical knowledge and access to resources to make a suitable and safe design. In this research no experiments will be performed. Nor will a new software or calculation method be derived. Therefore, this research will not contribute to the factor of knowledge. However, the research will contribute to the resources. In fact, the main target audience of the design approach are the initiators: the people that actually reuse existing girders in new designs. The decision framework will be a valuable resource for them as it guides them through the design process and make implementation of reusing girders easier, more efficient and above all accessible.

§3.2.6 Interaction

To make it even more complex the preconditions are not independent but related to each other on system level. So, on the level of the national construction market. For instance, if the design feasibility increases, the implementation of reuse becomes less risky, easier and financially more profitable. As a result, the willingness of the initiator increases and more parties will be interested. This influence on the market of supply and demand has an effect on the price. So, in the end it influences the financial feasibility again. Nonetheless, it should be pointed out that this is according to the economic market theory. For reuse the market of supply and demand is heavily affected by to be demolished girder bridges. However, with an increased feasibility the option of reuse will sooner occur when a bridge has to be demolished.

§3.2.7 Design phases

From the literature review it can be concluded that most differences between a project with new girders and a project with reusing existing girders arise during the preliminary design. Therefore, this phase will be the main focus of the design approach. During the system design the project is analysed and mapped out. The basic requirements as location of the bridge, type of bridge etc. are identified. Moreover, this phase also includes the interaction between many stakeholders. Therefore, although no guidelines will be provided on steps during the system design, this phase is still to some extend part of the scope as input from this phase is used during the preliminary design. The final and execution design will not be part of the scope of the decision framework. The reason is that before this phase the decision to reuse existing girders should already been made. Moreover, it is expected that these phases do not significantly differ from the traditional process.

§3.2.8 Overview

Table 3.1 provides an overview of the scope. A check mark indicates that the factor is included in the scope and a cross indicates that the factor is outside the scope of this research. A checkmark and a cross indicate that the factor is to some extend included. Figure 3.4 gives a graphical representation.

Table 3.1: Overview scope of decision framework.

Design feasibility		Construction market		Initiator		Design phase	
Technical	✓	Supply and demand	✗	Relevance	✗	System	✓/✗
Environmental	✓	Stakeholder management	✓/✗	Knowledge	✗	Preliminary	✓
Financial	✓	Planning	✓/✗	Resources	✓	Final	✗
						Execution	✗

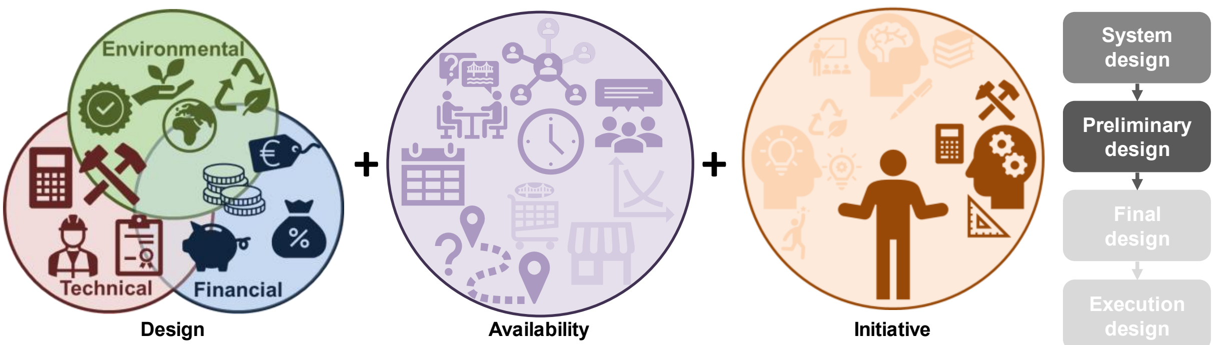


Figure 3.4: Visualisation of scope of design approach.

§3.3 Layout

The general layout of the framework is shown in Figure 3.5. The implementation of reuse starts at the system design, where the requirements from the project, client and stakeholders are identified. With these requirements search criteria are formulated and suitable girders are identified. Next design alternatives are developed during the preliminary design phase. In case more than two alternatives are possible with girders from the search a hierarchy is made. Two alternatives with most potential are worked out first. If these alternatives do not fulfil all structural requirements another alternative is worked out. If non of the alternatives is able to meet the structural requirements, the search criteria can be adapted and new alternatives can be made. An alternative with new girders is made as back-up option in case the alternative with existing girders turns out to be unsuitable.

There are three possible outcomes of the structural analysis:

- Structurally feasible: alternative is suitable and meets all structural requirements.
- Structural maybe feasible: there is a change that the alternative is feasible, but significant modifications are needed. These modifications can relate to strengthening, repair or protection. Shortening in width or length direction as well as small reparations do not belong to this category. These options are directly considered in the structural analysis. In case of substantial modifications the alternative is put on a temporary hold. Only when no other alternative is able to directly fulfil the structural requirements, the possibilities are examined. If it turns out that strengthening, repair or protection measures are possible the alternative is feasible and the analysis can continue. The necessary modifications should be included in further analysis. If it turns out that measures are not possible, the alternative is unfeasible and no further analysis have to be performed.
- Structurally unfeasible: alternative is not able to meet critical structural requirements. The alternative is not suitable and no further analysis have to be performed.

After the structural analysis an environmental and financial analysis are performed on the structural feasible alternatives. The outcome of these analysis is used together with other criteria in an MCA. Based on this analysis the most suitable design alternative is chosen and the preliminary design phase is finalized.

The main difference between the approach to implement reuse and a traditional approach is that working with existing girders, is working with what is available, while traditionally all elements are specified and manufactured according to the requirements best suited for the project. Traditionally the structural analysis of an alternative is an iterative procedure, in which prestressing and girders size are adapted until the alternative has sufficient capacity. With existing girders the structural analysis is a verification calculation. If the result is insufficient capacity the characteristics cannot be adjusted. An iterative process is only applied to investigate more invasive measures as strengthening or shortening.

Consequently, design alternatives with reuse might be unfeasible for application, while alternatives with new girders rarely turn out to be unfeasible. So, these alternatives are all fully developed, while in the approach with reuse the development of alternatives can stop in an early stage. However, most likely this is not only due to the adaptions possible, but also due to the experiences and guiding principles in choosing a robust girder from the start. This early stop compensates for the larger number of alternatives, because traditionally only a few design alternatives are developed.

Furthermore, traditionally the differences between design alternatives are quite large, otherwise they end up being the same. With reuse, depending on the girder availability, alternatives can be quite like each other. The reason is that girders originating form different viaducts might have the same dimensions. However, during the structural (and other) analysis minor differences will reveal. These differences can be crucial to the feasibility and cannot be changed. As a result, the initially investigated alternatives can turn out unfeasible. In this case other alternatives can be investigated.

So, it can be concluded that both processes are iterative, but in a different way. With new elements the iteration process is on a detailed level within the design alternative. With reused elements the iteration process is on a higher level within the design alternative and on an even higher level between the design alternatives.

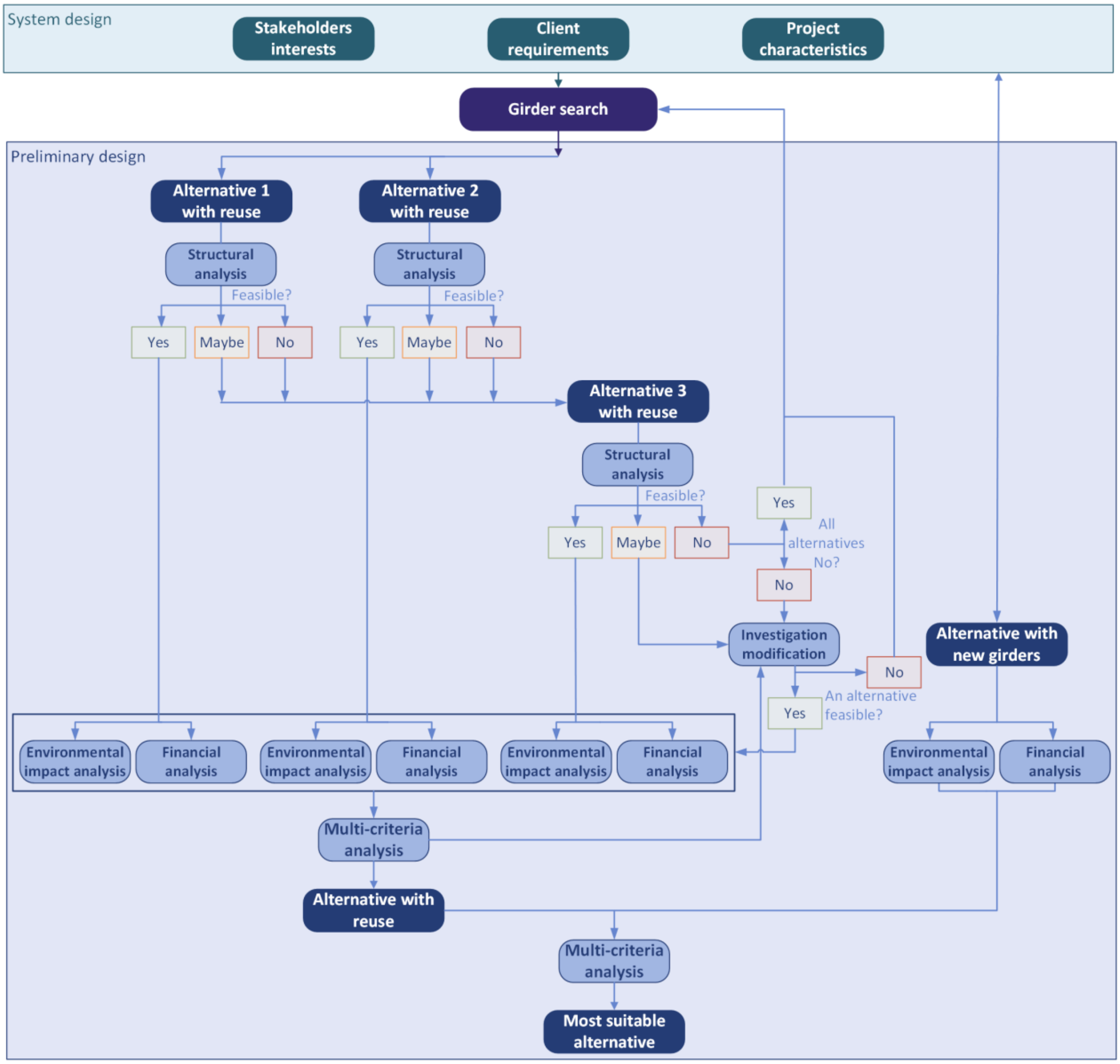


Figure 3.5: General layout framework of design approach.

§3.4 Girder search

This paragraph provides the guiding principles for the girder search. First the timing in the design process is discussed, followed by the influence of the strictness of the search criteria. Next the span division is discussed. In the following sections the search criteria: minimum and maximum span length, girder type, profile height and release date are discussed. Section eight provides a conclusion.

§3.4.1 Timing in design process

The follow up on the system design is the girder search. Based on the formulated requirements suitable girders can be searched for. For this search available platforms will be used (see §3.2.4). Since, not all girders are suitable for design, search criteria are needed to limit the possibilities. In this way, only one or a few potentially suitable design options are found, which are further explored during the preliminary design. Figure 3.6 shows the addition of the girder search in the traditional V-model for design.

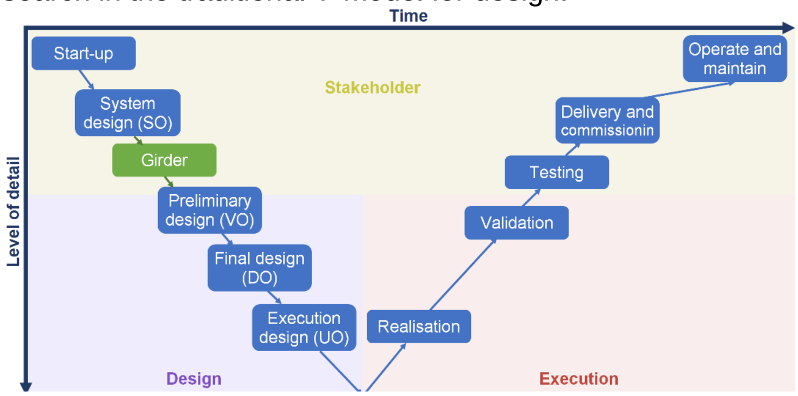


Figure 3.6: Girder search implemented in V-model of projects.

To give a more practical view the girder search can be compared with online shopping for a sweater. A customer surfs to a web shop for clothes and chooses the category of sweaters. To limit the number of results they can use a filter for size, colour and they can exempt certain types of sweaters. If a customer needs the sweater before a certain date the delivery date can be selected as filter. For girders, customers go to an exchange platform and choose the category of bridge girders. As filter or search criteria they are able to choose the range of length, type of girders, number of girders needed and construction height. Also, the availability is important, because the girders need to be available when the execution starts. So, the search criteria are the filters that make the design process more efficient by reducing the number of results and making a preselection on potentially suitable girders.

§3.4.2 Error and strictness of criteria

A difficult aspect is the strictness of search criteria. Continuing the example. A customer selects a certain size, however for some sweaters another size will fit as well. As a result, the customer may miss out the best sweater for him. On the other hand, without selecting a size, it costs more time to go through the results. Hence, a balance should be found between the strictness of search criteria and the time willing to take for searching or in case of girders the time used for design and calculations.

In statics this phenomenon is described with a type I and type II error. With a type I error a girder is assumed to be suitable based on the search criteria, however during preliminary design it turns out that the girder is not able to withstand the loads. Thus, the girder cannot be used in the actual design and another existing girder should be used or a new girder should be produced. With this false positive time and resources are wasted on design calculations and drawings.

With a type II error a girder is assumed not suitable for design and will not show up in the girder search. However, if the girder had shown up in the search and further calculation had been performed it would have been feasible. In other words, with this false negative reuse is not implemented, while it was possible. So, opportunities for circularity are left unused and primary resources are depleted unnecessary. The options are shown in Table 3.2.

Table 3.2: Type of errors possible in search criteria.

Girder suitability	No, not suitable	Yes, suitable
Assumed suitable	Type I error: false positive. *Shows up in search, but according to calculations not suitable.* Waste of time and resources	Correct decision *Shows up in search and according to calculations suitable.* Bridge with existing girders
Assumed unsuitable	Correct decision. *Does not show up in search, but according to calculations not suitable.* Bridge with new girders	Type II error: false negative. *Does not show up search, but according to calculations suitable.* Waste of primary resources

These search criteria are defined early in the design process based on limited information. Setting the requirements too strict results in many type II errors, which is an unfortunate development. Setting them too loose results in many type I errors. From an environmental aspect this does not matter. Also, from a technical point it is not a problem, because unsuitable girders will be dismissed in further design steps. However, it results in a longer design process, which negatively influences the financial costs. It also demotivates initiators because they spend time and effort in a design that turns out unfeasible. It should be noted that the strictness of search criteria is not uniform, but project dependent. If the system design is more detailed, search criteria will be more detailed. As a result, the statistically distribution will be smaller and the number of errors as well. This is shown in Figure 3.7b.

In conclusion search criteria are needed to limit the waste of time and resources, but type II errors should be avoided as much as possible. So, the search criteria should exclude the most unsuitable girders, but not all unsuitable girders will be excluded after this phase. This is shown in Figure 3.7c. Further in the design process more detailed cut-off criteria will be used to exclude these unsuitable girders.

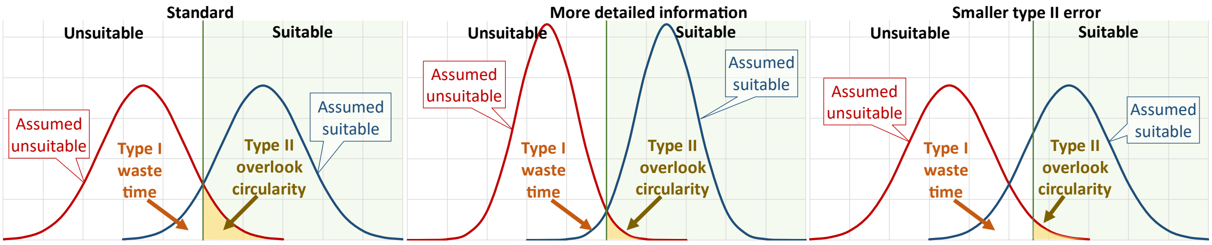


Figure 3.7 a) Standard *Figure 3.7 b) Smaller distribution* *Figure 3.7 c) Moved to limit type II errors*

Figure 3.7: Probability curves with type I and type II errors.

§3.4.3 Span division

In most cases a bridge or viaduct is divided in multiple spans. Since, the divisions in number of spans and length of spans is a determining factor in the search for suitable girders it is included as intermediate step between the requirements from the system design and the formulation of search criteria. Location, client and execution are the main influencing factors for the span division.

The location includes the geographical location and the surrounding of the bridge.

- Soil: the quality of the soil at the location of the bridge, determines the type of foundation needed at the piers. Heavy foundations require a lot of material, which results in high costs, high environmental impact and often longer construction time. Thus, in case heavy foundations are needed, fewer intermediate supports are preferred, so longer spans.

- Clearance height: the clearance height needed underneath the bridge or viaduct and the vertical alignment of the road over the structure influence the maximum profile height allowed. To meet the required clearance a bridge can be placed higher, however this involves an inclination towards the bridge, which requires a lot of soil. Again, this impacts the costs and environmental impact. So, in case of clearance height limitations a lower profile height is required. A lower profile height can be realised with shorter spans. It should be noted that the type of girder also has an influence.

- Area underneath the bridge: the function of the area under the bridge can also set requirements to the location of intermediate piers. If the bridge crosses a single road the main span is located in the middle and no intermediate support in the middle is allowed. While with a double road an intermediate pier can be placed in the middle and two main spans are located on both sides of the pier.

Boundaries for the span division might also follow from the client. These can be already formulated in the start-up of the project or might come from the list of requirements formulated during the system design or stakeholder analysis. A possible requirement for the span division is symmetry. Designs are often based on symmetry, since people have a general preference for predictability and repetitiveness. As a result, in most designs symmetry is implemented [130]. Nonetheless, a design process with reuse is reversed compared to the traditional process. A design with reuse starts with the available materials and aesthetical perfection is not directly pursued. As a result, other girder options might be available that result in an asymmetric design. However, this imperfection can have certain aesthetic qualities as well, but acceptance from the client is required [131]. Certainly, if the client still aims at certain (traditional) aesthetical concepts these should be implemented. Nevertheless, it should be tried to convince the client that these requirements are not needed or perhaps can be less strict.

The last influencing factor is execution. Many different span lengths or small variations in span length complicate the construction process and the design of construction details. Moreover, it may elongate the design process as more calculations have to be performed. So, in case of many spans a certain degree of repeatability is preferred.

From the literature review it can be concluded that most girders have a span between 12,5 and 30 [m]. Therefore, a new design should also be based on these girder dimensions. Girders between 12,5 and 18 [m] are seen as small. These girders are not used as main spans, but only used to fill gaps. So, a maximum of two spans is bridged with small girders. Girders with a span between 20 and 30 [m] are used as main span.

In this way the maximum number of spans can be calculated by dividing the total bridge length by 20. The outcome should be rounded down. The minimum number of spans can be calculated by dividing the total bridge length by 30. This outcome should be rounded up. In case of heavy foundations, one should opt in the direction of the minimum number of spans. In case of heigh vertical clearance one should opt more in the direction of the maximum number of spans. The output of this step is a list of possible span divisions.

§3.4.4 Minimum and maximum girder length

The minimum and maximum span length are search criteria that follow from the span division. These lengths from the span division are not directly the minimum and maximum length of the girders, because transition joints are present and girders can be shortened. Therefore, the range for the length of girders is larger. The minimum length of girders is found by deducting 0,5 [m] from the minimum span length. This value is based on the maximum width of a flexible transition joint [132]. Although, other types of joints and connections are possible a flexible transition joint is common in the Netherlands.

The maximum length of girders is found by increasing the maximum span length by 23%. So, the limit on shortening is 23% of the shortened length or 19% of the original length. As can be seen in Figure 3.8 the maximum shortening allowed to keep reuse environmentally feasible is higher than the structural limit. Therefore, the structural limit is the decisive factor. Financial effects are not considered, because the financial feasibility depends on client requirements (see §3.2.3). An explanation to the curves is provided in Appendix A: Maximum shortening.

§3.4.5 Girder type and origin

In this stage it is impossible to determine quantitative criteria to exempt certain girders from the search. However, it is known that T-girder are not capable of withstanding collision loads. Also inverted T-girders with a small bottom flange as HNP, HIP and VIP girders are not good at withstanding these loads. In case large design accidental loads are expected, these girders can be exempted from the search. Furthermore, in case a slender girder is needed and the length over height ratio should be larger than 28, box girders should be search for.

A girder originating from a highway viaduct has potential for a new highway viaduct but can also potentially be used in lower function. Nonetheless a girder originating from a pedestrian bridge, will not be suitable for reuse as bridge deck girder in a highway viaduct. Therefore, the type of the original bridge is used as search criteria. The principle is shown in Figure 3.9. Three categories of bridge girder sources are distinguished.

- National and regional roads: on national highways and provincial roads the intensity of heavy traffic exceeds 200.000 or 2.000.000 vehicles a year [133], [134], [135]. For these cases a negligible or no reduction in traffic load is allowed, according to NEN-EN 1991-2. Most of these structures have a design life of 100 years (belonging to consequence class 1). For this category of bridges only girders originating from a similar type of structure have potential. In relation to the older traffic load models, this corresponds to class 60 or A.
- Local roads: bridges and viaducts in roads owned by municipalities (local roads) are exposed to lower intensities of heavy traffic and have design life of 50 years. In relation to older traffic load models, this corresponds to class 45, 30, B or C.
- Pedestrian and bicycle paths: for pedestrian and cyclist bridges a separate load model is available. In older standards no load models are specified for these types of bridges. Only load model D may relate as this is for light vehicles.

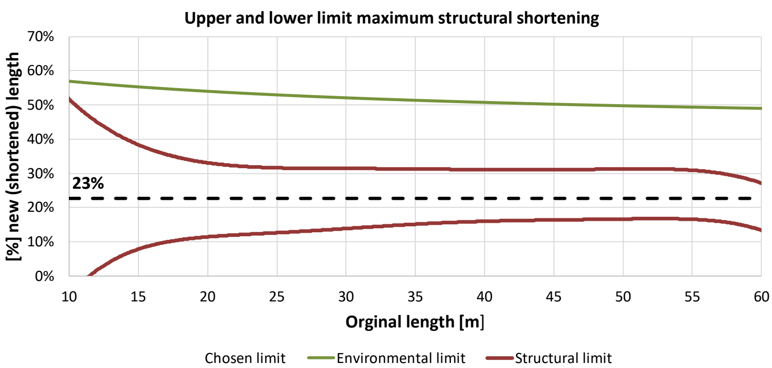


Figure 3.8: Maximum shortening possible.

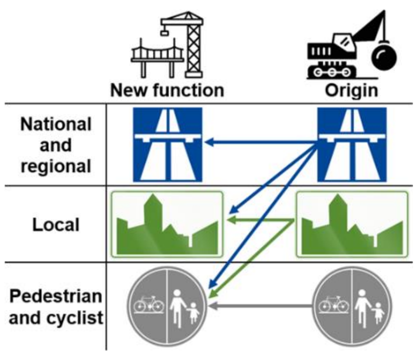


Figure 3.9: Criteria origin of girder.

§3.4.6 Profile height

Based on the girder length, type and origin only a small range of profile heights will be available (see Figure 2.11). To avoid execution and design difficulties at (intermediate) supports the same profile height is preferred for all spans. This search criterion is optional and not directly required but can be useful if many girders are available. In this case a first search can be done with the minimum profile height, based on the L/H-ratio. If no or too little alternatives are found an additional search can be performed with a higher profile height or with a combination of profile heights.

§3.4.7 Release date

The last search criterion is the release date. The supply of girders on exchanging platforms can be divided in directly available and available in the future. The directly available girders are already released from the existing structure and temporarily stored somewhere. Ideally the supply in this category is small, because it involves a lot of storage costs and space.

The future available girders are part of a still existing structure that is on a list for deconstruction. The deconstruction date should be before the execution date of placing the girders in the new structure. In addition, extra time is needed. Factors that require extra time:

- Transportation of girders from old location to storage/adaption facility and new location.
- Additional material testing (if required).
- Removal of the in-situ concrete deck (for inverted T-girders).
- Strengthening, shortening or modification the girders (if required).
- Risk factor: delay in deconstruction may not lead to a delay in construction.

Transportation can be quick, but if additional facilities or temporary road closures are needed, this takes extra time. Material testing and inspection should be done preferable before deconstruction, but some parts may only be accessible after deconstruction. The removal of the in-situ deck takes quite some time, as the reinforcement should be kept intact. A rough estimation is two weeks. This is based on a bridge consisting of 40 girders of 1,2 [m] wide, a deck thickness of 250 [mm], a length of 20 [m], an average rate of hydro demolition of 25 [ft^3/h] [71] and 4 machines working 8 hours a day. Sawing the girders to shorten them in length or width direction will not take much time, however personal and machines should be available. Based on these factors the time between release and reuse is at least a month.

§3.4.8 Overview search criteria

Figure 3.10 provides an overview of the search criteria and influencing factors. The input factors from the system design are project dependent. Despite this input also external input is used in the search criteria. This input is based on current research, knowledge and experience. If in the future more information and experience is present these factors should be updated.

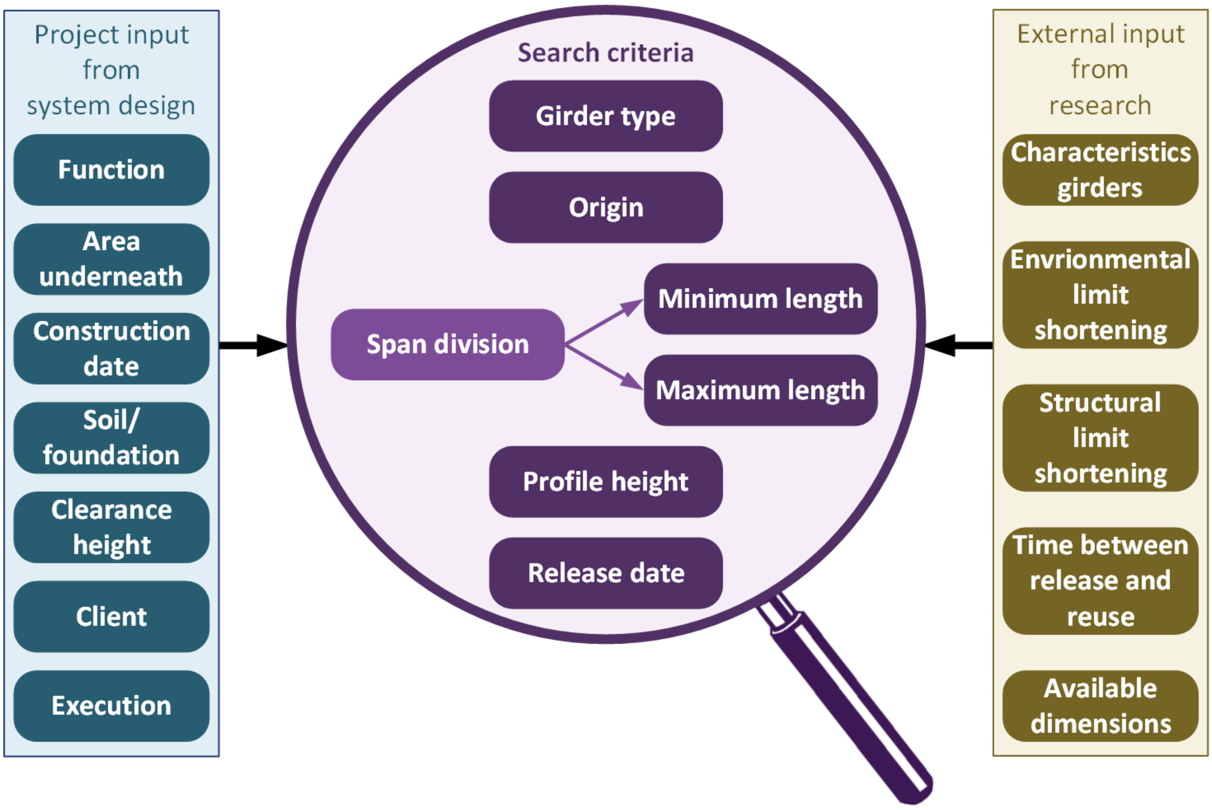


Figure 3.10: Overview search criteria and influencing factors on these criteria.

§3.5 Roadmap structural analysis

Up to now potential suitable girders are identified and design alternatives are developed. Subsequently the structural feasibility of the alternatives has to be determined. For this analysis a road map is developed. First the order of the required steps is discussed. In the next sections each step is discussed in detail.

§3.5.1 Steps

The intention of the structural assessment is to determine the feasibility of the alternative. In this early stage it is important to assess the most critical steps as quickly as possible. This limits the time wasted if an alternative turns out to be unfeasible.

Figure 3.11 gives an overview of the required steps. First the needed information is gathered. If the girders need to be shortened in the new configuration, it should be checked if this is possible. Most prefabricated prestressed concrete girders are equipped with splitting reinforcement. This reinforcement is present at the ends of the girder and is used to introduce the prestressing forces to the concrete. If the girders are shortened, this reinforcement is cut off and the concrete itself should resist the tensile splitting and spalling stresses that develop. This is the subsequent step in the analysis, because there are no modifications possible that replace splitting reinforcement. Moreover, this step only requires information about the prestressing force and concrete quality of the girder.

The other verification steps relate to the capacity of the girder, which can only be verified if the design loads are known. Shear capacity is often identified as critical criteria for reuse. Therefore, this step is performed before the bending moment capacity. If the capacity is insufficient the alternative is put on a temporary hold. Although strengthening might be a solution, first the other alternatives are worked out. The reason is that strengthening makes alternatives less attractive and it requires more detailed calculations.

Although durability does not include any structural calculations, it is an important step in the structural verification process. The durability assessment provides information about the quality of the girders as well as reparation needed. Moreover, the inspection also provides information that is needed in the structural calculation. Consequently, durability is also mentioned in the roadmap. Nonetheless this assessment is very different from the other steps and therefore discussed in a separate paragraph, §3.6 .

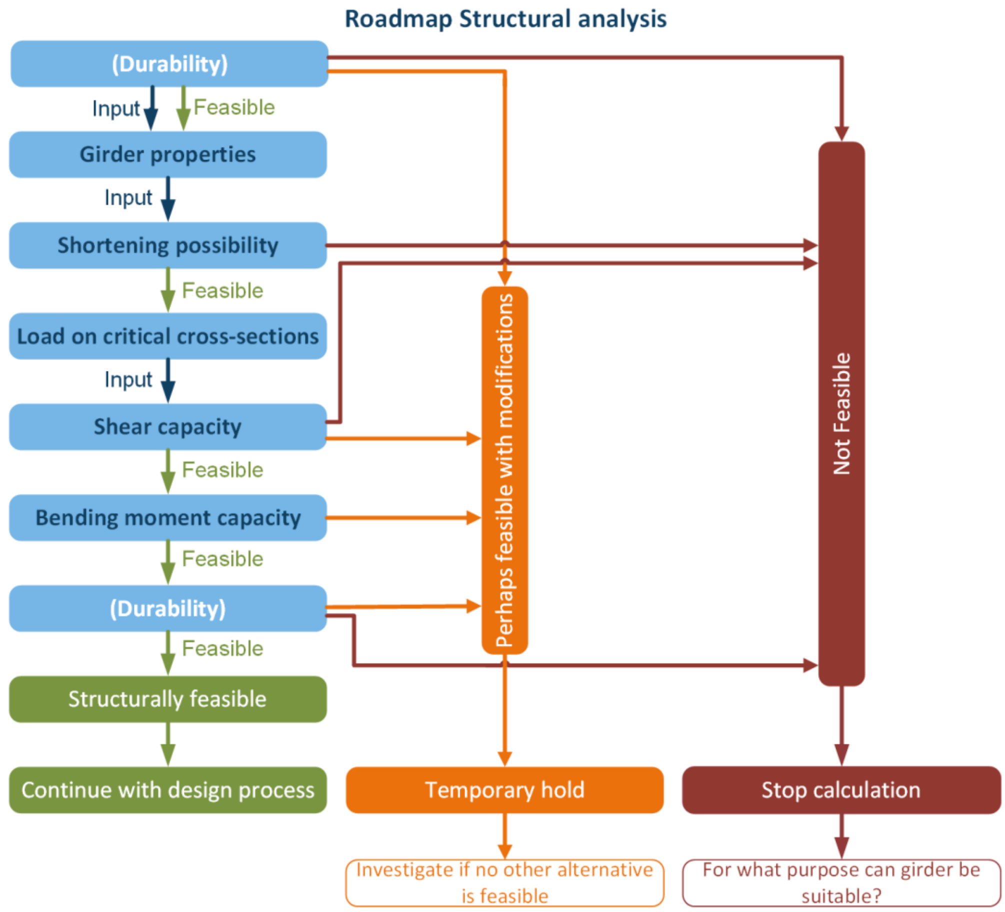


Figure 3.11: Roadmap that gives an overview of the main steps in the structural analysis.

§3.5.2 Girder properties

Input is required on concrete quality, girder type, length, prestressing tendons and shear reinforcement. An overview of the most important characteristics is given in Table 3.3. For concrete quality the concrete class of girder and deck can be used as input or the main characteristics as compressive strength, elastic modulus and tensile strength can be used. The cross-sectional dimensions are derived from the girder type or in case of deviating girders derived from drawings or inspections. For prestressing the stress in the prestressing tendons as well as the occurred and not yet occurred losses have to be known or assumed. Moreover, the average tendon diameter and eccentricity are required. If the girders need to be shortened a more detailed tendon layout is needed. Regarding shear reinforcement the diameter, steel type and spacing are needed. For the bending moment capacity the elastic modulus of the prestressing steel is required.

There are three levels to get input, which are shown in Figure 3.12. The first one is recent inspections or material tests. This input is most reliable and preferred in the structural calculation as it directly comes from the girders in current state. To gather this information material tests or inspections have to be performed, which take time. Moreover, depending on the type of test advanced equipment is needed. This is further discussed in the durability analysis in §3.6 . The second type of source are design drawings and specifications. This information is also specific for the structure. However, due to time dependent effects characteristics have changed. Therefore, these findings are often combined with assumptions based on research and tests on comparable structures. Since, this information is not element/structure specific, it is less reliable. In case almost no information is available the assumption will be based on minimum values, which is often conservative. Nonetheless, this information is most easy to access.

Table 3.3: Overview input characteristics.

Girder characteristics	Concrete characteristics	Reinforcement characteristics	Prestressing characteristics
Dimensions	Compressive strength	Cover	Area of tendons
	Tensile strength	Spacing	Location of tendons
	Elastic modulus	Diameter	Residual stress (and loss)
	Rupture strain	Yield strength	Elastic modulus

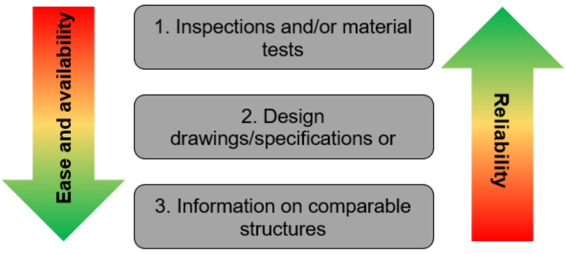


Figure 3.12: Information sources.

§3.5.3 Shortening possibilities

An overview of steps to investigate the shortening possibilities is shown in Figure 3.13.

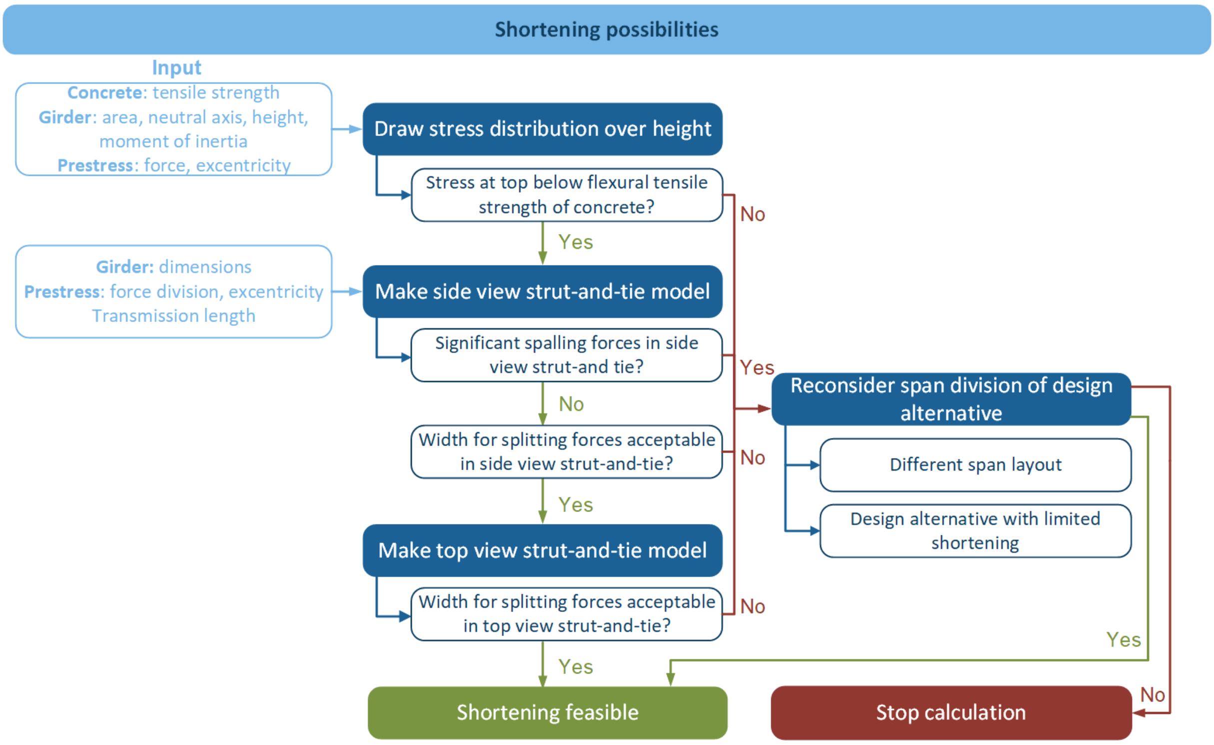


Figure 3.13: Roadmap shortening possibilities.

First it is verified with Equation 3.1 if the stress at the top remains below the flexural tensile strength of concrete. Otherwise, the girder is not suitable for reuse in the project of interest because cracks will occur at the top.

Equation 3.1: maximum (tensile) stress at top of girder.

$$\sigma_{top} = -\frac{F_p}{A} + \frac{F_p \times e \times (h-z)}{I} \leq \frac{f_{ctm}}{\gamma_c}$$

F_p = prestressing force currently present [N]
A= area girder [mm^2]
e= new eccentricity of prestressing force [mm]
h = height of girder [mm]
z= neutral axis of girder [mm]
f_{ctm} = flexural tensile strength concrete [N/mm^2]
γ_c= material factor concrete [-]

Strut-and-tie models are based on truss analogy and provide insight in complex stress patterns, which occur for instance at the application of prestressing forces [136]. In reality the prestressing force is introduced by multiple tendons over the transmission length, which can be calculated with Equation 3.2. In the strut-and-tie model the prestressing force is introduced all at one by two forces (one from the straight tendons and one from the curved tendons). The introduced forces have to find their way to a uniform stress distribution. The location from which on the stress distribution is uniform is called the Bernoulli (B) region. In between the point of application and the B region a disturbed (D) region is present. The length of this region is calculated with Equation 3.3. The stress distribution in D regions is schematized using compressive struts and tensile ties like a truss. Figure 3.14 provides a graphical representation. Vertical tensile ties at the beginning of the girder are spalling forces (shown in yellow), vertical tensile ties halfway the disturbed region are splitting forces (shown in green). In total the splitting forces should be in equilibrium as well as the spalling forces. So, the sum of the yellow forces should be zero and the sum of green forces should also be zero.

In a similar way the distribution of the forces in width direction can be schematized in a top view of the girder. Nonetheless in most cases the side view will be dominant.

Normally strut and tie models are used to calculate the amount of reinforcement needed. Therefore, the tensile forces are divided by the yield strength of reinforcement steel to find the required area of steel. In this case the tensile force is divided by the design tensile strength of concrete to find the area of concrete needed to resist the force. The height of the area follows from the model, so the width can be calculated. Next it has to be interpretated if this width is reasonable. So, if it is reasonable to assume that the force in the girder divides over at least the calculated width.

The spalling forces occur at the end of the girder. Most likely these forces are more difficult to be distributed. Therefore, spalling forces should be avoided as much as possible. For splitting forces more space is available. According to the case study between 14% and 34% of the transfer length is needed to distribute the splitting forces that occur. This transfer length is approximately half the length of the disturbed region.

If the splitting or spalling stresses are too high the only option is to review the span division. Does the girder really need to be shortened? Or are there options with less shortening, increasing the total length or increasing the span of concern and reducing another. Reducing the prestress by reducing the width of the beam and thereby cutting off prestressing tendons in the bottom flange is not a suitable solution. From the results of the case study it can be concluded that due to the change in cross-sectional properties a comparable situation occurs.

Equation 3.2: Transmission length, equation 8.15 from NEN_EN 1992-1-1. Background of equation: bond stress equals the force to transmit divided by the area available for transmission. This is the circumference of the steel tendon multiplied by the transmission length.

$$l_{pt} = \alpha_1 \times \alpha_2 \times \emptyset \times \frac{\sigma_{pm0}}{f_{pt}}$$

$\alpha_1 = 1{,}25$ for sudden release, $\alpha_1 = 1{,}0$ for gradual release
$\alpha_2 = 0{,}25$ for circular cross-section (wires), $\alpha_2 = 0{,}19$ for 3- and 7 wire strands
$\emptyset =$ nominal diameter of prestressing tendon [mm]
σ_{pm0}= stress at in tendons at moment of shortening [N/mm^2]
f_{pt}= bond strength [N/mm^2] $\quad f_{pt} = \eta_{p1} \times \eta_1 \times f_{ctd}(t)$
$\eta_{p1} = 2{,}7$ for wires, $\eta_{p1} = 3{,}2$ for 3- and 7 wire strands
$\eta_1 = 1{,}0$ with good bonding circumstances, $\eta_1 = 0{,}7$ otherwise
$f_{ctd}(t) =$ design tensile strength of concrete

Equation 3.3: Length of disturbed region.

Distribution over height: $L_{dis} = \sqrt{h^2 + (0{,}8 \times l_{pt})^2}$ Distribution over width: $L_{dis} = \sqrt{\frac{w^2}{2} + (0{,}8 \times l_{pt})^2}$

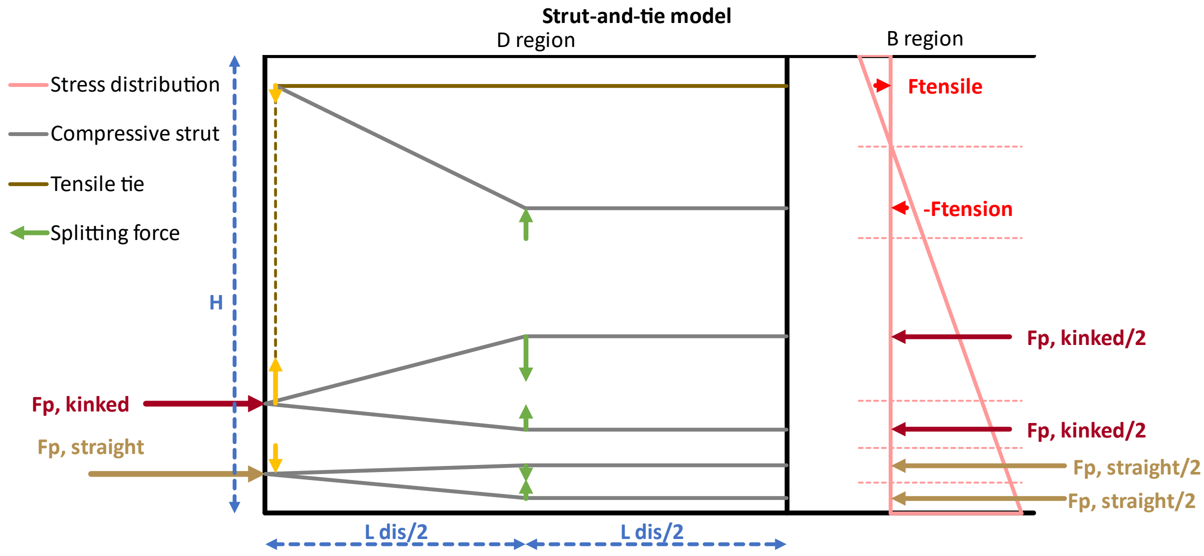


Figure 3.14: Principle of strut-and-tie model on a side-view of the girder.

§3.5.4 Design load on critical cross-sections

Since the structural calculation provides an indication of the structural feasibility, only the following loads are considered: prestress, self-weight, asphalt, traffic. The application of girder in a new construction is divided in stages:

1: After placing the girders at the support, the girders are subjected to self-weight and prestress. Contrary to new girders this prestress force is reduced by time-dependent losses. The total system is statically determined.

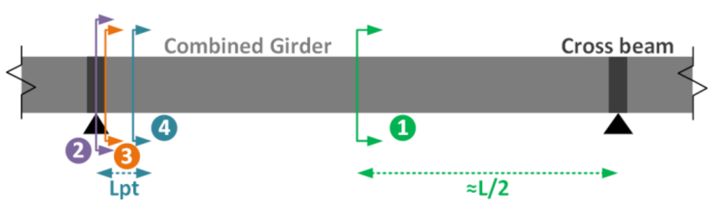
2: A new in-situ deck is casted on top. The additional self-weight is carried by the girders, because the deck cannot bear any load yet. The total system is statically determined.

3: The deck is loaded with the weight of asphalt and traffic, which is carried by the combined action of girders and deck. The system is statically determinate if a flexible transition joint is used. The system is statically indeterminate if a continuous connection is created at the intermediate supports. In this case the girders work together in longitudinal direction as well and hogging moments occur at the intermediate supports.

In Table 3.4 four critical cross-sections with the critical verification are identified. On these locations the maximum design load is calculated. Each load is considered separately to enable the use of different partial safety factors during verification. The governing sagging bending moment occurs near midspan. The shear force at the supports and next to the supports is needed, because at the support often a crossbeam is present, which can increase the shear capacity. Next to the support, no crossbeam is present. So, while the shear force in this area is lower, it can still be the critical factor.

Initially it is sufficient to consider a statically determinate system. If the bending moment capacity is insufficient, the loads in a statically indeterminate system are required. In a statically indeterminate system hogging bending moment occur. The maximum hogging bending moment can occur at the supports or after the introduction of the prestressing force, because for hogging bending moment the prestressing force is unfavourable.

Table 3.4: Critical cross-sections and verifications needed.



Cross-section	Checks
1	Sagging bending moment
2	Hogging bending moment (only SI system) and shear force
3	Shear force
4	Hogging bending moment (only SI system)

To calculate the design load different software or hand calculation procedures can be used. In the case study of this research the longitudinal load distribution is based differential equations and SCIA engineer. The traffic load distribution in width direction is based on a load spread under 45° until the neutral axis and SCIA engineer. More explanation can be found in Appendix E: Structural calculation.

§3.5.5 Shear capacity

Figure 3.15 provides a roadmap to verify the shear capacity.

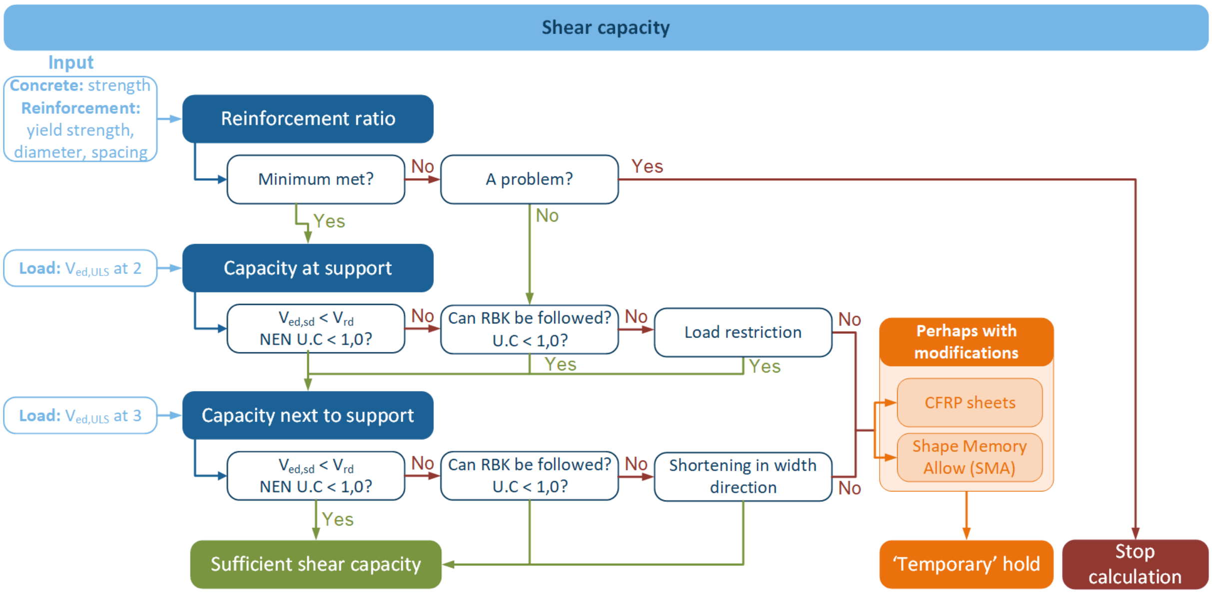


Figure 3.15: Road map shear capacity

If the calculation procedure of NEN-EN-1992-1-1 results in too little shear capacity the calculation procedure of the RBK might be useful. It should be kept in mind that currently no guidelines or standards for reuse exists. It might not be fair to demand the standards for new on an existing element. On the other hand, demanding only the RBK requirements might result in a reduction of life span, which is also unwanted. Therefore, the consideration on which standard to use and how to interpret them should be decided for each project individually. It should be noted that currently a lot of research is going on in the field of shear capacity. Also, in the new Eurocode the equations regarding shear capacity will change.

First the minimum reinforcement ratio is verified with Equation 3.4. The NEN-EN-1992-1-1 requires this minimum reinforcement ratio as a safety measure to make sure that the structure warns before failure. The RBK does not provide regulations regarding minimum reinforcement ratio.

Equation 3.4: Minimum shear reinforcement ratio.

$$\rho_w = \frac{A_{sw}}{s \times b_w \times \sin(\alpha)} \geq \frac{0{,}08 \times \sqrt{f_{ck}}}{f_{yk}}$$

A_{sw} = area shear reinforcement [mm²] for regular stirrups: $A_{sw} = 0{,}5 \times \pi \times \phi^2$
b_w = width of web [mm]
s = spacing stirrups [mm]
d= effective height [mm]
α= angle of stirrups, usually 90°
f_{yk}= characteristics yield strength of reinforcement [N/mm²]
f_{ck}= characteristic concrete compressive strength [N/mm²]

Next the shear resistance of concrete and reinforcement is calculated with Equation 3.5 and Equation 3.6. In cross-section 2 (at the support) σ_{cp}=0 because the prestressing force is not yet introduced. In cross-section 3 (just next to crossbeam), the prestressing force is partly introduced. It is assumed that the prestressing force linearly increases from 0 to F_p over the transfer length. However, in case of a statically indeterminate structure near the supports cracking occurs at the top, while the prestressing force is located at the bottom. So, instead of closing the cracks, the prestressing force has a negative influence. Therefore σ_{cp}=0 as well. From cross-section 4 onwards the full prestressing force is present. The concrete quality of deck and girder differ; therefore the resistance of both parts is calculated separately. However, for the size effect the combined effective height is used.

In case a crossbeam is situated at the support the shape of cross-section 2 (at the support) is rectangular and b_w is equal to the girder width. For the remaining cross-sections in statically indeterminate structures b_w is equal to the width of the web. The reason for this is that near the support the tensile zone is located at the top, so cracks occur in the web. Near the midspan the tensile zone is located at the bottom, however this zone is higher than the flange height, so still cracking will occur in the web as well.

In the RBK b_w is replaced by $b_{w,average}$, which is calculated with Equation 3.7 and indicated in Figure 3.16. Only in most cases the cross-section of the girder is like cross-section II of Figure 3.17. So, there are no consequences for the girder and the increased width can only be used for the deck. Only for inverted T-girders where at the ends a hammerhead is present the width (b_w) of the girder can increase. With a hammerhead the width of the web is larger at the end of the girder. So, the cross-section changes over the verification area. Therefore, first the verification area is projected. This is shown in Figure 3.17. A hammerhead is for example used in Gamma girders from Betonson or HNP girders from Spanbeton.

Equation 3.5: Minimum shear resistance of concrete only.

$$V_{rd,c} = \left(0{,}035 \times k^{\frac{3}{2}} \times \sqrt{f_{ck}} + 0{,}15 \times \sigma_{cp}\right) \times b_w \times d \leq 0{,}5 \times b_w \times d \times f_{cd} \times v$$

k = size effect = $1 + \sqrt{\frac{200}{d}} \leq 2$

$\sigma_{cp} = \frac{F_p}{b_w \times d} < 0{,}2 \frac{f_{ck}}{\gamma_c}$

b_w = smallest width of cross-section in tensile zone

d= effective height

$v = 0{,}6 \times (1 - \frac{f_{ck}}{250})$ reduction factor for concrete cracked due to shear

Equation 3.6: Shear resistance of shear reinforcement only.

$$V_{rd,s} = \frac{A_{sw}}{s} \times z \times f_{yd} \times \cot(\theta)$$

A_{sw} = area of stirrups = $2 \times \frac{1}{4} \times \pi \times \emptyset^2$

s = spacing stirrups

z = internal lever arm $z \approx 0{,}9d$

θ = angle of compressive strut

Equation 3.7: width according to RBK for concrete shear resistance.

$$b_{w,average} = \frac{A_{net,projected}}{d} \leq 1{,}25 \times b_{w,min}$$

$A_{net,projected}$ = net concrete area. Area in effective height zone of projected cross-section where widening is only considered under a 45° angle.

d = effective height

$b_{w,min}$= smallest width of cross-section

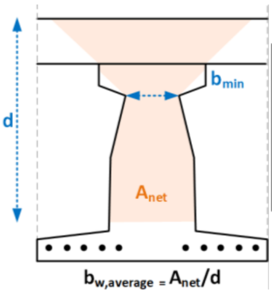


Figure 3.16: Net area according to RBK.

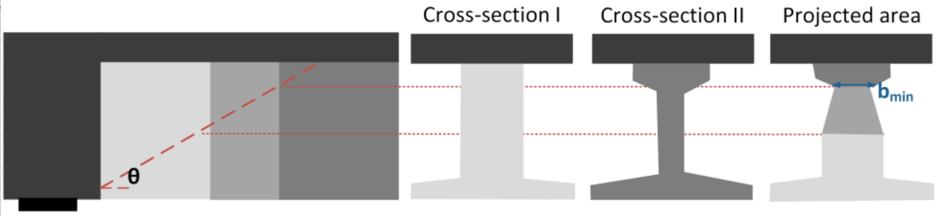


Figure 3.17: Projection verification area according to RBK.

For the resistance of the reinforcement according to NEN-EN 1992-1-1 the angle θ is taken as 21,8°. In this way the maximum number of stirrups is used. This causes the largest force in the compressive strut, however due to the height of the girders this will not be a limiting factor. For the verification according to the RBK θ is taken as 30°.

According to the RBK the total shear resistance of the cross-section is the sum of the resistance of concrete and reinforcement. Nonetheless the resistance can only be combined in case the minimum reinforcement ratio is met. Otherwise, the shear capacity is the capacity of concrete only. For new structures (following the NEN-EN 1992-1-1) the shear resistance is the resistance of concrete only. In case concrete is not able to withstand the shear load, the whole load should be taken by the reinforcement. So, the shear capacity is the maximum of the resistance of concrete and reinforcement.

With a statically determinate structure, the bending moment near the supports is zero. As a result, the girder remains uncracked in this area. So, in theory only tensile splitting failure and no flexural shear failure can occur in these areas. Uncracked concrete has a higher shear capacity compared to cracked concrete. So, the shear capacity calculated with Equation 3.5 might be an underestimation. Therefore, if this capacity is insufficient and the following points are applicable a distinction can be made between flexural shear and tensile splitting failure.

- The girders originate from a statically determinate structure.
- The girders are going to be used again in a statically determinate structure.
- No tensile stresses at the top of the girder due to prestress.

Nonetheless, it should be noted that calculation procedures for the shear capacity in uncracked regions are still in development. Moreover, the equations and procedures described below only give an indication and do not show the complete procedure and background. So, if this procedure is followed the RBK and NEN-EN-1992-1-1 should be consulted.

Tensile splitting failure occurs when the principal stress in the concrete reaches the concrete tensile stress. The RBK and NEN-EN-1992-1-1 provide a comparable equation, shown in Equation 3.8, to calculate the capacity. To derive this capacity the circle of Mohr can be used. According to the RBK, although flexural shear failure can theoretically not occur in these areas experiments have shown that similar calculation procedures can be used. Therefore, flexural shear failure also has to be checked with Equation 3.9.

Equation 3.8: concrete capacity tensile splitting failure in uncracked areas.

$$V_{Rd,c} = \frac{I \times b_w}{S} \times \sqrt{{f_{ctd}}^2 + \sigma_{cp} \times f_{ctd}} \leq 0{,}5 \times b_w \times d \times f_{cd} \times v$$

I = moment of inertia [mm^4]
b_w = width of cross-section at considered location
S = first moment of area [mm^2]
f_{ctd} = design tensile strength of concrete [N/mm^2]
σ_{cp} = compressive stress in concrete due to prestress at considered location (so consider the transfer length). [N/mm^2]

Equation 3.9: concrete capacity flexural shear failure in uncracked areas.

$$V_{Rd,c} = \left(0{,}12 \times k \times (2 \times f_{ck})^{1/3} + 0{,}15 \times \sigma_{cp}\right) \times b_{w,gem} \times d$$

When the shear capacity at the supports (cross-section 2) turns out to be insufficient the only option without significant modifications is a load restriction on the bridge. This option can be suitable for small regional bridges. In this way the design traffic load can be calculated by subtracting the design loads for self-weight, prestress and asphalt load from the capacity. The remaining capacity is available for the design traffic load and can be converted to a maximum vehicle weight and/or axle load allowed on the structure. Traffic signs, as shown in Figure 3.18, should be placed to indicate this restriction.

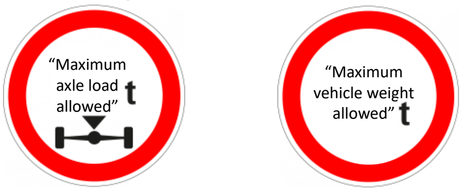


Figure 3.18: Traffic signs to indicate limit access of the structure for heavy vehicles.

When the shear capacity next to supports (cross-section 3) turns out to be insufficient another solution can be decreasing the width of the girders. By shortening the flanges of the girders the capacity remains the same, because the web is the determining factor. However, the girders can be placed closer to each other. So, the load that has to be carried by each girder decreases. The reduction in traffic and asphalt load is approximately linear, but the reduction in self-weight and prestress is not. Therefore, the reduction in width needed is calculated by an iterative procedure. The principle of shortening in width direction is indicated in Figure 3.19.

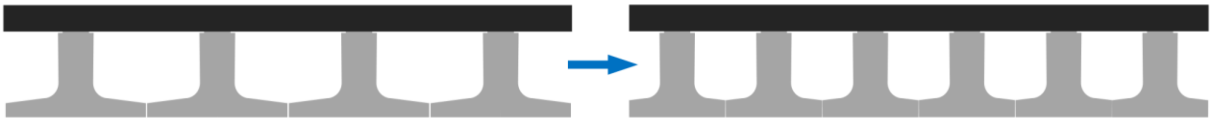


Figure 3.19: Shortening in width direction by cutting off a part of the flanges.

If the shear capacity remains insufficient or the proposed solutions are not suitable, the alternative is put on temporary hold. Strengthening of the girders by CFRP-sheets or Shape Memory Allow might be possible, but this requires detailed calculations as well as more material input. Therefore, these options are only considered in case no other alternative is structurally feasible.

§3.5.6 Bending moment capacity

The roadmap for the bending moment verification is shown in Figure 3.20.

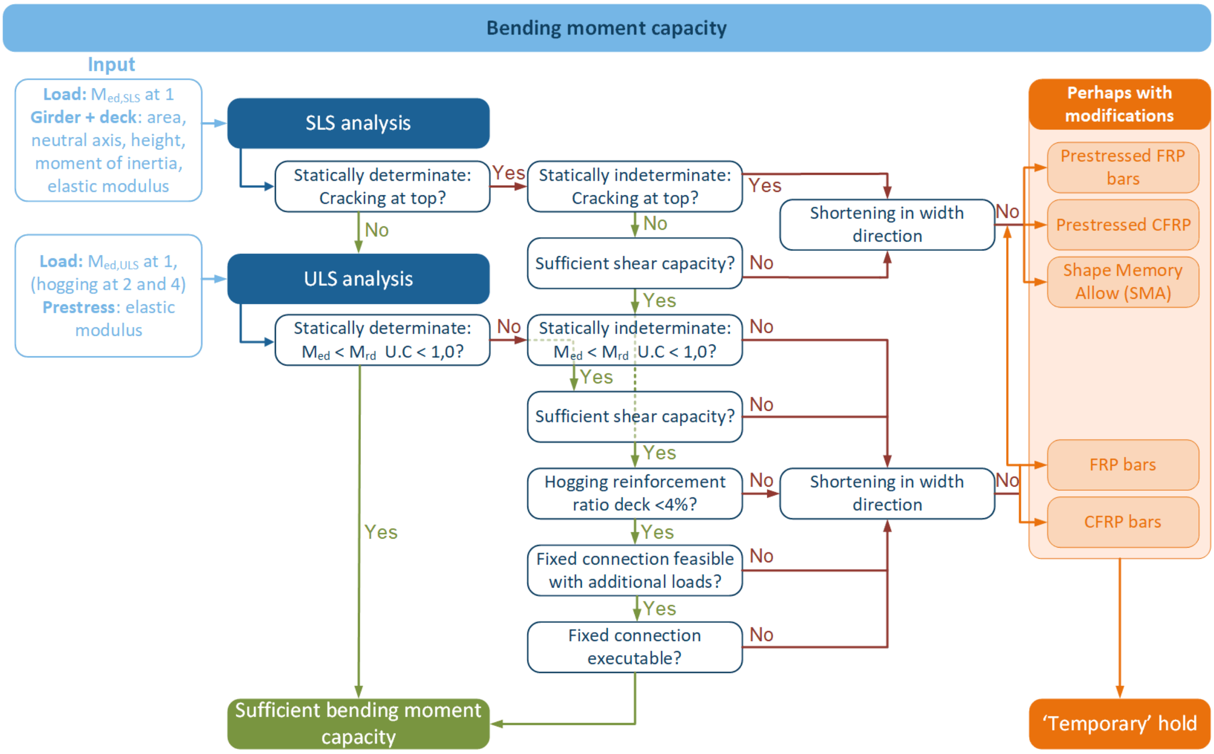


Figure 3.20: Roadmap bending moment capacity.

First, the serviceability limit state (SLS) is checked for a statically determinate structure at cross-section 1 (near midspan) with Equation 3.10. The bottom of the girder should not crack.

Equation 3.10: serviceability limit state verification.

$$\sigma_{bottom} = -\frac{F_p}{A_{girder}} - \frac{F_p \times e \times z_{girder}}{I_{girder}} + \frac{M_{self\ weight} \times z_{girder}}{I_{girder}} + \frac{M_{load} \times z_{combined} \times E_{girder}}{EI_{combined}} \leq \frac{f_{ctm}}{\gamma_c}$$

F_p = minimum prestressing force [N]
A= area [mm^2]
e= eccentricity of prestressing force [mm]
z = neutral axis [mm]
I= moment of inertia [mm^4]
E = elastic modulus [N/mm^2]
M= bending moment (near) midspan [Nmm]

f_{ctm} = flexural tensile strength concrete [N/mm^2]
γ_c= material factor concrete [-]

Next the bending moments in the ultimate limit state are analysed. The ultimate bending moment capacity is determined based on the rupture strain. The bending moment capacity is reached, when the concrete reaches it rupture strain in compression. The calculation is an iterative procedure, which is complicated by the non-uniform cross-section and concrete characteristics. In addition, the bottom part of the cross-section (the girder) is already loaded by the self-weight and the prestress. The steps are summarized in Equation 3.11. Moreover Figure 3.21 provides a graphical representation of the procedure.

Step 1: A stress in the prestressing steel is assumed. Subtracting the initial stress gives the additional stress in the prestressing steel from which the additional tensile force is calculated.

Step 2: With the help of the stress strain diagram of the prestressing steel the assumed strain can be found. By subtracting the initial strain, the additional strain is found.

Step 3: With horizontal force equilibrium the compressive force that should be provided by the concrete can be found.

Step 4: It is first assumed that the rupture strain of the in-situ deck is governing [137] and that the compressive capacity of the deck is fully used. In this way the magnitude of the compressive force in the deck can simply be calculated by multiplying the area by the design compressive strength. The residual compressive force should be delivered by the girder.

Step 5: The height of the compressive zone in the girder is determined by dividing the force over the width of the web and stress in the concrete. Since, the width of the web is used it should be verified if the compressive zone remains in this area. Because the strain capacity of the girder is not fully used the values for α and β are iteratively changed as well. These are based on the maximum strain found in the girder in the previous iteration.

Step 6: The stress-strain diagram over the height of the cross-section is drawn. This is based on the two known points: at the top of the cross-section section the rupture strain of concrete is reached and at the end of x_u the strain is zero. From this diagram the additional strain in the prestressing steel is derived.

Step 7: the strain at the bottom side of the deck should be higher than the strain at the peak stress (ε_{c3}). Otherwise, the full capacity of the deck is not used and the assumption is false. If the assumption is false α and β for the deck have to be iteratively changed as well.

Step 8: The steel strain found in the step 6 should be analogous to the strain found in step two. If not, a new stress in the steel is assumed and the above-mentioned steps are followed again, until the condition is satisfied.

Step 9: The total strain in the girder is found by adding the strain generated by self-weight and prestress. The total strain at the top of the girder should be below the rupture strain. Otherwise, the girder is governing instead of the deck.

Step 10: Finally, moment equilibrium around the bottom of the girder provides the bending moment capacity.

Equation 3.11: Iterative procedure to calculate bending moment capacity.

1. $\Delta F_p = (\sigma_{p,assumed} - \sigma_{p,intial}) \times A_p$
2. $\Delta\varepsilon_{p,assumed}$ follows from σ, ε diagram
3. $\sum F_{horizontal} = 0 \rightarrow F_{concrete} = \Delta F_p + F_{p,initial}$
4. $F_{girder} = F_{concrete} - F_{deck}$
   $F_{deck} = h_{deck} \times w_{deck} \times f_{cd,deck}$
5. $x_{u,girder} = \frac{N_{girder}}{\alpha \times b_{web} \times f_{cd,girder}}$
6. $\Delta\varepsilon_p = \frac{ds + h_{deck} - x_{u,girder}}{x_{u,girder}} \times \varepsilon_{cu,deck}$
7. $\varepsilon_{bottom\ deck} \geq \varepsilon_{c3}$ → otherwise adapt α and β
8. $\Delta\varepsilon_p = \Delta\varepsilon_{p,assumed}$ →otherwise new iteration
9. $\varepsilon_{top\ girder} \leq \varepsilon_{cu}$
10. Calculate bending moment $\sum F \times z$

F = Force
P = Prestressing
σ = stress
A = area
ε= strain
h = height
w = width
f = strength
Z = lever arm

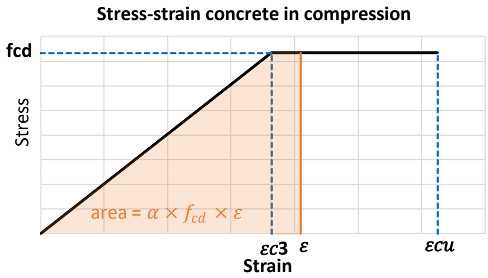


Figure 3.21 a) Stress-strain diagram of concrete. Values depend on concrete characteristics.

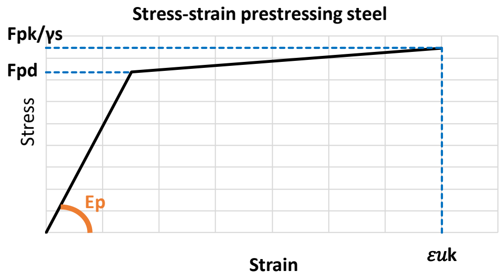


Figure 3.21 b) Stress-strain diagram of prestressing steel. Values depend on prestressing steel characteristics.

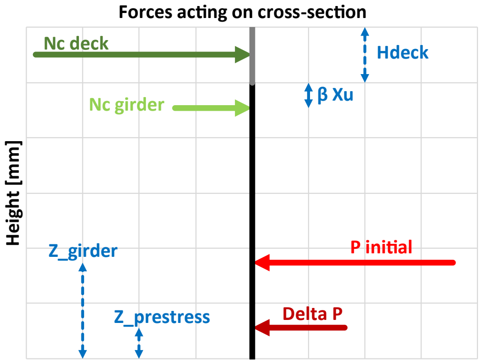


Figure 3.21 c) Forces on the cross-section. These should be in horizontal equilibrium. Moreover, they should match with the strains from figure d.

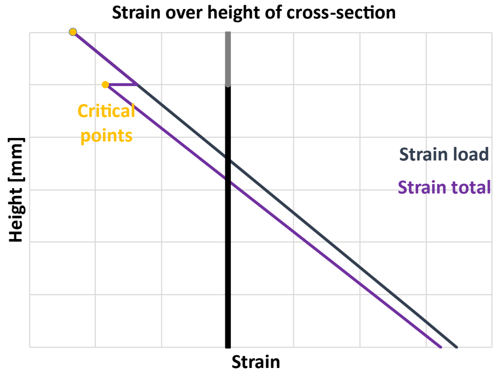


Figure 3.21 d) Strain diagram over height of the cross-section. Two critical points are marked. At these points the strain should be smaller than ε_{cu}.

Figure 3.21: Graphs and figures used to calculate bending moment capacity.

If the girders have insufficient capacity in SLS or ULS a statically indeterminate system should be checked. Although a statically indeterminate system leads to lower bending moments, the shear forces slightly increase. In addition, if the shear capacity is based on the capacity of concrete, the shear capacity reduces as well. This is because the prestressing force in Equation 3.5 cannot be included. Therefore, the shear capacity has to be verified again. Another point of attention is the hogging moment near the supports. As a result, tensile forces occur at the top that should be resisted by the reinforcement in the in-situ deck.

The in-situ deck will be new. So, instead of verifying if sufficient reinforcement is present, the amount of reinforcement can be determined based on Equation 3.12. The maximum design hogging bending moment is divided by the internal lever arm, which gives the tensile force that the reinforcement should resist. So, this force is divided by the yield strength of the reinforcement. Next a rebar diameter and spacing can be chosen. Since the whole deck is under tension, the reinforcement can be placed in two rows on top of each other. In most cases three layers will not be possible, because of the minimum spacing and cover requirements. In addition, the reinforcement ratio should remain below 4%. However, it is expected that these requirements will in most cases not lead to any problems.

Equation 3.12: Reinforcement in deck.

$$M_{ed} < M_{rd} \qquad \frac{M_{hog,ed,ULS}}{b_{girder}} < A_s \times f_{yd} \times z$$

$M_{hog,ed,ULS}$ = design hogging bending moment [Nmm]
b_{girder} = width of girder [mm]
z= internal lever arm [mm] $\approx 0{,}9 \times d$ (d = effective height)
A_s= amount of reinforcement [mm^2/mm] $= \frac{0{,}25 \times \pi \times \phi^2}{s}$
$\rho_{required} = \frac{A_s}{1000 \times h_{deck}} \leq 4\%$

A point that may cause problems is the additional loads. In a statically indeterminate system temperature, shrinkage and foundation settlements may cause additional load in the system. In statically determinate structure these factors do not play a role if sufficient space is available for the structure to move. Therefore, especially in a statically indeterminate structure these factors should be investigated.

The final point is the execution, which significantly differs for a statically determinate and indeterminate system. In a statically indeterminate structure a single crossbeam should connect the girders in longitudinal direction, while a statically indeterminate structure requires a flexible connection between the girders in longitudinal direction.

If the bending moment capacity is still insufficient or a statically indeterminate system is not possible shortening the girders in width direction can be a solution. In general, shortening in width direction is beneficial for the bending moment capacity. Although the capacity of the girder decreases, because tendons are cut off the load reduces more. As first indication the self-weight of the girder can be kept the same and only the load on the girder can be adapted until the girder fulfils. Since the cross-sectional properties and loads etc. all changes the shortened girder should be treated as kind of new alternative.

If the bending moment capacity remains insufficient or the proposed solutions are not suitable strengthening of the girders by FRB bars, CFRP bars or shape Memory Allow might be possible. These methods should be applied prestressed to improve the capacity in serviceability limit state. For the ultimate limit state, the methods can be applied without prestress.

§3.6 Durability

A step in the structural analysis is the durability. To assess the durability a roadmap is developed. It is worth mentioning that durability should be rather broadly interpreted. Besides the residual lifespan, durability is also about the current state of the girders, needed repairs and the material characteristics. First the context of durability is discussed together with the timing of the steps regarding the other steps of the structural analysis. Next the roadmap of durability is presented after which the steps of this roadmap are discussed in more detail. Finally, reparation and protection measures are discussed.

§3.6.1 Context and time in structural assessment

The input needed in the structural calculation is already discussed in §3.5.2. The first level of input source is based on information from inspections and material testing. This relates to durability because it is about the current state and characteristics of the girder. If this information is available, it can directly be used in the structural verification. Moreover, it can be used to assess the element on deterioration and residual life span. So, in this case durability should be assessed in advance to all the other steps of the structural verification. Furthermore, if critical points are identified additional material tests or destructive testing can be considered. Further analysis can include the planning and costs of these additional tests, reparations or protective measures.

However, it is also likely that girders are offered on an exchange platform to become available, but no inspections or tests are performed yet. Contingent upon insufficient or no information is available about the current state of the girders. To assess the durability, time and money consuming tests are needed, which may even be impossible in this stage as the girders are still part of an existing structure. In this case, it is more appropriate to assess the durability when there is more certainty about the suitability of the girders in the project of concern. Consequently, input about the girder characteristics should be based on the two lower levels of input sources: drawings and assumptions (see Figure 3.12). For some characteristics the assumptions may be insufficiently reliable. These characteristics should definitely be verified in the durability assessment. Based on inspections and testing it should be verified if the condition of the girder is sufficient and fits with the assumptions made. Finally, if the characteristics differ from the assumptions the impact on the structural capacity should be analysed. If the capacity is lower than expected strengthening or modification might be needed. But if the capacity is higher than expected proposed modifications may not be needed after all.

In conclusion, the timing of the durability assessment within the structural analysis depends on the information availability. It can be the first or last step, but most likely it will be a combination where some information is available, but additional material tests or inspections are needed.

§3.6.2 Steps

To derive the steps in the durability assessment CUR recommendation 72 and 121 [138], [139] are used as reference together with knowledge about damage and deterioration mechanisms as well as maintenance and repair options. CUR 121 can be used to determine the lower boundary of the expected residual life span of existing structures. CUR 72 provides information on inspections and investigations. Figure 3.22 provides a roadmap, that gives a general overview of different steps and possible procedures needed for reuse.

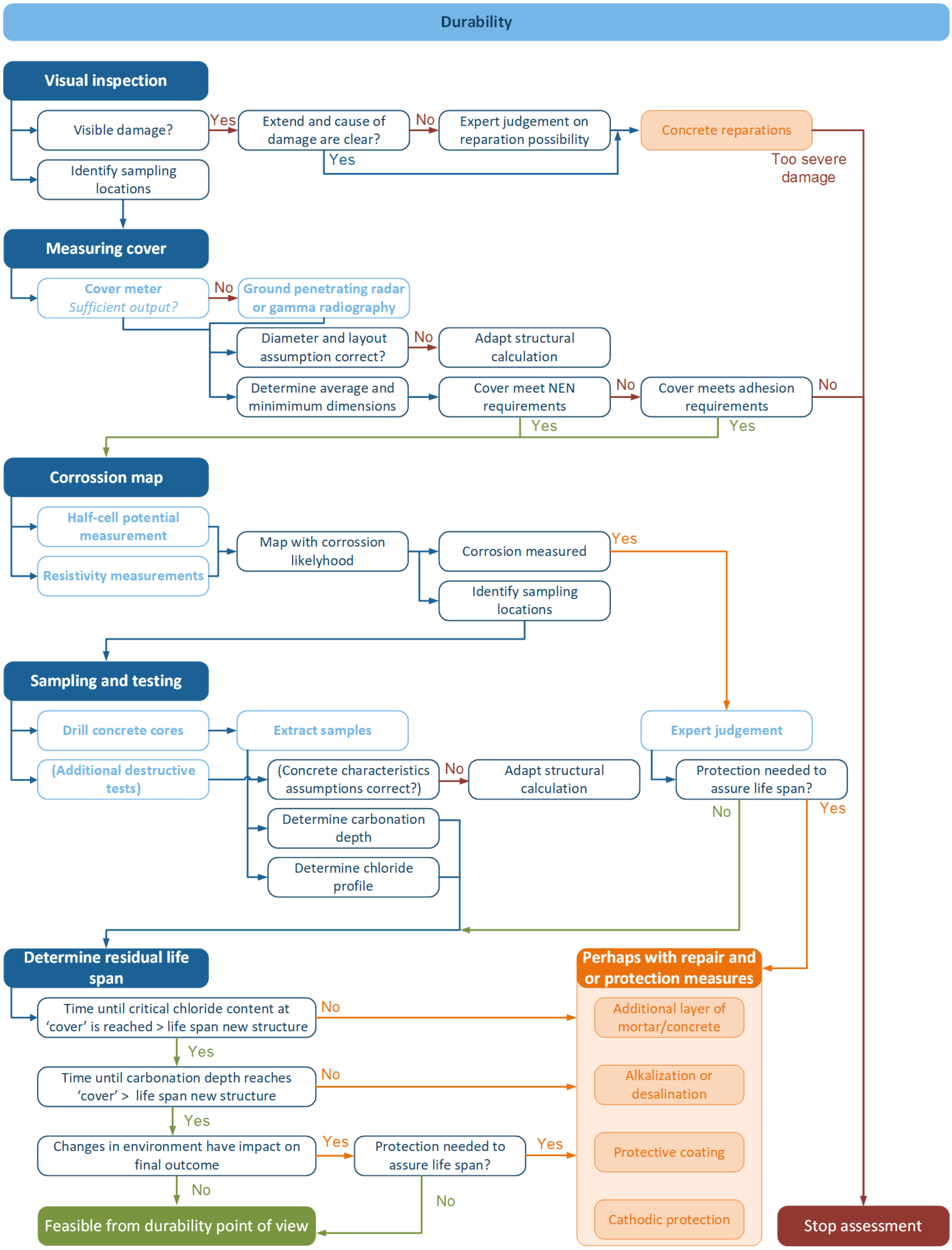


Figure 3.22: Roadmap durability.

§3.6.3 Visual inspection

The first step in the durability assessment is a visual inspection. This visual inspection together with the archived information on the structure is used to assess the general condition of the structure. Damage can manifest itself as cracks, delamination, spalling, rust spots or leakage. On damaged location the extend should be investigated. So, for example in case of cracks the pattern, width and depth should be identified. The possible mechanisms behind the damage should be identified as well. These can be grouped in corrosion, loading situation and concrete degradation due to chemical, biological or physical influences.

Corrosion can lead to rust water, cracks and eventually spalling of concrete and the exposure of rebars. Damage due to loading situation relates to cracks due to overloading, imposed deformation or accidents. Due to overloading, shear or flexural bending cracks might be visible. There are different chemical, physical and biological effects that deteriorate the concrete. If concrete is attacked by acids, hydration products may dissolve which leaves an open microstructure that is prone to further attack and possibly reduced strength. Sulphate attack and alkali-silica reaction results in an expansion reaction. With alkali-silica reaction a map cracking pattern is observed as well. Cracking, spalling and disintegration may also be caused by freeze-thaw attacks. If for example alkali-silica reaction is observed, it is not advised to reuse the girders. This deterioration mechanism is caused by reactive aggregates present in the concrete; hence it can only be slowed down and not stopped. For other degradation mechanisms repair and protection measures depend on the extend of damage.

Based on the visual inspection, the location for sampling and testing are chosen. The sampling locations should be divided over the element. Valuable sample locations come from areas where damage is visible or expected or locations where high loads are likely to occur or likely to have occurred. When after the visual inspection uncertainty in the extend or causes of damage remains expert judgement and additional tests might be needed.

§3.6.4 Non destructive tests on cover and reinforcement

A follow-up on the visual inspection is measuring the cover to the reinforcement. A cover meter, which measures the distance and diameter of a reinforcing bar by electromagnetic search, is the easiest and most used method. Due to the ease of measuring, many measurements can be performed. In case this method does not provide sufficient results more advanced and expensive methods as ground penetrating radar or gamma radiography can be used. The final objective is to verify or determine the reinforcement layout and derive an average, minimum and standard deviation of the concrete cover. The reinforcement layout is not only useful for the structural calculation, but it also indicates the locations where no concrete cores should be drilled and it can be used for the assessment of corrosion probability.

Furthermore, it can be checked if the minimum cover requirements for new structures are met. In general, durability is the governing criteria for the cover thickness. However, in this analysis the residual life span is determined with the cover that is present. So, durability requirements are met as long as the residual life span can be guaranteed. Nonetheless there is also a minimum that refers to the adhesion between concrete and reinforcement. This minimum is often the diameter of the reinforcement and should be always met. However, it is expected that this minimum cover will be met in almost all existing structures.

To get an indication of the corrosion risk of the reinforcement, half-call electrical potential measurements and concrete resistivity measurements are combined. A drop and shift to negative potential indicates corrosion initiation. In addition, with a low concrete resistivity, corrosion is more likely. In this way the reinforcement is mapped out and critical locations are pointed out. On these locations samples can be taken for further investigations. If corrosion is measured it does not directly impose a threat to the residual life span of an element. This depends on environmental conditions. Therefore, expert judgement is advised.

§3.6.5 Residual life span expectation

If existing girders are reused in a new structure, the girders should be able to fulfil their function until the end of the life span of the structure. So, the residual lifetime of the girders should at least be equal to the required lifetime of the structure. In most cases corrosion is the governing failure mechanism. The moment when corrosion starts to degrade the structure, is called the start of the propagation phase and the end of the initiation phase. In the initiation phase aggressive substance migrate into the structure and attack the protective environment around the rebars. In the propagation phase the rebars start to corrode and the corrosion becomes visible. The CUR recommendation 121 indicates this end of the initiation phase as the end of the residual lifetime of an element. Regarding to the definition of NEN 8700, where the end of the residual lifetime is reached if the corrosion of the main reinforcement reaches the level that the safety of the structure is insufficient, this is conservative. However, for reuse this approach is not indicated as conservative. Since, it is not wanted that the elements are in critical condition after reaching the required life span.

Corrosion of the rebars can be chloride induced or carbonation induced. Chloride induced corrosion results in local or pitting corrosion. Due to chloride ingress the passive film around the rebar is locally attacked. For this reaction oxygen and water are needed. The chlorides can originate from seawater, de-icing salts, contaminates or from accelerators added for curing. The chloride concentration is modelled with Fick's law. The equation is shown in Equation 3.13. By determining the chloride concentration at different depth from drilled cores the values for initial chloride content, surface content and diffusion rate can be determined.

Carbonation induced corrosion results in uniform corrosion. Due to a reaction of carbon dioxide with the calcium hydroxide in concrete calcium carbonate is formed, which lowers the pH. If this reaches the reinforcement other reaction products are formed. These products brake down the protective layer around the reinforcement and cause corrosion. These reactions occur at the water-air frontier. So, in wet concrete or completely dry concrete reinforcement does not corrode. Compared to chloride ingress carbonation occurs more uniform, so carbonation moves as a front towards the rebars. Therefore, carbonation is expressed as depth. The movement of the front can be modelled with Equation 3.14. Based on tests with phenolphthalein the carbonation depth can be determined, which can be used to determine the carbonation coefficient.

According to CUR 121 the end of the lifetime is reached when the chloride content at the rebar surface is higher than the critical content or when the carbonation front reaches the rebars. The average critical content is 0,5% chloride on cement weight. CUR 121 advises to use the average concrete cover minus 5 mm. This 5 mm reduces the chance of occurrence of corrosion propagation before the end of the lifetime to 30%. This is seen as acceptable level for existing structures, because uncertainties in design and execution are eliminated. For new construction an acceptable failure probability is 10%. The question raises: which value is appropriate for reuse? There are no uncertainties in the manufacturing of the elements anymore, however uncertainties in the new design and damage during execution are still there. If a failure probability of 10% is required, the safety marge on the concrete cover should be 18 [mm]. Also, regarding the conservative method used, this might be over conservative. If a safety marge of 10 [mm] is applied the probability of failure is 21%, which is approximately in the middle of new and existing. Hence, in this road map the concrete cover minus 10 [mm] is used and is indicated as 'cover'.

Equation 3.13: Chloride penetration model.

$$C(x,t) = C_s - (C_s - C_i) \times \mathrm{erf}\left(\frac{x}{\sqrt{4 \times D \times t}}\right)$$

$C(x,t)$ = chloride content at time t [year] and depth x [mm]
C_i = initial chloride content [chloride ions/cement mass]
C_s = surface chloride content [chloride ions/cement mass
D = diffusion coefficient [mm^2/year]

Equation 3.14: carbonation front.

$$x_c = K \times \sqrt{t}$$

x_c = carbonation depth [mm]
K = carbonation coefficient [mm/$\sqrt{year}$]
t = time [year]

So, the residual life span is estimated by determining the time until the chloride content at concrete cover minus 10 [mm] reaches the critical content of 0,5% and the time until the carbonation front reaches the concrete cover minus 10 [mm]. The minimum of these two is the residual life span. If this residual life span is larger than the required lifetime of the new structure, the alternative is structurally feasible from a durability point of view.

A final remark should be made about the differences in environment between the existing structure and the new structure. The coefficients used in the modelling of the carbonation front and chloride penetration are not only depended on the concrete characteristics, but also on the environmental conditions. For example, in a chloride rich environment (near the sea) the surface content will be higher and chloride penetration will be faster. Therefore, the differences between the environment around the existing structure and the new structure should be compared. If the environments are comparable or the new environment is less severe the same coefficient can be used. If the environment is less severe the residual life span is certainly not reduced, so using the same coefficient is conservative, but safe. However, if the environment is more severe, the coefficient might need to be increased with a certain factor to assure the residual life span. The comparison of the environment can be based on the durability classes, but also within durability classes differences may occur.

§3.6.6 Additional research

Due to uncertainties in input for structural assessment or based on the condition of the girders additional research might be required. With regard to the strength characteristics of the concrete, concrete cores can be drilled and tested. Samples may be gathered and investigated under the microscope to determine cement type and the occurrence of secondary formations as ASR or ettringite. Most research is non destructive or semi destructive. In many cases cores have to be drilled, but these holes can easily be repaired and do not affect the structural capacity of the girder. However, determining the residual stress in prestressing tendons with a non-destructive test is very complicated. Although, it is possible [140], it should only be done if absolutely necessary. It may also be possible to test deconstructive if sufficient girders are available.

§3.6.7 Reparations

Different methods exist to protect or repair damaged concrete. In this section a few examples are mentioned. An additional layer of mortar can be applied to provide an additional barrier against carbonation and chloride ingress. Moreover, the already present content of chloride and carbonation is reduced, because of diffusion to the additional cover. A quite similar approach is used with alkalization when the pH of the concrete is raised by applying an alkaline paste on the surface. With electrochemical chloride extraction a high voltage is used to remove chlorides from the concrete. With cathodic protection an impressed current is applied to protect the structure. In this case the structure should be permanently connected to a power supply, which is from a financial and environmental point of view not preferred. For other degradation mechanisms there are also possibilities with coatings.

§3.7 Road map environmental impact analysis

The incentive of reusing existing prefabricated concrete girders in new bridges is reducing the environmental impact and contributing to a more circular economy. To assess the effect of this innovation the environmental impact should be quantified in some way. So, an environmental impact assessment is performed on the structurally feasible alternative(s) with reuse and the alternative(s) with new girders. An overview of steps is given in Figure 3.23.

As already mentioned in §2.6.4 and §2.7 the environmental focus is often solely on the protection of the environment. Attention for other aspects as depletion of natural resources is lacking. CB'23 tried to tackle this problem with the Kernmeetmethode. Even though this method is not yet fully operation nor fully applicable for the circularity level of reuse, it is used as inspiration for this environmental impact analysis.

The Kernmeetmethode of Figure 2.21 includes six indicators. The most familiar indicator, the fourth one, aims at the protection of the environment. This effect is assessed by calculating the Environmental Cost Indicator (ECI). Since, the protection of the environment is relevant for all levels of circularity the ECI-value is also calculated in this environmental impact analysis. The approach for this calculation is discussed in the first section.

The first three indicators aim at the protection of material stock. This is a useful aspect for reuse, because an important aim of reuse is reducing the depletion of natural resources. These factors are used as basis for the analysis on material origin and use discussed in the second section. The fifth and sixth factor relate to the protection of existing value and are meant for higher levels of circularity only. Hence, these factors are not included in this design approach.

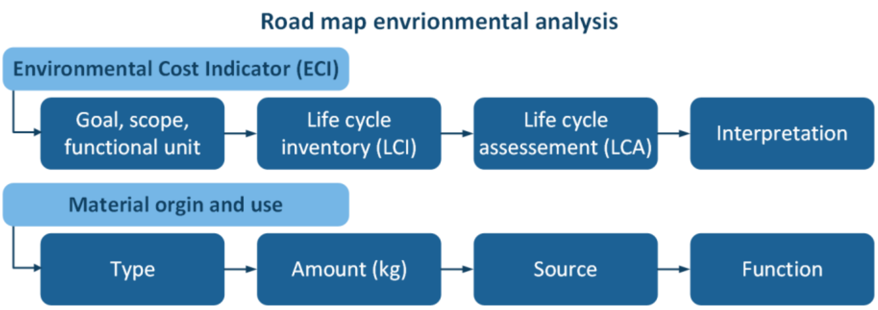


Figure 3.23: Road map environmental impact analysis.

§3.7.1 Environmental Cost Indicator (ECI)

The ECI-value is calculated by performing a Life Cycle Assessment (LCA) in four steps:

Step 1: Goal scope and functional unit
The aim of the ECI-value is comparing the alternative with reuse with each other and with an alternative with new girders. In this way the benefits of reuse can be identified. This ECI-value is used in the project team in the MCA to find the best suitable option. However, it can also be used to convince clients or stakeholders.

The basis of comparison, the functional unit, is a bridge deck that fulfils on the critical requirements from the system design and consists of prefabricated concrete girders, an in-situ deck and crossbeams at the supports.

All phases except the use phase (B1-B5) are included in this LCA. Thus, the production (A1-A3), construction process (A4-A5), end of life (C1-C4) and benefits and loads beyond system boundary (D). Maintenance in the use phase (B2) will only be included if additional maintenance is needed due to the modification to existing girders.

The LCA includes the 11 environmental impact categories that are mandatory in an ECI-calculation: abiotic depletion non fuel, abiotic depletion fuel, global warming, ozone layer depletion, photochemical oxidation, acidification, eutrophication, human toxicity, fresh water ecotoxicity, marine water ecotoxicity and terrestrial ecotoxicity.

In this analysis category three data from “Milieu database” of half products is used. Category three data not validated and brand independent. It is the least reliable type of data [141]. Nonetheless since this data is used in the calculation of all alternatives, still a reliable comparison is made. For elements (girders) that are reused the unintentional reuse factor H of 20% is applied as discussed in §2.7.2. It should be noted that the data used is based on more recent fabricated girders. Currently, the material input in girders originates from recycling and fabrication processes that are more sustainable compared to decades ago. Therefore, it is likely that the initial environmental impact of existing girders is higher than new girders. Nonetheless, during the construction of the existing girders no life cycle assessment was performed. Moreover, the reuse factor of 20% is also an indication and the data is an average.

Step 2: Life Cycle Inventory (LCI) analysis
An overview of the life cycle inventory analysis is provided in a process three in Figure 3.24. Phase A1, A2 and A3 relate to the manufacturing of the materials. On these stages the reuse factor is applied for the girders. A4 accounts for transport between manufacturing site and construction site. For reused girders this implies transport between location of existing structure, storage and new construction site. It is assumed that the environmental impact of this transport is comparable to transport in the traditional process. This is reasonable as storage locations will be searched for in the vicinity of the location of the existing structure or the location of the new structure. Together with the quite small country, transport distances for different projects will be comparable and will not be significantly larger than traditional distances. In the construction stage, A5, the elements are combined. For reuse and new elements this process is similar.

Most effects in the use phase are similar for all alternatives and hence there is no need to include them. However, if a certain alternative requires a special coating that has to be replaced every 10 years to be able to meet durability requirements or strengthening materials require maintenance these effects should be included in B2. In phase C1 and C2 the elements are again removed from the structure. On phase C3, C4 and D the reuse factor is again applied. These phases relate to the waste processing.

Apart from longer girders or more girders shortening in length or width direction is assumed to have no additional impact on the ECI-value. The reason for this is that also the concrete that is cut off, goes through all the life cycle stages. Only the use phase is negligible, but there are no influencing effects in this phase for the girders. After transport from original location to storage (A4) the girders are partly demolished (C1) and the concrete is transported (C2) to waste processing (C3, C4, D). So, all stages are relatively similar.

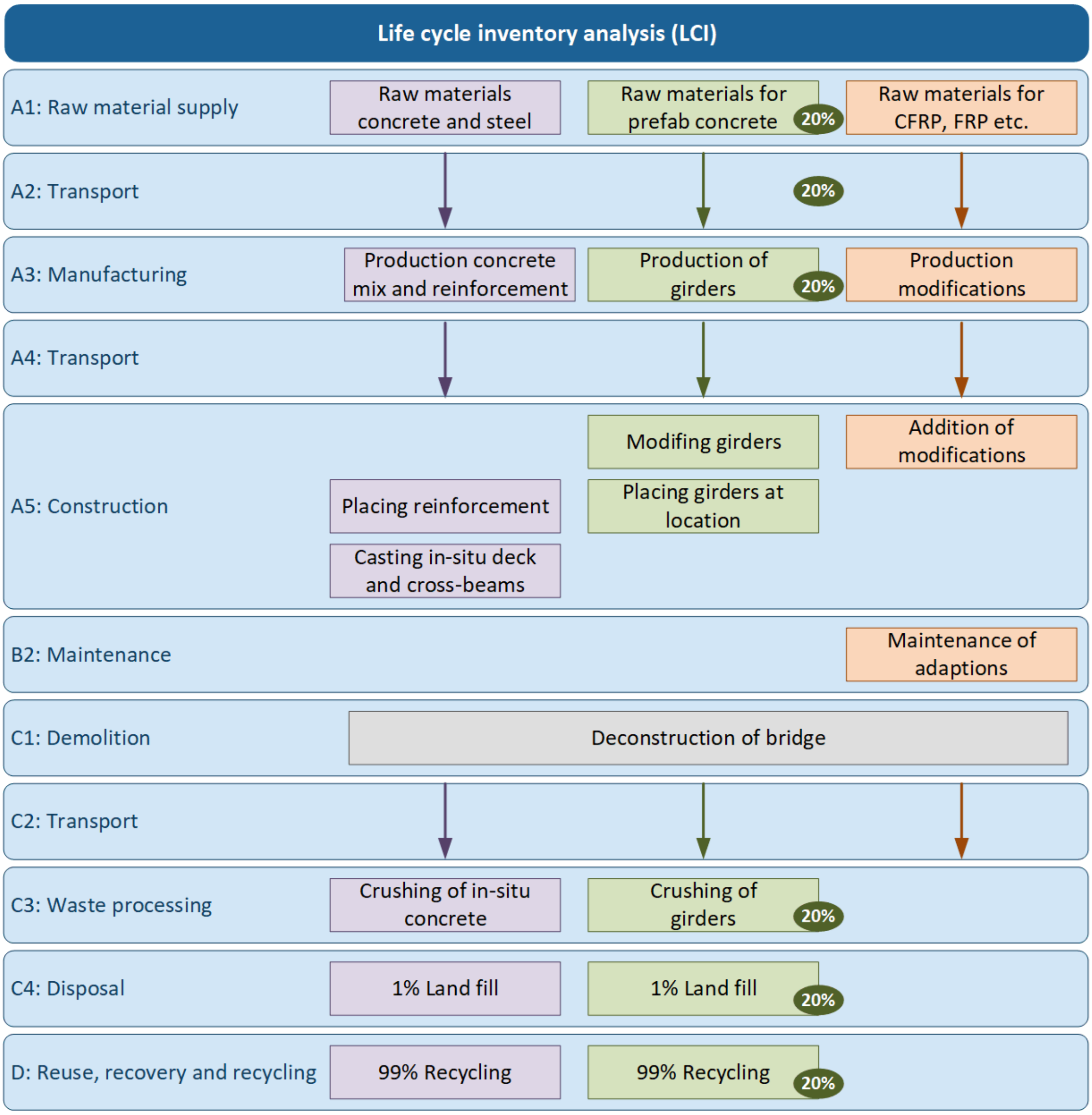


Figure 3.24: Life Cycle Inventory (LCI) analysis. In purple the processes relating to the in-situ deck, in green the processes relating to the prefabricated girder and in orange the processes relating to the adaptions to the girders. The 20% indicates the reuse factor that is applied on existing girders.

Step 3: Life cycle assessment
With an overview of the processes the ECI can be calculated, with Figure 3.25. On the left the equation for the alternatives with reused girders is shown and on the right the equation for alternatives with new girders. The approaches are similar only no reuse factor is applied with new girders and no modifications are needed. In the equation references are made to Appendix B: Data for Environmental impact analysis.

In this appendix the available data is converted. The data available on the in-situ concrete decks [142] is measured in a functional unit of 1000 x 1500 x 230 [mm] and is for this analysis converted to [m^3]. So, to calculate the ECI-value of the concrete deck the area of the bridge deck and height of deck is needed.

Analyses on prefabricated concrete bridge girders are available for a length of 25, 35 and 45 [m] [142]. These ECI-values are measured per [m] girder length and are linearly related. With interpolation the ECI-value per [m] length can be derived for other girder length as well. So, in the calculation the number of girders and length of girders is needed. For the ECI-values of strengthening materials no key figures are yet available.

In most projects a crossbeam is needed at the supports. In the Netherlands in infrastructural project in-situ concrete of concrete class C30/37, with CEM III is standard. It is assumed that around 265 [kg/m^3] reinforcement is present in a crossbeam. This is based on 250 [kg/m^3] for structural reasons and an additional 6% for practical reasons. So, the ECI-value per [m^3] crossbeam is calculated with the ECI-value per [m^3] C30/37 with CEM III [143] and the ECI-value per ton reinforcement grid [144]. This value should be multiplied with the volume of the crossbeams.

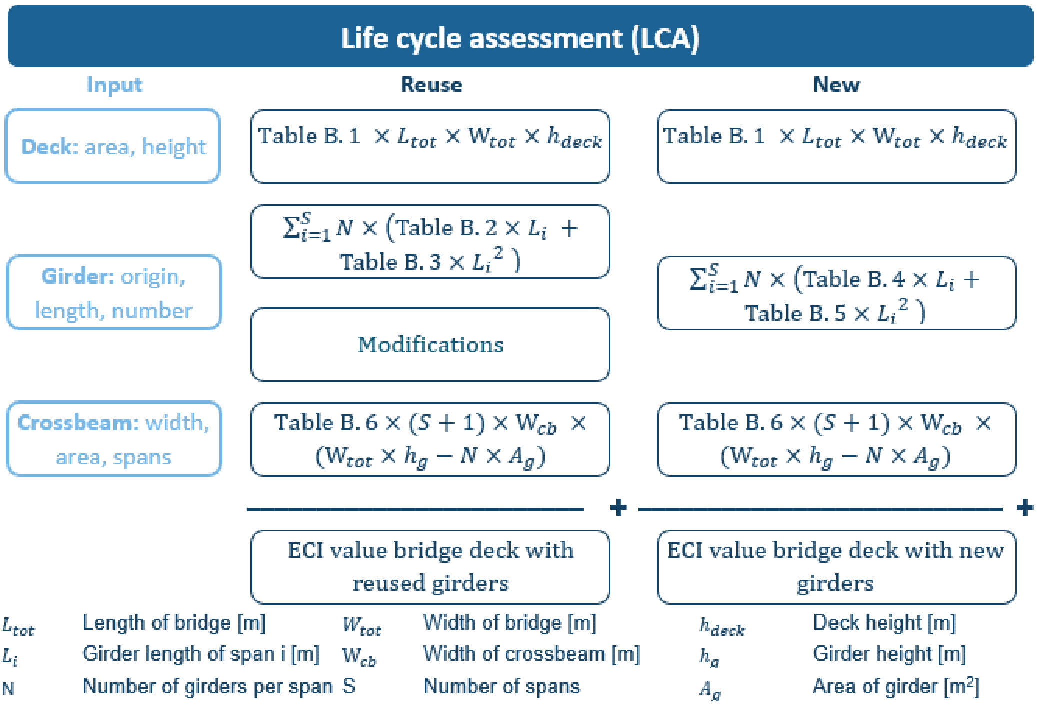


Figure 3.25: ECI-value of bridge deck.

Step 4: Interpretation of results

In the last step the results of the LCI have to be reviewed. The results from the alternatives will differ, does this difference can be explained? Another point of review is the effect of the bridge deck on the ECI-value of the complete structure. For example, reuse alternative A with 3 spans of 20 [m] and reuse alternative B with 2 spans of 30 [m]. The ECI-value of alternative B will be higher, due to a higher girder profile needed. However, this alternative requires only one intermediate support, while alternative A requires two.

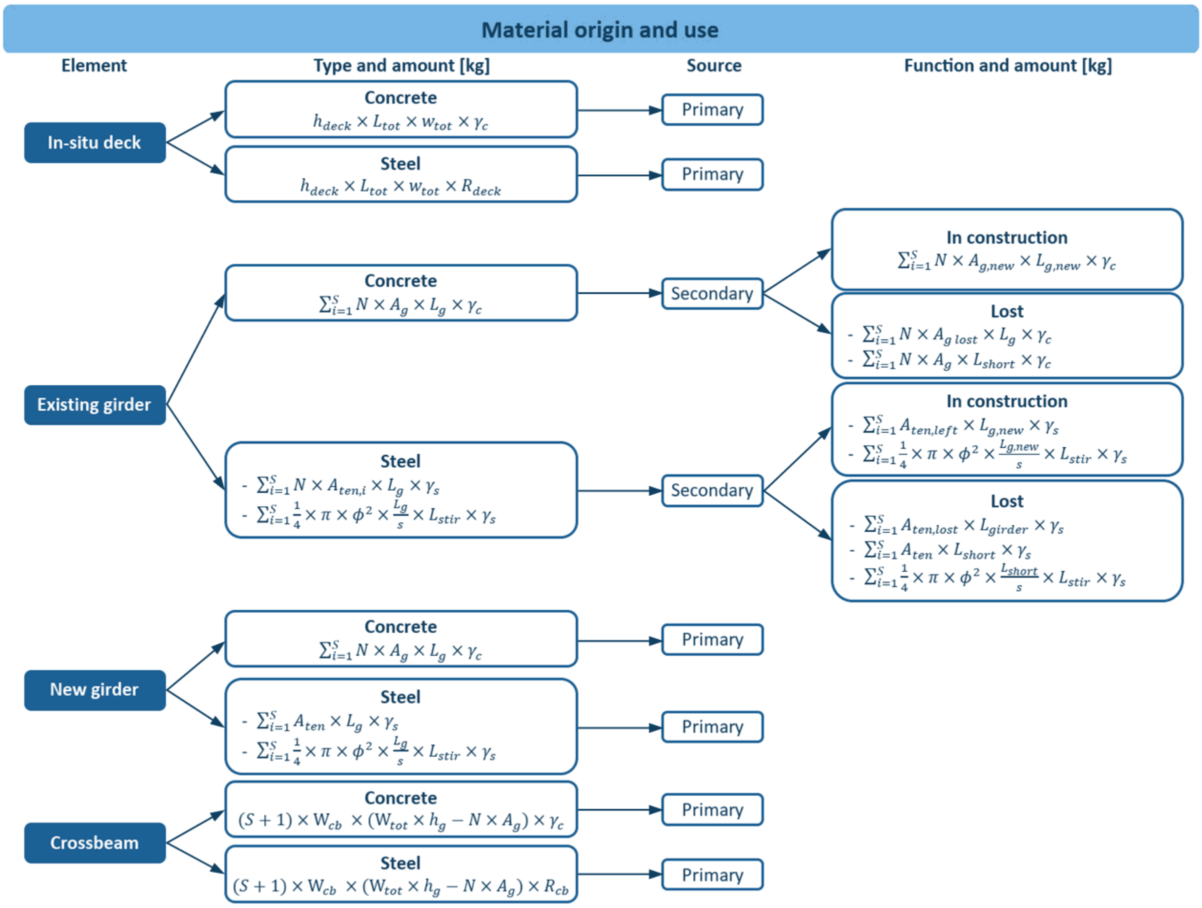
The influence of the bridge deck on the ECI-value of the complete structure, reduces because of reuse. So, the effect of an extra intermediate pier will be more significant compared to alternative with new girders. This effect is not quantified in this research, but it should be noted. Quantification is largely project dependent as it depends on for example soil conditions and aesthetic demands.

§3.7.2 Material origin and use

To assess the depletion of natural resources the material in- and output put and output is analysed. For each alternative the total mass [kg] of steel and concrete is calculated. To distinguish new and existing girders the source of the material is indicated. So primary or secondary. The secondary input is further divided into 'in construction' or 'lost due to modifications'. This is done to include the effects of shortening in width or length direction.

So, the in-situ deck consists of primary concrete and primary steel for the reinforcement. New girders also consist of primary concrete and primary steel for stirrups and prestress. Existing girders consist of secondary concrete which is partly used in construction and partly lost in modifications. Secondary prestressing steel and steel for stirrups is present, which is also partly used in construction and partly lost. For shortening in width direction it is assumed that no steel for stirrups is lost, because most steel is present in the web of the girder. The total procedure is shown in Figure 3.26.

This analysis is inspired on the 'Kernmeetmethode'. From the first indictor, input material the first sub-indicator, type, is used. The other two sub-indicators: physical scarcity and socio-economic scarcity relate to the raw material input, which is too specific for this more generic analysis. In the second and third indicator the output material is specified. These indicators are more applicable to higher levels of circularity. However, these indicators are more or less combined in specifying how much of the material from existing girders is used in the new structure and how much lost due to modifying the girders.



γ	Density [kg/m³]	L	Length [m]	A	Area [m²]	ϕ	Stirrup diameter [m]
c	Concrete = 2400 [kg/m³]	tot	Total bridge	g	Girder in span i	s	Spacing stirrups [m]
s	Seel =7850 [kg/m³]	g	Girder in span i	ten	Prestressing tendons in span i	w_{tot}	Width bridge [m]
R	Reinforcement in concrete [kg/m³]	stir	Stirrup in cross section	lost	lost due to shortening in width direction	h_{deck}	Deck height [m]
deck	Deck	short	Cut off	left	Left after shortening in width direciton	N	Number of girders per span
cb	Crossbeam = 265 [kg/m³]	new	After shortening			S	Number of spans

Figure 3.26: Steps to analyse material origin and use.

§3.8 Road map financial analysis

To compare design alternatives the costs of the structurally feasible alternative(s) with existing girders and the alternative(s) with new girders should be estimated. Developing a road map like the other analysis is not possible. The reason is that costs depend on many external factors, which are time and location dependent. For instance, fluctuating material prices or market situations. Currently, implementing existing girders in new structures is still in the development phase, which involves high costs. As discussed in §3.2.1 the costs will decrease in the growth phase. Furthermore, the development phase involves limit experiences and references. Consequently, there is a lot of uncertainty and no indicative values can be derived.

By addressing the attention points and discussing the main differences between a traditional process and a process with reuse guidance is provided for the financial analysis. To make sure that the cost estimation is complete, the entire execution process is walked through and expenditures are attached to all elements. Accordingly, the execution process is discussed in the first three sections. In the concluding section the aspects relating to financing are discussed.

§3.8.1 Deconstruction

For reusing existing girders the process starts with the deconstruction of the existing structure. From a financial perspective the main influencing factors are duration, the infrastructural network around the existing structure and the equipment needed. In §2.4.1 the process of deconstruction is discussed. In a traditional demolition process multiple machines are used, to demolish the structure as fast as possible. Although these machines are expensive, the crane needed to hoist entire girders from the structure is more expensive. Besides, a crane is more immobile compared to traditional demolishing machinery. Combined with the caution needed the process takes much longer. Consequently, the costs of machinery rise. This also holds for conventional sawing and cutting equipment since these need to be available for a longer period.

During deconstruction the infrastructure underneath the existing structure cannot be used. Depending on the importance of the infrastructure closures of roads or train tracks might only be possible in weekend and night sessions. In a traditional demolition process often a single closure is sufficient, however with the longer deconstruction process this might not be the case. As a result, machinery has to be available for a longer period. In addition, the machinery has to be moved to the right position and removed from the road at the start and end of each closure, which consumes extra time again. The same holds for the measures to redirect traffic. Besides, there might be a restriction to the number of road closures due to hindrance for users.

Due to the longer duration the location of the existing structure will be later available for construction, which might be a problem if a new structure is needed on that location. This relates to the question of when to remove the in-situ deck. If the existing structure is not in use anymore and there is no rush for deconstruction, the deck can be removed in advance. Under other conditions the deck has to be removed at the storage or modification location. So, the girders are extracted from the structure with the deck on top. In this way, especially with long girders, transportation and hoisting weight can become an issue. A crane with a larger capacity or exceptional transport might be needed, which rises the costs again.

§3.8.2 Transport, storage and modifications

Next the girders need to be transported to a storage or testing facility where they are made ready for application in a new structure. This phase replaces the traditional transport of debris and elements to waste processing facilities. Due to small size of elements the transport is arranged with dumpers, whereas girders should be transported on trailers. The transport is comparable to the delivery of new girders, for which often due to the large girder lengths special transport arrangements are required. For example, guiding vehicles are needed.

Ideally the storage location is at the new construction site. In this way only one transport is needed, which reduces the costs. Otherwise, the girders have to be (un)loaded and put on transport twice. Moreover, rent costs might be associated with storage. At the storage location the girders are fixed up and modified or strengthened whenever needed. These measures are discussed in §2.4.2, §2.4.3 and §2.4.4. Moreover, additional tests and inspection may be performed. Depending on the measures and tests needed this can be a considerable costs item. It might even be the case that extra girders are extracted to test destructively. In this case additional costs are made in the previous phases as well.

§3.8.3 Constructing new structure

Compared to a traditional design the previous steps replace the manufacturing phase. The costs for manufacturing are apart from the type and length of the girder primarily influenced by the prices of raw material, the demand and project size.

The next steps are comparable for the traditional process and the process with reuse. The new or existing girders should be transported to the project location. As already discussed, transport to the project location is not needed if the existing girders are stored at this location. At the construction site the girders are placed at the support. Next the reinforcement for the crossbeams and the deck is placed and concrete is casted. With the reuse of existing girders some additional formwork may be needed. Prefabricated girders are executed with a small notch at the top. At this notch formwork for the deck can easily be placed. Even if the existing girders are originally equipped with a notch this notch is most likely not present anymore. With removing the cast in-situ deck the concrete in the notch is not removed or the part of the girder around the notch is removed. Consequently, formwork has to be applied with an additional support. This is indicated in Figure 3.27.

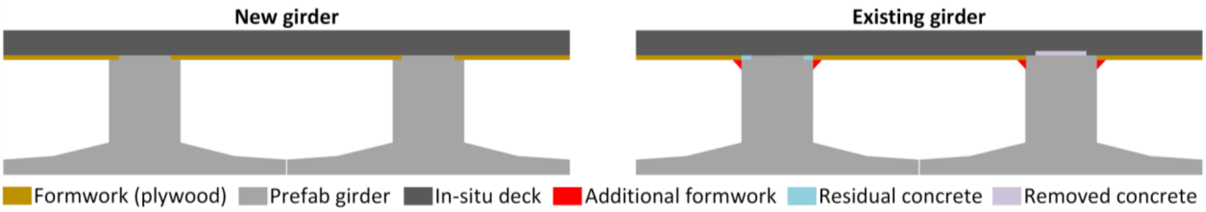


Figure 3.27: Additional formwork needed with reusing existing girders.

§3.8.4 Financing

A final aspect related to the financial analysis is financing and liability. These aspects are not of relevance only in the event that deconstruction of the existing structure and the construction of the new structure are projects issued by the same client and executed by the same contractor. So, in most cases a division in liability and financing between parties is needed. For example, it should be determined which part of the deconstruction belongs to the existing structure and which part belongs to the new structure. Contributing all costs to the deconstruction is not fair, as the deconstruction is more expensive than demolition. However, accounting all the costs to the new project is also not fair, because the existing structure should be removed anyway. The best solution is the middle way, whereby the traditional demolition costs are addressed to the existing structure and the additional costs for deconstruction to the new structure.

Though, this is easy to say, it is difficult to put into practice. First of all, how much would traditional demolition cost? The parties responsible for the existing structure and the ones responsible for the new structure should agree on these ‘fictional’ costs. However, as already said costs are uncertain and depend on many factors. This factor is further complicated by the competitive nature of the construction industry. So, the parties involved in the existing structure will estimate lower costs for traditional demolition and higher costs for deconstruction compared to the parties involved in the new structure.

The process is further complicated by the liability division. First, who extracts the girders? But also, who is responsible for damage that occurs? And who says that damage was not already there, before extracting the girders? Furthermore, who guarantees the residual life span? Nevertheless, these questions are outside scope of this research.

§3.9 Multi-criteria analysis alternatives

In the final step the best suitable design alternative is chosen by means of multi-criteria analyses (MCAs). All alternatives left are able to meet the critical requirements. Therefore, noncritical requirements are weighed against each other in two trade-off matrices (TOMs) or MCAs. First a decision is made on the best alternative with reuse. Of course, this decision is only needed if there are multiple feasible design alternatives with reuse. Next this design alternative is weighted against the design alternative(s) with new girders.

In the first section the steps of MCA's are explained. In the second and third section the assessment criteria are discussed. This is followed by the weights used to combine the scores to a final score. Finally, the decision is discussed.

§3.9.1 Set up of the MCA

An MCA can be divided in six steps [145].

Step 1: Establish the decision context
The aim of the MCAs is to compare bridge decks consisting of prefabricated concrete bridge girders and find the most suitable alternative for the project of concern. A small part of the project team has worked on the alternatives with existing girders. This group chooses the most suitable alternative with existing girders by performing an MCA. The second MCA in which existing and new girder are considered is performed more collectively. So, for the final decision a larger part of the team is involved.

Step 2: Identify the alternatives
During the girder search alternatives are developed. Some of these alternatives may not reach the MCA as they turn out the be structurally unfeasible. However, in some cases the alternatives that reach the MCA might be split up in sub-alternatives due to different modifications possible. It is also imaginable that during the MCA it turns out that investigating modifications or adaptions to a certain alternative might be beneficial. As a result, a potential new alternative is developed, which should be structurally verified and environmentally and financially analysed. In the final MCA the best suitable alternative with reuse of existing girders is compared with one or more alternatives with new girders.

To score the alternatives a basic description should be provided. In this description at least the girder type, height and width should be mentioned, as well as the span length/division. Besides, the source and state of girders and modifications needed is valuable information.

Step 3: Identify the criteria
According to the standard format of Van Hattum and Blankevoort the following criteria should be included: contract requirements; execution; planning; maintainability; interfaces, risks and opportunities; spatial quality and design; integral safety; costs; and sustainability [90]. Nonetheless, due to the application of a TOM to a specific element the criteria of spatial quality and design and integral safety are left out. All alternatives will score comparable on these criteria.

The seven main criteria left are further specified and divided into possible sub-criteria in the next two sections. Specification of the criteria ease the scoring procedure and prevents differences in interpretation. However, the interpretation of criteria is project dependent and should be discussed within the decision-making team. The number of sub-criteria should be kept minimal to keep the process manageable and prevent double counting. Moreover, it should be noted that the sub-criteria derived are potential criteria, which can be used. Which criteria are used in the analysis depend on the differences between alternatives and the project requirements.

A prerequisite for the criteria is operationality: each alternative can be assessed on each criterion [145]. This is the reason for two MCAs in the framework. Alternatives with existing girders can be assessed on different criteria than alternatives with new girders. Other conditions that should be considered in developing criteria are mutual independence and double counting.

Step 4: Score the alternatives on the criteria
A scoring scale of 1 to 4 is used for criteria that cannot be measured quantitatively. 1 is bad, 2 is reasonable, 3 is good and 4 is great. Scores on criteria that can be quantitatively measured, such as costs, are also converted to this scale to make summation to a final score possible. For a fair comparison the best alternative scores ‘great’, so 4 points. The least favourite alternative scores ‘bad’, so 1 point. The scoring of the other alternatives depends on the difference between the best and worst score and will be interpolated, which is explained with two examples in Table 3.5.

If only two alternatives are compared in the analysis it is better to use a scale of 1 or 2. The most favourable alternative gets 2 points on that criterion and the other 1.

In case the differences between alternatives are small, it might seem unfair to use a scale of 1 to 4, because the alternatives are comparably ‘great’ or ‘bad’. Nonetheless using a different scale would make the scoring procedure more subjective and complicated, because for each criteria a suitable range should be chosen. Therefore, in case a criterion is not distinctive, it is excluded from the analysis. In other cases, it is choses to use the same scale on all criteria but the effect is not neglected. When the scores of all alternatives on a certain criterion are close to each other, this criterion is off less relevance compared to other criteria. As a result, the weight of the criteria is low. This is discussed in §3.9.4. Nonetheless, to be able to derive the weight of the criteria it is important to keep the ‘real’ value in mind. Therefore, besides the score also the ‘real’ value or some remarks should be added in the analysis.

Table 3.5: Calculating the score belonging to quantitative criteria.

Example 1				**Example 2**			
Alternative	Calculation		Score	Alternative	Calculation		Score
1. €50	0	$4-0$	4	1. €50	0	$4-0$	4
2. €75	$\frac{\text{€}75-\text{€}50}{\text{€}33{,}33}=0{,}75\rightarrow 1$	$4-1$	3	2. €75	$\frac{\text{€}75-\text{€}50}{\text{€}116{,}67}=0{,}21\rightarrow 0$	$4-0$	4
3. €120	$\frac{\text{€}120-\text{€}50}{\text{€}33{,}33}=2{,}1\rightarrow 2$	$4-2$	2	3. €120	$\frac{\text{€}120-\text{€}50}{\text{€}116{,}67}=0{,}60\rightarrow 1$	$4-1$	3
4. €150	3	$4-3$	1	4. €400	3	$4-3$	1
Max difference:	$\text{€}150-\text{€}50=\text{€}100$			*Max difference:*	$\text{€}400-\text{€}50=\text{€}350$		
Costs per point:	$\text{€}100/3=\text{€}33{,}33$			*Costs per point:*	$\text{€}350/3=\text{€}116{,}67$		

Step 5: Assign weights to criteria and combine the scores to a final score
First an average score on the main criteria is derived by summing the scores on the sub-criteria. The final score is found by adding the averages scores on the main criteria together. In both summations weight factors can be applied on the sub/main criteria. Although these weights are largely project dependent, they are discussed in the fourth section.

Step 6: Examine the results and choose the most suitable alternative
In the end the project team should decide on the most suitable alternative for the project. This is discussed in the last section.

§3.9.2 Criteria for alternatives with reusing existing girders
This section focuses on the criteria for the MCA between alternatives with existing girders. The seven main criteria are placed in the context and possible sub-criteria are derived. A summary is given in Table 3.6.

Contract requirements
Contract requirements are related to the requirements and wishes of the client and stakeholders. All alternatives fulfil on the critical requirements, but some alternatives may fulfil these requirements better or are able to fulfil additional noncritical requirements. The main variables are profile height and span division.

An example of a project that requires a bridge of 100 [m]. There is space available for a bridge of maximum 120 [m]. Alternative A perfectly matches with the requirement of 100 [m] and scores 4 points. Alternative B with a span of 120 [m] scores 1 point. Alternative C has a span of 105 [m] and scores 3 points. The span division may also relate to wishes for symmetry or other aesthetic aspects.

Execution
Execution criteria are related to the ease of construction. Possible sub-criteria are support conditions and modifications. Support conditions relate to the system needed (statically determinate or indeterminate) and the crossbeam requirements. For instance, an alternative that requires a statically determinate connection and does not require a crossbeam score great, a 4. While an alternative with a statically indeterminate connection and an extra wide crossbeam scores only 1 point.

The modification criterion is related to shortening and strengthening. Shortening in length or width directions has less impact, compared to strengthening. But an alternative without shortening or strengthening is of course preferred and scores best.

Planning
For a bridge deck in which existing girders are reused the required research and fix up are the main influencing factors on the planning. Girders that are released close to the start of the execution and need additional inspections or material test get a lower score compared to girders that are already released and inspected. So, the release date influences the planning as well, but this aspect is directly related to the research and therefore no separate sub-criteria is needed. Moreover, the release date is already included in the search criteria, so all alternatives are feasible regarding to planning.

Attention should be paid to possible direct relation with criteria in interfaces, risks and opportunities. If research is required, it is likely that assumptions are used for the characteristics. As a result, the structural feasibility of an alternative might be uncertain. If this is the case, it is better to only use the criteria of uncertainty and not apply the criteria of planning as it would result in double counting. In this way the criterion of planning is only relevant if research and fix ups are only time related. There should be no doubts about the structural feasibility or state of the girder, but for example research is needed for certification.

Maintainability
For reuse of girders maintainability is more related to the residual life span of the girders and the assurance of this life span. The age of the girders is not a suitable criterion, because it is about the state of the girders and the ability to meet the requirements. Another sub-criterion is the maintenance that arise due to modification or adaptations to the girder.

Interfaces, risks and opportunities
The origin of the girder can be a useful sub-criterion. Girders that are released close to the new project location are preferred over girders that have to be transported over a large distance. In addition, girders that are released from a project within the organisation are preferred over girders that originate from a governmental authority. Girders released by an organisation that has no collaboration history with Van Hattum and Blankevoort will score even lower. This sub-criterion should not be used to compare the deconstruction methods of the existing viaducts, as this is directly related to the costs.

Another sub-criterion is the reliability and feasibility. Of course, the structural feasibility is already verified, however there is still some uncertainty as the structural assessment is most likely based on some assumptions. Therefore, there is a risk that the alternative appears unfeasible in a later stage. This risk depends on the number of assumptions and the influence of these assumptions. It also depends on the robustness. If there is margin in the capacity or modifications or adaptations are possible that increase the capacity the risk is lower.

Costs
The result from the financial analysis is also a valuable criterion. Apart from the costs of acquisition and execution, the costs related to research and storage should be included. It is not useful to split the costs in different sub-criteria, because it is about the amount of money spent and it does not matter on what aspects the money is spent.

In financial analysis only the costs of the bridge deck are estimated. However, a viaduct or bridge also needs foundations, intermediate piers and abutments. If alternative A only has three spans, while alternative B has four, the costs for intermediate piers will significantly differ. This aspect should then be included in the MCA with a sub-criterion of impact on the structure. So even if the costs estimation in the financial analysis show that the costs for both alternatives are roughly the same, from a financial aspect alternative A is preferred.

Sustainability
The results form the environmental analysis are used as input for this criterion. The ECI-value and the material input can be used as sub-criteria. Another sub-criterion is the global circularity. This criterion relates to the question if the girders are suitable in a ‘bigger’ project. So, are the girders only suitable for this or a comparable project? Or results reuse of these girders in a bridge-deck with overcapacity in relation to its function and other structural elements? In this way there might be potential to use the girders in more complex or bigger project for which other girders are not suitable. A second question is how likely it is that such a project will occur on short notice?

In most cases the ECI-value and material input already include global circularity effects. Because if an alternative has over-capacity and can also be used in larger projects the ECI-value as well as the material input will be higher. Nonetheless, the difference might not be significant. Besides, it might be the case that there are two alternatives A and B. Alternative A has a higher environmental impact compared to B due to strengthening or shortening of the girders. However, without strengthening or shortening the girders of alternative A cannot be used in any project. The girders from alternative B are suitable for many other projects. To avoid that alternative B gets a better score on environmental impact the sub-criterion of global circularity is added to the MCA.

By including the sub-criteria of global circularity, it is not needed to include the criteria of impact on the structure, just like with costs. In general, only alternatives with a low global circularity will have an adverse impact on the environmental impact on the other elements of the structure. The reason is that additional environmental impact on other elements in the structure is only possible in case of heavy girders or many intermediate piers. In this way the girders will be better suitable for another project and will score low on this criterion.

§3.9.3 Criteria for both alternatives with reusing existing girders and new girders

This section focuses on the criteria in the MCA between an alternative with existing girders and alternatives with new girders. So, the same main criteria are placed in context and possible sub-criteria are derived. Some sub-criteria are similar, however due to different context the score can be different. A summary is given in Table 3.6. It should be noted that the interpretation of criteria and division in sub-criteria for this MCA are much more project related.

Contract requirements
The sub-criteria relating to contract requirements are similar to the ones mentioned in the previous section. It is possible that the client prefers reuse or new, but this should not be included in the sub-criteria, but in the weight of the criteria.

Execution
The criterion of execution is left out of this MCA, because a standard execution method is followed for decks consisting of inverted T-girders with an in-situ deck. As already discussed, the execution reusing existing girders start earlier, because girders have to be extracted from an existing structure. However, these aspects are already included in criteria of planning and maintainability. Furthermore, due to operationality sub-criteria cannot be related to the modifications or adaptations. However, if in the future the design approach is extended and different type of prefabricated concrete decks are considered (inverted T, box girder) changes in execution method are possible.

Planning
For planning the main differences between reuse and new are related to the duration of design and detailed calculation and the duration of execution. This criterion can be further divided in sub-criteria but can also be assessed in general. However, this is project dependent.

Maintainability
Again, the life span is used as sub-criteria. In the previous MCA it was about the assurance of the life span. In this MCA the life span itself is considered. The new alternative has a certain design life span, the alternative with reuse can exactly meet this life span or has a slightly lower or higher residual life span. Moreover, due to uncertainty in the life span additional measures or inspection might be needed.

Interfaces, risks and opportunities
The uncertainty in material delivery is an important sub-criterion. For the alternative with reuse this relates to the release date and certainty of assumed quality. For the alternative with new girders it relates to the deliver possibilities and time. The opportunity for a ‘green image’ is also mentioned as sub-criteria. This might seem not operational, but also in an alternative with new girders attention can be paid to sustainable innovations.

Costs
For the costs the results from the financial analysis are used. In addition cashflow, the timing of costs, can be used as sub-criteria. The reason for this criterion, is that it is known that generally the expenses have to be made earlier for reuse compared to new. Similar to the previous MCA the impact on the structure can be included.

Sustainability
Again, the results from the environmental analysis are used. Compared to the previous MCA, ECI-value and material input and origin are combined in a single sub-criterion. The alternative in which existing girders are reused will score best on both criteria compared to alternatives with new girders. Due to this direct relation, it is not needed to include both sub-criteria. However, both the material input and the ECI-value should be mentioned in the remarks because it influences the weight factor of the criterion. The criterion of global circularity is left out. If the alternative with reuse has a low global circularity it will result in small differences in environmental impact or costs compared to the alternative with new girders. As a result, using new girders will be more attractive and the existing girders can be used in a more appropriate project. As a replacement the impact on the structure can be used as sub-criterion. With this sub-criterion for example the impact of additional intermediate piers is included. If an alternative with reuse uses larger spans and fewer intermediate piers are needed, this is beneficial.

Table 3.6: Main and sub-criteria for MCA analyses.

	MCA existing girders	MCA existing and new girders
Contract requirements	Profile height Span division	Profile height Span division
Execution	Support conditions Modifications	-
Planning	Research and fix-up	
Maintainability	Life span Modifications	Life span Control measures
Interface, risks and opportunities	Origin Reliability and feasibility	Material availability and delivery Opportunity for circularity
Costs	Cost indication Impact on structure	Cost indication Cash flow Impact on structure
Sustainability	ECI-value Material input Global circularity	ECI-value + Material input Impact on structure

§3.9.4 Weighing factors
Weight factors reflect the importance of the criteria, which depends on the differences between the alternatives, the client requirements and market conditions. If a criterion is divided in multiple sub-criteria first the weight of the sub-criteria is derived. Below some considerations are provided as guidance:

- Contract requirements: the weight of span division can be double compared to the profile height if the client has a strong wish for symmetry and not all alternatives have a symmetric span division, while the difference in profile heights is small.

- Execution: modifications might be more important than support conditions. Nonetheless, if alternatives need comparable modifications, the weight of this criterion should be low.

- Interface, risks and opportunities: in the current situation, where reusing existing girders is still in the innovation phase, reliability and feasibility are more important than the origin. Due to limit experiences the risks are already high, so the most reliable alternative is preferred. Parties involved in innovation projects want to gain information and experience. As a result, the communication chain can be small and involved parties are likely to be cooperative. Consequently, the origin is not so important. However, in the future with more experiences and knowledge the risks on reliability and feasibility will decrease. Moreover, due to an increased competitiveness in this business the origin becomes more important.

- Costs: the weight of the sub-criteria mostly depends on the differences between alternatives. If the difference in costs is high, the sub-criteria is more important and vice versa.

- Sustainability: in the current situation the importance of global circularity is low. When existing girders are reused in a new structure, this can already be considered as a mile stone. Nonetheless, in the future when more girders are available and reusing existing girders is frequently implemented the importance of global circularity increases. The weight of ECI-value might be increased if client provides a fictional discount on a low ECI-value. Or the weight of material input can be increased if the client aims at a certain percentage of reused materials.

Next the score for each main criterion is calculated. This is the average score calculated by dividing the total weighted score by the total weight. An example of the principle is shown in Table 3.7.

Table 3.7: Calculating the score for a main criterion.

	Weight	Alternative 1		Alternative 2		Alternative 3	
		Score	Weighted score	Score	Weighted score	Score	Weighted score
Sub-criteria 1	1	1	1	4	4	3	3
Sub-criteria 2	2	3	6	2	4	4	8
Sub-criteria 3	1	4	4	1	1	2	2
Main criteria	4	**2,75**	11	**2,25**	9	**3,25**	13

Subsequently the weights for the main criteria should be chosen. To provide guidance again some considerations are discussed. Especially, in the comparison between reuse and new the weights mainly depend on the project characteristics and client requirements, which depend on the developments in the construction industry as well. For example, before the introduction of TOMS costs used to be the decisive criteria [146]. Nowadays the focus is slowly moving to sustainability as clients are willing to pay extra for a more sustainable alternative. Consequently, the weight of this criterion should be increased. However, if a client is more focussed on costs, the weight factor on costs should be increased.

Again, the differences between alternative also plays a role in assigning the weights. If the difference in expected costs for the alternatives is small, the weight of this criteria should be small as well. In the alternatives with reuse risks and planning will probably get a higher weight. In the future when reusing girders is frequently applied and more experience is gained the focus will shift more to sustainability. So, again for this MCA the weights may depend on the developments in the construction industry.

Finally, the final score is calculated in a similar way as shown in Table 3.7. Only for the total score there is no need to calculate the average.

§3.9.5 Decision

Due to the numbers, it is easy to forget that the scores are subjective and so are the weight factors. Therefore, the final score should not directly follow in a decision on the most suitable alternative. Instead, the project teams have to digest the results and may adapt the scores, weighing factors or criteria. So, an MCA is kind of iterative process. In a sensitivity analysis the influence of ambiguity or disagreements between people on the final score is investigated [145].

4. Case study: a bridge in Arnhem Meinerswijk

Parallel to the development of a design approach a case study is performed. The objective of this case study is to verify, adapt and expanded the framework to a design approach that makes implementation of reused girders more common practice. Therefore, the build up of the framework is followed. However, to explore possibilities and improve the framework sometimes other steps are performed or a different order is followed. In the first paragraph the system design is discussed by introducing the project and the project requirements. In the next paragraph the girder search and design alternatives are described. Then the preliminary design phase is started by working out the design alternatives. This phase is ended by MCA in paragraph eight. Finally, the case study is reviewed.

§4.1 Basis for design

The case study is about a to be build bridge at Meinerswijk in Arnhem. The total span length of this bridge is 107 [m], which is divided over 5 spans. The width of the bridge is 6 [m] of which 5 [m] is used by traffic. The bridge provides access to a residential area located at a peninsula. The structure belongs to consequence class 2 and has a required life span is 50 years.

The main permanent loads are the weight of the girders, weight of the deck and weight of asphalt layers. The weight of the girders and deck is based on the reinforced concrete density of 24 [kN/m^3]. For the weight of asphalt layers 3,22 [kN/m^2] is used.

The main variable load is traffic. For this load only the first load model of NEN-EN 1991-2 is used. The values and sizes are shown in Figure 4.1.

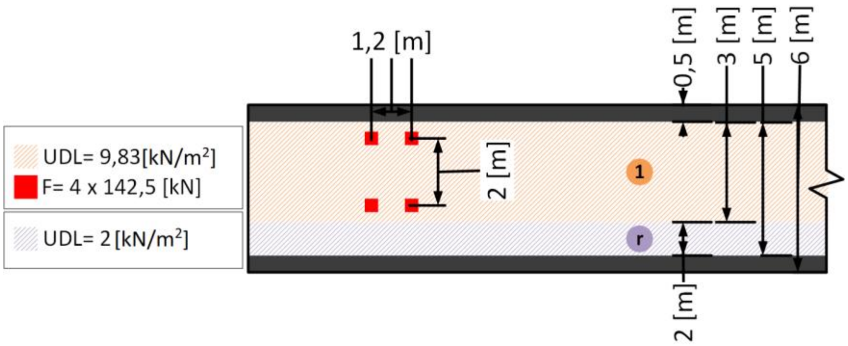


Figure 4.1: Traffic load based on load model 1 of NEN-EN 1991-2.

For these loads the partial safety factors from Table 4.1 are used.

Table 4.1: Partial safety factors for main loads.

Load	Favourable	SLS	ULS
Prestress	0,9	1,0	1,0
Permanent	0,9	1,0	1,2
Traffic	0	1,0	1,35

All details about the location and loads can be found Appendix C: Basis for design.

§4.2 Girder search and alternatives

The next step is girder search. In the first section available girders are matched with search criteria. Next design alternatives are generated. In the final two sections the characteristics of the girders are discussed.

§4.2.1 Search criteria

Before the start of this case study three search criteria are derived: girder type, minimum length and maximum length. For the last two criteria first the span division should be determined. The total span length is 107,21 [m]. The minimum number of spans is calculated by dividing the span length by the maximum girder length and rounding this value up. So, the minimum number of spans is $\frac{107}{30} = 3{,}6 \rightarrow 4$. The maximum number of spans is calculated in a similar manner, but by dividing the span length by the minimum girder length and rounding the value down. So, the maximum number of spans is $\frac{107}{20} = 5{,}35 \rightarrow 5$. There is no need to opt in the direction of minimum or maximum number of spans, because there are no profile height limitations for vertical clearance, no heavy foundations are expected and transport is not a critical factor. In the most optimal case girders can be transported over water, but also over land there are sufficient possibilities. Therefore, four and five span possibilities are considered in Table 4.2.

Table 4.2: Possible span division.

Option	Span 1 [m]	Span 2 [m]	Span 3 [m]	Span 4 [m]	Span 5 [m]
1a	12,5 -18,0	23,7 - 27,4	23,7 - 27,4	23,7 - 27,4	12,5 - 18
1b	12,5 - 18,0	24,6 - 30,0	22,0	24,6 - 30,0	12,5 – 18
1c	12,5 - 18,0	20,6 - 26,1	30,0	20,6 - 26,1	12,5 – 18
1d	21,4	21,4	21,4	21,4	21,4
2	22,0 – 30,0	22,0 – 30,0	22,0 – 30,0	22,0 – 30,0	

The minimum span length for each option can be calculated by subtracting 0,5 [m]. For the maximum length no specific rule is yet derived. However, an increase of 20% is used as first assumption [22]. The last search criterion is type of girder. Since there are no profile height limitations or specific requirements no girder type can be exempted from the search.

It can be concluded that due to the flexible requirements almost every available girder is potentially suitable. Regarding the current market situation with limited girder supply, this is favourable. However, in the future with more supply, it results in many design alternatives and a waste of time in case these alternatives turn out to be unfeasible. Therefore, more search criteria are appreciated.

§4.2.2 Available girders

Between 2020 and 2027 Rijkswaterstaat is widening the highway A9 between junction Badhoevedorp and Holendrecht [147]. In this project 11 viaducts consisting of prefabricated girders are (if possible) selectively demolished. In this way HNP and HIP girders built between 1968 and 1973 by Spanbeton are released for reuse [35]. Most likely RVB 1967 and VOSB 1963 are used during this period. Since the A9 is a national highway most likely traffic class 60 is used. With these potentially available girders for each span division from Table 4.2 a design alternative is developed. These are presented in Table 4.3. Information about the origin of the girders is provided in Appendix D: Existing viaducts.

Alternative 1a makes use of HIP800 girders of 18,53 [m] in the side spans and 22,62 [m] in the middle spans. These girders date from 1973. The total length is 104,92 [m]. This leaves 2,08 [m] short. However, extra space at the supports or a shortening of the design span is possible. In the original situation the girders are used in a statically indeterminate system.

In alternative 1b HNP750 girders of 23 [m] are used in span 1,2, 4 and 5 and a 16 [m] long girder is used in the middle span. These girders originate from two viaducts constructed in 1968. The total length equals 108,05 [m]. So, the middle girder has to be shortened or the span length has to be increased. In the original situation the girders are used in a statically indeterminate system.

In alternative 1c girders from two different viaducts are used. Spans 1 and 5 consist of HIP 800 girders, from 1969 with a length of 19,5 [m]. In span 2 and 4 the same type of girder is used, but with a length of 21,5 [m]. The middle span is realised with 28 [m] long HIP 1100 girders from 1972. Again, the total length is 1 [m] too long. So, one or some girder spans have to be shortened or the span length has to be increased. In Figure 4.2 two possibilities to combine girders with different heights are shown. The first option is to level the heights by varying the height of the in-situ deck layer. In this case the in-situ deck layer on the HIP 800 girder has to be increased with 300 [mm]. This results in a significant increase in self-weight. Another option is to level the height at the supports. This results in a complex situation at the supports. Therefore, this alternative is identified as less suitable than the others and is not further investigated in this case study.

This consideration can be included in the girder search criteria by adding a height criterion. Based on a length over height ratio a preferred profile height can be chosen. If no girders of this height are available the search can be broadened to other profile heights and more complex alternatives.

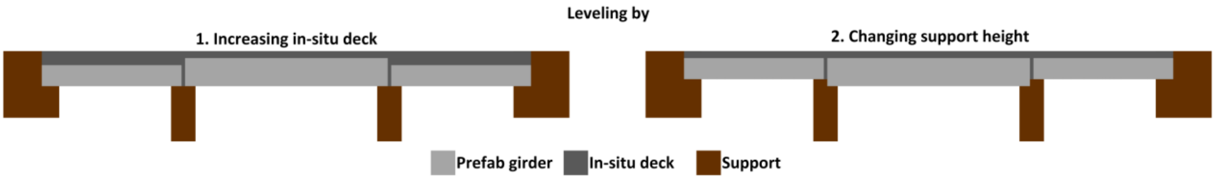


Figure 4.2: Options to deal with different profile heights.

Alternative 1d consists of 4 spans of HIP800 girders with a length of 21,26 [m] and originating from 1969. The total length equals 106,3 [m], which is almost equal to the required span length. The small difference should not be a problem. In the original situation the girders are used in a statically indeterminate system.

Alternative 2a consists of 4 spans of HIP1100 girders with a length of 28 [m] and originating 1972. To match with the required span length all girders should be shortened by 1,25 [m]. However, if the capacity allows it, it is more economic to only shorten one or two girders with 4,79 [m] or 2,4 [m] respectively. In the original situation the girders are used in a statical determinate system. From a global circularity concept this alternative is less attractive. Since, it is likely that these girders can easily be used in a new highway viaduct. Moreover, the girders are higher, which is less attractive for this situation regarding the space available to bridge the height differences. In addition, apart from the dimension no information is available on these girders. As a result, this alternative is not further worked out in this case study.

Table 4.3: Design alternatives with available girders.

Alternative	Girder	Span 1 [m]	Span 2 [m]	Span 3 [m]	Span 4 [m]	Span 5 [m]
1a	HIP 800	18,53	22,62	22,62	22,62	18,53
1b	HNP750	23,00	23,00	16,05	23,00	23,00
1c	HIP 800 HIP 1100	19,50	21,50	28,00	21,50	19,50
1d	HIP 800	21,26	21,26	21,26	21,26	21,26
2a	HIP 1100	28,00	28,00	28,00	28,00	28,00

§4.2.3 HIP-girder

The cross-section of a HIP girder is shown in Figure 4.3. the number of ribs on the side depends on the height of the girder. Spanbeton used three mould sizes to cast the girders: 800 [mm], 1100 [mm] and 1400 [mm] [137]. So, for a 500 [mm] heigh girder the 800 [mm] mould was filled just below the side ribs. For a 600 [mm] heigh girder the mould was filled up until the first rib etc. So, the 800 [mm] size girder has four side ribs. To model and perform calculations the shape of the girder is simplified and the side ribs are considered by reducing the width of the web over the height where ribs are present. The model is shown in Figure 4.4 and Figure 4.5. The standard height for the in situ-deck is 160 [mm] for all girder sizes [35]. Concrete class C55/67 is assumed for the girder and C30/37 will be used for the new deck. Originally K600 is used for the girder and K300 for the deck. The original elastic moduli are 39 [Mpa] and 29 [Mpa] respectively.

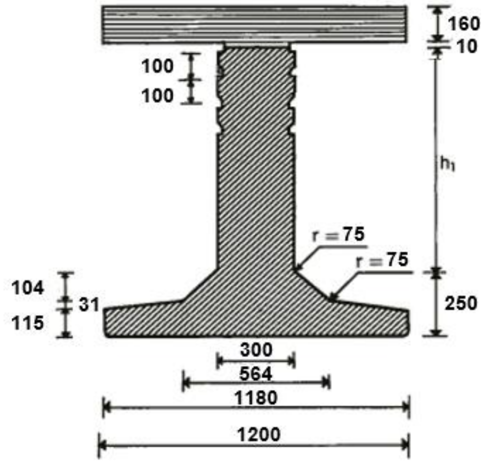


Figure 4.3: Cross-section HIP girder from Span Beton [11], adapted. Sizes in [mm].

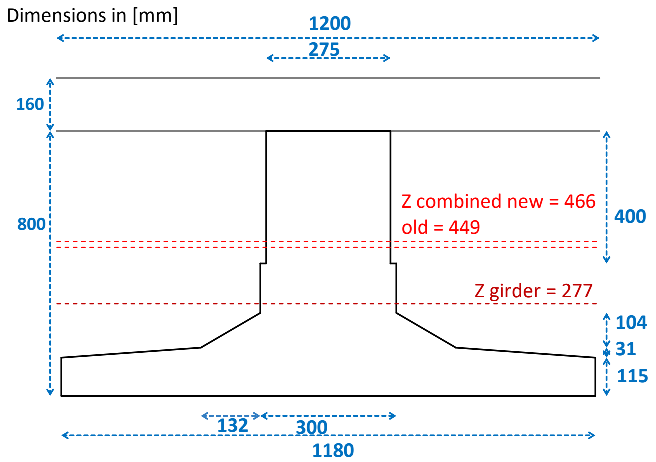


Figure 4.4: Model of cross-section of HIP-800 girder.

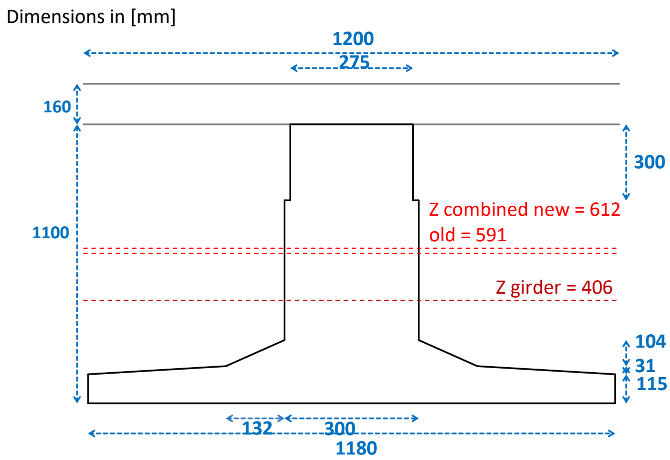


Figure 4.5: Model of cross-section of HIP-1100 girder.

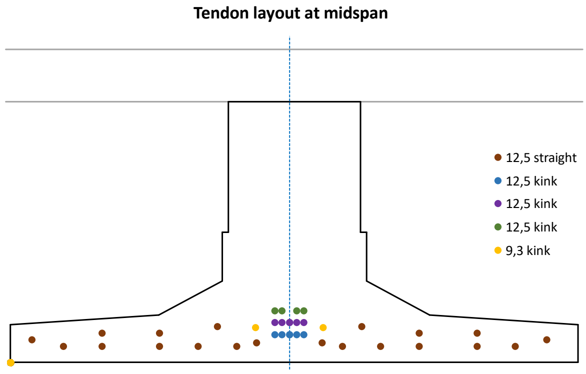
Prestressing

Spanbeton used QP190 prestressing strands with a diameter of 12,5 [mm] and an area of 94 [mm^2]. A general tendon layout is used in which the number of tendons could be varied depending on the prestressing force needed and the height of the girder. The tendon layout is shown by a cross-section at the support and at midspan in subfigures a and b of Figure 4.6 to Figure 4.8. In subfigures c the tendons are replaced by one equivalent tendon.

An initial stress of 1212 [N/mm^2] in the prestressing strands is assumed. This is 65% of the tensile strength, which is the maximum stress allowed according to the standard of RVB 1962/1967. This stress is present after the direct prestress losses such as friction have occurred. Theses girders are approximately 50 years old. Therefore 99% of shrinkage losses, 98% of creep losses and 93% of relaxation losses have occurred (see §2.3.6). Following RVB 1962/1967 the maximum strain due to shrinkage is 25×10^{-5} for a structure in open air. The strain due to creep depends on the ratio between the strength of the concrete at the moment of application of the prestress and the compressive stress after applying the prestress. Assuming a maximum ratio of 1, the maximum strain due to creep is 143×10^{-5}. This total strain is multiplied with the elastic modulus of prestressing steel to calculate the stress loss. For the elastic modulus 195 [Gpa] is assumed, which results in a stress loss of 0,3 [%] of the initial stress. Relaxation results in 14% stress loss according to RVB 1962/1967.

Considering these losses the currently present maximum stress in the prestressing steel is 1051 [N/mm^2]. This stress should be used in calculations where prestressing has a negative influence. For calculations where prestressing works favourable the minimum stress should be used. Considering that all losses have occurred the minimum stress in the prestressing steel is 1039 [N/mm^2], however 1025 [N/mm^2] is used since this is the maximum allowed according to RVB 1962/1967 [46]. Based on this stress the prestressing loads shown in subfigure d of Figure 4.6 to Figure 4.8 are calculated.

Prestressing layout of HIP 800 girder of 22,62 [m] length



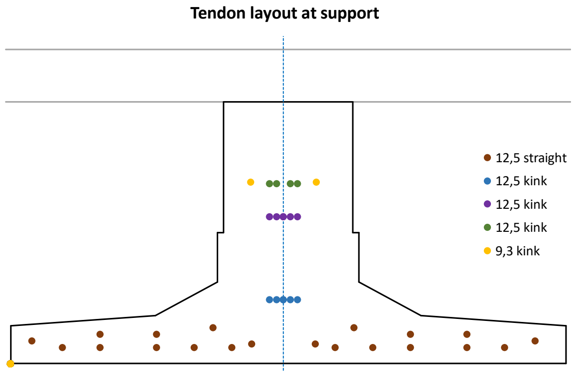


Figure 4.6 a) Tendon layout at midspan *Figure 4.6 b) Tendon layout at support*

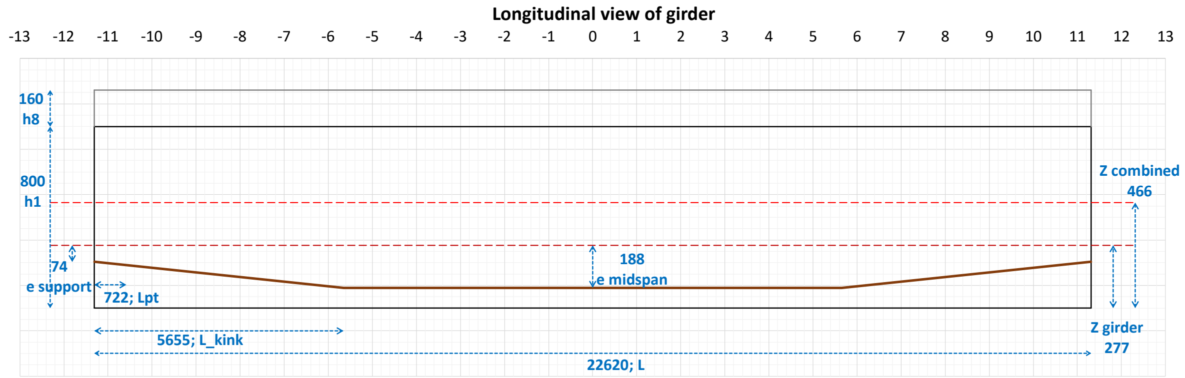


Figure 4.6 c) Longitudinal view of simplified prestressing layout, all strands are replaced by one equivalent tendon.

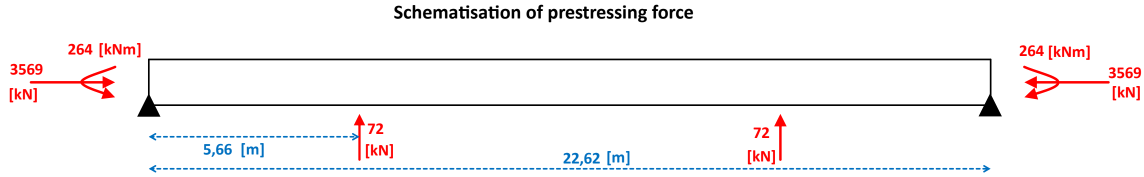


Figure 4.6 d) Prestressing forces acting on girder.
Figure 4.6: Prestressing layout of existing HIP 800 girder of 22,62 [m] length.

In sub-figure c the length over which the prestressing force is introduced is shown as L_{pt}. This transmission length is calculated with Equation 4.1. For the stress at tendon release 71,5% of the ultimate tensile strength of the prestressing steel is used [46]. Moreover, good bonding circumstances and a sudden tendon release are assumed. The tensile stress at the moment of application of the prestress is assumed to be 2,5 [N/mm^2] [62]. This value is divided by 1,5 to find the design tensile strength.

Equation 4.1: Transmission length, equation 8.15 from NEN_EN 1992-1-1. Same equation as Equation 3.2, but σ_{pm0} and $f_{ctd}(t)$ differ.

$$l_{pt} = \alpha_1 \times \alpha_2 \times \emptyset \times \frac{\sigma_{pm0}}{f_{pt}}$$

$\alpha_1 = 1{,}25$ for sudden release, $\alpha_1 = 1{,}0$ for gradual release
$\alpha_2 = 0{,}25$ for circular cross-section (wires), $\alpha_2 = 0{,}19$ for 3- and 7 wire strands
$\emptyset$ = nominal diameter of prestressing tendon [mm]
σ_{pm0}= stress at tendon release [N/mm^2]
f_{pt}= bond strength [N/mm^2] $\quad f_{pt} = \eta_{p1} \times \eta_1 \times f_{ctd}(t)$
$\eta_{p1} = 2{,}7$ for wires, $\eta_{p1} = 3{,}2$ for 3- and 7 wire strands
$\eta_1 = 1{,}0$ with good bonding circumstances, $\eta_1 = 0{,}7$ otherwise
$f_{ctd}(t)$ = design tensile strength at moment of prestress application

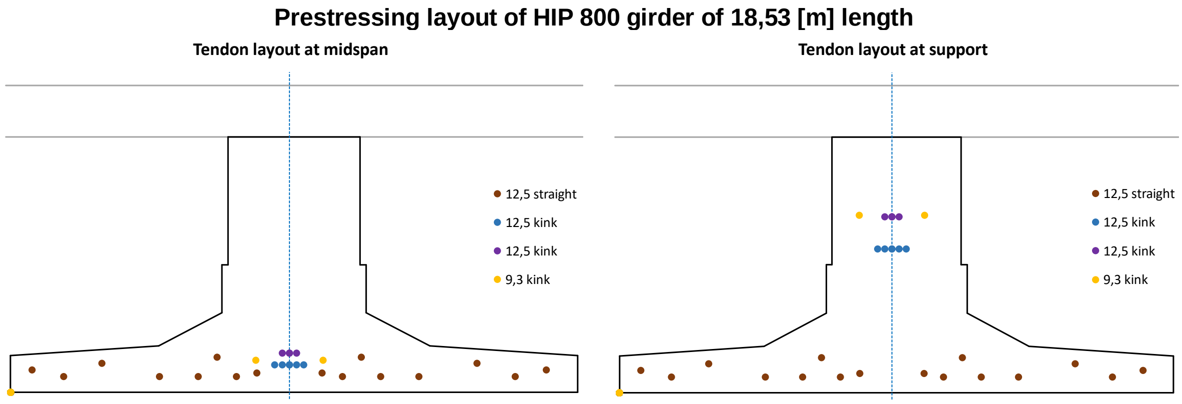


Figure 4. 7 a) Tendon layout at midspan.
Figure 4. 7 b) Tendon layout at support.

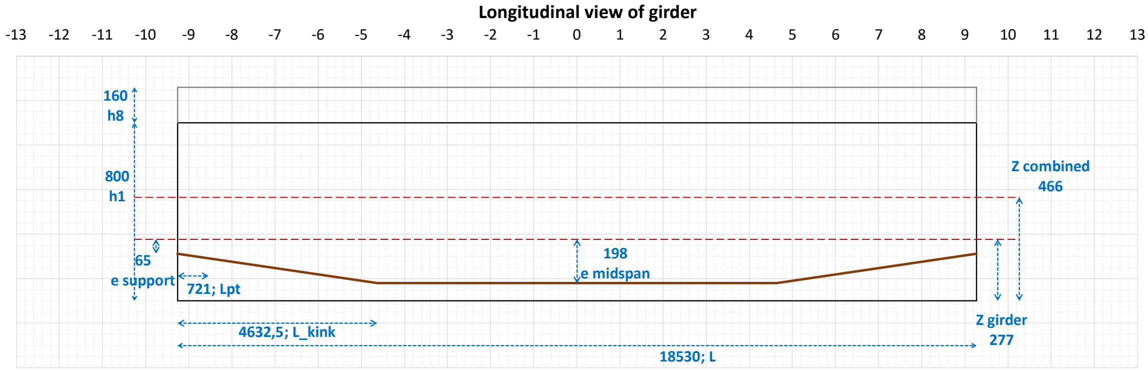


Figure 4. 7 c) Longitudinal view of simplified prestressing layout, all strands are replaced by one equivalent tendon.

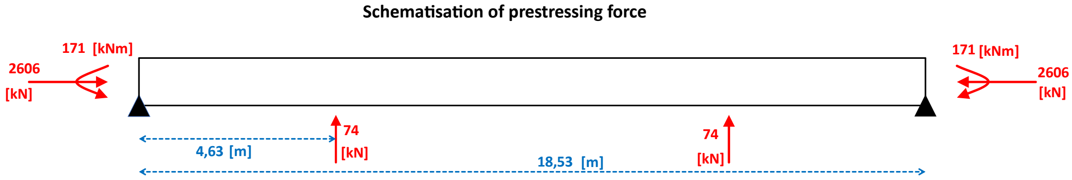


Figure 4. 7 d) Prestressing forces acting on girder.
Figure 4. 7: Prestressing layout of existing HIP 800 girder of 18,53 [m] length.

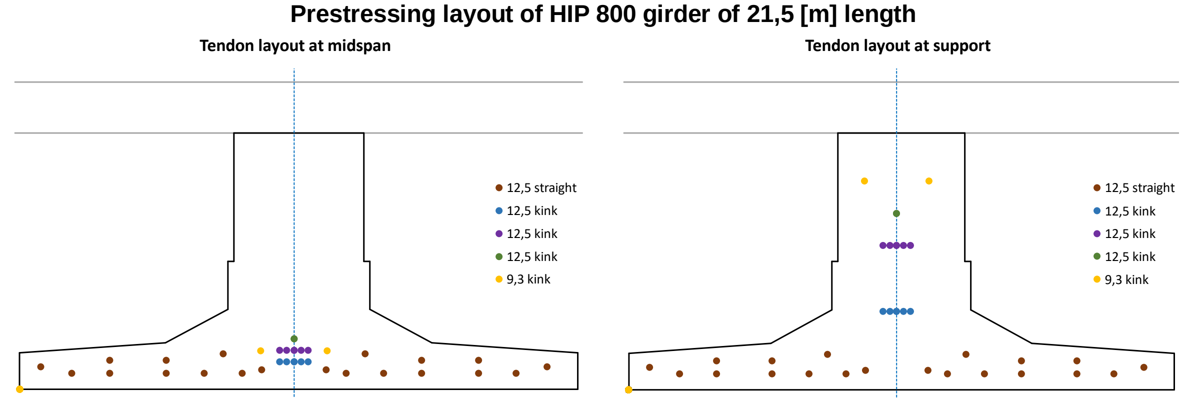


Figure 4.8 a) Tendon layout at midspan.

Figure 4.8 b) Tendon layout at support.

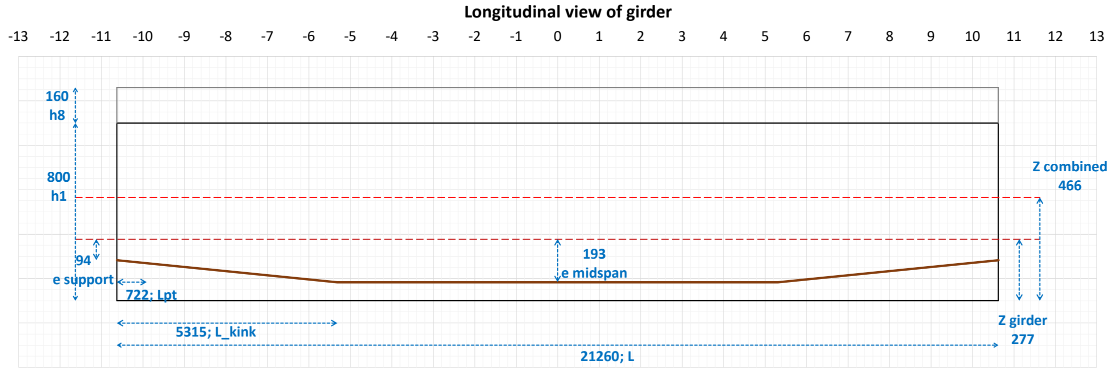


Figure 4.8 c) Longitudinal view of simplified prestressing layout, all strands are replaced by one equivalent tendon.

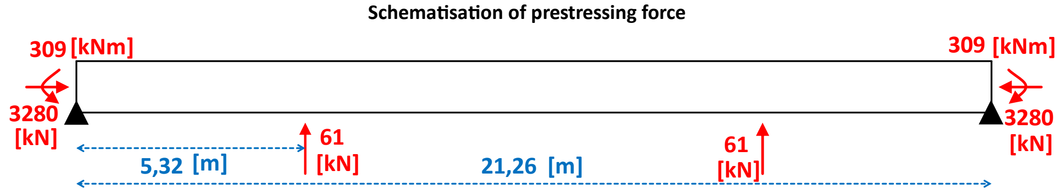


Figure 4.8 d) Prestressing forces acting on girder.

Figure 4.8: Prestressing layout of existing HIP 800 girder of 21,5 [m] length.

Reinforcement

During the period in which the viaducts are build regulations regarding shear reinforcement changed. Moreover, it is not exactly known when the girders are manufactured and information is contradictory. According to the available information of Rijkswaterstaat stirrups are present with spacing of 300 [mm]. However according to a folder of Span Beton of 1970 stirrups have a diameter of 8 [mm] and a spacing of 50 [mm] [137]. Nevertheless between 1971 and 1974 stirrups of 8, 10 or 12 [mm] are used with a spacing of 300 [mm]. After 1974 the spacing in the end zone is reduced to 100 [mm]. It is assumed that stirrup diameter is 8 [mm] and the spacing is 300 [mm]. However, a reduction of the spacing to 100 [mm] in the end zones is also investigated. If the alternative turns out to be feasible this assumption should be verified by construction drawings from archives or non-destructive testing with a cover meter. Steel quality QR40 is used.

§4.2.4 HNP-750 girder

The cross-section and model of the cross-section of the HNP girder is shown in Figure 4.9 and Figure 4.10. The standard height for the in situ-deck is 200 [mm]. The concrete strength classes are similar to the previous girder. So, the assumed strength for the girder is C55/67 and C30/37 for the new in-situ deck.

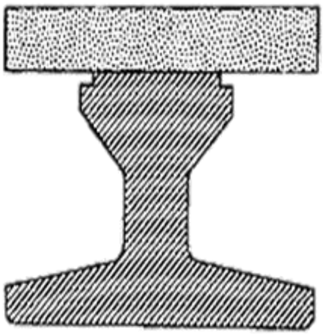


Figure 4.9: Cross-section HNP-girder from Span Beton [35].

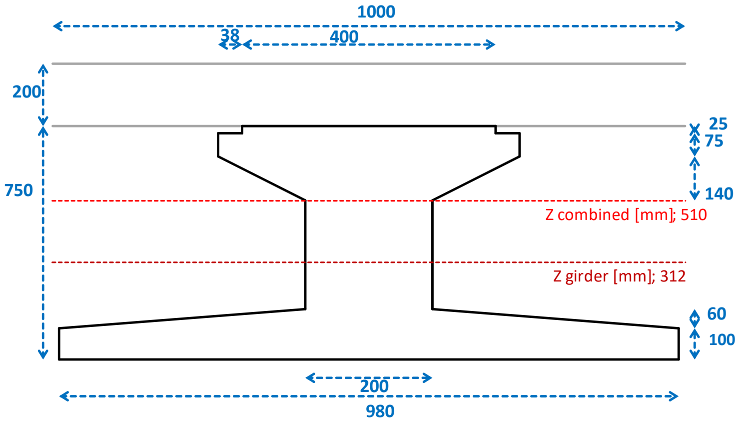


Figure 4.10: Model of cross-section of HNP-750 girder.

Prestressing

Steel quality QP190 is used for the prestressing steel. In the girders of 23 [m] 29 strands are present [74]. This is derived from a drawing in which the tendon layout at midspan is presented. The tendon layout at the support is unknown. In the model a diameter of 9 [mm] is assumed for the strands. This assumption is based on a 9 that is mentioned on the drawing, but it is unclear if this represents diameter. With a nominal diameter of 9 [mm] the steel area becomes 50 [mm^2] [148]. The assumed stress in the prestressing steel is similar to the previous girders. Due to the smaller diameter the transmission length becomes 553 [mm]. Based on this information the tendon layout and prestressing load is shown in Figure 4.11. No information is available about the 16,05 [m] long girder.

Reinforcement

HNP girders are not equipped with stirrups. Only hairpins with a diameter of 6 [mm] and a spacing of 500 [mm] are used. The steel quality used is QR40 [35].

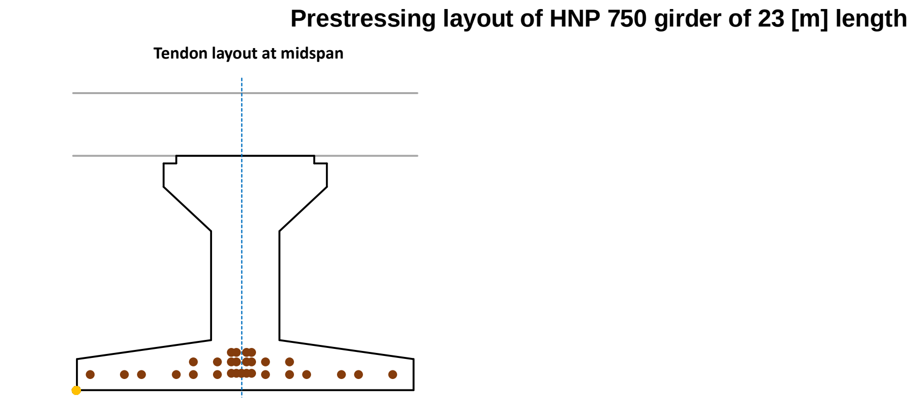


Figure 4.11 a) Tendon layout at midspan.

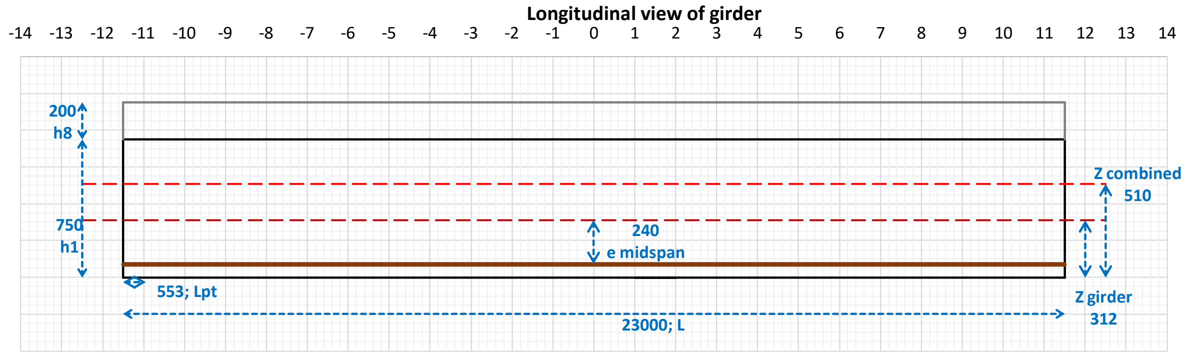


Figure 4.11 c) Longitudinal view of simplified prestressing layout, all strands are replaced by one equivalent tendon.

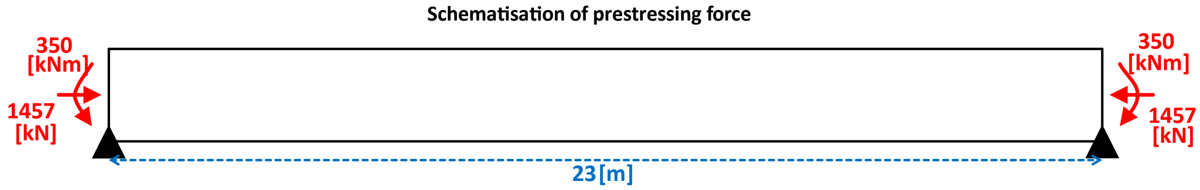


Figure 4.11 d) Prestressing forces acting on girder.

Figure 4.11: Prestressing layout of existing HNP 750 girder of 23 [m].

§4.3 Alternative 1a

In this paragraph the structural analysis of alternative 1a is briefly discussed. This design alternative consists of HIP 800 girders with lengths of 18,53 and 22,62 [m]. The derivation of all results and more detailed calculations and information can be found in Appendix E: Structural calculation. First the design loads are discussed followed by the shear capacity, bending moment capacity and shortening possibilities.

§4.3.1 Loads on structure

Figure 4.12 presents the design loads on the critical cross-sections for both girder lengths present in the alternative. In addition, both statically determinate and indeterminate structure system are evaluated.

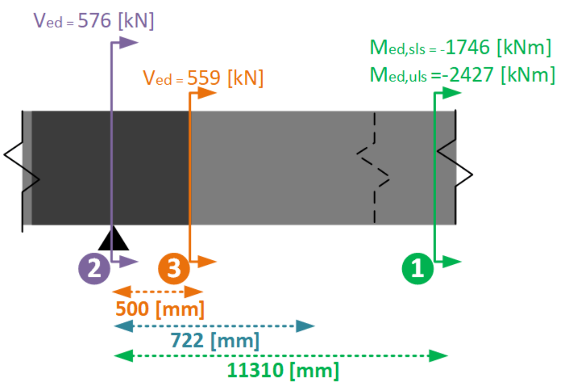


Figure 4.12 a) Design loads on 22,62 [m] girder in statically determinate system.

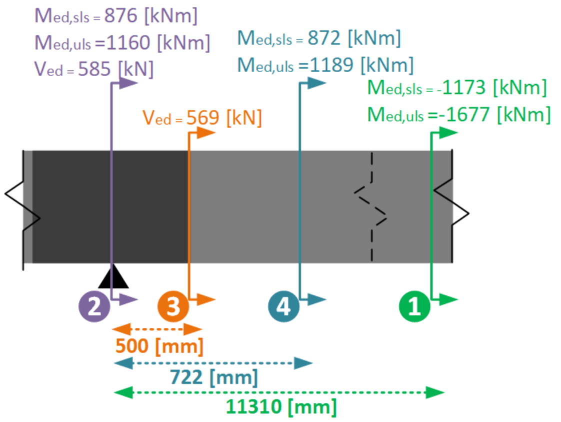


Figure 4.12 b) Design loads on 22,62 [m] girder in statically indeterminate system.

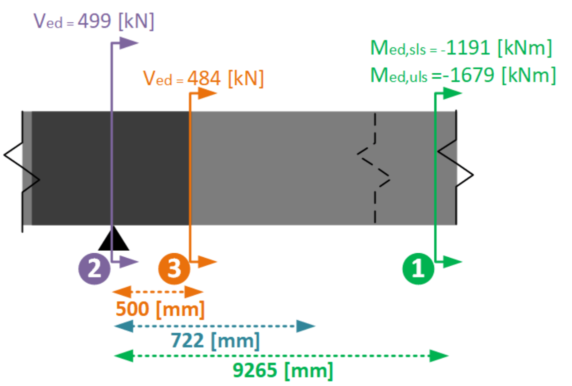


Figure 4.12 c) Design loads on 18,53 [m] girder in statically determinate system.

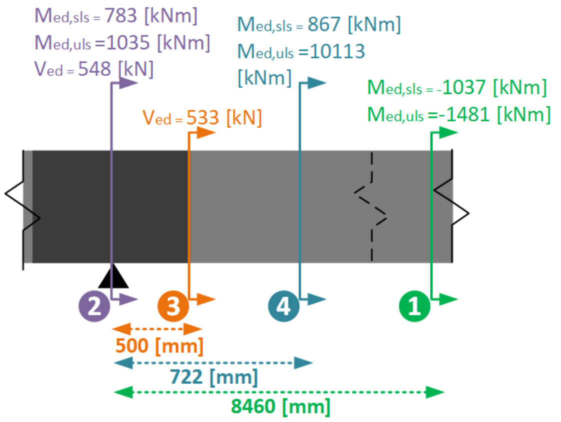


Figure 4.12 d) Design loads on 18,53 [m] girder in statically indeterminate system.

Figure 4.12: Design loads on critical cross-section of girders in design alternative 1a.

§4.3.2 Shear capacity

Table 4.4 shows the verification on shear capacity. It can be concluded that the minimum shear reinforcement ratio is not met. Even if the spacing at the supports is 100 [mm], the ratio is not met in the remaining part of the span. The calculation process is continued, since the RBK does not require a minimum reinforcement ratio and this ratio can be discussed for reusing existing elements.

Furthermore, it can be concluded that if the spacing near the supports is 100 [mm] the girders are able to meet the other NEN requirements. If the spacing is 300 [mm] the girders are not able to meet NEN nor the RBK requirements. To make the girders suitable the shear capacity can be increased with CFRP sheets or the load on the girder can be reduced. In this case study only the last option is considered. Since, the capacity at the support and next to the support is insufficient a combination of restriction on the traffic load and reducing the width of the girder is needed.

To give an indication of the load restriction. In the Netherlands the maximum allowed vehicle load is 50 [tons]. Assuming this load is linearly related to the vehicle load model, the maximum allowed vehicle load is 33 [tons]. Since, the bridge is only used to access a residential area and exceptional transport can be realized over the water this may be a suitable solution. In addition, the girders should be shortened in width direction to 790 [mm]. So, around 200 [mm] has to be cut on both sides.

Table 4.4: Summary of shear verification for girders in design alternative 1a.

		Standard	NEN		RBK	
		Spacing	S=100 [mm]	S=300 [mm]	S=100 [mm]	S=300 [mm]
			Shear reinforcement ratio: $\frac{A_{sw}}{s \times b_w} \geq 0,1483$ [%]			
		[%] present	0,3656	0,1117	-	-
Length [m]	Cross-section	System	Shear capacity: $U.C = \frac{V_{ed}}{V_{rd}} \leq 1$			
22,62	2. At support	SI	0,82	1,22	0,60	1,22
		SD	0,80	1,20	0,59	1,20
	3. Next to support	SI	0,79	2,38	0,91	3,44
		SD	0,78	1,60	0,65	1,56
18,53	2. At support	SI	0,76	1,14	0,56	1,14
		SD	0,70	1,04	0,51	1,04
	3. Next to support	SI	0,74	2,23	0,85	3,22
		SD	0,68	1,68	0,61	1,63

§4.3.3 Bending moment capacity

First a serviceability limit state analysis at midspan is performed. In a statically determinate system tensile stresses develop at the bottom of the girder, but no cracking occurs. Often, limit prestress is allowed hence this system is possible. However, there is not a lot of margin available. In a statically determinate system no tensile stresses develop. In case the girders are shortened in width direction to 790 [mm] the situation is similar. In the ultimate limit state verification all situations fulfil, since the unity checks stay below 1,0. The results are summarized in Table 4.5.

Table 4.5: Summary of sagging bending moment verification for girders in design alternative 1a.

Length [m]	System	SLS: minimum stress [N/mm^2]		ULS: U.C. $=\frac{M_{ed}}{M_{rd}} < 1,0$	
		Width 1200 [mm]	Width 790 [mm]	Width 1200 [mm]	Width 790 [mm]
22,62	SD	+2,48	+1,26	0,64	0,58
	SI	-1,80	-1,42	0,41	0,37
18,53	SD	+0,26	-0,68	0,59	0,54
	SI	-0,89	-1,73	0,51	0,47

In a statically indeterminate system the deck should be reinforced with 2781 [mm^2/m] steel. This corresponds to a reinforcement ratio of 1,74 [%], which stays below the maximum of 4 [%]. This amount can be achieved with two layers of reinforcement with diameter 12 [mm] and spacing 75 [mm]. In this way the minimum spacing requirements are fulfilled.

§4.3.4 Shortening possibilities

The girders do not have to be shortened to be applied in the new project. Nonetheless in favour of the design approach shortening is investigated. The maximum shortening according to the search criteria is 23%. For these girders shortening until this maximum will be possible. Due to a changed stress distribution over the height a tensile stress of 1 [N/mm^2] develops at the top of the girder. Furthermore, strut-and-tie models in a side and top view, to distribute the prestressing force respectively over the height and width, indicate that the distribution over the height is governing. Based on strut-and-tie models in side views it is estimated that between 18 and 28 [%] of the transfer length is needed to resist the tensile splitting forces.

§4.4 Alternative 1b

In this paragraph the structural analysis of alternative 1b is briefly discussed. This design alternative consists of HNP 750 girders with lengths of 23,00 [m] and 16,05 [m]. However, in this analysis only the girders of 23,00 [m] are considered, because on the girders of 16,05 [m] no information is available. The derivation of all results and more detailed calculations and information can be found in Appendix E: Structural calculation. First the design loads are discussed followed by the shear capacity.

§4.4.1 Loads on structure

In Figure 4.12 the design loads on the four critical loads on the 23,00 [m] girder are shown. Both a statically determinate and statically indeterminate system are evaluated.

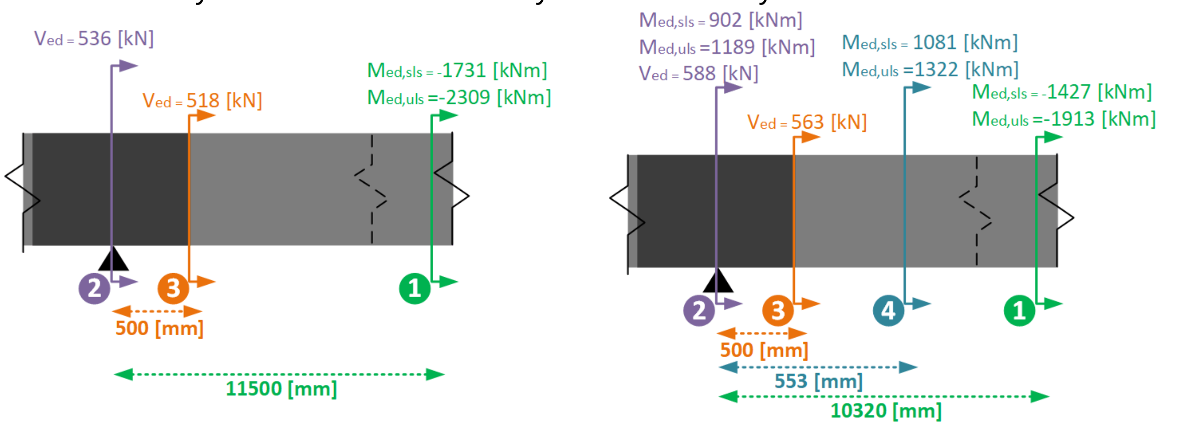


Figure 4.13 a) Design loads in statically determinate system. *Figure 4.13 b) Design loads in statically indeterminate system.*
Figure 4.13: Design loads on critical cross-section of girders in design alternative 1b.

§4.4.2 Shear capacity

Table 4.7 shows the verification on shear capacity. The minimum shear reinforcement cannot be met; hence no reinforcement is present. Moreover, the shear capacity is insufficient to such an extend that the only solution is strengthening of the girder. A potentially suitable strengthening method is the use of CFRP sheets. Nonetheless, the use of strengthening measures makes the alternative less attractive and significantly complicates the structural analysis. As a result, in accordance with the framework, the alternative is put on a 'temporary hold'. Only, in case no other alternative is feasible without strengthening, a further investigation in strengthening can be performed. For this case study this further investigation is not performed. On the one side, because other design alternatives are feasible without strengthening. On the other hand, detailed calculations on strengthening measures are outside the scope of this research.

Table 4.6: Summary of shear verification for girders in design alternative 1b.

	Shear reinforcement ratio: $\frac{A_{sw}}{s\times b_w} \geq 0,1483$ [%]		
	[%] present	0	
	Standard	**NEN**	**RBK**
Cross-section	**System**	**Shear capacity: $U.C = \frac{V_{ed}}{V_{rd}} \leq 1$**	
2. At support	SI	1,50	1,50
	SD	1,36	1,36
3. Next to support	SI	2,70	2,48
	SD	1,72	1,65

§4.5 Alternative 1d

In this paragraph the structural analysis of alternative 1d is briefly discussed. This design alternative consists of HIP 800 girders with a length of 21,26 [m]. The derivation of all results and more detailed calculations and information can be found in Appendix E: Structural calculation. First the design loads are discussed followed by the shear capacity, bending moment capacity and shortening possibilities.

§4.5.1 Loads on structure

In Figure 4.14 the design loads on the critical cross-section on the girder are shown for a statically determinate and indeterminate system.

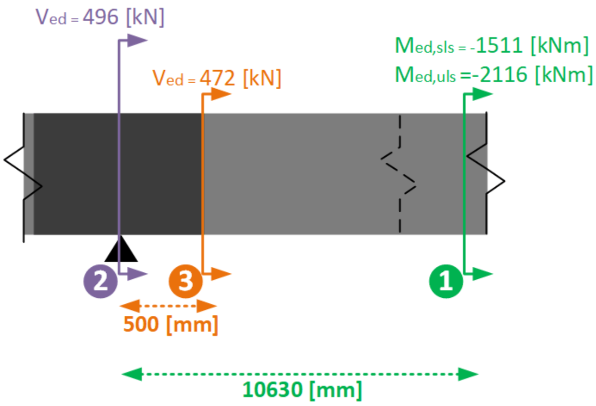


Figure 4.14 a) Statically determinate system.

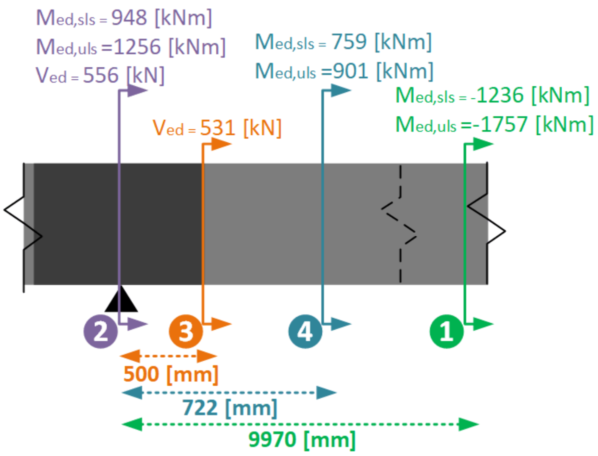


Figure 4.14 b) Statically indeterminate system.

Figure 4.14: Design loads on critical cross-section of girders in design alternative 1d.

§4.5.2 Shear capacity

Table 4.7 shows the verification on shear capacity. Similar to alternative 1a the minimum shear reinforcement ratio is not met. With a spacing of 100 [mm] near the supports al other NEN requirements are met. If the spacing near the supports is 300 [mm] the unity check at the supports is 1,03 in a statically determinate system. In a more detailed calculation model of the bridge this unity check can presumably be reduced to below 1,0. Than the width of the girder should be reduced to 750 [mm] to reduce the unity check next to the support to 0,91.

Table 4.7: Summary of shear verification for girders in design alternative 1d.

	Standard	NEN		RBK	
	Spacing	S=100 [mm]	S=300 [mm]	S=100 [mm]	S=300 [mm]
	Shear reinforcement ratio: $\frac{A_{sw}}{s \times b_w} \geq 0,1483$ [%]				
	[%] present	0,3656	0,1117	-	-
Cross-section	System	$U.C = \frac{V_{ed}}{V_{rd}} \leq 1$			
2. At support	SI	0,78	1,16	0,57	1,16
	SD	0,69	1,03	0,51	1,03
3. Next to support	SI	0,74	2,22	0,63	3,21
	SD	0,66	1,42	0,56	1,39

§4.5.3 Bending moment capacity

According to the serviceability limit state analysis a 1200 [mm] wide girder can only be used in a statically determinate system if limit prestress is allowed, because tensile stresses develop at the top. However, no cracking occurs. In a statically indeterminate system no tensile stresses develop.

According to the shear capacity shortening of 750 [mm] in a statically determinate system is might be needed. From the SLS analysis on bending moments this is possible; hence no tensile stresses develop. If the shortening is not needed, but a statically determinate system is required without tensile stresses developing a shortening to 1000 [mm] is possible. In the ultimate limit state analysis, all above mentioned situations fulfil and the unity checks stays well below 1,0. The results are summarized in Table 4.8.

Table 4.8: Summary of sagging bending moment verification for girders in design alternative 1d.

System	SLS: minimum stress [N/mm²]			ULS: U.C. $=\frac{M_{ed}}{M_{rd}} < 1,0$		
	Width 1200 [mm]	Width 750 [mm]	Width 1000 [mm]	Width 1200 [mm]	Width 750 [mm]	Width 1000 [mm]
SD	+1,29	-0,10	-0,24	0,6	0,54	0,56
SI	-0,78	-	-	0,57	-	-

In a statically indeterminate system the deck should be reinforced with 2938 [mm²/m²] steel. This corresponds to a reinforcement ratio of 1,84 [%], which is below the maximum of 4 [%]. This amount can be achieved with two layers of reinforcement with diameter 12 [mm] and spacing 75 [mm]. In this way the minimum spacing requirements are fulfilled as well.

§4.5.4 Shortening possibilities

The girders do not have to be shortened to be applied in the new project. Nonetheless in favour of the design approach shortening is investigated. The maximum shortening according to the search criteria is 23%. For these girders this maximum will be (close to) critical due to tensile stresses that develop at the top. Due to the changing stress distribution tensile stresses at the top of the girder already start to develop between 6 and 9% of shortening. These tensile stresses reach the cracking stress between 22 and 25% of shortening. From side view strut-and-tie models it is estimated that between 20 and 26% of the transfer length is needed to resist the tensile splitting forces.

§4.6 Environmental impact analysis

The environmental impact analysis is performed on the structurally feasible alternatives. The structural feasibility of alternative 1a depends on the layout of the shear reinforcement. Therefore, this alternative is split in two sub-alternatives, an alternative with 1200 [mm] wide girders and an alternative with girders shortened in width direction to 790 [mm]. Alternative 1b is not considered, because this alternative is structurally unfeasible. For alternative 1d three sub-alternatives are considered: 1200 [mm] wide girders, 1000 [mm] wide girders and 750 [mm] wide girders. To make a comparison between reusing existing girders and using new girders an alternative with new girders is considered.

The input, especially for the alternative with new girders is discussed in the first section. The environmental impact analysis is divided in the Environmental Cost indicator (ECI), discussed in the second section and the material origin and use, discussed in the third section. In this paragraph references are made to tables in Appendix F: Environmental impact analysis.

§4.6.1 Input

For the environmental impact analysis of the alternatives with reusing existing girders most input can be derived from previous analyses. However, in the previous analyses no alternative with new girders is considered; hence to be able to make a comparison a general design with new girders is developed. For each element: girders, deck and crossbeams the guiding principles and input is discussed.

For the girders the required input for the ECI calculation is the span length and quantity. The span lengths in the alternatives in which existing girders are reused are already discussed. The number of girders depends on the width of the girders. For the alternative with new girders the span division from the system design is used. So, two spans of 16,11 [m] and 3 spans of 25 [m]. Based on the spans of 25 [m] and the indications graphs from Haitsma HRP 800 girders with a in situ deck of 250 [mm] are used as starting point [40].

In addition, the area of the girder, area of tendons, stirrup diameter and stirrup length are needed for the indicator material origin and use. Based on the cross-sectional dimensions the area of the girder is determined and the length of stirrups is roughly estimated. The area of tendons is based on an estimation of the required prestressing force. The stirrup diameter and spacing are based on an estimation of the shear force. The calculations are available in the first section of Appendix F: Environmental impact analysis.

For the in-situ deck the input is the height of the deck and the total width and length of the bridge. All alternatives, except alternative 1a with a girder width of 790 [mm], can exactly meet the required width of 6 [m]; because this total width is a multiple of the girder width. For 790 [mm] this does not apply and the bridge becomes 6,24 [m]. Additionally, for the material origin and use the amount of reinforcement in the deck is needed. For reinforcement 170 [kg/m^3] is assumed. The background of this assumption is provided in the second section of Appendix F: Environmental impact analysis. The assumption is based on a statically determinate system, where no hogging bending moments occur. In case of hogging bending moment additional reinforcement in the deck is needed. However, this potential increase has a negligible effect on the outcome because the reinforcement has a minor effect on the material origin and use.

§4.6.2 Environmental costs indication

The ECI calculation is performed in accordance with the roadmap. So, to calculate the value the input from Table 4.9 is used in Figure 3.25. The detailed tabled results for each life cycle and environmental impact category can be found in the third section of Appendix F: Environmental impact analysis.

Table 4.9: Input values for ECI calculation.

Input	Alternatives					
	New girders	1a 1200 [mm]	1a 790 [mm]	1d 1200 [mm]	1d 1000 [mm]	1d 750 [mm]
Total length [m]	107,2	104,9	104,9	106,3	106,3	106,3
Deck height [m]	0,25	0,16	0,16	0,16	0,16	0,16
Girder length [m] Number of girders [-]	25 15	22,62 15	22,62 24	21,26 25	21,26 30	21,26 40
Girder length [m] Number of girders [-]	16,11 10	18,53 10	18,53 16			
Crossbeam width [m]	1,0	1,0	1,0	1,0	1,0	1,0
Number of spans	5	5	5	5	5	5
Height girder [m]	0,8	0,8	0,8	0,8	0,8	0,8
Area girder [m^2]	0,41	0,36	0,31	0,36	0,34	0,31

In Figure 4.15 the total ECI-value and value of each element is shown. To be able to compare the values with related and future projects the ECI-value is converted to [m^2] of bridge deck in Table 4.10. Though this ECI-value heavily depend on the girder span; the average span length is mentioned as well.

From these figures it can be concluded that the average ECI-value of a bridge deck in which existing girders are reused is 51% lower compared to a bridge deck with new girders. In case the girders keep their original width the reduction is 59%. This is in line with the project of 'hergebruik liggers 2.0', in which an ECI reduction of 61% is calculated. In this calculation only the girders and the in-situ deck are considered [22], which is comparable to the elements considered in this analysis. For the bridge deck with new girders, girders account for 76% of the ECI-value. For the alternatives in which existing girders are reused this is slightly lower and varies between 58 and 72%.

The ECI-value of alternative 1a and 1d are almost equal. This is logical because the alternatives only differ in span division and not in girder size or deck-height. Alternative 1a has a slightly lower (0,8%) ECI-value compared to alternative 1d because the total span length is 1,38 [m] smaller. Consequently, the average span length is smaller, however the ECI value per m^2 is €0,10 higher. This is due to the span division and the same number of crossbeams. Alternative 1a has 3 larger spans and 2 smaller spans, while alternative 1d has equal spans. A larger span, results in a higher ECI-value per [m] length.

If the girders are shortened in width direction the ECI-value increases because more girders are needed. The results from alternative 1d show that a width reduction of 16,7% results in a ECI increase of 17,4% and a width reduction of 37,5% results in a ECI increase of 41,5%. So, a reduction in width leads to a significant increase in ECI-value. The theoretical maximum width reduction is a reduction to the width of the webs. For the HIP 800 girders this is 300 [mm], which is equal to a reduction in width of 75%. For this theoretical situation a linear relationship based on the above-mentioned values is assumed between the percentage in width reduction and the increase in ECI-value. In this case the reduction of 75% of the width would imply an ECI increase of 85%. This is equal to an ECI value of €25.085, -, which is still 25% lower than the ECI value of the alternative with new girders. Therefore, it might be concluded that shortening in width direction has no environmental limit.

Due to the shortened flanges, there is less space between the webs of the girder. So, as a result there is less concrete needed to cast the crossbeam, which causes a lower ECI-value of the crossbeams. Nonetheless, the influence of the crossbeams on the total ECI-value is not more than 8%.

The ECI-values of the decks are lower compared to the alternative with new girders. This is due to the lower height of the in-situ deck. Nonetheless this height is not investigated and just assumed to be equal as in the existing structure. Therefore, the ECI value of the alternatives with reusing existing girders might be slightly underestimated.

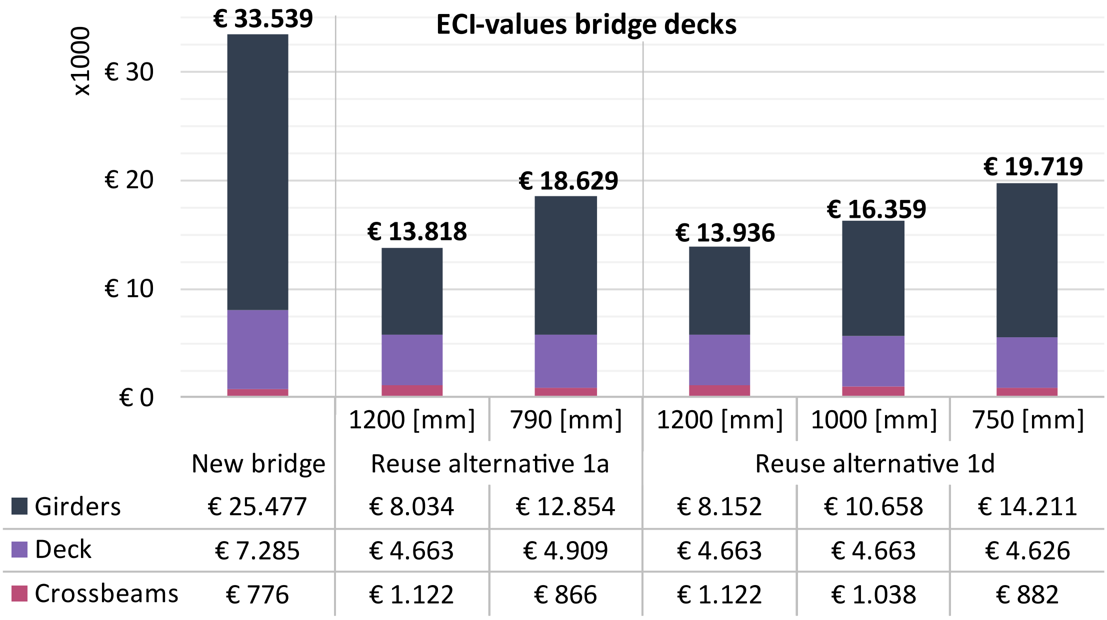


Figure 4.15: ECI-values per element of different alternatives.

Table 4.10: ECI-values per m^2 bridge deck

Alternative	New bridge	Alternative 1a		Alternative 1d		
Width girder	1200 [mm]	1200 [mm]	790 [mm]	1200 [mm]	1000 [mm]	750 [mm]
Average span	21,4 [m]	21,0 [m]		21,3 [m]		
ECI-value per m^2	€52,14	€21,95	€28,09	€21,85	€25,65	€30,92

An overview of the division over life cycle stages is given in Figure 4.16. Most benefit of bridge decks with reused elements is gained in life stage A1 to A3. These phases account for the raw material supply, transport to factory and the manufacturing. Since, the girders originate from an existing structure only 20% of the traditional environmental impact is account for. The same holds for life stage C3 and C4, which account for waste processing and disposal. The impact of these stage is also lowered with around 50%, however the impact of these stages on the total ECI-value is less than 1%. The positive effects are partly counteracted by the reduced positive effect in life cycle stage D. The effects in the other life cycles: A4, A5, C1 and C2 are comparable or higher. The effects are higher if the girders are shortened in width direction. In this case more transport is needed and more concrete is used, so more material to process.

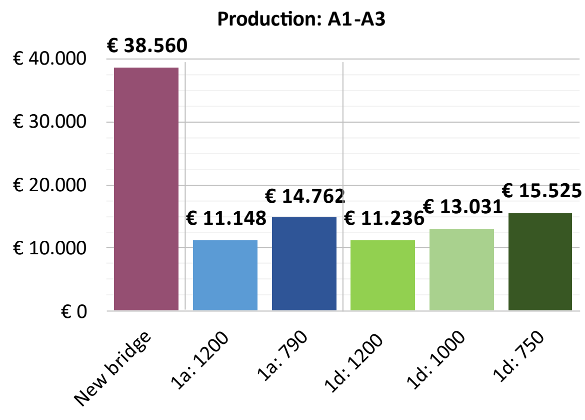


Figure 4.16 a) Product stage: material supply, transport and manufacturing.

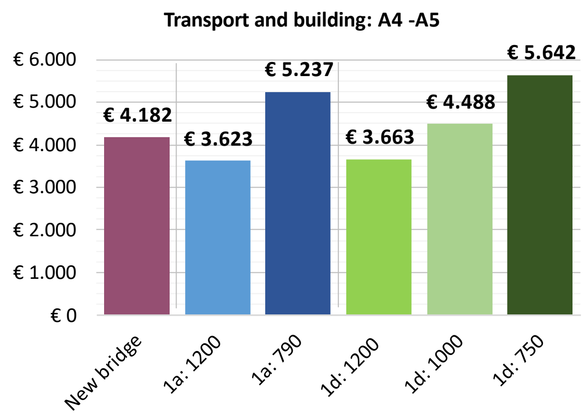


Figure 4.16 b) Product stage: transport and construction.

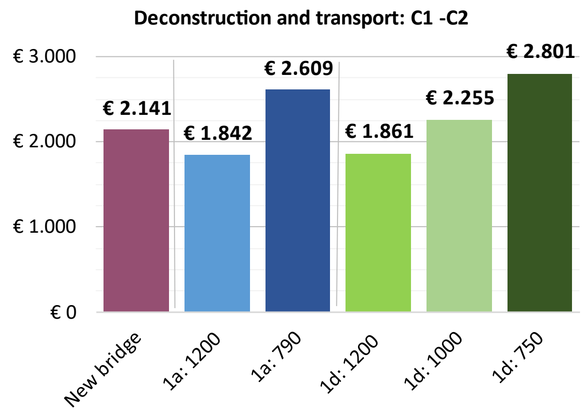


Figure 4.16 c) End of life stage: deconstruction and transport.

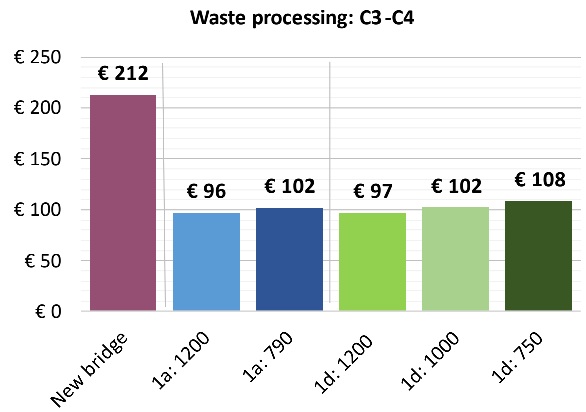


Figure 4.16 d) End of life stage: waste processing and disposal.

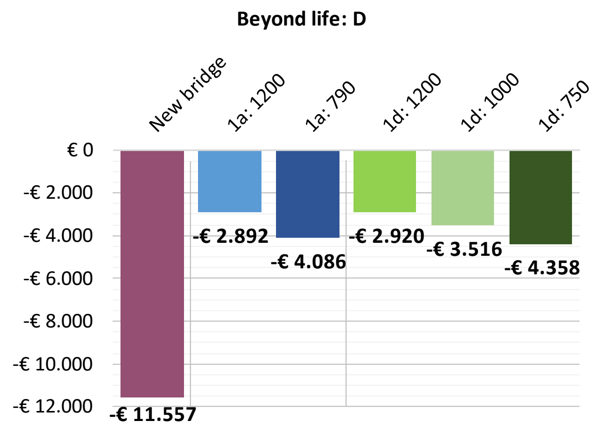
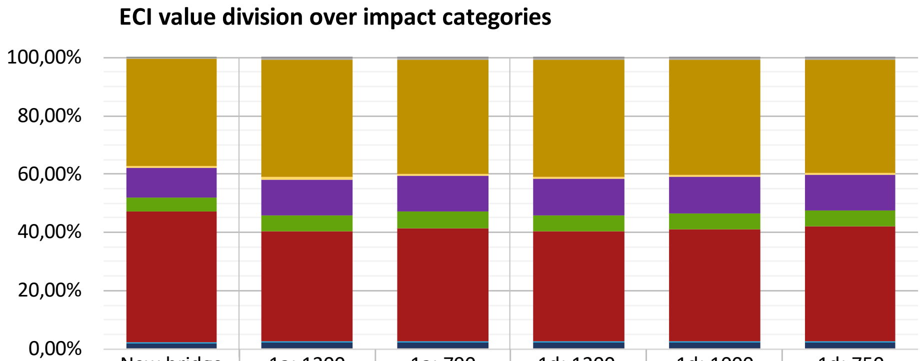


Figure 4.16 e) Beyond life cycle: reuse, recovery and recycling.

Figure 4.16: ECI-values per life cycle stage of different alternatives.

An overview of the division over environmental impact categories is given in Figure 4.17. From this overview it can be concluded that the division over the impact category is not significantly affected by reuse. Only the impact on human toxicity slightly decreases, while the impact of global warming increases. The human toxicity impact category is based on the chemicals with toxic impact on humans that are emitted to the environment. Most chemicals will be released during production and waste processing. As already discussed, the effect of these life cycle stages is reduced due to reuse of existing girders. This explains the decrease of the proportion of human toxicity. The global warming impact category relates to the emission of greenhouse gasses. Due to the use of equipment and transportation vehicles especially during transport, construction and deconstruction this impact category has a large influence. With reusing girders the share of these stages is rather increased than decreased. Therefore, it is reasonable that the proportion of global warming has increased.



	New bridge	1a: 1200	1a: 790	1d: 1200	1d: 1000	1d: 750
Abiotic depletion, non fuel (AD)	0,00%	0,00%	0,00%	0,00%	0,00%	0,00%
Abiotic depletion, fuel (AD)	0,56%	0,68%	0,67%	0,68%	0,67%	0,67%
Global warming (GWP)	36,6%	40,3%	39,16%	40,26%	39,56%	38,82%
Ozone layer depletion (ODP)	0,00%	0,00%	0,00%	0,00%	0,00%	0,00%
Photochemical oxidation (POCP)	0,66%	0,80%	0,80%	0,80%	0,80%	0,80%
Acidification (AP)	10,4%	12,5%	12,34%	12,48%	12,41%	12,33%
Eutrophication (EP)	4,53%	5,47%	5,48%	5,47%	5,48%	5,49%
Human toxicity (HT)	45,2%	37,7%	39,06%	37,73%	38,53%	39,41%
Ecotoxicity, fresh water (FAETP)	0,15%	0,18%	0,18%	0,18%	0,18%	0,18%
Ecotoxicity, marine water (MAETP)	1,77%	2,09%	2,07%	2,09%	2,08%	2,07%
Ecotoxicity, terrestric (TETP)	0,25%	0,29%	0,24%	0,29%	0,27%	0,24%

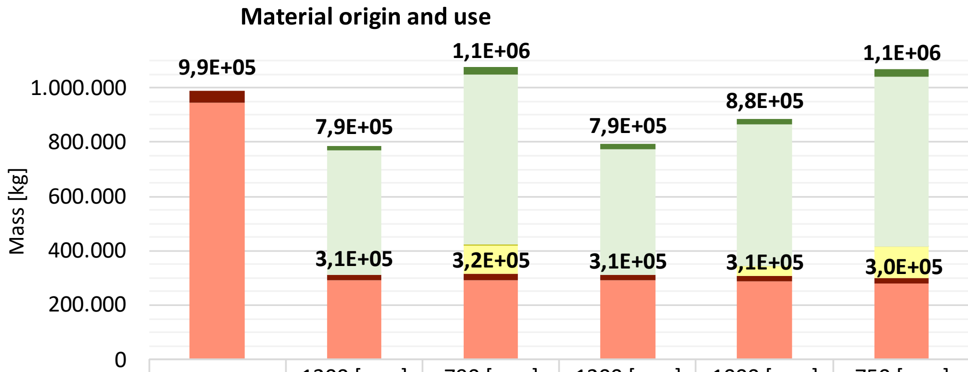
Figure 4.17: ECI-values per impact category of different alternatives

A final remark should be made about the impact on the total structure. In all alternatives 5 spans are used. So, in each alternative 3 intermediate piers are used. Also, the same girder height is used, so the load on the foundations and piers will be equivalent. So, it can be concluded that there is no effect on the total structure that should be considered.

It is good to mention that for alternatives in which existing girders are reused, it will be sooner attractive to use larger spans and less intermediate piers. With larger spans, the girders need to be higher, which increases the ECI-value of the girders. Therefore, it can be more beneficial to use smaller spans and an extra intermediate pier. Nonetheless due to implementation of reuse the impact of the girder in the total ECI-value is smaller. Consequently, the impact of the foundations and intermediate piers is larger. So, using larger girders and less intermediate piers can be beneficial.

§4.6.3 Material origin and use

In accordance with the roadmap the amount of material, origin and use is visualized in Figure 4.18. The division between steel and concrete from primary and secondary resources is made. Due to shortening in width direction some secondary steel and concrete does not end up in the construction. This 'wasted' amount is shown in yellow. From the figure it can be concluded that reusing girders result for this case study in a reduction of more than 65% on primary resources for the bridge deck. The total material amount is reduced, due to the reduction of deck height, but as already mentioned this is not investigated. In case of shortening in width direction, the total material amount increases.



	New bridge	Reuse alternative 1a 1200 [mm]	Reuse alternative 1a 790 [mm]	Reuse alternative 1d 1200 [mm]	Reuse alternative 1d 1000 [mm]	Reuse alternative 1d 750 [mm]
Secondary steel in construction	0	16983	25867	17388	20182	25654
Secondary concrete in construction	0	456480	628032	462600	519192	625632
Secondary steel wasted	0	0	4540	0	941	2509
Secondary concrete wasted	0	0	102336	0	35928	114518
Primary steel	42515	22112	21948	22112	21760	20961
Primary concrete	946207	289920	293174	289920	286728	278798

Figure 4.18: Amount of material used in different alternatives.

§4.7 Financial analysis

The financial analysis on the structurally feasible alternatives is performed in the same manner as the environmental impact analysis. So, alternative 1a is split in two sub-alternatives, an alternative with 1200 [mm] wide girders and an alternative with girders shortened in width direction to 790 [mm]. For alternative 1d three sub-alternatives are derived: 1200 [mm] wide girders, 1000 [mm] wide girders and 750 [mm] wide girders. In this analysis only the costs for the 1200 [mm] wide girders and 1000 [mm] wide girders are estimated, because this provides sufficient information for a comparison.

For the alternative with new girders the same guiding principles as discussed in §4.6.1 are used. However, during the performance of the case study the system design changed. Instead of two spans of 16,1 [m] and three spans of 25 [m], five equal spans of 21,4 [m] are proposed. As a result, instead of HRP-800 girders, HRP-700 girders can be used. Consequently, this cost estimation is based on HRP-700 girders.

After discussing the guiding principles for the cost indication in the first section, the results are presented in the second and compared in the last section.

§4.7.1 Elements in costs indication

In this section the attention points discussed in §3.8 are applied on the design alternatives. The deconstruction process of the existing viaducts is discussed in Appendix D: Existing viaducts.

For both alternatives it is assumed that deconstruction of the viaducts will not be on the critical path of the planning. So, sufficient time is available to remove the in-situ deck before extracting the girders from the viaducts. This time is available because, traffic over the viaducts (on the highway) is not substantially hindered. The demolition of the existing viaducts and building of new viaducts is divided into different phases. By moving the lanes and making use of parts of the new viaducts and parts of the existing viaducts traffic can always continue. Furthermore, parallel to the deconstruction of the existing viaducts additional construction works for widening of the highway are performed, which will take longer than deconstruction of the viaducts.

Contrary to the highway traffic over the viaducts, roads underneath the viaducts have to be closed during deconstruction. The road underneath the existing viaduct of alternative 1a can be closed for a certain period, without significant hindrance. Therefore, a longer period is not an issue and deconstruction can be performed during regular working days. It is a local road with a low traffic intensity and traffic can easily be redirected with a small detour. The road underneath the existing viaduct of alternate 1d is of higher importance for the infrastructural network. Therefore, this road cannot be closed during normal working days and deconstruction must be done during weekend and night closures. Due to the longer deconstruction process, multiple closures are needed.

The next step is storage and modification. For the cost estimation, two transports are accounted for: from existing structure to storage and from storage to Meinerswijk. It is likely that the girders can be stored at the project site in Meinerswijk as sufficient space is available. If the modifications and investigations can also be performed at this location, the costs of transport will reduce. The length of the girders is around 22 [m], therefore guiding vehicles are needed. However, the transport length is not exceptionally long. For the 1200 [mm] options of alternative 1a and 1d the girders only need fix up. So, the shear reinforcement is straightened, girders are cleaned and small damages are repaired. For the girders shortened in width direction, the girders are cut. Consequently, the concrete that has been cut off has to be removed and exposed reinforcement has to be covered to ensure the durability.

For the formwork of the in-situ deck extra costs are included due to the lost notch. As already indicated the height of the in-situ deck is not investigated in this research. With a lower deck-height, the amount of reinforcement increases, which has a significant effect on the costs. For this analysis a 280 [mm] deck is used as guiding principle, because this is frequently applied in practice.

§4.7.2 Results

The cost estimation of the alternative with new girders is shown in Table 4.11. The estimation for the alternatives with existing girders is shown in Table 4.12. A detailed cost estimation can be found in Appendix G: Financial analysis.

In Figure 4.19 the results are graphically shown. The costs of demolition are separately considered because it is unclear to which extend the extraction of the girders from the existing structure should be accounted for in the cost estimation of the new bridge. Currently, Rijkswaterstaat has contracted VeenIX for the deconstruction of the viaducts. It is possible, that VeenIX extracts the girders from the structure and sells them. Another possibility is that Rijkswaterstaat does a request for a reduction in work to VeenIX. The budget that is released can be used by Van Hattum en Blankevoort to cover a part of the deconstruction costs. In this way the regular demolishing costs are compensated.

Table 4.11: Cost indication alternative with new girders.

Design alternative / Cost element		New
Manufacturing, delivery and placing girders	€	299.780
Crossbeams	€	29.485
In-situ concrete layer	€	126.818
Total	**€**	**456.083**

Table 4.12: Cost indication alternatives in which existing girders are reused.

Design alternative / Cost element		1a: 1200		1a: 790		1d: 1200		1d: 1000
Demolition	€	150.750	€	207.050	€	383.950	€	489.800
Deconstruction	€	38.180	€	49.500	€	39.740	€	48.820
Transport to storage	€	88.712	€	140.768	€	135.768	€	167.024
Fixing and durability assessment	€	147.371	€	400.907	€	149.163	€	322.837
Transport to new location	€	88.712	€	140.768	€	88.712	€	104.712
Placing girders in new structure	€	85.180	€	137.000	€	85.180	€	104.840
Crossbeams	€	33.820	€	33.820	€	33.820	€	33.820
In-situ concrete layer	€	161.761	€	183.903	€	165.499	€	176.847
Total	**€**	**791.486**	**€**	**1.293.716**	**€**	**1.081.832**	**€**	**1.448.700**

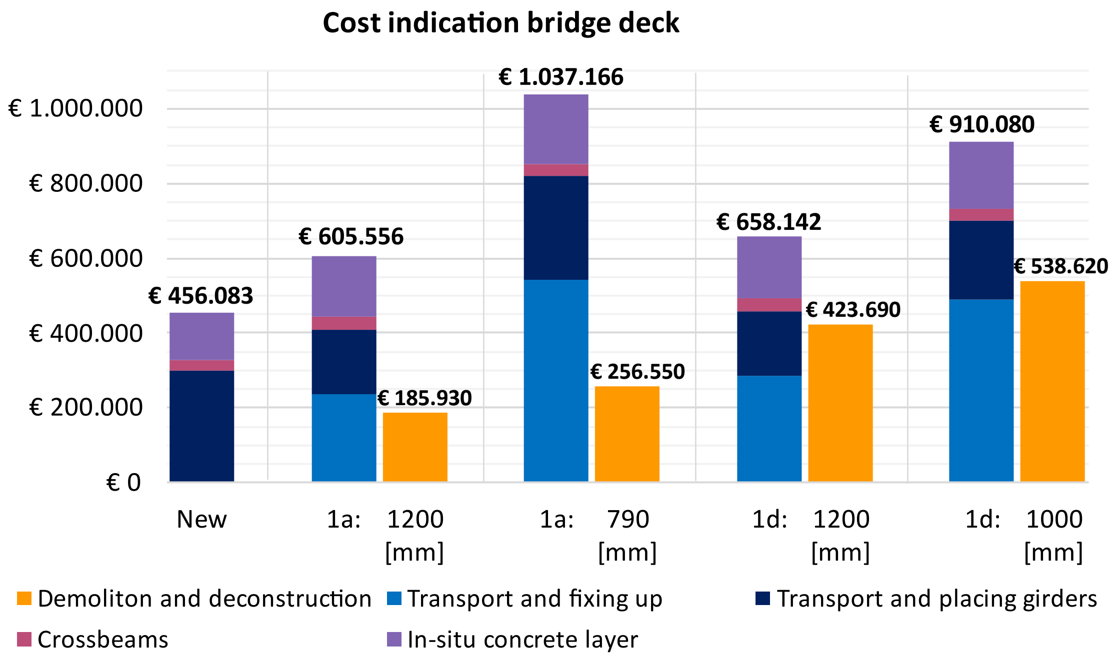


Figure 4.19: Cost indication design alternatives.

§4.7.3 Comparison

The costs for demolition include removing the in-situ deck, disconnecting the girders and removing the crossbeams. These costs increase when more girders are needed and explains the differences between the girder widths. Also, road closures are included. As a result, the demolition costs for alternative 1d are more than 150% higher than for alternative 1a.

The costs for deconstruction and placing the girders are related to hoisting actions and depend on the number of girders. Also, transportation depends on the number of girders. Although alternative 1a and 1d with 1200 [mm] wide girders need the same number of girders, the costs for transportation to storage 1d are significantly higher, because transported is divided over multiple weekend and night closures.

The small difference in the costs for fixing up and modifications between the alternatives with 1200 [mm] wide girders can be explained by the small difference (1,38 m) in span length. In this cost category €75.000 is included for additional research and structural calculation. The costs for the alternatives shortened in width direction are significantly higher due to the additional activities needed.

The 13% higher costs for the crossbeams in the alternatives with reuse of existing girders compared to alternatives with new girders is explained by the girder height. In the alternative with new girders, the height of the girders is 700 [mm], while in the other alternatives the girder height is 800 [mm]. Consequently, a smaller area of formwork and less concrete is needed in the alternative with new girders, which leads to lower costs.

The costs for the in-situ deck are 28% to 45% higher for alternatives with reuse compared to alternatives with new girders. This difference is primarily caused by the additional formwork needed because the notch in the formwork is gone (see §3.8.3). For the alternatives with smaller girders, this effect is intensified by the extra girders needed. The difference between alternatives 1a and 1b is again explained by the difference in total span length.

Even compared to the cheapest alternative in which existing girders are reused and without considering additional demolition costs, using new girders is still 33% cheaper. The difference in costs for crossbeams and in-situ deck only account for 12% of the difference. So, the girders alone are already 29% more expensive. For the alternatives with girders shortened in width direction the costs for modifications are already higher than manufacturing, transporting and placing new girders.

Nonetheless it should be noted that there is much more experience with a cost estimation for a bridge deck with new girders. As a result, this cost indication is more certain compared to the alternatives in which existing girders are reused. For example, €75.000 is included in the fixing and durability assessment for additional research and structural calculation. However, this entirely depends on the state of the girders and research needed. So, this indication can be an over or under-estimation.

In conclusion, the alternatives with reuse are 33% to 218% more expensive compared to an alternative with new girders. The degree depends on the modifications and the extend to which demolition is included.

§4.8 Multi-criteria analysis and decision

The final steps of the case study are two multi-criteria analysis (MCA) to find the most suitable option. First the decision context and alternatives are discussed. Next the MCA to find the most suitable alternative with existing girders is discussed. This is followed by the analysis to decide between using new girders or reusing existing girders. In the final section the decision is discussed. In the analysis the procedure from §3.9 is used.

§4.8.1 Alternatives in MCA

Until now alternative 1a and 1d are considered in all analysis as these alternatives are identified as structurally feasible. However, the feasibility depends on the layout of the stirrups. Consequently, different sub-alternatives are derived. So, these sub-alternatives are based on different starting points; hence comparing all these sub-alternatives in an MCA is not operatable. In all probability, the stirrup layout can be derived from archives or otherwise with simple non-destructive and non-invasive tests. These tests are possible while the girders are still part of the existing structure. Therefore, it is recommended to derive this layout before a decision is made about the best suitable option. Nonetheless, this is outside the scope of this case study.

To provide insight in the MCA procedure the assumption is made that the stirrup spacing is 100 [mm] around the supports. In this case all the sub-alternatives are feasible but shortening in width direction to 790 [mm] (for alternative 1a) or 750 [mm] (for alternative 1d) is not required for the shear capacity. Consequently, these sub-alternatives are not considered. Nonetheless, a distinction is made between statically determinate and indeterminate system. This distinction is needed because of differences in execution and feasibility. This structural system does not significantly affect the environmental impact or cost indication; hence these analyses did not consider this aspect. The benefits of shortening the girders of alternative 1d in width direction to 1000 [mm] are only applicable in a statically determinate system; according only this system is considered. In conclusion, in the first MCA 5 sub-alternatives are compared. The most suitable alternative is compared in the next MCA with the alternative with new girders.

§4.8.2 MCA between alternatives with existing girders

First the MCA between the alternatives with reusing existing girders is performed. Only the sub-criteria of §3.9.2 that are relevant for this case study are used:

- Contract requirements: profile height is similar for all alternatives and no requirements regarding span division. So, this main criterion is excluded.

- Execution: the support conditions as well as the modifications differ for the alternatives. The criteria are of equal importance, therefore both sub-criteria are included with an equal weight factor.

- Planning: release dates of girders are close too each other, besides the aspects of planning are not sufficiently investigated to include the criterion. So, this criterion is excluded from the analysis.

- Maintainability: a durability assessment is not yet performed. Consequently, the sub-criterion of life span cannot be applied. However, due to the comparable origin of the girders, no significant differences between the alternatives are expected. In none of the alternatives strengthening is applied, therefore the maintainability of modifications is excluded.

- Interface, risks and opportunities: origin of girders is similar, so this sub-criterion is excluded but the sub-criterion of reliability and feasibility is included. The alternatives differ in reliability and feasibility of shear as well as bending moment capacity in SLS and calculation procedure.

- Costs: cost indication is included but impact on structure not, as this is not a decisive criterion for these alternatives. For the costs indication the demolition and deconstruction costs are added to the total estimation. However, €100.000 is subtracted as these are the assumed traditional demolition costs. This is assumption is based on demolition costs of €150, - [1/m^2] and the bridge area of the new bridge.

- Sustainability: only ECI-value is included. Primary material input is comparable for the alternatives and global circularity is with current market conditions not yet of importance.

The scores and remarks on the (sub) criteria are shown in Table 4.13. By assigning weight factors to the main criteria the total score is derived in Table 4.14. The weight factor for sustainability is 1, because the maximum difference in ECI-value is only 17%. The maximum difference in costs is 85%, which is five times a much compared to the ECI-value. Therefore, the weight factor on costs is 5. The execution is not considered as highly important an gets a weight factor of 2. The interfaces risks and opportunities are especially in current market situation very important. Therefore, a weight factor of 8 is used.

Although alternative 1a in a statically determinate system scores 4 out of 4 on 3 of the 4 criteria, the total score is the lowest. This is reasonable as the feasibility of this alternative is highly questionable. The statically determinate options of alternative 1d have a joint third place. With 1000 [mm] wide girders the feasibility increases compared to the 1200 [mm] wide girders, but this benefit is counteracted by additional costs and modifications. The second place is for alternative 1d in a statically indeterminate system. This alternative scores similar to the winner, alternative 1a in a statically indeterminate system, but has higher costs. Therefore, alternative 1a in a statically indeterminate system is identified as the most suitable option with reusing existing girders. Even if other weight factors are applied, this alternative comes out best.

A final remark if the stirrup reinforcement near the supports is 300 [mm] instead of 100 [mm]. In this case, only two alternatives can be considered. Alternative 1a with 790 [mm] wide girders and alternative 1d with 750 [mm] wide girders. The feasibility of alternative 1a is questionable because additional load restrictions are needed. Probably, these load restrictions are not accepted by the client. Consequently, additional strengthening measures are needed. These measures will significantly increase the costs, which counteracts the initial benefit of this alternative compared to alternative 1d. In conclusion, an MCA based on 300 [mm] spacing of stirrups will probably end with alternative 1d with 750 [mm] wide girders.

Table 4.13: Scores on sub-criteria and average score on main criteria in the MCA between alternatives with reusing existing girders under the assumption that the stirrup spacing near the supports is 100 [mm].

Main criteria	Sub-criteria (+ weight)	Alternative 1a statically determinate 1200 [mm]		Alternative 1a statically indeterminate 1200 [mm]		Alternative 1d statically determinate 1200 [mm]		Alternative 1d statically indeterminate 1200 [mm]		Alternative 1d statically determinate 1000 [mm]	
		Remarks	Score	Remarks	Score	Remarks	Score	Remarks	Score	Remarks	Score
Execution	Support conditions Weight = 1	Statically determinate is preferred	4	Statically indeterminate is not preferred	1	Statically determinate is preferred	4	Statically indeterminate is not preferred	1	Statically determinate is preferred	4
	Modifications Weight = 1	No modifications	4	No modifications	4	No modifications	4	No modifications	4	Shortening in width direction	1
	Average		4		2,5		4		2,5		2,5
Interface, risks and opportunities	Reliability and feasibility	Tensile stresses in SLS close to cracking	1	Additional loads due to statically indeterminate not considered	3	Tensile stresses in SLS	2	Additional loads due to statically indeterminate not considered	3	No additional uncertainties	4
Costs	Cost indication	New structure: €605.556 deconstruction €185.930 Total: ≈€700.000	4	New structure: €605.556 deconstruction: €185.930 Total: ≈€700.000	4	New structure: €658.142 deconstruction: €423.690 for Total: ≈€1.000.000	3	New structure: €658.142 deconstruction: €423.690 Total: ≈€1.000.000	3	New structure: €910.080 deconstruction: €538.620 Total: ≈€1.300.000	1
Sustain-ability	ECI-value	€13.818	4	€13.818	4	€13.936	4	€13.936	4	€16.359	1

Table 4.14: Average score on main criteria combined to total score in MCA between alternatives with reusing existing girders under the assumption that the stirrup spacing near the support is 100 [mm].

Main criteria	Weight	Alternative 1a statically determinate 1200 [mm]		Alternative 1a statically indeterminate 1200 [mm]		Alternative 1d statically determinate 1200 [mm]		Alternative 1d statically indeterminate 1200 [mm]		Alternative 1d statically determinate 1000 [mm]	
		Score	Weighted score	Score	Weighted score	Score	Weighted score	Score	Weighted score	Score	Weighted score
Execution	2	4	8	2,5	5	4	8	2,5	5	2,5	5
Interface, risks and opportunities	8	1	8	3	24	2	16	3	24	4	32
Costs	5	4	20	4	20	3	15	3	15	1	5
Sustain-ability	1	4	4	4	4	4	4	4	4	1	1
Total		40		53		43		48		43	

§4.8.3 MCA analysis between alternative with existing girders and new girders

In the second MCA the best suitable alternative from the previous analysis, alternative 1a with 1200 [mm] wide girders in a statically indeterminate system, is compared with the alternative with new girders. Again, only the sub-criteria of §3.9.3 that are relevant for this case study are used:

- Contract requirements: profile height is included. It is not a strict requirement, but with higher girders the road alignment is more complicated. This is due to houses on the east side of the bridge.

- Planning: this criterion is excluded because this is outside the scope of this case study.

- Maintainability: because no durability assessment is performed this criterion is excluded.

- Interface, risks and opportunities: both sub-criteria of material availability and delivery and opportunity for circularity are included. The criterion of material availability and delivery is of slightly higher importance; hence a weight factor of 3 is applied compared to a weight factor 2 on the criterion of opportunity for circularity.

- Costs: cost indication is included but impact on structure not, as this is not a decisive criterion for these alternatives. Cashflow is also excluded because this aspect is outside the scope of this case study. For the cost indication once again, in the alternative with existing girders the demolition and deconstruction costs are added to the total estimation. However, €100.000 is subtracted as these are the assumed to be the traditional demolition costs.

- Sustainability: the combined criteria of ECI-value and material input is included. Impact on the structure is excluded because it is not a decisive criterion for these alternatives.

The scores and remarks on the (sub) criteria are shown in Table 4.15. By assigning weight factors to the main criteria the total score is derived in Table 4.16. The contract requirements are of least importance, wherefore the weight factor is 1. The interfaces, risks and opportunities and the costs are of comparable importance and get weight factor 4. Sustainability is of high importance as it follows from the guiding design principles. Combined with the global environmental objects this criterion gets a weight factor of 5.

It can be concluded that the alternative with new girders scores best, although the difference is small. Even if the weight-factors are changed the differences remain small but in favour of the alternative with new girders. This alternative scores better on all criteria, except the sustainability. However, the difference in the criterion of interfaces, risks and opportunities is small.

A final remark if the stirrup reinforcement near the supports is 300 [mm] instead of the assumed 100 [mm]. In this case, as discussed in the previous section probably the alternative with new girders has to be compared with the alternative 1d with 750 [mm] wide girders. As a result, the material availability and delivery would be slightly more certain. Therefore, this criterion would get a lower weight factor. Nonetheless, the costs significantly increase and the environmental benefit is reduced. Thereupon, the alternative with new girders would certainly get the highest score.

Table 4.15: Scores on sub-criteria and average score on main criteria in the MCA between alternative with reusing existing girders and alternative with new girders under the assumption that the stirrup spacing near the supports is 100 [mm].

Main criteria	Sub-criteria (+ weight)	Alternative 1a statically indeterminate 1200 [mm]		Alternative with new girders	
		Remarks	Score	Remarks	Score
Contract requirements	Profile height	The profile height of the girders is 800 [mm].	1	If equal span division is used 700 [mm] heigh girders are possible, which is preferred for existing houses	2
Interface, risks and opportunities	Material availability and delivery Weight = 3	Uncertain as currently demolition and deconstruction of existing viaducts is started. Feasibility not certain because additional loads for statically indeterminate structure not considered	1	No problems expected	2
	Opportunity for circularity Weight =2	Good opportunity to gain experience, suits with guiding principles of Meinerswijk	2	Traditional approach, no innovations	1
	Average		**1,4**		**1,6**
Costs	Cost indication	New structure: €605.556, deconstruction €185.930 Total: ≈€700.000	1	Total €456.083	2
Sustainability	ECI-value + Material input and origin	ECI-value: €13.818 Primary material input: 3,1E+05 [kg] Secondary material input: 4,8E+05 [kg]	2	ECI-value: €33.539 Primary material input: 9,9E+05 [kg]	1

Table 4.16: Average score on main criteria combined to total score in MCA between alternative with reusing existing girders and alternative with new girders under the assumption that the stirrup spacing near the support is 100 [mm].

Main criteria	Weight	Alternative 1a statically indeterminate 1200 [mm]		Alternative with new girders	
		Score	Weighted score	Score	Weighted score
Contract requirements	1	1	1	2	2
Interface, risks and opportunities	4	1,4	5,6	1,6	6,4
Costs	4	1	4	2	8
Sustainability	5	2	10	1	5
Total		**20,6**		**21,4**	

§4.8.4 Conclusion

From the results it can be concluded that under current circumstances reusing existing girders in the bridge for Meinerswijk is not the most suitable option. The additional costs and risks do not outweigh the sustainability benefits. In this way this case study confirms the current market situation of the reusing existing girders. The innovation is still in the development phase and not yet ready to enter the market. Although the scoring is subjective and the scores should be kept in context the small difference in final scores corresponds with the literature and assumptions made about the stage in the development phase. The innovation is at the end of the development phase so almost ready to the enter the market.

There are several conceivable scenarios in which reusing existing girders would be the most suitable option:

- Performing more detailed calculations to investigate if additional loads due to a statically indeterminate system affect the feasibility of the alternative.
- Communication with Rijkswaterstaat about the availability of the girders and the state of the girders. Moreover, in this way planning and durability aspects may be addressed.
- Communication with the municipality of Arnhem or Rijkswaterstaat about possible subsidizes for implementation of reusing existing girders. In this way the project can be seen as innovation project that contributes to the development of this innovation by providing knowledge and experience. For an innovation project the importance of costs decreases, while the importance of opportunity for circularity and sustainability increase.
- At Meinerseiland two bridges are required. The one investigated in this case study is the larger bridge that provides the main access to the island. There is also a small pedestrian, bicycle and emergency vehicle bridge needed on the other side of the island. In this smaller bridge reusing existing girders is sooner a suitable option. On the one hand, because the risks are smaller. On the other hand, less girders are needed, which is also beneficial for the costs.

On a wider scale governmental regulations can also stimulate reusing existing girders. For example, by setting penalties on the use of primary resources. In this way the difference between costs of alternatives with and without reusing existing girders are smaller. Consequently, the alternative in which existing girders are reused is sooner identified as the best option.

In conclusion, unfortunately the most suitable option for this case study is using new girders. Nonetheless, with stimulating measures and or increased knowledge this will soon change.

§4.9 Review

Besides investigating the possibility of reusing existing girders in Arnhem Meinerswijk the objective of the case study is to support the development of the design approach. To see to which extend this aim is achieved a review is conducted. The observed barriers are discussed in the first section. This is followed by the validity and the limitations of the case study.

§4.9.1 Barriers observed

During the case study barriers for reuse mentioned in the literature and described in §2.6 are observed. These barriers are discussed based on the scope of the design approach, which is discussed in §3.2 and divided into feasibility of design, availability and initiative.

In the case study all three subjects of feasibility, technical, environmental and financial, are addressed. The structural analysis showed that the shear capacity is the governing criteria, which is consistent with the literature. Design alternative 1b was excluded from further analysis due to insufficient shear capacity. Depending on the partly unknown stirrup layout in alternative 1a additional measures may be needed to guarantee sufficient shear capacity. The same occurs with alternative 1d, although only minor modifications might be needed. In addition, regulations and opinions on the shear verification differ, which makes the assessment even more complicated. Moreover, the standards for minimum stirrup reinforcement ratio for new structures cannot be met. For existing structure this minimum is not required. But who decides that the standards for existing structures are sufficient for new structures with reused elements? Because the structural safety should be guaranteed for the next life span. In this way the case study disagrees with literature that indicates that the lack of regulations is a simple argument to not implement reuse.

Another point of technical feasibility came forward during the financial analysis. Deconstruction compares more time and or a higher crane capacity compared to demolition. Consequently, depending on the surroundings of the existing viaduct the costs might substantially increase. Although, both alternatives make use of girders that originate from the same project and are of comparable sizes the costs for deconstruction significantly differ. This is only due to the surroundings of the existing viaducts. This marks the importance of this factor, which was not considered before. Another barrier observed regarding this aspect is the limited information and experience in deconstruction, which leaves a lot of certainty in the analysis.

In this case study the environmental benefit of reuse clearly comes forward in the ECI-value and material input. However, the barrier that these benefits are not translated in financial benefits is observed. From the financial perspective reusing existing girders is not a competitive alternative. In this way this case study confirms what was known beforehand that the financial feasibility of reuse is currently not very high but is expected to improve in the near future. So, this case study cannot be used as example for future cases because the financial feasibility depends on many time-related factors and movements in the construction industry.

In the scope the availability in included to some extend regarding stakeholder management and planning. The experiences from the case study show that most of the stakeholders are willing to implement reused girders. For example, the willingness for reuse was already indicated in the masterplan of the area. However, the resources to turn the ambition into reality is difficult. This relates to the resources in the initiative of the scope. In fact, it is the main objective of the design approach. So, hopefully this barrier is partly removed by the introduction of the design approach. Nonetheless, this case study shows that changing the design approach is not sufficient. Other aspects as the communication in the supply chain have to be modified as well. For example, the interaction between deliverer and user (contractor) of the existing girders and the planning regarding the durability assessment and storage.

§4.9.2 Case study validity

The objective of the design approach is to provide a strategy that is generally applicable to bridge designs. Of course, every construction project is unique and no average project exists. Accordingly, this case study has unique features, which should not end up in the design approach. To avoid generalization difficulties, which is the main disadvantage of a case study [149], the most unique features of the case are discussed. By taking notice of these characteristics and there influence the validity of the case study is proved.

Project requirements

In this case study the freedom of design is large. For example, there are no requirements regarding span division. Only the total span length is established and even on this requirement margin available. Also, construction of the deck and transportation of the girders to the new location are not identified as limiting factors. For many bridge designs this will not be the case and the area underneath the bridge governs the span division and total span length. The surroundings of the new bridge can also impose limiting factors.

Although this might be identified as a tread to validity, it was in fact beneficial for the research. With strict requirements, the girder search would probably have ended with no potentially suitable girders. So, no design alternatives and no case at all but without these strict requirements more design alternatives were possible. As a result, the design approach also includes a way how to deal with many design alternatives and provides a way to select the most suitable ones. This suits with the expectation for the future that more girders become available. Moreover, by including other requirements that could have been there, the design approach is applicable for both projects with limited and with a lot of design freedom.

Project elements

In this case study only the bridge deck is considered, while a bridge consists of many more elements like foundations, abutments, intermediate piers and supports. In addition, a bridge has many interfaces as it is part of larger infrastructural network and part of the surrounding. However, in this case study the span division and girder height corresponded to the original design alternative with new girders. Moreover, as soon as the choice for prefabricated concrete girders is made, it does not matter if existing or new girders are used. Therefore, this unique feature does not form a threat to the validity of the case study as the influence on the other elements and interfaces is limited.

Project type

Although many projects in the construction industry are tender projects, this case study is not. It is part of a larger project of Kondor Wessels to redevelop the area and Van Hattum en Blankevoort is subcontracted for the construction of the bridges in the area of concern. However, the design approach in the early phases of system and preliminary design do not significantly differ. The main difference will be in the MCAs. With a tender project the decision criteria and weight of the criteria will follow from the client. In this case study no specific decision criteria or weight factors were available. Nonetheless possible factors that influence criteria and weight factors are considered. So, no conclusion derived from the case study on this topic are implemented in the design approach. As a result, this is not a thread for the validity.

Duration of project
The last unique characteristics is the duration. For this case study a lot of time was available to investigate the possibility to reuse existing girders, because the project was on a temporary hold due to objections to permits. While in general not a lot of time is available, it is not a thread to validity. The objective of the case study is to assist in the development of a design approach that speeds up the process of implementing reuse. However, to develop this approach and to investigate the opportunities more time is needed. In conclusion, this extra time was needed for this case study, but this time is not generalized in the design approach.

§4.9.3 Limitations on case study
Implementing reuse adds many more dimensions to construction projects, which are already multidimensional from nature. Circularity and reuse involve many interactions within the construction market and is even more iterative compared to traditional projects. Besides, the research behind requires thorough investigation as well. Consequently, not all elements could be investigated in the case study.

First it should be noted that the case study did not start from the beginning of the project. The system requirements where partly available. Furthermore, due to the delay the case study is mostly performed in the time when the project was on a temporary hold. As a result, the case study did not consider any interfaces with other project elements or stakeholder management. Another noteworthy point is that due to the delay some system requirements changed when the project was officially resumed. These changes are not considered in the case study. Consequently, this study cannot be used one to one on the official project.

The other limitation of the case study is the relation between the existing structure and the new structure. Only at the start and at the end of the case study the connection between the existing and new structure was considered. Information about the potentially suitable girders was searched for in the initial phase of the research. During this phase the project of the highway A9 was found. In this project 11 viaducts consisting of prefabricated girder decks had to be demolished. This demolishment or deconstruction is planned between 2023 and 2024. The start of the construction of the bridges in Meinerswijk was not expected to be before the end of 2023. Therefore, it was concluded that regarding planning a match was possible.

Rijkswaterstaat, the owner of the existing viaducts, wants to reuse the girders. So, there were already initiatives going on to extract girders from the viaducts and reuse them. Worth mentioning is that these initiatives for reuse started quite late when the construction works were already contracted and planned. Consequently, releasing the girders from the building contract was difficult. However, this aspect of building contracts is not considered in this research. Based on the information available the durability of the girders is assumed sufficient. Since, the initiative would not have been started if the girders are known to be in a critical condition. Furthermore, some information about the girder types, span lengths and prestress was available. Thus, at the start of the case study attention was paid to the link between existing structure and new structure and information was gathered.

In the next phase of the research the focus was on developing alternatives with the information from the initial phase, hence no information is exchanged between existing and new plans. Due to the research and development of the design approach this phase took longer than in a traditional project. As a result, the project of the A9 continued as well. Only in the final phase when the costs estimation is performed the link is again made to the existing structure. In this phase the deconstruction method and storage came forward again.

The link between existing and new structure would be investigated in the next steps, if the alternative with reuse turned out to be the best suitable option and more time was available for the case study. In that case contact should be made with Rijkswaterstaat and the demolition contractor. In addition, transport, storage and certification arrangements should be made. Furthermore, if not yet performed the durability of the girders should be verified. In conclusion, this case study pays limited attention to the durability aspects in the structural analysis and the connections in the supply chain.

A final remark is about the difference between the existing and new situation. The loads in both situations are calculated to make a comparison. However, this comparison was impeded by the differences in safety factors and calculation methods. Insight in capacity differences and the robustness of the original designs could have been gathered if capacity calculation were also performed according to the original standards, used when existing viaducts are built.

5. Discussion

In this chapter the findings of this research are discussed. First the barriers and limitations are addressed. Next the application of the design approach is discussed. The final paragraph provides a foresight to the future.

§5.1 Barriers and limitations

In §4.9 the barriers and limitations that specifically apply to the case study are discussed. In this paragraph the barriers observed in the development of the design approach and the limitation of the research itself are discussed. The first section provides a review on the scope. Next the steps in the design approach are walked through. So, the girder search is followed by the structural, durability, environmental impact, financial and multi-criteria analysis.

§5.1.1 Scope

This design approach aims at preparing the design process for the implementation of reusing existing girders in new designs. Therefore, this design approach is founded on the assumptions that the end of the development phase for the innovation of reusing existing girders is approaching. After this phase, the construction market should be ready to implement the innovation. Only in this way environmental objectives are achieved and companies can gain competitive positions. Nonetheless, it can be discussed how far the development really is, because also in this research barriers are faced. Only when these barriers are overcome the innovation is ready to be implemented on large scale. Some barriers are already mentioned in §4.9, others are mentioned in the upcoming sections and complicated the development of the design approach.

One of the complicating factors is the limited information and experiences available. Although, many articles are available, the majority of literature provide only general information, address similar aspects or refer to the same projects. In addition, there are only two available reference projects in the Netherlands, which both originate from the same innovation project (SBIR) of Rijkswaterstaat. Furthermore, a lot of research is still going on. So, the final results are not yet available. Therefore, this research might be conducted too early, with as a result a low level of detail. However, a high level of detail is also not required in this phase. When the innovation enters the market, there should be basis on how to implement it. During the introduction phase a lot of experiences and information will be gained. This can be used to adapt, modify and extend the design approach. So, as almost all processes in the construction industry, the development of a design approach can be seen as an iterative process, in which this research provides the first step.

Another point of discussion is the reliability of the research, since it is based on limit information and a single case study, in which multiple alternatives are considered. However, the approach is not about exact values nor about specific procedures, therefore reliability is not of high relevance. It is about providing guidance to users and addressing attention points. Furthermore, uncertain or time dependent factors are excluded from the approach and only mentioned as example or indication. Furthermore, due to the low level of detail and the variety of elements uncertainties are not used as basis for new procedures.

A final point of discussion related to the scope is the interaction between interfaces and elements. This research focuses on bridge decks, but a bridge deck is not an element on its own. Although the interaction is not specifically investigated the interfaces and influences on other elements are addressed if significant influence is expected. For example, in the girder search interaction with the surroundings is addressed. Also, in the environmental impact analysis the impact on remaining parts of the structures is considered and the impact on the structure is a sub-criterion of the costs in the MCA. Furthermore, it should be noted that the type of bridge deck has the most significant impact on the other bridge elements and the interaction with the surroundings. So, these relations are comparable for the alternatives with reused girders and alternatives with new elements, when a prefabricated girder bridge of similar type is used.

§5.1.2 Girder search

Especially in the girder search assumptions are made about the future availability of girders and the exchange platforms. Nonetheless, the search criteria derived are general. Information about the girder length, profile type, profile height and release data will be available in every supply and demand situation. Furthermore, information availability will only increase over time. So, in the future additional or stricter criteria are possible. As a result, less girders are identified as potentially suitable, however a larger proportion turns out to be actually suitable. So, time is saved, while unnecessary primary resources are prevented.

The guiding principles on span division are based on average historical data about bridges in the Netherlands. However, in the future the average length of girders will change. First of all, because the inventory of bridges in the Netherlands is still going on. Secondly, in this research girders that become available in the near future are considered. However, with the developments in the prefabrication industry longer and longer girders are made. Therefore, later in the future longer girders will become available. Also, to derive the environmental limit of shortening, environmental impact data is used. This data is time dependent because shadow costs, calculation methods and regulations might change. Therefore, these factors need review from time to time.

To determine the structural limit of shortening in length direction strut-and-tie models are used to evaluate splitting and spalling forces. These models are used in a way, they are not designed for. In addition, this type of modelling is not done before. So, no information was available to verify the method. On the other hand, the method is only used to get an indication and no detailed values or critical decision are made. To make derive more detailed guidelines and procedures more advanced modelling and verification is needed.

§5.1.3 Structural analysis

The traffic load distribution and shear capacity are the main points of discussion for the structural analysis. Due to the broad scope of this research the loads and load distribution over the bridge deck could not be investigated in detail. As a result, the traffic loads on the girders might be overestimated, while horizontal loads are not considered. However, in the initial design phase, which is the main focus of this research, it is unwanted to perform a lot of detailed structural calculations, as it might be a waste of time when the alternative turns out to be unfeasible. Nonetheless, a more detailed calculation could have provided insight in the influence of the simplification. In this way detailed calculations are not included in the design approach but provide the backbone of guidelines. So, for example recommendation about the reserve capacity needed in this initial phase could have been derived.

The shear capacity is identified as the most critical factor in the structural analysis. The assessment of the shear capacity was also not straightforward. Different standards can be used, NEN-EN-1992-1 for new structures or the RBK for existing structures. In addition, literature suggests that it is possible for girders to meet the existing regulations for shear capacity. However, calculations in the case study showed that this was not always possible. Therefore, the assessment used or the loads considered might be too conservative. Another option is that the literature is too optimistic. Also, the age of the girders has influence, as shear regulations have changed over time. Girders manufactured after 1974 are likely to have sufficient shear capacity. In this way, reusing girders has more potential if girders manufactured after 1974 are used. This would imply that the problem of shear capacity occurs now but will be over when all viaducts before 1974 are demolished or deconstructed. Nonetheless, a new Eurocode is coming in which shear calculations significantly change again. This might also affect the newer girders. As a result, the barrier remains.

§5.1.4 Durability assessment

This research is unable to encompass the entire durability assessment. First of all, there are no experiences from the case study that can be used. The reason is that in some reference projects the girders are not yet released from the existing structures or only the final result is presented. So, if the life span of the girders is guaranteed or not. Therefore, this research is mostly based on the assessment of existing structure and applying this on reused elements. However, assessing the structural safety and residual lifetime of an existing structure, is different than reusing existing elements in new structures because the lifetime should be guaranteed and a higher level of safety is required. Nonetheless, due to a lack of regulations on reusing existing elements it is undetermined what this level of safety should be.

In this research to determine the residual life span the middle way is chosen between the level required for new structures and the level required in the assessment of existing structures. This seems reasonable as there are less uncertainties compared to new structures, because of the already proved performance during the previous service life of the girders. In addition, due to the durability assessment characteristics are determined with more certainty. The level of safety should be higher compared to an existing structure. First, the environmental conditions may change, due to a change of location. Furthermore, uncertainties arise due to the new design and possible damage during execution. In addition, due to a longer service life the probability of severe environmental conditions increases.

Finally, it is difficult to determine the scheduling on when to assess which characteristics. However, as already mentioned this is most likely highly project dependent. It would also depend on the development of the exchange platforms; what kind of information is offered here. It will be most optimal if the durability can be assessed in the first step of the structural analysis or even be included in the girder search. So, for example if the life span and durability class can be added as search criteria. Another point of discussion is who takes responsibility for the assessment and certification? Although not all interactions are addressed this research provides an overview of unavoidable steps, which are independent on the executor, the schedule and the type of girder.

§5.1.5 Environmental impact analysis

During the girder search it is demonstrated that shortening the girders in length direction does not impose a threat to the environmental feasibility of implementing reuse, because factors that govern the structural feasibility are governing. From an environmental perspective the environmental impact reduction is zero if the girder is shortened to around 65% of the original length. The case study shows that shortening in width direction almost linearly increases the ECI-value of an alternative with existing girders, but still leads to a substation reduction in environmental impact. Without shortening the reduction in ECI-value is around 60% and even with maximum shortening the reduction is around 25%. These reductions are consistent with the literature and the expectation. However, in both investigations only a single modification is included and the combined impact is not investigated. So, in other words the impact is investigated on a two-dimensional scale: shortening in one direction versus reduction in ECI-value. However, it could be investigated on a three-dimensional scale. So, shortening in width and length direction on x and y and reduction in ECI-value on the Z-axis. In addition, strengthening the girders might also significantly affect the environmental feasibility. However, these possible strengthening methods are not investigated.

Since, the interfaces between elements and environments is excluded from the scope the environmental impact of the bridge deck cannot be put in perspective. By analysing the environmental impact of the complete bridge, the significance of the environmental impact of the bridge deck could be increased. Nonetheless this is only possible with a detailed calculation in a later stage of the design. A general indication is not possible, as the environmental impact of the total structure depends on specific location dependent factors as soil characteristics and the aesthetics of intermediate piers.

The last aspect of the environmental impact is global circularity. It is important that the global environmental impact of all structures together is as low as possible. Therefore, existing girders should be used in the most suitable project, especially in the future when more and more existing girders are reused in new bridge decks. The most suitable project is the project in which compared to other potential projects, the girders have the least overcapacity as possible and need the least modifications as possible. The definition of global circularity is complicated. Most aspects and influencing factors discussed until now depend on the specific project requirements and characteristics and the girder properties and availability. However, as the name suggests, global circularity has a broader context and the characteristics and availability of surrounding projects have an influence as well.

The environmental indicators used in this research are not able to quantify global circularity. Both indicators do not significantly change, when a bridge deck is slightly over-dimensioned for his purpose. Modifications have a negative effect on these indicators; however these are not necessarily negative for global circularity. Since, girders might only be suitable for reuse in new structures with modifications. Currently to address this aspect a subjective criterion is used in the MCA. In this way structural and environmental considerations can be combined. However, for clarity and uniformity a quantitative indicator is preferred. In the future this indicator might be derived from structural boundaries combined with reference values on material input and ECI-value. For example, Girder type A is only structurally feasible with modification B and a design alternative with girder type A is considered globally circular if the ECI reduction compared to an alternative with new girders is more than €15, - per [m^2]. Another option is to introduce a tender procedure to obtain girders. In this way, the owner of the existing structure receives bids for new structures where the girders can be applied. The bid with the lowest environmental impact wins and this project receives the girders. However, this makes the already complicated procedure even more complicated and involves additional costs and time.

§5.1.6 Financial analysis

This research shows that currently due to more uncertainty, extra time for demolition and material investigations the costs of an alternative with reused girders is much higher compared to an alternative with new girders. Earlier, it was stated that the financial feasibility is based on how much a client wants to pay extra for a lower environmental impact. Although the environmental impact is substantially reduced, this will not yet outweigh the additional costs. The additional costs are only in proportion if the project is seen as innovation. With subsides or investments in circularity implementing reuse of existing girders is possible. However, it is not competitive for use on large scale. In this way the outcome is consisted with the scope, which indicates that reusing existing girders is still an innovation that is not yet ready to enter the market.

According to this research the financial feasibility has to be increased, to make more frequent implementation possible. By gaining more experiences in deconstruction and assessment of elements the costs and uncertainty in the costs for alternatives with reused girders will reduce. Moreover, if material costs increase or penalties are introduced on raw material use alternatives with new girders become more expensive and the difference between reuse and new decreases as well. Governing authorities can play an important role. Apart from introducing penalties or demanding certain percentages of reuse they can help to gain knowledge by subsiding innovation projects. For example, in the case study financial stimulating measures are possible of for example Rijkswaterstaat, which owns the existing viaducts or the Municipality of Arnhem.

§5.1.7 Multi-criteria analysis (MCA)

The final step of the design approach is an MCA. This research proposes the use of two analysis to prevent comparing apples and oranges within the criteria. Also, possible sub-criteria and interpretations on criteria are provided. However, only recommendations can be derived because an MCA is always highly project dependent.

§5.2 Application of the design approach

In this paragraph the application of the design approach is discussed. First the use and users of the approach are discussed. Next some recommendations are done for other actors in the construction industry.

§5.2.1 Field op application

The design approach is intended for developers, engineers and project teams that have to design a bridge deck consisting of prefabricated concrete girders, especially inverted T-girders. In a traditional approach the characteristics of the elements in a design are optimized to the specific requirements of the project. By using the developed design approach the focus will shift from optimizing new girders to investigating the possibilities with reusing existing girders. Therefore, this design approach should be used in the first stages of the project. So, during the system and preliminary design phases.

This change of approach is needed to move forward to a circular economy, in which depletion of natural resources is prevented as much as possible and existing elements are reused. By using this design approach all possible options with reused girders are examined. At the end of the approach the most suitable alternative is selected. This selection is based input defined by the project and the client. Moreover, since a design with new girders is considered as well, it always results in a suitable option.

This design approach provides guidance, but it is not a step-by-step plan that can be used without reconsiderations. The approach requires project specific input and reflection. Moreover, in many steps attention points or considerations are addressed that aim for a discussion within the team.

Currently, reusing existing girders is still in its infancy. Consequently, there are a lot of ongoing developments. As a result, the approach needs to be regularly reviewed to match with the state-of-the art knowledge and experiences. Relatedly, the feasibility of designs with existing girders is still low, which might be used as unfounded excuse to wait with the implementation of considering reuse of existing elements in the design. It is important, that design options with existing elements, although still unfeasible, are already considered. In this way reusing existing elements will become more familiar, which is beneficial in two ways. First, more experiences and knowledge are gained which speeds up the development of the innovation and consequently makes the innovation sooner market ready. Secondly, when the innovation is market ready the wide scale implementation is sooner realised because it is already common practice.

§5.2.2 Recommendation for the industry

Reuse of existing girders is still in the innovation phase and not yet completely ready to enter the market. The innovation is market ready when the financial and structural feasibility is increased and exchange platforms are widely available. Clients and governmental organisations have a major influence on these developments. Governmental organisations by working out exchange platforms and subsidising research to gain more experience and knowledge in the deconstruction and assessment of elements. Clients by discussing the future of elements released from constructions with demolition contractors. In addition, when project teams start to consider the options of reuse and subsides are possible reuse will also become more familiar and knowledge is gained. When the implementation of reuse gains competitiveness this subsides can be scaled back.

When reusing existing girders is feasible and it results in a competitive alternative to using new girders, the client can play an important role in formulating the requirements. By making the requirements as least strict as possible, alternatives with reused girders are sooner suitable. Moreover, if clients give clear preference to reuse and are willing to pay extra, the implantation of reuse will become more common practice. Not only project teams will focus more on reuse, alternatives with reuse will also become more attractive as the weight of sustainability in the multi-criteria can increase. In tender procedures the same result can be achieved when the Most Economically Advantageous Tender (MEAT) or in Dutch ‘Economisch Meest Voordelige Inschrijving’ (EMVI) method is used. By using awarding criteria on sustainability or more specifically on reusing existing elements tenderers are forced to shift their focus to reusing existing elements. These awarding criteria can be evaluated by scoring or a fictional reduction on the tender price.

Governmental organisations might have an influence as well by setting regulations for reuse. For example, by requiring a certain percentage of reused elements in new designs. In this way using existing materials and elements becomes a critical requirement. Design alternatives that do not meet these requirements are not feasible. However, defining such a requirement is difficult. Because suitable existing elements should be available. If no design alternative with existing elements is feasible, the requirement cannot be met. Requiring a certain percentage of reused elements or materials may solve this problem but brings back an already existing problem. In this way, a backup option is to use recycled or recovered materials in new designs. However, in this way the focus can shift again to lower levels of circularity as these options are closer to the traditional process and require less adaptions.

§5.3 Future on the design approach

Altogether in this research many aspects of reuse are addressed. Some aspects are investigated in detail, while others are compared to literature or only addressed as attention points. However, reuse is extremely, almost infinitely multi-dimensional. Moreover, due to the time dependent factors investigating and relating all aspects together requires endless research. So, the construction sector has to adapt, modify and learn by doing. As a result, more research and modifications to the design approach are possible and discussed in this paragraph. First possible modifications, extensions and investigations are discussed. Next possible future use of the design approach is discussed.

§5.3.1 Modifications, extensions and recommendations

First, regular reviews are needed to modify the input of this design approach to experiences, change of regulations and updated data. Also, modifications may be needed to suit the approach to exchange platforms that become available. In this research excel sheets are developed and used for the structural verification. The roadmap and excel sheets could be combined in an automated tool. In this way the assessment of the structural feasibility can be quicker and when an alternative turns out to be unfeasible less time is wasted. Furthermore, this research focuses on inverted T-girders, but could easily be extended to other type of prefabricated girders. Although this research only uses alternatives with existing girders or alternatives with new girders, the approach could be extended to alternatives that combine new and existing girders.

In this research the influence of the height of the in-situ deck is not investigated. The deck-height influences the structural capacity as well as the environmental impact and financial costs. Therefore, it is recommended to investigate this influence and find the optimal deck-height. The second recommendation is related to splitting and spalling stresses in concrete that occur if girders are shortened in length direction and the prestressing force has to be introduced without splitting and spalling reinforcement. In this research strut-and-tie model are used, but these models provide insufficient insight in the stress distribution in concrete. Therefore, it is recommended to investigate this stress distribution in more detail, with for example finite element analysis to produce more detailed guidelines about the structural limit for shortening.

§5.3.2 Change of use

The intention of this research is to guide project teams trough the steps in the design phase that are needed to implement reuse. By reviewing the results the research has also potential to be used in a different setting. If an exchanging platform, for example the Bruggenbank, is further deployed, a form of this design approach can be included. The design approach can be the basis of some sort of material passport. In this passport information about the release date, length, profile height, prestress, reinforcement, concrete quality, residual life span, strengthening possibilities, shear capacity, bending moment capacity and shortening possibilities can be included. In this way, most steps of the structural analysis are moved to the girder search. Consequently, additional search criteria regarding the load and lifetime of the new structure should be included. As a result, a lot of time in the design process is saved as the structural feasibility does not need to be verified. Although the capacity and residual life span should be determined in advance still a lot of time is saved in general. With the approach from this research different alternatives are structurally verified. When an alternative turns out to be unfeasible more or less the same verification has to be performed in another project where the girders are again indicated as potentially suitable. To conclude, including the approach in the girder search would improve the efficiency of the total process and thereupon stimulates the more common implementation of reuse.

Finally, an idealized picture for the future is provided. A tool will be available in which all girders that are already available or become available are included. Based on the information about these girders, the tool calculates the capacity and shortening possibilities. Next, the software generates all possible span divisions and options. Moreover, the software provides an environmental impact and costs estimation. Before a new bridge is designed, this software is used to find the most suitable option with existing girders. This idealized picture is based on structural Panda, which is a software that can be used to generate all possible building layouts. With this software costs and environmental impacts of structurally feasible options are assessed and the most suitable alternative is selected [150].

6. Conclusion and recommendations

This final chapter first provides conclusions, after which recommendations for further research are made.

§6.1 Conclusion

In this research a design approach is developed that pursues a more frequent implementation of reusing existing bridge girders in new design. This approach enters a new field of research and can be indicated as the first step in the follow-up phase of research in reusing existing girders in new bridge deck designs. Current projects and investigations aim at proving that the innovation of reusing existing girders is feasible. This development phase is almost completed and reusing existing girders in new designs is possible. Consequently, in the next phase it needs to be indicated how this innovation should be applied. So, the innovation is almost ready to be introduced on the market. However, the market should be ready to apply the innovation and know how to use it. Speeding up this introduction process is important because many girders will be released in the upcoming years. By implementing reuse of existing girders these girders can be saved from demolition. This is highly valuable because raw material input is saved and environmental emissions are prevented, which contributes to the environmental goals of a reduction of primary resources by 50% in 2023 and a circular economy in 2050.

So, the next phase of research aims as preparing the construction market and providing the necessary tools and guidance to use this innovation. Since, reusing existing elements differs in many aspects from using new elements the construction market has to adapt. One of the elements in this market is the design phase, which is the aim of this research.

The developed design process guides designers/engineers through the different steps of the design and addresses attention and discussion points. In parallel a case study is performed to verify the approach and extend it with practical insights. Hitherto, only innovation projects are conducted to demonstrate the possibilities of using existing girders. In this case study, the aim was to derive an efficient and largely applicable procedure to implement these possibilities. In this way the first sub-research question is answered.

What are the main attention points in a process with reusing existing girders compared to using new girders?

The main focus of the approach is the preliminary design phase, because in this phase most differences occur. Since this preliminary design phase depends on the requirements of the system design, the system design is included as well. Compared to the traditional process the system design should concentrate not only on formulating the requirements, but also investigating the flexibility of the requirements. Requirements that follow from the system design are used together with available information about existing girders to find potentially suitable girders in the girder search. This girder search is an additional intermediate step between system and preliminary design.

Identical to a traditional preliminary design alternatives are generated. Subsequently, equivalent to the traditional process structural calculations are performed. Traditionally these are used to determine the characteristics of the girders, but with reused elements the calculations are used to verify the feasibility of an alternative. Moreover, a durability assessment is needed, which is comparable to assessing an existing structure. This assessment is not comparable to a traditional approach, because in a traditional approach new certificated elements are used.

In the next step the environmental feasibility of implementing reuse is assessed by comparing the environmental impact of design alternatives based on existing girders with design alternatives based on new girders. In a traditional process this step is comparable to the environmental cost indicator (ECI-value). However, this research proposes and additional indicator of material input and origin. In this way the relevant environmental objectives to protect the environment and prevent depletion of natural resources are both included. For both indicators a road map is provided.

Although it might be expected that the environmental feasibility is threatened by modifications to the girder. This research shows that shortening the girders in length or width direction does not impose a direct threat to the environmental feasibility of implementing reuse.

The financial feasibility of implementing reuse is determined in by comparing the costs of design alternatives based on existing girders with design alternative based on new girders. This financial analysis is comparable to a traditional cost estimation. However, the major difference is that with reusing existing girders the costs estimation starts already with the deconstruction of the existing structure. This deconstruction process is more complicated and involves additional costs compared to demolition. These additional costs should be included in the costs for the new construction.

In the final steps of the design approach decisions should be made. In a traditional process this is done by a single multi-criteria (MCA) analysis. However, in a process with reusing existing elements two MCA-analysis are needed. This research provides attention points and possible sub-criteria for these analyses. The first analysis is used to find the best suitable alternative with reused girders. The final decision between using existing or new girders is based on an analysis where amongst other it is decided if the lower environmental impact weighs up against the additional financial costs. The environmental impact and financial analysis provide insight in the differences, but the final consideration is mostly influenced by the client.

During these different steps girders are identified as potentially suitable and alternatives as feasible or unfeasible. This relates to the second sub-research question.

What prerequisites must an existing girder meet to be eligible for reuse?

The first selection on suitability occurs during the girder search. By defining search criteria related to girder type, origin, minimum length, maximum length, profile height and release date potentially suitable girders are selected. Next potentially suitable alternatives are developed with these selected girders. Nonetheless, if the girders are really eligible for reuse in the project of concern is verified during the structural analysis. In this roadmap based on girder properties and durability investigations the girder characteristics are determined. Based on these characteristics it is, if needed, verified if the girders can be shortened until the required length. Next the main loads on the girders are determined after which the shear capacity and bending moment capacity is verified. Girders with insufficient capacity, are not directly identified as unfeasible. First possibilities of shortening in width direction are investigated. Only, when this is not a suitable option and no other alternative is feasible strengthening of the girders might be investigated. When the structural analysis shows that the alternative is feasible, the approach is continued with an environmental, financial and multi-criteria analysis. In these steps it is not needed to prove the feasibility of an alternative. The aim of these analysis is to be able to find the most suitable alternative for the project of concern.

However, to become a circular economy it is important to look beyond a single project. So, the view needs to be widened, which relates to the final sub-research question on global circularity.

How can the design process of one bridge take into account the global overall implementation of reuse of existing girders?

When reusing existing girders is implemented on a large scale, girders should be reused in the best suitable project to ensure that the overall environmental impact of all structures together is as low as possible. This research suggests that the importance of this aspect increases over time with the more frequent implementation and is not sufficiently ensured in the ECI-value and material input and origin. Therefore, the sub-criterion of global circularity is added to the multi-criteria analysis between alternatives for reuse. In this criterion the question is asked if the girders are suitable for other larger projects. In case reusing existing girders results in a bridge-deck with overcapacity in relation to its function and other structural elements the girders have more potential to be used in more complex or bigger projects. In these projects other girder types may not be suitable. However, it should also be reviewed how likely it is that such a project will occur on short notice.

So, in conclusion:

"How should the design process of concrete bridges be adapted to make reuse of existing concrete bridge girders more common practice?"

This research indicates that the first phases of the traditional design approach should be adapted to make implementation of reusing existing girders more common practice. First, searching for girders should be included as intermediate step between system and preliminary design. Furthermore, the setup of the structural analysis should be changed and a durability assessment needs to be added. Moreover, the environmental impact analysis should be extended with an additional indicator and the financial analysis needs to consider additional aspects. Finally, instead of a single multi-criteria analysis, two analyses are needed to find the most suitable design alternative for the project.

Overall, this research provides a setup of a design approach that shifts the view from using new girders to reusing existing girders. By providing roadmaps, attention points and possible procedures this approach guides project teams trough each step of the design. In closing, this research is a first step in preparing and adapting the construction market to implement the innovation of reusing existing girders in new structures.

§6.2 Recommendation for further research

In this paragraph recommendation for further research are made. These recommendations address separate topics that might stimulate the implementation of reusing existing girders. These topics do not necessarily have to be included in a future version of the design approach. Recommendation for the design approach itself are provided in §5.3

Regulations
First of all, it is recommended to investigate regulations for reuse. Currently no standards for reuse exists and this complicates the process om implementing reuse on large scale. Partly because many existing elements are not able to meet the regulations for new elements, which is also confirmed in this research. Especially the regulations regarding shear capacity should be investigated because shear capacity is identified as a critical factor. On the other side, there are aspects which are only applicable to reusing existing elements and not to new elements. For instance, the durability assessment. On these aspects there is no basis at all and without any clarity the uncertainty increases and the willingness to implement the reuse of existing girders decreases.

Splitting and spalling stresses
Secondly, it is recommended to conduct research into splitting and spalling stresses in concrete that occur when girders are shortened and splitting reinforcement is cut off. Tensile splitting and spalling stresses occur, because the prestressing force has to be transferred to the concrete. In this research an attempt is made to model these stresses with strut-and-tie models. However, more advanced research is needed to verify these models and to derive guidelines or calculation procedures to determine the maximum shortening length of girders. From this research it is expected that this maximum shortening depends on the stresses and stress patterns that occur, the tensile strength of existing girders and the length needed to transfer the prestressing force to the concrete. Further research could be in form of Finite Element Modelling (FEM) or experiments with girders.

Financial stimulating measures
This research showed that reusing existing bridge girders is not yet financially viable. Consequently, reuse of existing bridge girders will not be implemented on a regular basis. Currently most research and projects are focused on making reuse a competitive alternative for new by gaining experience. In many literature as well as is in this research some financial stimulating measures are devised. Nonetheless the effectiveness of these measure and the possibilities to implement these measures in society are unknown. Therefore, further research into financial measures that stimulate the implementation of reuse is recommended.

Stakeholder management
The design approach is developed as tool to implement reuse in the design process. This design approach is used internally in the project team to investigate possibilities for reuse and choose the most suitable alternative. Nonetheless, as discussed earlier implementing reuse is multidisciplinary. Therefore, to make reuse of existing bridge girders more common practice not only the design process should be modified, but also the planning and interactions in the supply chain. Therefore, it is recommended to not only develop exchange platforms to bring supply and demand together, but also develop tools or approaches to deal with the additional aspects of reuse. For example, a planning tool or approach on how to deal with the time needed for durability assessment and demolition. Or an approach on how to manage the building contracts for demolition and reuse.

Global circularity

The final recommendation relates to the aspect of global circularity. So, how to ensure that the combined environmental impact of all structures is as low as possible. In this research global circularity aspects are included in a subjective sub-criterion in the multi-criteria analysis. Nonetheless, it is recommended to investigate the possibilities of quantification of this criterion, comparable to an ECI-calculation. In this way a uniform procedure is available. This research indicates that this quantitative indicator may be derived from structural boundaries combined with reference value on material input and ECI-value. However, to be able to derive such an indicator, more experiences and knowledge is needed. Combined with the fact that the importance of global circularity is currently low and will slowly increase over time, it is recommended to wait with the development of this indicator until the end of the market introduction phase of the innovation of reusing existing girders or the start of the growth phase.

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Appendix A: Maximum shortening

The length of girders can be shortened. However, this shortening is not endless. Above a certain maximum it leads to structurally unsuitable girders or it is not environmentally attractive anymore. In this appendix the environmental limit and structural limit of shortening are investigated. In advance it should be noted that these analyses are only used as indication for the maximum. The analysis is not exact, because they are based on many assumptions and average data.

A.1 Environmental limit

The environmental limit is investigated by a cost benefit analysis. Environmental Cost Indicator (ECI) values that are used are mostly based on class three data from the 'Milieu database'. It is first assumed that shortening until the minimum length over height ratio (L/H-ratio) is environmentally feasible. So, for example inverted T-girders have a L/H-ratio between 20 and 28. Therefore a girder with a profile height of 800 [mm] can at least be shortened until $0{,}8 \times 20 = 16\ [m]$. Further shortening is classified as 'over-dimensioning', which is environmentally feasible until the environmental benefit of reuse is equal to the environmental costs of shortening (or over-dimensioning).

Subtracting the costs from the benefits gives a final equation in the form of a parabola: $C_1 \times \Delta L^2 + C_2 \times \Delta L + C_3 = 0$. The coefficients depend on the length L [m], which together with the shortening length, ΔL [m] remain as variables.

A.1.1 Benefit

The reuse factor (H) for unintentional reuse in the environmental impact assessment is 20% of phase A1, A2, A3, C3, C4 and D. So, in these phases the ECI-value of a reused girder is only 20% of the original ECI-value. This original ECI-value is comparable to the ECI-value of a new girder. Other phases, such as A4 (transport) are not considered, because these are similar for both new and existing girders. The ECI-value increases with the height of the profile. So, longer girders have a higher ECI-value per meter length. Based on the ECI-values for 25 [m], 35 [m] and 45 [m] prefabricated girders [142] a linear relationship between ECI-value and length is found. This is shown in Figure A.1.

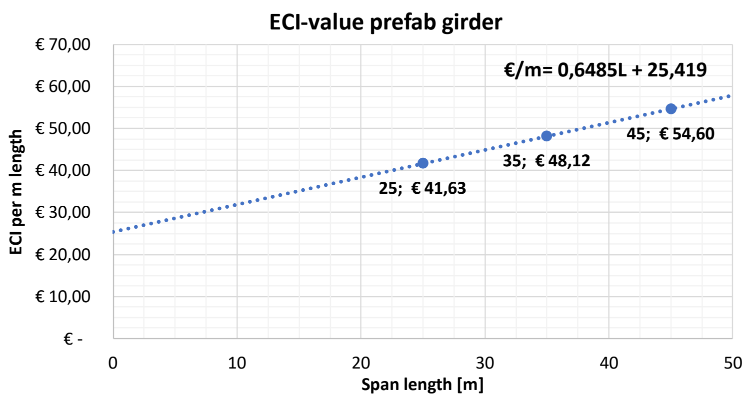


Figure A.1: ECI-value of prefabricated girder per [m] span length.

- ECI-value of new girder = $€_{ECI,girder\ [\frac{1}{m}]} \times L = 0{,}6485L^2 + 25{,}419L$
- ECI-value of reused girder = $0{,}2 \times €_{ECI,girder\ [\frac{1}{m}]} \times (L + \Delta L) = 0{,}1297(L + \Delta L)^2 + 5{,}0838L$
- Benefit = ECI-value new girder – ECI-value old girder
  $Benefit = 0{,}5188L^2 + 20{,}3352L - 0{,}2594L\Delta L - 0{,}1297\Delta L^2 - 5{,}0838\Delta L$
  $C_1 = -0{,}1297$ $\quad C_2 = -0{,}2594L - 5{,}0838$ $\quad C_3 = 0{,}5188L^2 + 20{,}3352L$

A.1.2 Loss of benefit

The girder could have been used somewhere else where shortening is not needed. This loss is only theoretical. Based on the reuse factor, 80% of the ECI-value could be saved on the shortened length.

- $Costs = 0{,}8 \times €_{MKI,girder}\left[\frac{1}{m}\right] \times \Delta L = 0{,}5188\Delta L^2 + 0{,}5188L\Delta L + 20{,}3352\Delta L$

  $C_1 = 0{,}5188$ $\quad C_2 = 0{,}5188L + 20{,}3352$ $\quad C_3 = 0$

A.1.3 Costs of removing concrete deck

If girders are not going to be reused, the in-situ deck and girders will be demolished together with large machines. By reusing prefab girders the in-situ deck has to be removed carefully. This results in a higher environmental impact. For this effect only phase C3 is considered. No ECI-values are known about this process. Therefore, the values are based on ECI-values for demolishing concrete. The ECI-value to demolish a ton of concrete with small machines is €6,38. In case large machines are used this value is €2,68 [151]. Taking the difference and converting the unit gives an ECI-value of €8,88 per m^3 concrete. The deck height is often between 160 and 250 [mm]. For this analysis an average of 200 [mm] is used. The width of most beams is 1200 [mm].

- $Costs = h_{deck} \times w_{girder} \times (L + \Delta L) \times €_{MKI,demolition}\left[\frac{1}{m^3}\right] = 2{,}1312 \times (L + \Delta L)$

  $C_1 = 0$ $\quad C_2 = 2{,}1312$ $\quad C_3 = 2{,}1312L$

A.1.4 Recycling concrete

Due to shortening concrete will be released. This has to be processed and recycled. The amount of concrete is the area of the cross-section multiplied with the shortened length. The area depends on the profile height and type of girder. For this analysis only inverted T-girders are considered. The profile height versus the area for different profile types is shown in Figure A.2. The red line in this figure shows the average for girders between 500 and 1500 [mm]. Only these profile heights are used, because these are most common for span lengths between 12,5 and 30 [m]. This formula for the area is rewritten to the variable length instead of profile height, by applying a L/H ratio of 20. This is the minimum L/H ratio of for inverted T-girders. The ECI-value for processing and recycling concrete is €3,68 per m^3 concrete. This value is based on the ECI-value of phase C1, C2, C3, C4 and D of concrete mixtures [143].

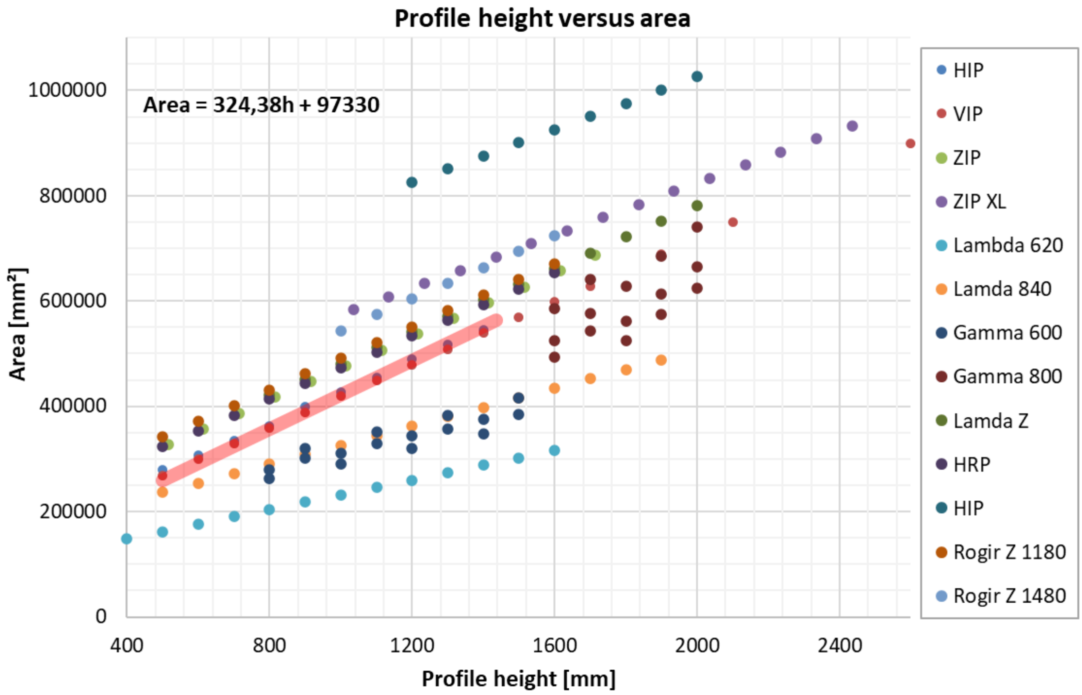


Figure A.2: Profile height (h) [mm] versus area [mm²] of different inverted T-girders

- $Costs = A_{cross-section} \times \Delta L \times €_{MKI,recycling} \left[\frac{1}{m^3}\right]$

  $A_{cross-section} = (324{,}38 \times h + 97330) \times 10^{-6} = 16{,}219 \times 10^{-3} \times L + 97330 \times 10^{-6}$

  $C_1 = 0$ $\quad C_2 = 0{,}0596L + 0{,}3582$ $\quad C_3 = 0$

A.1.5 Extra soil needed

Due to over-dimensioning the profile height is higher than needed. As a result, extra soil is needed, because inclination towards bride or viaduct cannot be increased. For this analysis a maximum inclination of 3% is used. The width of most girders is 1,2 [m] and the maximum L/H ratio of 20 is used again. The ECI-value for moving sand is used, which is €3,16 [152].

- $Costs = Volume_{soil} \times €_{MKI,soil} \left[\frac{1}{m^3}\right]$

  $Volume_{soil} = 0{,}5 \times \Delta h \times \frac{1}{incl} \times \Delta h \times w_{girder} \approx 20 \times \Delta h^2 = 0{,}05\Delta L^2$

  $Costs = 0{,}158\Delta L^2$

  $C_1 = 0{,}158$ $\quad C_2 = 0$ $\quad C_3 = 0$

A.1.6 Actions for shortening

With shortening extra actions are required, which involves environmental impact as well. For example, concrete has to be cut. However, these actions have a very small influence. For example, a concrete saw has an ECI-value of €6,21 per hour. Therefore, they are neglected in this analysis.

A.1.7 Heavier foundation

Due to over-dimensioning the self-weight of the structure is larger. As a result, a heavier construction under the bridge deck is needed. In this analysis three elements will be considered: end beams, bridge piers and foundations piles. For each element first the additional weight on the concrete is calculated. Next it is assumed that this weight introduces a normal force into the element.

End beam

- With higher inverted T-profiles only the height of the web increases. Most profiles have a web thickness of 300 [mm]. However, also smaller thickness between 140-250 [mm] exists. Therefore, average is taken as t = 280 [mm] = 0,28 [m]. So, extra concrete = $\Delta h \times t \times L\ [m^3]$.
- Using the same L/H ratio (named R) of 20 and substituting = $\Delta h \times t \times L = \frac{\Delta L}{R} \times t \times L = U \times \Delta L \times L\ [m^3]$.
- Density of concrete is: $\gamma_{conc} = 24\ [kN/m^3]$ so extra weight of bridge girder = $\gamma_{conc} \times U \times \Delta L \times L\ [\frac{kN}{[m]girder}]$.
- Extra weight of bridge deck per meter width = $\frac{\gamma_{conc} \times U \times \Delta L \times L}{w} = \gamma_{conc} \times \frac{U}{w} \times \Delta L \times L\ [\frac{kN}{m}]$
- Because of extra weight end beams need to have a larger area. Often concrete class C45/55 is used with design concrete strength of $f_{cd} = \frac{45}{1{,}5} = 30 \left[\frac{N}{mm^2}\right] = 30.000\ [\frac{kN}{m^2}]$

  Extra area $= \frac{\gamma_{conc} \times t \times \Delta L \times L}{f_{cd} \times 1000 \times w \times R}$
- Extra volume of end beams per girder = $\frac{\gamma_{conc} \times U \times \Delta L \times L}{f_{cd} \times 1000 \times w} \times w = \frac{\gamma_{conc} \times U \times \Delta L \times L}{f_{cd} \times 1000} = H_{ex} \times U \times \Delta L \times L$

Bridge piers

- Additional volume on bridge piers = $U \times \Delta L \times L + H_{ex} \times U \times \Delta L \times L$.
- This introduces extra weight of = $U \times \Delta L \times L \times \gamma_{conc} + H_{ex} \times U \times \Delta L \times L \times \gamma_{conc} = U \times \Delta L \times L \times \gamma_{conc} \times (1 + H_{ex})$
- Extra area needed = $\frac{U \times \Delta L \times L \times \gamma_{conc} \times (1+H_{ex})}{f_{cd} \times 1000} = H_{ex} \times U \times \Delta L \times L \times (1 + H_{ex})$
- Volume needed = $H_{ex} \times U \times \Delta L \times L \times (1 + H_{ex}) \times h_{pier}$

Piles

- Additional weight on piles =
$$\left(U \times \Delta L \times L + H_{ex} \times U \times \Delta L \times L + H_{ex} \times U \times \Delta L \times L \times (1 + H_{ex}) \times h_{pier}\right) \times \gamma_{conc}$$
- Extra area needed:
$$H_{ex} \times \left(U \times \Delta L \times L + H_{ex} \times U \times \Delta L \times L + H_{ex} \times U \times \Delta L \times L \times (1 + H_{ex}) \times h_{pier}\right)$$
$$= H_{ex} \times (U \times \Delta L \times L) + H_{ex}{}^2 \times (U \times \Delta L \times L) \times (1 + h_{pier}) + H_{ex}{}^3 \times (U \times \Delta L \times L) \times h_{pier}$$
- Volume needed:
$$\left(H_{ex} \times (U \times \Delta L \times L) + H_{ex}{}^2 \times (U \times \Delta L \times L) \times \left(1 + h_{pier}\right) + H_{ex}{}^3 \times (U \times \Delta L \times L) \times h_{pier}\right) \times L_{pile}$$

Total volume needed

$$[H_{ex} \times U \times \Delta L \times L \times (1 + h_{pier} + L_{pile})] + [H_{ex}{}^2 \times U \times \Delta L \times L \times (2 \times h_{pier} + L_{pile})] + [H_{ex}{}^3 \times U \times \Delta L \times L \times (h_{pier} \times L_{pile})]$$

For this analysis 6 [m] is assume for the height of the pier and 15 [m] for the length of the pile.

This volume is multiplied with the ECI-value of €26,56 per [m^3] concrete C45/55 with CEM III [143]. This results in the following values for the parabola.

$C_1 = 0$ $\quad C_2 = 6{,}55 \times 10^{-3} L$ $\quad C_3 = 0$

A.1.8 Cost benefit

For a girder of 25 [m] in the new situation (shortened) this results in the costs and benefits from Figure A. 3. The intersection is located at 12,77 [m]. This means that a girder with a maximum length of 37,77 [m] can be used. So, the girder can be shortened with 34%. From this figure it can be concluded that the heavier foundation has a negligible influence. In fact, only the loss of benefit has a significant influence on the costs, but this type of costs was only theoretical and is not a direct impact. In Figure A.4 this percentage is set out for other girder lengths. When the length of the girder increases the maximum % of shortening allowed decreases. According to this analysis the maximum shortening allowed from an environmental point of view for girders between 12,5 and 30 meters is between 33 and 35%. To derive the search criteria of maximum length the ratio should be derived regarding the span length. This is shown in Figure A.4 b.

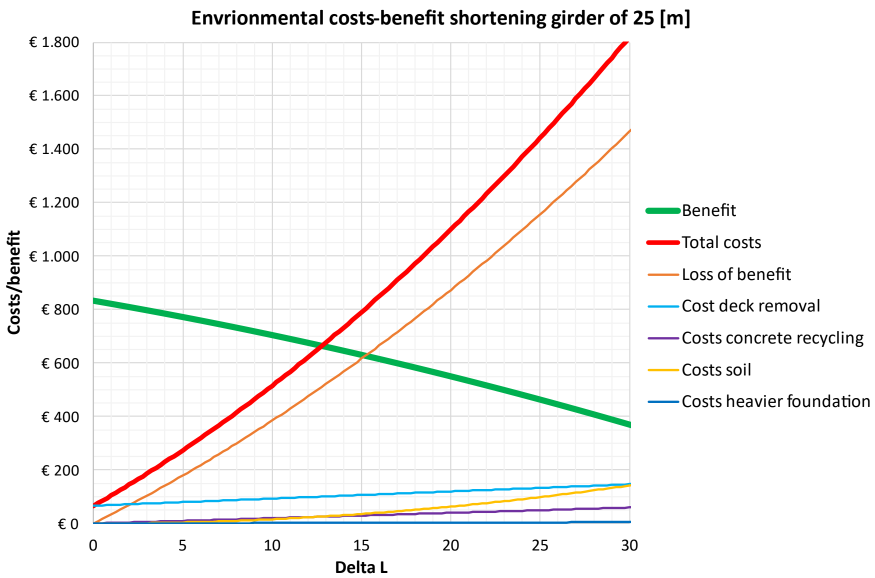


Figure A. 3: Cost-benefit analysis for girder shortened until 25 [m]

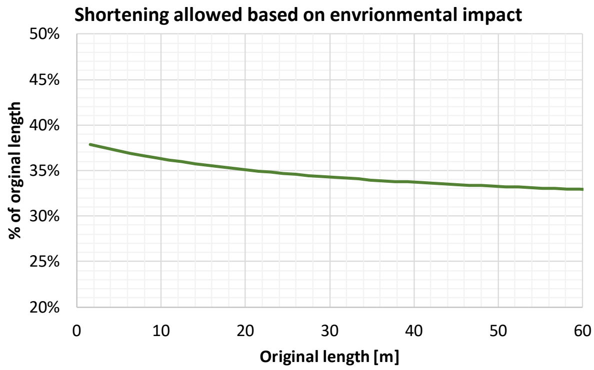


Figure A.4 a) shortening % based on original length

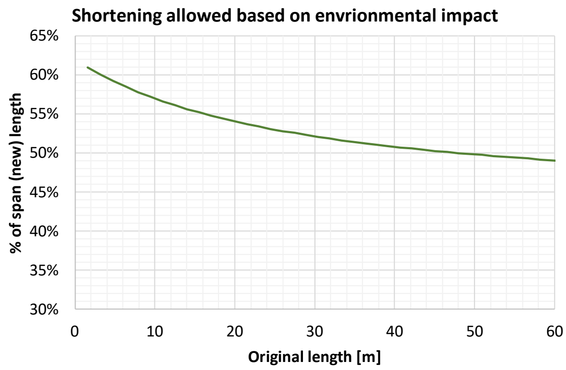


Figure A.4 b) shortening % based on new shortened length

Figure A.4: Environmental shortening limit

A.2 Structural limit

This analysis is based on the results of the strut-and-tie models made in the case study. Shortening until the centre of gravity of the prestressing force is lowered to the point that the prestressing force causes tensile stresses at the top of the cross-section seems feasible. So, shortening until $\sigma_{top} = \frac{P}{A} + \frac{P \times e \times z_{top}}{I_{girder}} = 0$.

From this follows that the prestressing force should remain within the core of the cross-section. The maximum eccentricity allowed is, $e_{max} = \frac{I_{girder}}{A \times z_{top}}$ and depends on the girder type only.

The length after which e_{max} is reached depends on the tendon layout. Since, this layout is highly variable an upper and lower limit are assumed. First all cables are replaced by one equivalent cable with a kink at 25% of the length. Without shortening the cable is assumed to be near the neutral axis of the girder at the support. As minimum height 70% of the height of the neutral axis is assumed and as maximum 90%. After the kink the eccentricity of the cable does not change anymore. The minimum cover to prestressing cables is 50 [mm]. However, since more cables are present not all cables can be located at 50 [mm]. So, the minimum height is 75 [mm]. The maximum is taken as 95 [mm]. This value is based on the maximum in HIP girders from Spanbeton [137]. In Figure A.5 the zone in which the tendon is located is marked. The upper limit is used to derive the maximum length when e_{max} is reached and the lower limit is used to derive the minimum length when e_{max} is reached. This is calculated for inverted T-profiles and plotted against the average length of these girders in Figure A. 6. This average length is based on the average L/H-ratio of 24. To derive a search criterion the shortening percentage is converted to the percentage of the new length in Figure A.7. In this figure also the average between the lower and upper limit is given. This average is used as criteria.

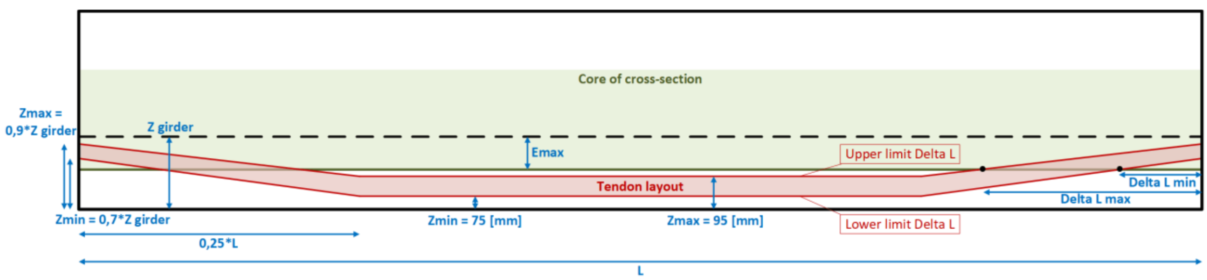


Figure A.5: Side view of girder. Equivalent prestressing tendon is located in the red marked zone.

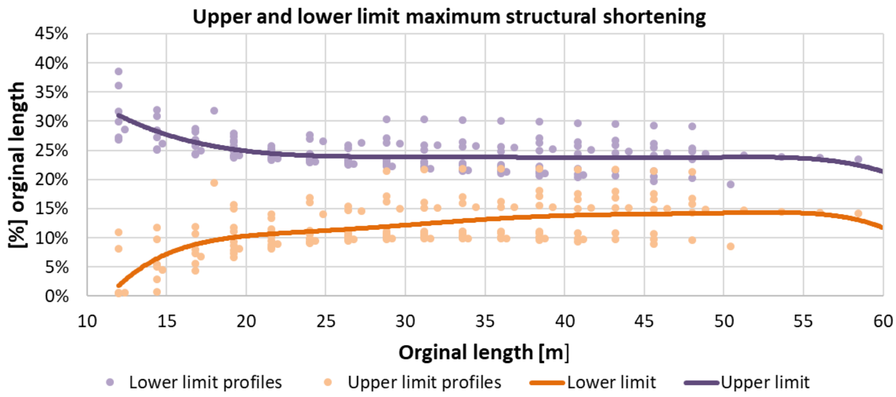


Figure A. 6: Upper and lower limit of structural shortening for different inverted T-profiles. Same profiles are used as in Figure A.2.

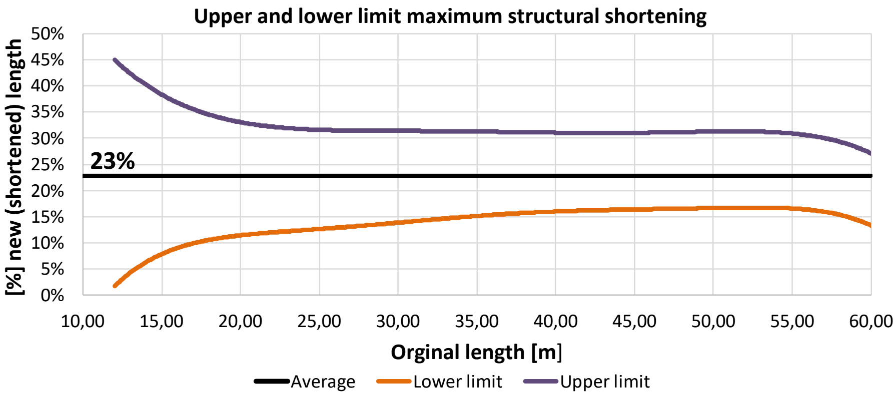


Figure A.7: Upper and lower limit of structural shortening for different inverted T-profiles.

Appendix B: Data for Environmental impact analysis

This appendix provides the ECI-values that can be used in the Environmental impact analysis. In Table B.1 the ECI-values for a in-situ concrete deck are shown. The values are converted to m^3, so can be multiplied with the area and height of the deck. Table B.2 and Table B.3 are both needed to calculate the ECI-value of reused girders. For new girders Table B.4 and Table B.5 can be used. The values in Table B.1 to Table B.5 are based on a LCA report of SGS Search Consultancy and belong to category three data of milieu database [142]. Table B.6 is based on a LCA report of SGS Search Consultancy about concrete mixtures [143], a LCA report of Nationale Milieudatabase on concrete structures [144] and the assumption of 250 kg reinforcement per m^3 of concrete.

Table B.1: ECI-values in-situ concrete deck per [m^3].

Impact category	Total	A1	A2	A3	A4	A5	C1	C2	C3	C4	D
Total	**4,53E+01**	3,69E+01	1,90E+00	1,05E+00	5,73E+00	2,53E+00	2,74E+00	1,93E+00	3,97E-01	1,90E-02	-7,88E+00
Abiotic depletion, non fuel (AD)	1,42E-04	1,09E-04	7,30E-06	6,43E-07	2,20E-05	4,64E-06	1,11E-06	7,42E-06	3,80E-07	2,54E-08	-1,10E-05
Abiotic depletion, fuel (AD)	3,09E-01	2,15E-01	1,89E-02	1,94E-02	5,71E-02	1,91E-02	2,28E-02	1,92E-02	4,72E-03	3,01E-04	-6,75E-02
Global warming (GWP)	2,17E+01	1,76E+01	8,03E-01	7,39E-01	2,42E+00	1,09E+00	1,03E+00	8,14E-01	1,96E-01	6,93E-03	-3,07E+00
Ozone layer depletion (ODP)	9,34E-04	3,54E-04	8,87E-05	3,07E-05	2,68E-04	7,68E-05	1,12E-04	9,01E-05	1,36E-05	1,38E-06	-1,01E-04
Photochemical oxidation (POCP)	3,37E-01	3,51E-01	1,89E-02	4,90E-03	5,71E-02	2,83E-02	4,17E-02	1,92E-02	4,38E-03	2,96E-04	-1,88E-01
Acidification (AP)	5,89E+00	4,32E+00	2,78E-01	1,08E-01	8,41E-01	4,49E-01	6,26E-01	2,83E-01	7,22E-02	4,12E-03	-1,09E+00
Eutrophication (EP)	2,49E+00	1,52E+00	1,25E-01	5,45E-02	3,77E-01	2,12E-01	3,16E-01	1,27E-01	3,68E-02	1,74E-03	-2,86E-01
Human toxicity (HT)	1,36E+01	1,22E+01	5,77E-01	1,11E-01	1,74E+00	6,87E-01	6,58E-01	5,88E-01	7,71E-02	5,10E-03	-3,07E+00
Ecotoxicity, fresh water (FAETP)	6,84E-02	3,94E-02	5,65E-03	8,49E-04	1,70E-02	3,36E-03	3,07E-03	5,74E-03	4,29E-04	4,23E-05	-7,19E-03
Ecotoxicity, marine water (MAETP)	8,87E-01	5,36E-01	6,78E-02	1,39E-02	2,05E-01	4,12E-02	3,48E-02	6,90E-02	5,48E-03	4,84E-04	-8,70E-02
Ecotoxicity, terrestric (TETP)	7,69E-02	7,10E-02	1,36E-03	1,25E-03	4,12E-03	2,56E-03	7,33E-04	1,39E-03	6,93E-04	1,01E-05	-6,26E-03

Table B.2: Base ECI-values prefabricated reused girder per [m] length.

Impact category	Total	A1	A2	A3	A4	A5	C1	C2	C3	C4	D
Total	**9,57E+00**	7,38E+00	1,26E-01	6,22E-02	1,83E+00	1,23E+00	8,11E-01	6,19E-01	2,58E-02	1,54E-03	-2,52E+00
Abiotic depletion, non fuel (AD)	3,75E-05	2,34E-05	4,85E-07	3,81E-08	7,05E-06	3,87E-06	3,28E-07	2,37E-06	2,45E-08	2,08E-09	-1,29E-06
Abiotic depletion, fuel (AD)	8,21E-02	4,08E-02	1,25E-03	1,15E-03	1,83E-02	7,28E-03	6,78E-03	6,20E-03	2,96E-04	2,48E-05	-2,16E-02
Global warming (GWP)	4,30E+00	2,43E+00	5,33E-02	4,37E-02	7,85E-01	4,12E-01	3,06E-01	2,63E-01	1,28E-02	5,66E-04	-9,79E-01
Ozone layer depletion (ODP)	2,50E-04	6,59E-05	5,92E-06	1,80E-06	8,73E-05	2,63E-05	3,30E-05	2,90E-05	9,39E-07	1,13E-07	-2,93E-05
Photochemical oxidation (POCP)	1,39E-01	9,01E-02	1,25E-03	2,92E-04	1,83E-02	1,07E-02	1,24E-02	6,20E-03	2,83E-04	2,41E-05	-6,53E-02
Acidification (AP)	1,44E+00	7,05E-01	1,88E-02	6,38E-03	2,69E-01	1,62E-01	1,85E-01	9,03E-02	4,65E-03	3,32E-04	-3,31E-01
Eutrophication (EP)	5,89E-01	2,43E-01	8,20E-03	3,23E-03	1,22E-01	7,70E-02	9,35E-02	4,08E-02	2,36E-03	1,41E-04	-7,84E-02
Human toxicity (HT)	5,29E+00	3,77E+00	3,84E-02	6,56E-03	5,48E-01	5,40E-01	1,96E-01	1,89E-01	4,96E-03	4,17E-04	-1,02E+00
Ecotoxicity, fresh water (FAETP)	1,90E-02	8,63E-03	3,76E-04	5,03E-05	5,42E-03	1,70E-03	8,98E-04	1,85E-03	2,76E-05	3,45E-06	-2,12E-03
Ecotoxicity, marine water (MAETP)	2,15E-01	9,27E-02	4,51E-03	8,14E-04	6,53E-02	1,88E-02	1,03E-02	2,21E-02	3,51E-04	3,21E-05	-2,33E-02
Ecotoxicity, terrestrial (TETP)	1,91E-02	1,48E-02	9,19E-05	7,37E-05	1,32E-03	2,13E-03	2,16E-04	4,51E-04	4,46E-05	8,18E-07	-1,89E-03

Table B.3: Inclination ECI-values prefabricated reused girder per [m] length[2].

Impact category	Total	A1	A2	A3	A4	A5	C1	C2	C3	C4	D
Total	**2,71E-01**	1,80E-01	4,20E-03	2,08E-03	5,97E-02	3,49E-02	2,69E-02	2,00E-02	8,19E-04	4,63E-05	-5,77E-02
Abiotic depletion, non fuel (AD)	1,00E-06	5,70E-07	1,62E-08	1,27E-09	2,30E-07	9,70E-08	1,09E-08	7,75E-08	7,90E-10	6,10E-11	-3,50E-08
Abiotic depletion, fuel (AD)	2,34E-03	1,00E-03	4,20E-05	3,80E-05	5,95E-04	2,25E-04	2,25E-04	2,00E-04	1,01E-05	7,30E-07	-4,90E-04
Global warming (GWP)	1,23E-01	6,30E-02	1,78E-03	1,46E-03	2,50E-02	1,27E-02	1,02E-02	8,45E-03	4,00E-04	1,68E-05	-2,22E-02
Ozone layer depletion (ODP)	7,53E-06	1,62E-06	1,96E-07	6,10E-08	2,75E-06	8,35E-07	1,10E-06	9,35E-07	2,58E-08	3,31E-09	-6,60E-07
Photochemical oxidation (POCP)	3,70E-03	2,10E-03	4,20E-05	9,60E-06	5,95E-04	3,30E-04	4,10E-04	2,00E-04	9,10E-06	7,10E-07	-1,47E-03
Acidification (AP)	4,18E-02	1,79E-02	6,10E-04	2,13E-04	8,75E-03	5,05E-03	6,15E-03	2,95E-03	1,50E-04	1,00E-05	-7,50E-03
Eutrophication (EP)	1,75E-02	6,30E-03	2,80E-04	1,07E-04	3,90E-03	2,40E-03	3,10E-03	1,32E-03	7,60E-05	4,20E-06	-1,81E-03
Human toxicity (HT)	1,33E-01	8,70E-02	1,28E-03	2,20E-04	1,85E-02	1,36E-02	6,45E-03	6,10E-03	1,60E-04	1,23E-05	-2,36E-02
Ecotoxicity, fresh water (FAETP)	5,39E-04	2,06E-04	1,25E-05	1,67E-06	1,80E-04	4,80E-05	3,05E-05	5,95E-05	8,90E-07	1,02E-07	-4,80E-05
Ecotoxicity, marine water (MAETP)	6,26E-03	2,31E-03	1,51E-04	2,77E-05	2,15E-03	5,50E-04	3,40E-04	7,20E-04	1,14E-05	1,47E-06	-5,40E-04
Ecotoxicity, terrestric (TETP)	4,85E-04	3,60E-04	2,98E-06	2,47E-06	4,30E-05	5,40E-05	7,25E-06	1,42E-05	1,44E-06	2,44E-08	-4,30E-05

Table B.4: Base ECI-values prefabricated new girder per [m] length.

Impact category	Total	A1	A2	A3	A4	A5	C1	C2	C3	C4	D
Total	**2,99E+01**	3,69E+01	6,31E-01	3,11E-01	1,83E+00	1,23E+00	8,11E-01	6,19E-01	1,29E-01	7,70E-03	-1,26E+01
Abiotic depletion, non fuel (AD)	1,33E-04	1,17E-04	2,43E-06	1,90E-07	7,05E-06	3,87E-06	3,28E-07	2,37E-06	1,22E-07	1,04E-08	-6,43E-06
Abiotic depletion, fuel (AD)	2,56E-01	2,04E-01	6,25E-03	5,75E-03	1,83E-02	7,28E-03	6,78E-03	6,20E-03	1,48E-03	1,24E-04	-1,08E-01
Global warming (GWP)	1,44E+01	1,21E+01	2,67E-01	2,19E-01	7,85E-01	4,12E-01	3,06E-01	2,63E-01	6,40E-02	2,83E-03	-4,90E+00
Ozone layer depletion (ODP)	5,49E-04	3,30E-04	2,96E-05	8,98E-06	8,73E-05	2,63E-05	3,30E-05	2,90E-05	4,70E-06	5,65E-07	-1,47E-04
Photochemical oxidation (POCP)	5,07E-01	4,51E-01	6,25E-03	1,46E-03	1,83E-02	1,07E-02	1,24E-02	6,20E-03	1,41E-03	1,20E-04	-3,26E-01
Acidification (AP)	4,38E+00	3,52E+00	9,38E-02	3,19E-02	2,69E-01	1,62E-01	1,85E-01	9,03E-02	2,33E-02	1,66E-03	-1,65E+00
Eutrophication (EP)	1,61E+00	1,21E+00	4,10E-02	1,61E-02	1,22E-01	7,70E-02	9,35E-02	4,08E-02	1,18E-02	7,05E-04	-3,92E-01
Human toxicity (HT)	2,05E+01	1,88E+01	1,92E-01	3,28E-02	5,48E-01	5,40E-01	1,96E-01	1,89E-01	2,48E-02	2,08E-03	-5,09E+00
Ecotoxicity, fresh water (FAETP)	5,53E-02	4,32E-02	1,88E-03	2,51E-04	5,42E-03	1,70E-03	8,98E-04	1,85E-03	1,38E-04	1,73E-05	-1,06E-02
Ecotoxicity, marine water (MAETP)	6,08E-01	4,63E-01	2,25E-02	4,07E-03	6,53E-02	1,88E-02	1,03E-02	2,21E-02	1,76E-03	1,60E-04	-1,17E-01
Ecotoxicity, terrestrial (TETP)	7,92E-02	7,40E-02	4,60E-04	3,68E-04	1,32E-03	2,13E-03	2,16E-04	4,51E-04	2,23E-04	4,09E-06	-9,43E-03

Table B.5: Inclination ECI-values prefabricated new girder per [m] length^2.

Impact category	Total	A1	A2	A3	A4	A5	C1	C2	C3	C4	D
Total	**7,90E-01**	9,01E-01	2,10E-02	1,04E-02	5,97E-02	3,49E-02	2,69E-02	2,00E-02	4,09E-03	2,32E-04	-2,89E-01
Abiotic depletion, non fuel (AD)	3,36E-06	2,85E-06	8,10E-08	6,35E-09	2,30E-07	9,70E-08	1,09E-08	7,75E-08	3,95E-09	3,05E-10	-1,75E-07
Abiotic depletion, fuel (AD)	6,70E-03	5,00E-03	2,10E-04	1,90E-04	5,95E-04	2,25E-04	2,25E-04	2,00E-04	5,05E-05	3,65E-06	-2,45E-03
Global warming (GWP)	3,90E-01	3,15E-01	8,90E-03	7,30E-03	2,50E-02	1,27E-02	1,02E-02	8,45E-03	2,00E-03	8,40E-05	-1,11E-01
Ozone layer depletion (ODP)	1,52E-05	8,10E-06	9,80E-07	3,05E-07	2,75E-06	8,35E-07	1,10E-06	9,35E-07	1,29E-07	1,66E-08	-3,30E-06
Photochemical oxidation (POCP)	1,23E-02	1,05E-02	2,10E-04	4,80E-05	5,95E-04	3,30E-04	4,10E-04	2,00E-04	4,55E-05	3,55E-06	-7,35E-03
Acidification (AP)	1,17E-01	8,95E-02	3,05E-03	1,07E-03	8,75E-03	5,05E-03	6,15E-03	2,95E-03	7,50E-04	5,00E-05	-3,75E-02
Eutrophication (EP)	4,46E-02	3,15E-02	1,40E-03	5,35E-04	3,90E-03	2,40E-03	3,10E-03	1,32E-03	3,80E-04	2,10E-05	-9,05E-03
Human toxicity (HT)	4,88E-01	4,35E-01	6,40E-03	1,10E-03	1,85E-02	1,36E-02	6,45E-03	6,10E-03	8,00E-04	6,15E-05	-1,18E-01
Ecotoxicity, fresh water (FAETP)	1,42E-03	1,03E-03	6,25E-05	8,35E-06	1,80E-04	4,80E-05	3,05E-05	5,95E-05	4,45E-06	5,10E-07	-2,40E-04
Ecotoxicity, marine water (MAETP)	1,63E-02	1,16E-02	7,55E-04	1,39E-04	2,15E-03	5,50E-04	3,40E-04	7,20E-04	5,70E-05	7,35E-06	-2,70E-03
Ecotoxicity, terrestric (TETP)	1,95E-03	1,80E-03	1,49E-05	1,24E-05	4,30E-05	5,40E-05	7,25E-06	1,42E-05	7,20E-06	1,22E-07	-2,15E-04

Table B.6: ECI-values for crossbeam per [m^3].

Impact category	Total	A1+ A2 + A3	A4	A5	C1	C2	C3	C4	D
Total	**6,26E+01**	5,60E+01	2,23E+00	4,22E+00	3,10E+00	2,00E+00	1,63E+00	2,73E-02	-6,63E+00
Abiotic depletion, non fuel (AD)	1,66E-04	1,23E-04	8,18E-06	7,02E-06	1,42E-06	7,56E-06	1,92E-05	3,59E-08	-4,25E-06
Abiotic depletion, fuel (AD)	6,01E-01	4,80E-01	2,27E-02	3,96E-02	2,73E-02	2,01E-02	1,11E-02	4,41E-04	-5,20E-02
Global warming (GWP)	2,94E+01	2,39E+01	9,53E-01	1,91E+00	1,24E+00	8,49E-01	4,95E-01	9,87E-03	-2,63E+00
Ozone layer depletion (ODP)	1,39E-03	8,74E-04	1,07E-04	1,47E-04	1,33E-04	9,44E-05	3,66E-05	2,03E-06	-8,44E-05
Photochemical oxidation (POCP)	9,17E-01	7,69E-01	2,27E-02	4,66E-02	4,39E-02	2,01E-02	1,44E-02	4,23E-04	-2,12E-01
Acidification (AP)	9,57E+00	7,32E+00	3,21E-01	6,70E-01	6,68E-01	2,91E-01	2,92E-01	5,88E-03	-8,68E-01
Eutrophication (EP)	3,48E+00	2,43E+00	1,44E-01	2,92E-01	3,32E-01	1,30E-01	1,48E-01	2,50E-03	-2,31E-01
Human toxicity (HT)	2,23E+01	1,85E+01	6,72E-01	1,13E+00	7,47E-01	6,07E-01	6,13E-01	7,42E-03	-2,80E+00
Ecotoxicity, fresh water (FAETP)	2,18E-01	1,87E-01	7,82E-03	9,94E-03	3,67E-03	6,34E-03	3,06E-03	6,09E-05	8,73E-03
Ecotoxicity, marine water (MAETP)	1,90E+00	1,55E+00	8,65E-02	9,53E-02	4,14E-02	7,39E-02	4,87E-02	7,02E-04	9,63E-03
Ecotoxicity, terrestric (TETP)	8,88E-01	8,51E-01	1,76E-03	3,13E-02	9,55E-04	1,49E-03	1,86E-03	1,38E-05	1,40E-01

Appendix C: Basis for design

In this appendix the guiding principles and requirements for the case study about the to be build bridge at Meinerswijk are discussed. These principles and requirements are related to the specification from the client as well as to standards and guidelines. Together they provide the basis for the structural calculations needed for the constructive design. The case study is only about the design of the bridge deck. Other elements like the foundation and the abutments will not be dealt with. The case study focuses on the system and preliminary design. The final and execution design will not be considered and requirements that are specific for these phases are not included.

In the first chapter the location of the bridge and the redevelopment plans are discussed. Next the relevant standards and guidelines are mentioned. These provide the basis for the more detailed requirements needed for the structural calculation. The third chapter deals with the requirements for the design. Finally in the last chapter the load and load combinations for the bridge deck are discussed.

C.1 Location

Meinerswijk is a natural parc situated in the flood-plain area of river the Rijn approximately a kilometre from the city centre of Arnhem. The municipality wishes to redevelop this area into a flood-plain parc with room for nature, culture, recreation and living. The initiative started in 2007 and will be executed by Kondor Wessels. The subject of the case study is a bridge in this area that provides access to the residential area: 'Meinerseiland'.

This chapter provides a summary of the masterplan of the area [152], [153], which serves as system design for the complete area. The first two paragraphs give an overview of the current and new situation. Next 'Meinerseiland' is discussed. This followed by a description of the distributor road network. Finally, relevant general design principles are discussed.

C.1.1 Current situation

Figure C.1 gives an overview of the existing situation. At Meinerswijk (north side of the area) sport and recreation facilities are located. This area used to be a brick factory and the remaining buildings will be preserved, because of the historical value. On the east side of the 'Eldenseweg' a small harbour that used to belong to a timber company is located. Currently, it is used as water sports facility. In the past the ASM harbour was a shipyard, but at present it is a remote area with some nature. Next to the yard a festival terrain is situated, which is used only a couple times a year.

C.1.2 New situation: stadsblokken Meinerswijk

In the new vision the area is a robust flood plain park of 300 hectares, which can be divided in three parts: Meinerseiland, Harbour of Workum and ASM-site. Figure C.2 provides an overview.

Meinerswijk is transformed from an industrial area with some trees to a natural area with some houses. Around this area two side channels to the Rijn are dug. This important measure to lower the water level is part of the national program to make give rivers more space ('ruimte voor de rivier'). The connection downstream is permanent. So, in dry periods at least one meter of water is present. In case of high water levels, the other side channel is also filled with water and Meinerseiland becomes a peninsula. This is visualised in Figure C.3.

The Harbour of Workum is transformed to a public marina, called 'Stadsblokken'. It is a lively area with history, houses and boats. The ASM-site becomes a lively public area and provides room for housing, work, recreation and nature. Large festivals still take place at the festival location, but smaller festivals take place around the ASM yard.

Figure C.1: Overview of existing situation

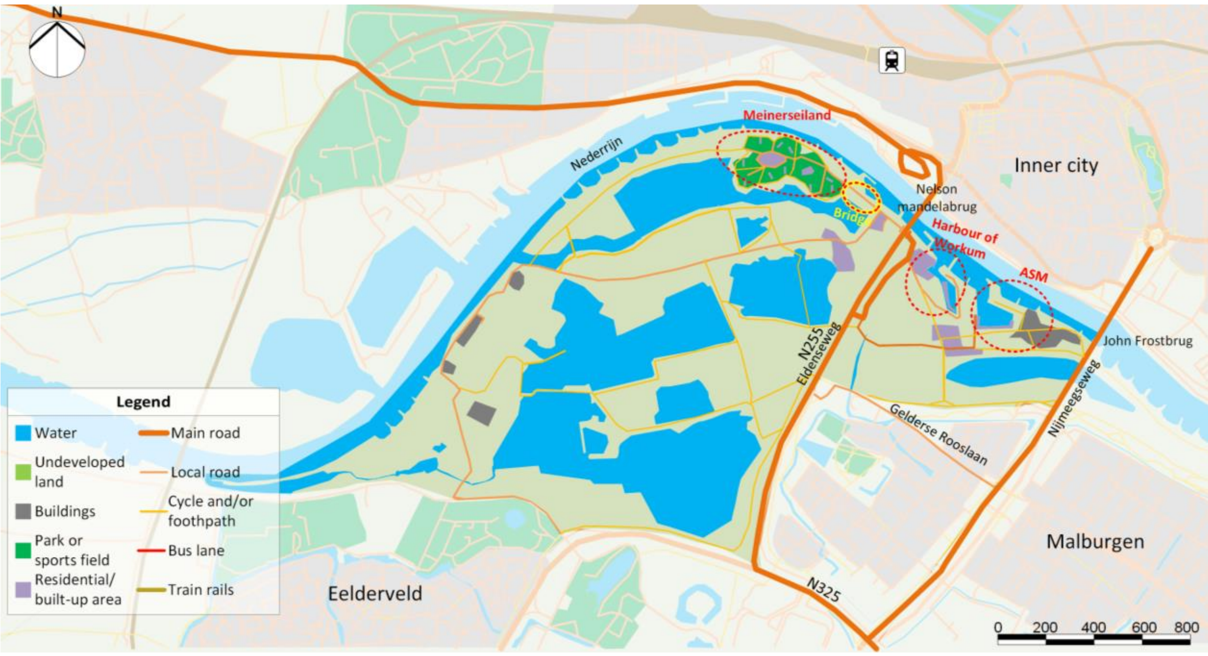


Figure C.2: Overview of new situation

Figure C.3 a) Low water level *Figure C.3 b) Medium water level* *Figure C.3 c: High water level*

Figure C.3: Meinerseiland at different water levels. The bridge of subject is circled in red [152].

C.1.3 Meinerseiland

At Meinerseiland a maximum of 80 houses with a maximum height of 7,5 meters are build. Existing houses keep their function but are renovated. Existing industrial buildings with historical value are kept and used for public functions. These public functions take between 1600 and 2500 [m^2]. The water and the banks of the side channel and the Rijn are used for recreational functions and accessible to everyone. The houses are located in the middle, at higher levels to prevent flooding. An impression is given in Figure C.4.

The soil that will be released from digging the side channels is used to raise the peninsula. Buildings and roads are located at least at +14.00 [m] NAP. This level is higher than the current decisive water level that could occur ones every 1250 years and is therefore safe. The maximum level is +20 [m] NAP, which ensures no significant changes for the environment.

The bridge at the east side (subject of this design) is the main access road to the peninsula. At the peninsula, the road branches into smaller secondary roads to houses and buildings. The profile of the main road is shown in Figure C.5. The concept of shared space is applied. The main road has a width of 3,8 meters and is made of asphalt. On both sides 60 centimetres of grass concrete pavement is applied to provide diversion options. So, the total paved width is 5 meters. In addition, on one side of the road an unpaved footpath is provided. At low water levels the island is also accessible for cyclist, pedestrians and emergency vehicles by a small bridge at the east side. With high water levels this bridge is flooded.

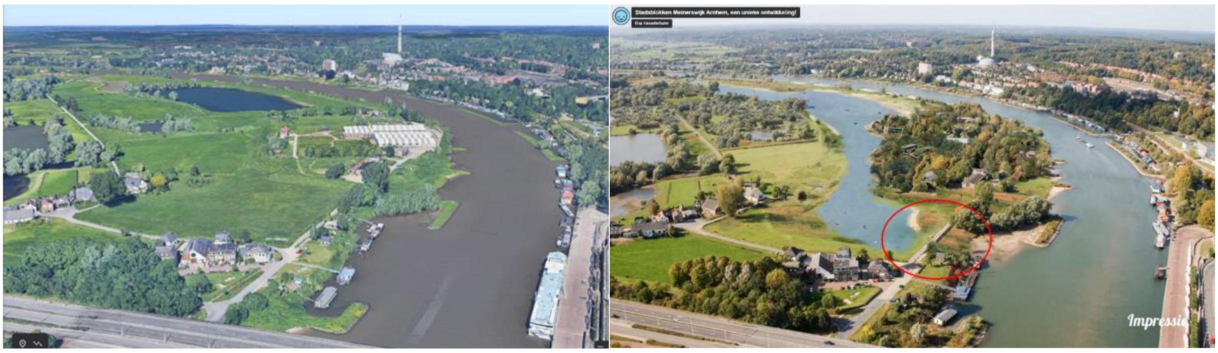


Figure C.4 a) Current situation, 2021 [155] *Figure C.4 b) New situation. Bridge circled in red [156]*

Figure C.4: Impression of east-side of Meinerseiland from Nelson Mandelabrug

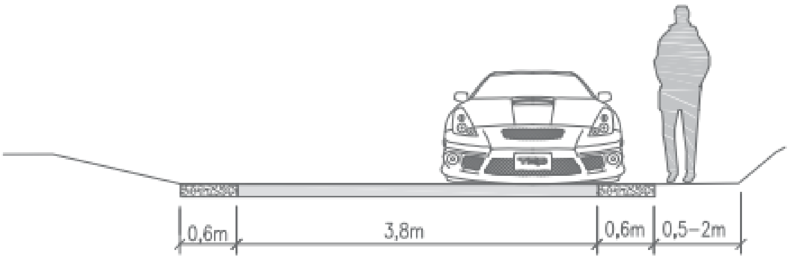


Figure C.5: Cross-sectional view of main road at Meinerseiland [153]

C.1.4 Access roads

At the Eldenseweg (N255) there is only an exit towards Meinerswijk for traffic from the inner city (north side). Traffic from the south side of Arnhem can access the area by using the ‘Gelderse Rooslaan’. Moreover, this traffic is forced to use a part of a bus lane. In addition, there are multiple intersections with cyclist. Because of predicted increase in traffic intensity, inefficiency of the existing network and dangerous intersections the access road network is improved.

At one intersection with the Eldenseweg motorized traffic can enter the area or leave the area in northern or southern direction. Traffic enters the area at the west side. Then the road branches towards the festival site or continues to the other areas. The second branch is towards the harbour of Workum. The road continuous and underpasses the Eeldenseweg. After this underpass Meinerseiland can be reached by the new bridge. The speed limit at the Eldenseweg around Meinerswijk is lowered from 70 [km/h] to 50 [km/h]. The speed limit at the entrance and exit road is also 50 [km/h]. In Meinerswijk the speed limit is 30 [km/h] since it is a residential area. So, the bridge is in the transition zone between 50 and 30 [km/h].

Using the definitions from the handbook of road design by CROW [157] at the location of the bridge the road can be classified as 'erftoegangsweg type I' with the focus of providing access to properties. The roads at the island can be classified as 'erftoegansweg type II', the access road to the area is classified as 'gebiedsomslutingsweg' or distributor road. The main function of the Eeldenseweg is to provide traffic flow and therefore this road is classified as 'stroomweg'.

C.1.5 Design principles

The masterplan and vision of the area provides many guiding principles for design [152], [153], [154]. A list of the principles relevant for the bridge design is provided.

- *Sustainability is a guiding principle. Obligation towards durable and sustainable design, because the area is a flood plain area inside the city centre. In this area river, nature development, infrastructure, living and recreation come together. The ambition is to build climate-proof, nature inclusive and energy neutral. Materials released in the area can be reused. In addition, circularity in new to build structures can play a significant role. There are many developments in the field of circularity. Something that is currently seen as innovative may be standard practice within a few years. However, for Meinerswijk innovation and being in the lead is prevailing standard.* This design principle suits perfectly with a bridge design based on existing girders.

- *Foster the current character of the area. The area should feel natural and not as designed.* A bridge design with existing girders can contribute to this feeling. Instead of a completely fresh and new look the bridge might look like its laying there for years already. Moreover, the design may result in an asymmetric bridge which contributes to the more natural and robust history of the area.

- *Inhabitants should always be able to reach their homes. Moreover, together with visitors they want to reach their homes or destinations inside the city as soon as possible.* To achieve this principle the bridge should have sufficient width and capacity to ensure a steady traffic flow without delays.

- *The area is developed as natural park. Consequently, pedestrians and cyclist are prioritized over car traffic. In favour of the environment car use and ownership is discouraged. This is possible, because of the availability of other modes of transport. So, other modes of transport and shared car use are preferred. However, the connection to the city is important and should be improved for all types of traffic.*

- *The bridges should be designed simple and horizontal. They should not draw attention in the river landscape. Instead, they should match with the wide river landscape with long sight lines.* This bridge design is in line with this principle because it will be a simple flat concrete girder bridge, so no pylons or high elements on the bridge are needed.

C.2 Reference documents

This chapter provides an overview of the standard and guidelines applicable to the project.

C.2.1 Standards

The standards shown in Table C.1 provide the basis for the design.

Table C.1: Standards used as basis for design.

	Standard	Addition	Date	Title
1.	NEN-EN 1990		2019 2019	Basis of structural design + National Annex
2.	NEN-EN 1991-1-1	C1, C11	2019 2019	Action on structures – Part 1-1: General actions – Densities, self-weight, imposed loads for buildings + National Annex
3.	NEN-EN 1991-1-4	A1, C2	2011 2020	Actions on structures – Part 1-4: General actions – Wind actions + National Annex
4.	NEN-EN 1991-1-5	C1	2011 2019	Actions on structures – Part 1-5: General actions – Thermal actions + National Annex
5.	NEN-EN 1991-1-7	C1, A1	2015 2019	Actions on structures – Part 1-7: General actions – Accidental actions
6.	NEN-EN 1991-2	C1	2015 2019	Action on structures – Part 2: Traffic loads on bridges + National Annex
7.	NEN-EN 1992-1-1	C1 A1	2011 2016	Design of concrete structures – Part 1-1: General rules and rules for buildings + National Annex
8.	NEN-EN 1992-2	C1	2011 2016	Design of concrete structures – Concrete bridges – Design and detailing rules + National Annex

C.2.2 Guidelines

The guidelines shown in Table C.2 provide information used in the basis for design. These guidelines are not normative for this project. The ROK is not normative for this project, because the structure is not owned by Rijkswaterstaat. Nonetheless, some requirements from the ROK can be used as basis for design. For the bridge existing girders will be reused, whenever possible. The RBK and the background rapport provide valuable information regarding the characteristics of these existing girders.

Table C.2: Guidelines used as basis for design.

	Guideline	Publisher	Date	Title
9.	RTD 1001	Rijks-waterstaat	2021	Richtlijnen Ontwerp Kunstwerken (ROK) (Guidelines for design of civil objects)
10.	RTD 1006 [52]	Rijks-waterstaat	2022	Richtlijnen Beoordeling Kunstwerken (RBK) (Guidelines for assessment of civil objects)
11.	TNO-2022-R10927a [46]	TNO	2022	RBK 1.2 Achtergrondrapport Beton (Background rapport concrete)

C.3 Basic Requirements for design

This chapter deals with the basic requirements of the bridge. In the first paragraph the dimensions in length and width direction are discussed. This is followed by the traffic intensity that the bridge should manage. In the third paragraph the design life and consequence class of the bridge are discussed. This is followed by the material characteristics in the fourth paragraph. Finally, the durability requirements are discussed.

C.3.1 Bridge dimensions (profiles)

The bridge is located between Meinerswijk and De Praets. The total length that needs to be bridged is 107,21 [m]. In the existing drawing from the system design the span is divided in three mains spans of 25 [m] and two sides spans of 16,1 [m]. However, this division is not a strict requirement and variations are possible.

At both sides of the bridge the land level is + 14,00 [m] NAP. The maximum heigh water level will be +13,40 [m] NAP. To prevent the bridge from flooding the bottom of the bridge deck is situated around 13,90 [m] NAP. This level is not a strict requirement but, the bridge deck should not be in contact with the water. Especially not since in that case other exposure classes apply. Due to the profile height of the bridge the top of the road will be situated higher than the surround land. It will be around +15.05 [m] NAP. This is also not a strict requirement. However, in case the profile height increases more soil and road length is needed to bridge the height difference. Underneath the bridge the soil starts with a clay layer at +12.00 [m] NAP. The length dimensions and height levels are visualized in Figure C.6.

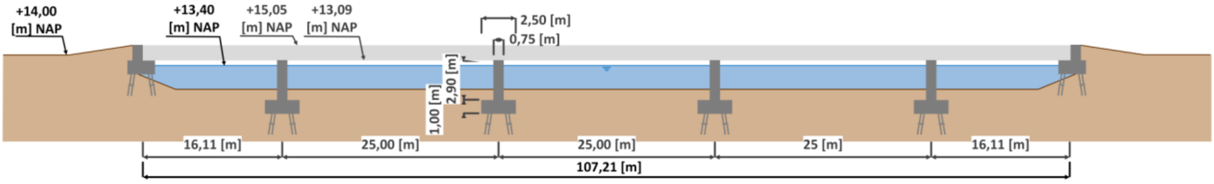


Figure C.6: Length profile of bridge. Measures in black are strict requirements. The measures in grey give an indication.

The width of the bridge is determined by the cross-section of the roads on both sides of the bridge. These profiles should match to ensure a continuous road. The road provides the transition between the distributor road Eeldensedijk and the property access roads at Meinerseiland and is classified as ‘Erftoegangsweg type I’. The common cross-sectional dimensions for this road type are shown in Figure C.7 and Table C.3.

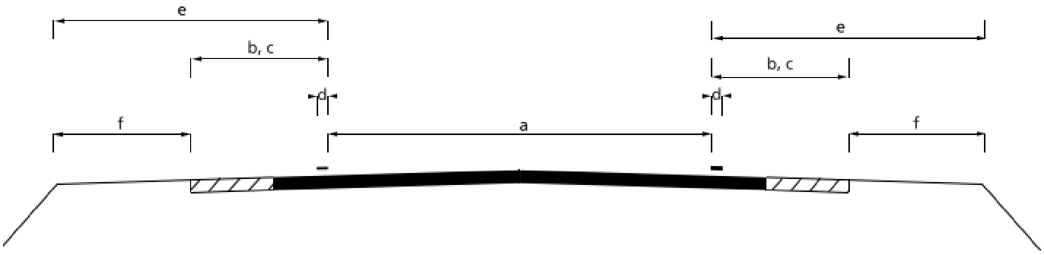


Figure C.7: Cross-sectional profile of 'erftoegangsweg type I' the dimensions are shown in Table C.3 [157]

Table C.3: Cross-sectional dimensions of road type: ‘erftoegangsweg type I’

Letter in Figure C.7	Name	Dimensions		
		Normal [m]	Minimum [m]	Maximum [m]
a.	Driving lane	3,50	3,50	4,00-4,50
b.	Diversion lane	0,50	0,25	1,25
c.	Suggestion lane	1,25	1,00	1,75
d.	Marking	0,10	0,10	0,10
e.	Obstacle free zone	1,50	1,50	-
f.	Outer berm	2,50	1,50	-

At Meinerseiland traffic can use the driving lane or the paved side path or exceptionally the berm area to pass each other. However, at the bridge this is not possible. Therefore, the bridge should have sufficient width to let two vehicles pass each other. In addition, cars should be able to pass cyclist without hindrance. However, a smooth transition at both sides of the bridge should be guaranteed. Since, the bridge is only used to access the island and the traffic intensities are low all traffic will use the same area. A traffic lane of 4 [m] is used with two diversion lanes of 0,5 [m]. Together this adds up to 5 [m], which is comparable to the paved width at the road at Meinerseiland. On both sides of the bridge 0,5 [m] will be reserved for railing. Therefore, the total width of the bridge will be around 6 meters. With this width cars and large vehicles can pass vehicles and two small vehicles can pass each other carefully as well. The cross-sectional profile is shown in Figure C.8.

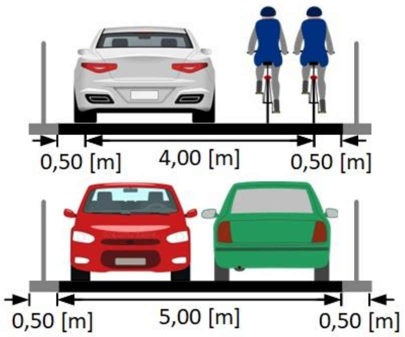


Figure C.8: Cross-sectional profile of bridge

C.3.2 Traffic intensity

Information about the traffic intensity is required to determine load reduction factors. This information is derived from research about the acoustic emission in the environment. In this research two situations for 2030 are investigated. The difference between these situations is the division of recreational space over 'Meinerseiland' and 'Stadsblokken'. For this design, the scenario in which the centre of gravity is located at Meinerseiland is used, because this causes a higher traffic intensity over the bridge. Instead of an intensity based on an average day including the weekends, the intensity in an acoustic emission research is based on a working day. As a result, the traffic intensity found can be considered as the worst-case scenario for 2030. It would have been more accurate to convert the intensity to a weekday intensity with a municipality dependent conversion factor. For Arnhem this factor for heavy traffic is 0,796 [158].

In this research three traffic categories are distinguished: light, medium weight and heavy. Cars, vans and trucks on four wheels belong to the light category. Busses and trucks with two axels to the medium weight category. Trucks with three or more axles, trucks with trailers and tractors with trailers belong to the heavy category. This categorization differs from the categorization used in the Eurocode for traffic load. According to the National Annex of NEN-EN 1991-2 vehicles heavier than 3500 [kg] should be considered as heavy traffic. The maximum weight of a van is 3500 [kg]. So, to translate the acoustic intensity to the load intensity the medium and heavy acoustic categories are both considered as heavy load category.

For the worst-case scenario 1700 vehicles a day are expected to pass the bridge in 2030. The division over the acoustic categories light, medium and heavy is 97,5%, 2,2% and 0,3% respectively. So, the total expected number of heavy load vehicles that passes the bridge in 2030 is approximately 16.000. Since, the bridge has a design life of 50 years this number should be multiplied with a correction factor of 1,5 to account for a future increase. Therefore, the design value of heavy traffic is 23.000 vehicles a year. More detailed values and calculations can be found in Figure C.9.

Traffic intensities

Table 1: Worst case scenario

Year	2030	
Vehicles	1700	[Vehicles/day]

Table 2: Definiton day, evening and night

Day division			
	Start	End	[hours]
Day	7:00	19:00	12,00
Evening	19:00	23:00	4,00
Night	23:00	7:00	8,00
Total			24,00

Table 3: Divison of traffic over day.

	Hour intensity	vehicles [vh/hour]
Day	7,0%	119
Evening	2,6%	44,2
Night	0,7%	11,9

Table 4: Division type of traffic over day

	Light	Medium	Heavy
Day	97,5%	2,2%	0,3%
Evening	98,0%	1,7%	0,3%
Night	96,4%	2,7%	0,9%

Table 5: Number of vehicles per hour. Calculation based on Table 3 and 4

	Light	Medium	Heavy
Day	116,03	2,62	0,36
Evening	43,32	0,75	0,13
Night	11,47	0,32	0,11

Table 6: Total number of vehicles a day. Calculation based on Table 2 and 5

	Light	Medium	Heavy	Total
Day	1392,30	31,42	4,28	1428,00
Evening	173,26	3,01	0,53	176,80
Night	91,77	2,57	0,86	95,20
Total	1657,34	36,99	5,67	1700,00
%	97,5%	2,2%	0,3%	100,0%

Table 7: Total number of heavy load vehicles a year. Calculation based on Table 6

Medium + Heavy vehicles a year	15.572
Factor for 50 years	1,5
For load model	**23.358**

Figure C.9: Calculation of heavy traffic intensity for load consideration

C.3.3 Design life and consequence class

The consequence class of the bridge is CC2, conforming the National Annex B1 of NEN-EN 1990. The bridge will be used by more than 2000 trucks a year but is not part of the main road infrastructure. Consequently, the design life of the load carrying structure is 50 years. The design working life of replaceable element is 25 years according to table 2.1 of NEN-EN 1990. So, this holds for the parapets and the bearing blocks. However, if the guideline from Rijkswaterstaat about bearing blocks (RTD 1012) is followed the working life of the bearing blocks should be 50 years. The design life of the joints is 40 years, however for replaceable elements in it the design working life is 10 years. Finally, the parapets have a design working life of 25 years as well. The design lives of the elements are summarized in Table C.4.

Table C.4: Design life of elements

Element	Design life [years]
Load bearing structure	50
Parapets	25
Bearing blocks	25-50
Joints total	40
Replaceable elements in joints	10

C.3.4 Material characteristics

Asphalt, concrete, reinforcing steel and prestressing steel are the main materials in this design. Therefore, their characteristics are described.

Density

Table C.5 gives an overview of the densities of the materials.

Table C.5: Material densities

Materials	Density [kN/m^3]
Reinforced concrete	25
Concrete	24
Asphalt	23

Concrete

In the design different strength classes will be used for different elements. The bridge is made of prefabricated concrete girders with a cast in-situ deck. The substructure is made of concrete abutments and piers and founded on prefabricated concrete piles. Starting point of this case study is the use of existing prefabricated girders. So, the strength class and material characteristics depend on the girders available. The design of existing girders is often based on old strength class K600, which is comparable to C40/50. However, with testing a higher strength, comparable to C55/67 is often found. For the cast-in-situ parts C30/37 is used. So, the in-situ-deck, the abutments and piers. The existing cast-in situ deck is often made with concrete class K300, which is comparable to C19/22. The foundations piles are made of C45/55. The characteristics of these strength classes are shown in Table C.6. A poison ratio of $\upsilon = 0{,}2\,[-]$ is used for uncracked concrete and of $\upsilon = 0\,[-]$ for cracked concrete. A coefficient of thermal expansion of $\alpha = 8 \times 10^{-6} \left[\frac{1}{K}\right]$ is used.

Table C.6: Concrete characteristics

Class	f_{ck} [MPa]	$f_{ck,\,cube}$ [MPa]	f_{cm} [MPa]	f_{ctm} [MPa]	$f_{ctk,\,0,05}$ [MPa]	$f_{ctk,\,0,95}$ [MPa]	E_{cm} [GPa]	ε_{c3} [‰]	ε_{cu3} [‰]
C19/22	19	22	27	2,1	1,5	2,8	29	1,75	3,50
C30/37	30	37	38	2,9	2,0	3,8	33	1,75	3,50
C40/50	40	50	48	3,5	2,5	4,6	39	1,75	3,50
C45/55	45	55	53	3,8	2,7	4,9	36	1,75	3,50
C55/67	55	67	63	4,2	3,0	5,5	38	1,82	3,13

f_{ck}	Characteristic cylindrical compressive strength after 28 days	$f_{ctk,\,0,95}$	Average axial tensile strength 95% fraction
$f_{ck,\,cube}$	Characteristic cubical compressive strength after 28 days	E_{cm}	Elastic modulus
f_{cm}	Average cylindrical compressive strength	ε_{c3}	Crushing strain when reaching f_{cd}
f_{ctm}	Average axial tensile strength	ε_{cu3}	Maximum value of crushing strain
$f_{ctk,\,0,05}$	Average axial tensile strength lower 5% fraction		

Reinforcement steel

For new reinforcement B500B is used. The characteristics are shown in Table C.7. The characteristics of the reinforcement applied in the existing girders should be derived from archives or material testing. Commonly applied reinforcement types are QRn40 and QRn48, for which the translated characteristics are shown.

Table C.7: Reinforcement steel characteristics

Type	f_{yk} [MPa]	f_{yd} [MPa]	ε_{uk} [%]	E_s [GPa]
B500B	500	435	5	200
QRn40	400	348		210
QRn48	480	417		210

f_{yk} Characteristic yield strength; ε_{uk} Maximum allowed strain
f_{yd} Design yield strength; E_s Modulus of elasticity

Prestressing steel characteristics
The type of prestressing steel depends on the existing girders. QP190 and QP 200 are regularly used in prefabricated girders. The characteristics of these types translated to current Eurocode formulation are shown in Table C.8.

Table C.8: Prestressing steel characteristics

	$f_{p,k}$ [MPa]	$f_{p,k}/\gamma_s$ [MPa]	$f_{pk,0,05}$ [MPa]	f_{pd} [MPa]	ε_{pu} [%]	σ_{inital} [MPa]
QP190	1864	1694	1619	1472	3,5	1212
QP200	1962	1784	1717	1561	3,5	1079

$f_{p,k}$ Tensile strength; $f_{p,k}/\gamma_s$ Tensile strength divided by material factor
$f_{pk,0,05}$ Tensile strength lower 5% fraction; f_{pd} Tensile stress at location of kink in stress-strain diagram
ε_{pu} Rupture strain; σ_{inital} Originally allowed initial stress

Partial material factors
The partial material factors are provided in Table C.9 and based on table 2.1 from NEN-EN 1992-1-1. A distinction is made between normal design situations and accidental combinations. The normal design situations include the permanent and temporary load combinations.

Table C.9: Partial material factors

Design situation	Concrete γ_c	Reinforcement γ_s	Prestressing γ_s
Normal	1,5	1,15	1,1
Accidental	1,2	1,0	1,0

C.3.5 Durability and concrete cover
The exposure classes are based on table 4.1 of NEN-EN 1992-1-1. The foundation piles are situated in permanent wet conditions because the top of the piles is situated below the ground water table. As a result, XC1 applies. In case of chloride containing soil XD2 should be applied as well. However, the soil conditions are not yet known.

The bridge girders and in-situ deck are cyclic wet and dry and exposed to splash water and de-icing salts. Therefore, they should be designed on XC4, XD3 and XF4. The same conditions hold for the intermediate bridge piers and the top side of the abutment. The sides of the abutment can be designed on XC2, because these elements are permanent in wet soil. In case the soil contains chlorides XD2 should be applied as well. The exposure classes for the different elements are shown in Figure C.10. Class XF is relevant for the concrete mix design but has no influence on the concrete cover.

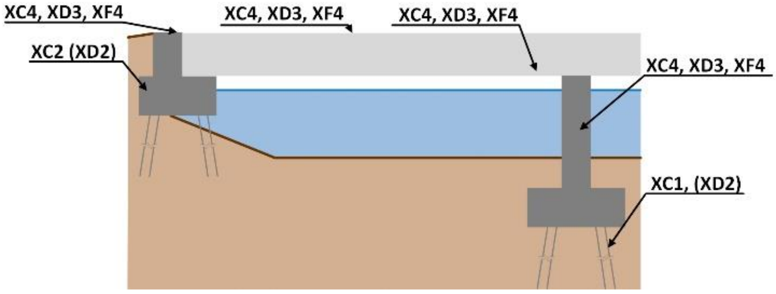


Figure C.10: Exposure classes in bridge

Apart from the environmental conditions the required concrete cover for durability requirements depends on construction class. A structure with a design life of 50 years belongs to class S4. However, with high concrete strength classes, this class might be reduced. The minimum concrete cover for the different elements is shown in Table C.10. These values are derived from table 4.3, 4.4 and 4.5 of the national annex of NEN-EN 1992-1-1.

Table C.10: Minimum concrete cover for durability requirements

Element	Construction class	Governing durability class	Minimum cover to reinforcement [mm]	Minimum cover to prestressing steel [mm]
Bridge deck	S4	XD3	40	
Bridge girder	S4	XD3	40	50
Abutment sides	S3, (S4)	XC2, (XD2)	20, (40)	
Abutment top	S4	XD3	40	
Intermediate pier	S4	XD3	40	
Foundation piles	S3	XC1 (XD2)	10 – 35	

The nominal concrete cover is calculated by adding an execution tolerance of 5 [mm] as shown in Equation C.1.

Equation C.1: Nominal concrete cover

$$c_{nom} = c_{min,} + \Delta c_{dev} \qquad \Delta c_{dev} = 5[mm]$$

C.4 Loads on the super structure

In this chapter the loads on the bridge deck are discussed. The following loads are consecutively considered: permanent, prestressing, traffic, braking and acceleration, accidental, guiding rail, wind, temperature, earthquake, snow, fire and fatigue. In the twelfth paragraph the traffic load combinations are described. Next in the thirteenth and fourteenth paragraph the ultimate limit state verification and serviceability limit state verification are described. In the last paragraph the partial load factors are discussed.

C.4.1 Permanent load

The permanent loads consist of the weight of the elements that are permanently on the bridge. The self-weight of the girders is modelled as uniformly distributed load. The value of this load is calculated by multiplying the cross-sectional area with the concrete density and dividing this over the width of a girder. The cross-sectional area is calculated based on the nominal dimensions. The permanent load due to the in-situ deck is also modelled as uniformly distributed load and calculated by multiplying the thickness of the layer with the concrete density.

The layers of asphalt generate a uniformly distributed load as well. A thickness of 140 [mm] is assumed, in accordance with the ROK. This thickness is again multiplied with the density. For guiding rails a line load of 1 [kN/m] is assumed.

The kerb edges are around 0,5 [m] wide. The load is modelled as a uniformly distributed load by multiplying the height with the concrete density. A height of 100 [mm] is used, which is the minimum required. In Table C.11 and Figure C.11 an overview is given.

Table C.11: Permanent loads of bridge deck

Element	Type of load	Area	Value
Girders	Uniformly distributed	Complete bridge	$\frac{area\ [m^2] \times 24 \left[\frac{kN}{m^3}\right]}{width\ girder\ [m]}$
In-situ deck	Uniformly distributed	Complete bridge	$height\ [m] \times 24 \left[\frac{kN}{m^3}\right]$
Asphalt	Uniformly distributed	Complete bridge	3,22 [kN/m^2]
Guiding rails	Line load	Sides of bridge	1 [kN/m]
Kerb edges	Uniformly distributed	0,5 [m] on both sides	2,4 [kN/m^2]

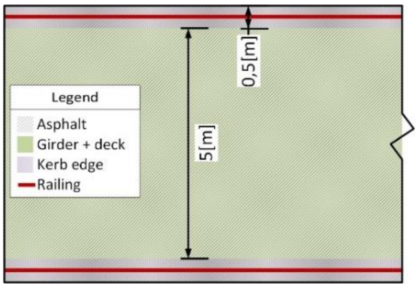


Figure C.11: Permanent loads present on deck. Top view bridge deck

C.4.2 Prestressing load

In traditional designs the prestressing force is based on the loads applied on the structure. However, in a design with existing girders the prestressing force is a given value and cannot be changed. The load can be derived from archive information, but attention should be paid to time dependent losses that have occurred. In accordance to the background report of the RBK from TNO, the size of the loss can be assumed 17% for girders before 1974 and 12% for girders after 1974. Nonetheless, for young girders (age less than 40 years) it should be taken into account that not all losses have yet occurred.

C.4.3 Traffic load

The four traffic load models from Eurocode are successively described.

Traffic load model 1

The first traffic load model consists of axle and uniformly distributed loads. It is used for global and local verifications and covers most of the traffic effects. First the bridge deck is divided in theoretical lanes according to table 4.1 NEN-EN 1991-2. The total width between the kerb edges is 5 meters. So, the bridge is divided into a lane of 3 meters and 2 meters of remaining area.

On the lane two axle loads and a uniformly distributed load is present. On the remaining area only a distributed load is present. The axle load is divided over two contact areas of 0,4 [m] by 0,4 [m]. The sizes of the load from table 4.2 of NEN-EN 1991-2 are shown in Table C.12. These loads should be multiplied with correction factors shown in Table C.13. A part of this correction factor is a reduction factor, because the number of heavy trucks is lower than 2.000.000. Figure C.12 gives a visualization.

Table C.12: Characteristic values of loads in load model 1

Position	Axle load Q_{ik} [kN]	Distributed load q_{ik} [kN/m^2]
Lane 1 (i=1)	300	9
Remaining area (i=r)	0	2,5

Table C.13: Correction factors

Position	Reduction factor axle load α_{Qi}	Reduction factor distributed load α_{qi}
Lane 1 (i=1)	0,95	$1{,}15 \times 0{,}95 = 1{,}09$
Remaining area (i=r)	-	0,8

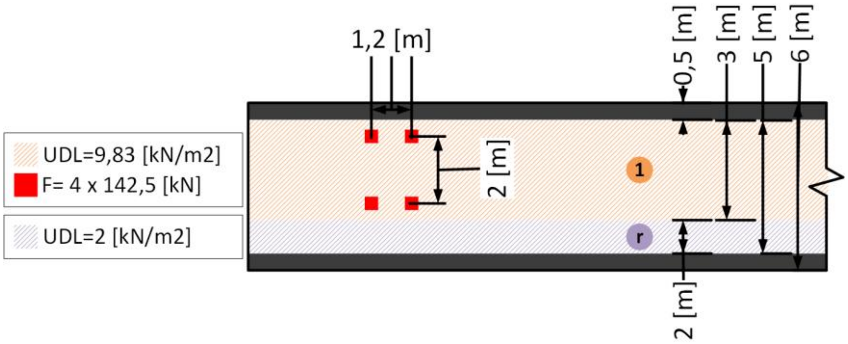


Figure C.12: Loads from traffic load model 1

The loads applied at the contact areas are spread in a 45° angle towards the centroidal axis of the bridge deck. This is visualised in Figure C.13.

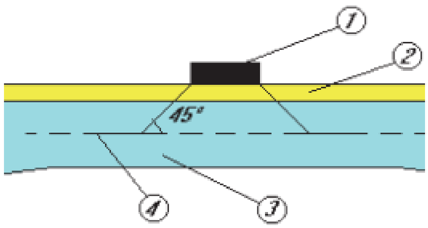


Figure C.13: Spread of axle load. Number 1 indicates the contact area. Number 2 the asphalt layers. Number 3 indicates the concrete bridge deck and number 4 indicates the centroidal axis of the bridge deck.

Traffic load model 2
In traffic load model 2 a single axle load is considered. This model is only used for verification of local effects and can be decisive for spans shorter than 7 [m]. In this bridge design the minimum span length is 12,5 [m]. Therefore, this model will not be considered.

Traffic load model 3
Traffic load model 3 deals with exceptional transports. This bridge is not part of the national road network and no exceptional transports are expected. The bridge only provides access to the residential area on the island. For very heavy transport instead of the bridge the water can be used. Therefore, this bridge will not be designed on exceptional transports.

Traffic load model 4
Traffic load model 4 considers loading due to a crowd and is used for global verification. Since the bridge is situated in a residential area it might be used for processions occasionally. Therefore, this load model will be considered. In this load model a uniformly distributed load of 5 [kN/m^2] is applied over the complete area of the bridge, including the kerb edges.

C.4.4 Horizontal loads due to braking, acceleration and centrifugal action
The braking load is calculated in Equation C.2. The horizontal acceleration and braking load that should be considered depends on the span length, which is not yet determined. The minimum span length is 12,5 [m], which corresponds with a load of 376 [kN]. The maximum span that can be used is 30 [m], which corresponds with a load of 430 [kN]. The braking and acceleration value is the same only the direction is opposite. The load is applied at the top side of the bridge deck and transferred to the supports. Since, the bridge has a straight profile no centrifugal force is considered.

Equation C.2: Formula for braking and acceleration force

$$\begin{aligned} Q_{lk} &= 0{,}6 \times \alpha_{Q1} \times 2 \times Q_{1k} + 0{,}10 \times \alpha_{q1} \times q_{1k} \times w_1 \times L \\ &= 0{,}6 \times 0{,}95 \times 2 \times 300 + 0{,}10 \times 1{,}09 \times 2{,}5 \times 3 \times L \\ &= 342 + 2{,}95 \times L \end{aligned}$$

$$\begin{aligned} 180 \times \alpha_{Q1}[kN] \leq Q_{lk} \leq 800[kN] \\ 180 \times 0{,}95[kN] \leq Q_{lk} \leq 800[kN] \\ 171[kN] \leq Q_{lk} \leq 800[kN] \end{aligned}$$

$$L = 12{,}5\ [m] \rightarrow Q_{lk} = 376\ [kN]$$
$$L = 30\ [m] \rightarrow Q_{lk} = 430\ [kN]$$

C.4.5 Accidental load
For accidental loads collision with kerb edges and guiding rails can occur. A collision with the substructure or bridge deck is not possible. At high water levels the bridge will cross the water, but no boat traffic is able to go underneath. At low water levels the bridge crosses the land, but since no roads are present, no vehicles can go underneath.

During accidents vehicles on the bridge deck can collide with the kerb edge on the sides. To account for this situation a 100 [kN] horizontal load is applied over 0,5 [m]. This load is applied 0,05 [m] from the top of the edge. Combined with this horizontal load a vertical load of 214 [kN] is applied, see Equation C.3. The load is spread into the kerb edge under a 45° angle. In Figure C.14 the load is visualized.

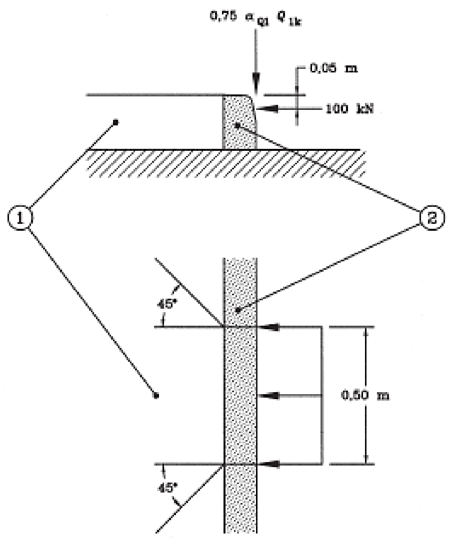


Figure C.14: Collision with kerb edge. At the top of the figure a side view and at the bottom a top view. Number one represents the footpath and number to the kerb edge.

Equation C.3: Vertical force that should be combined with horizontal collision load on kerb edge.

$$F_{vertical} = 0{,}75 \times \alpha_{Q1} \times Q_{1k}$$
$$= 0{,}75 \times 0{,}95 \times 300 = 214\ [kN]$$

Equation C.4: Vertical force that should be combined with horizontal collision load with guiding rails.

$$F_{vertical} = 0{,}5 \times \alpha_{Q1} \times Q_{1k}$$
$$= 0{,}5 \times 0{,}95 \times 300 = 143\ [kN]$$

A vehicle may also collide with the guiding rails. In this situation a similar load model is valid. A horizontal force is again spread over 0,5 [m] length. However, in this case the point of application is 1 [m] above the bridge deck or 0,1 [m] below the top of the guiding rails if this gives a lower value. However, the minimum height of the guiding rails is 1 [m], so this situation is not likely to occur. The magnitude of the force depends on the stiffness of the guiding rails. The stiffer the rails, the higher the load. The minimum force is 100 [kN] and the maximum 600 [kN]. This load is again combined with a vertical force, which is in this situation 143 [kN] according to Equation C.4.

C.4.6 Guiding rails

A variable line load of 3,0 [kN/m] is applied in horizontal or vertical direction on the guiding rail. The horizontal and vertical load only need to be considered separately and not together.

C.4.7 Wind load

The bridge is in Arnhem in the province Gelderland, which is situated in wind zone area 3. Terrain category 2 is valid, because the bridge is situated in a relatively unbuild area with low vegetation. The basic wind velocity is calculated with Equation C.5. This value is already based on a reference period of 50 years, so no correction factor is needed. To calculate the average windspeed first the terrain factor and roughness factor are calculated in Equation C.6 and Equation C.7. In Equation C.8 the average wind speed is calculated which is used in Equation C.11 together with the standard deviation from Equation C.9 and the turbulence intensity from Equation C.10 to calculate the wind pressure.

Equation C.5: Basic wind velocity

$$v_b = c_{dir} \times c_{season} \times v_{b,o} = 24{,}5 \left[\frac{m}{s}\right]$$

$c_{dir} = 1{,}0$ correction factor for wind direction
$c_{season} = 1{,}0$ correction factor for season
$v_{b,o} = 24{,}5 \left[\frac{m}{s}\right]$ fundamental value for wind speed based on wind zone 3

Figure C.15: Wind zone areas in the Netherlands based on [159]

Equation C.6: Terrain factor

$$k_r = 0{,}19 \times \left(\frac{z_0}{0{,}05}\right)^{0{,}07} = 0{,}21$$

$z_0 = 0{,}2\ [m]$ = for terrain category II

Equation C.7: Roughness coefficient

$$c_r(z) = k_r \times \ln\left(\frac{z}{z_0}\right) = 0{,}63$$

$z = z_{min} = 4\ [m]$ height above ground level

Equation C.8: Average windspeed

$$v_m(z) = c_r(z) \times c_0(z) \times v_b = 15{,}41 \left[\frac{m}{s}\right]$$

$c_0(z) = 1{,}0$ orography factor, for no curved terrain.

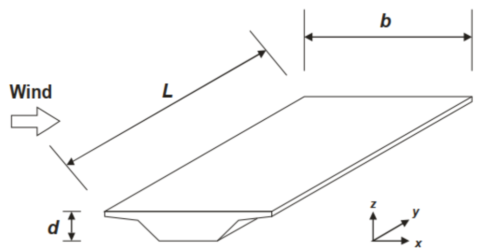


Figure C.16: Wind directions

Equation C.9: Standard deviation wind turbulence

$$\sigma_v = k_r \times v_b \times k_l = 5{,}13$$

$k_l = 1{,}0$ turbulence factor

Equation C.10: Turbulence intensity

$$I_v(z) = \frac{\sigma_v}{v_m(z)} = 0{,}33$$

Equation C.11: Wind pressure

$$q_p(z) = \left(1 + 7 \times I_v(z)\right) \times \frac{1}{2} \times \rho \times v_m^2(z) = 494 \left[\frac{kg}{m\,s^2}\right]$$

$\rho = 1{,}25 \left[\frac{kg}{m^3}\right]$ air density

Wind generates forces in x, y and z-direction. These directions are visualized in Figure C.16. The wind force in x-direction acts in horizontal direction along the length of the bridge. The magnitude of the force is calculated with the simplified method of NEN-EN 1991-1-4. The wind load in z-direction is calculated with the same equation, Equation C.12 and Equation C.13. The reference area for x-direction can be calculated by multiplying the length of the bridge element by the total height. This height is the sum of the height of bridge girders, deck, asphalt layers (140 mm), kerb edge (100 mm), rails (300 mm can be used). The reference area for z-direction is simply the length multiplied with the width. So, in total 642 [m^2]. The wind force in y-direction is taken as 40% of the wind load in x-direction. This load occurs simultaneously with the load in x-direction. In case the load in z-direction acts unfavourable it should also be included.

Equation C.12: Load factors

$$C = C_e \times C_f$$

$C_{f,x} = 1{,}33,\ C_{f,z} = 0{,}9$

$C_e = \frac{q_p(z)}{q_b} = 1{,}32$ → moet 3,03 zijn

$q_b = \frac{1}{2} \times \rho \times v_b^2 = 375 \left[\frac{kg}{m\,s^2}\right]$

$C_x = 1{,}76$
$C_z = 1{,}19$

Equation C.13: Wind force

$$F_w = \frac{1}{2} \times \rho \times v_b^2 \times C \times A_{ref}$$

$$F_{w,x} = 465 \left[\frac{N}{m^2}\right] \times \left(\frac{540 + h_{deck} + h_{profile}}{1000} \times 107\right) [m^2]$$

$$F_{w,z} = 314 \left[\frac{N}{m^2}\right] \times 642 [m^2] = 202\ [kN]$$

C.4.8 Temperature load

If the structure is statically determinate all deformations are free to occur. Therefore, temperature differences will not cause stresses in the structure. However, this under the condition that there is sufficient space to accommodate these deformation. If the structure is statically indeterminate some deformations are restricted and temperature differences result in stresses in the structure. Therefore, in this paragraph the temperature differences and related deformation or stresses are described. A distinction is made between the uniform temperature and the temperature gradient.

The uniform temperature causes shortening and elongation in the horizontal direction. The minimum and maximum temperature of the bridge are based on the minimum and maximum temperatures in the shadow. Next the temperature difference is calculated. Al the values from NEN-EN-1991-1-5 al already based on a reference period of 50 years, so no correction is needed. The temperature difference is related to the strain by the coefficient of thermal expansion. The contraction and elongation of the elements in length direction can be calculated by multiplying this strain with the span of the girders. Currently the dimensions are unknown, but the minimum and maximum are calculated. The calculation is shown in Equation C.14 to Equation C.17.

Equation C.14: Minimum and maximum temperature

$$T_{e,min} = T_{min} + 8 = -25 + 8 = -17°C \qquad T_{e,max} = T_{max} + 2 = 30 + 2 = 32°C$$

Equation C.15: Minimum and maximum temperature range for contraction and expansion

$$\Delta T_{N,con} = T_0 - T_{e,min} = 10 - -17 = 27°C \qquad \Delta T_{N,exp} = T_{e,max} - T_0 = 32 - 10 = 22°C$$

Equation C.16: Strain for contraction and expansion

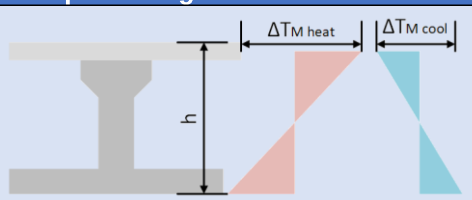
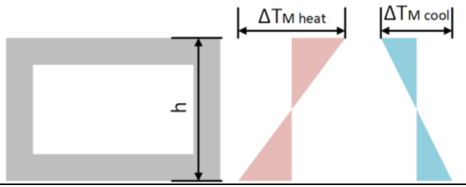
$$\varepsilon_{con} = \alpha_{con} \times \Delta T_{N,con} = 0{,}216 \left[\frac{mm}{m}\right] \qquad \varepsilon_{exp} = \alpha_{con} \times \Delta T_{N,exp} = 0{,}176 \left[\frac{mm}{m}\right]$$

Equation C.17: Minimum and maximum contraction and expansion.

$$\Delta L_{con} = \varepsilon_{con} \times L \qquad \Delta L_{exp} = \varepsilon_{exp} \times L$$
$$\Delta L_{con,min} = 0{,}216 \times 12{,}5 = 2{,}70[mm] \qquad \Delta L_{exp,min} = 0{,}176 \times 12{,}5 = 2{,}20[mm]$$
$$\Delta L_{con,max} = 0{,}216 \times 30 = 6{,}48[mm] \qquad \Delta L_{exp,max} = 0{,}176 \times 30 = 5{,}28[mm]$$

The linear temperature difference over the height of the bridge deck differs for the deck type. Both types: (inverted) T and box girders are considered separately because it is not yet known which type will be used. The values from Table C.14 are based on a wear layer of 50 [mm], conforming the standard. This value is also used in the thickness of the asphalt layer. The linear gradient results in a curvature, which can be calculated with Equation C.18. In reality, the temperature will not differ linearly, because of the different widths over the cross-section. As a result, also eigen temperature and stresses will occur. The total temperature difference is given in Table C.15.

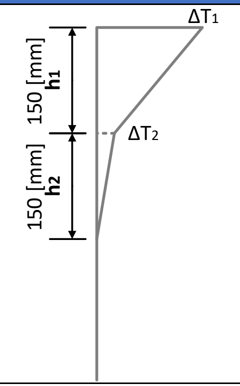
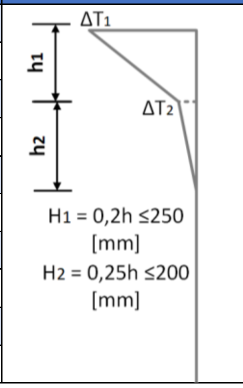
Table C.14: Linear temperature gradient

Type of girder	Temperature gradient	Values
(inverted)-T-girder		$\Delta T_{M,heat} = 15°C$ $\Delta T_{M,cool} = 8°C$
Box beam		$\Delta T_{M,heat} = 10°C$ $\Delta T_{M,cool} = 5°C$

Equation C.18: Curvature

$$\kappa(\Delta T_M) = \frac{\Delta T_M \times \alpha_{conc}}{h}$$

Table C.15: Temperature profile

h [mm]	Heating			Cooling down		
	Profile	ΔT_1 [°C]	ΔT_2 [°C]	Profile	ΔT_1 [°C]	ΔT_2 [°C]
500		10	6		-5	-3
600		10	5		-5	-3
700		10	4,75		-5	-2,75
800		10	4,5		-5	-2,5
900		10	4,5		-5	-2
1000		10	4,5		-5	-1,5
1100		10	4,5		-5	-1,3
1200		10	4,5		-5	-1,1
1300		10	4,5		-5	-0,9
1400		10	4,5		-5	-0,7

In case the structure is statically indeterminate, elongation and shortening will be restricted at the intermediate supports. Therefore, all elongations and shortenings should be accommodated at the end supports. So instead of approximately 5 [mm] per support 12 [mm] should be present at both end supports (see Equation C.19).

Equation C.19: Contraction and expansion end supports.

$$\Delta L_{con} = \varepsilon_{con} \times L \qquad \Delta L_{exp} = \varepsilon_{exp} \times L$$

$$\Delta L_{con} = 0{,}216 \times 107{,}21 = 23{,}16[mm] \qquad \Delta L_{exp} = 0{,}176 \times 107{,}21 = 18{,}89[mm]$$

With a statically determinate structure each span can curve independently of the others. However, when the spans are connected to each other they can only form one curve. However, due to the intermediate supports this curvature is restricted. The stresses that occur are calculated with Equation C.20. In case of heating at the top compression stresses develop and at the bottom tensile stresses. With cooling this is vice versa.

Equation C.20: Stresses due to linear temperature gradient in statically indeterminate structure

$$\sigma_{Tm,heat} = \pm \frac{\alpha_{con} \times \Delta T_{M,heat} \times E_c}{2 \times (1 - v)} \qquad \sigma_{Tm,cool} = \pm \frac{\alpha_{con} \times \Delta T_{M,heat} \times E_c}{2 \times (1 - v)}$$

$$\sigma_{Tm,heat,inverted\ T} = \pm 2{,}85\ [N/mm^2] \qquad \sigma_{Tm,cool,inverted\ T} = \pm 1{,}52\ [N/mm^2]$$

$$\sigma_{Tm,heat,box\ beam} = \pm 1{,}9\ [N/mm^2] \qquad \sigma_{Tm,cool,box\ beam} = \pm 0{,}95\ [N/mm^2]$$

C.4.9 Earthquakes

Earthquakes only have to be considered in case the bridge is classified in consequence class 3. This bridge belongs to consequence class 2. So, loads due to earthquakes are not considered in the calculation.

C.4.10 Snow load

According to Eurocode NEN-EN-1990 load due to snow does not have to be combined with traffic load or other variable loads. Since, snow load alone will not be decisive it will not be considered in the design calculation. In addition, according to the ROK snow load on a bridge deck does not have to be considered.

C.4.11 Fire

Fires will also not be considered in this design calculation. This is in line with the Dutch Bouwbesluit (Building Act), which states that new bridges do not have to be designed for accidental load of fire. However, requirement 0749 of the ROK states that new bridges should be designed for fire. This design should be risk based. Nonetheless this bridge is not part of the national infrastructural network. Beside the bridge does not consist of elements, like pylons or cables that have a higher risk in case of fire.

C.4.12 Combinations traffic load

Traffic introduces vertical and horizontal loads. These loads are combined in different ways in five traffic load groups according to NEN EN-1991-2 NB4. In each situation only one of these groups can occur. So, they cannot be further combined. A group of traffic load is considered as one variable in the ultimate and serviceability limit state verifications discussed in the next paragraphs. Group 3 only considers pedestrians and cyclist on the sidewalks. For this bridge, this combination will not be dominant and is therefore not further considered. In group 1b traffic load model 2 dominates and in group 5 traffic load model 3. Since, these load models are not considered in this design, the traffic groups will also not be considered. In addition, for the frequent values group 1a and 2 result in the same combination. The combinations are shown in Table C.16 and Table C.17.

Table C.16: Groups of traffic load characteristic value

	Driving line			**Kerb edges**
Group ↓	Vertical Traffic model 1	Vertical Traffic model 4	Horizontal brake and acceleration	Vertical
1a	1,0 x characteristic value		0,8 x characteristic value	0,4 x characteristic value
2	0,8 x characteristic value		1,0 x characteristic value	0,4 x characteristic value
4		1,0 x characteristic value		

Table C.17: Groups of traffic load frequent values

	Driving line			**Kerb edges**
Group ↓	Vertical Traffic model 1	Vertical Traffic model 4	Horizontal brake and acceleration	Vertical
1a	1,0 x frequent value		1,0 x frequent value	1,0 x frequent value
4		1,0 x frequent value		

C.4.13 Ultimate limit state verification (ULS)

In NEN-EN 1990 six types of ultimate limit state verifications are distinguished. For this bridge only ULS-STR is relevant. The definitions of the other types can be found in

Table C.18: Possible ultimate limit state verifications

Type	**Definition**
ULS-EQU	Loss of equilibrium. Only relevant in case a small variation has major influence or strength of materials or soil is determining factor. So, not relevant for this project
ULS-STR	Failure due to excessive deformation. This structural failure is considered.
ULS-GEO	Failure due to excessive deformation of soil. This will not be a problem for this project location
ULS-FAT	Failure due to time dependent factors
ULS-UPL	Loss of equilibrium in construction in soil due to upwards water pressure. This is not decisive for this bridge
ULS-HYD	Failure due to piping underground. Not relevant for bridges

For ULS-STR the load combinations are formulated in Equation C.21 to Equation C.23. The most unfavourable combination should be used for the verification. In the first formulation the self-weight acts as dominate factor. In the second formulation a variable load acts as dominant factor. The third formulation relates to accidental loads.

Equation C.21: ULS combination in which self-weight dominates.

$$\sum_{j\geq 1} \gamma_{G,j} G_{k,j} + \gamma_P P + \gamma_{Q,1} \psi_{0,1} Q_{k,1} + \sum_{i>1} \gamma_{Q,j} \psi_{0,i} Q_{k,i}$$

Equation C.22: ULS combination in which variable load dominates.

$$\sum_{j\geq 1} \xi_j \gamma_{G,j} G_{k,j} + \gamma_P P + \gamma_{Q,1} Q_{k,1} + \sum_{i>1} \gamma_{Q,j} \psi_{0,i} Q_{k,i}$$

Equation C.23: ULS combination with accidents

$$\sum_{j\geq 1} G_{k,j} + P + A + \psi_{1,1} Q_{k,1} + \sum_{i>1} \psi_{2,i} Q_{k,i}$$

The partial load factors and combinations factors (psi-values) can be found in C.4.15.

C.4.14 Serviceability limit state verification (SLS)

The ultimate limit state verification is based structural failure or loss of equilibrium. To ensure functionality during normal use, comfort for people and aesthetics the serviceability limit state is used. NEN-EN 1990 distinguishes three types of combinations in SLS. The characteristic combination leads to irreversible deformation. The frequent combination for reversible deformation and the quasi-permanent for the long-term effects.

Equation C.24: Characteristic SLS combination

$$\sum_{j\geq 1} G_{k,j} + P + Q_{k,1} + \sum_{i>1} \psi_{0,i} Q_{k,i}$$

Equation C.25: Frequent SLS combination

$$\sum_{j\geq 1} G_{k,j} + P + \psi_{1,1} Q_{k,1} + \sum_{i>1} \psi_{2,i} Q_{k,i}$$

Equation C.26: Quasi-permanent SLS combination

$$\sum_{j\geq 1} G_{k,j} + P + \sum_{i>1} \psi_{2,i} Q_{k,i}$$

The combinations factors (psi-values) can be found in C.4.15.

C.4.15 Partial factors

The partial load factors can be found in Table C.19. In case a load acts favourable it reduces the total effects on the structure. In case a variable load acts favourable it should not be considered in the combination, so the partial factor is 0.

The combination factors, or psi-values can be found in Table C.20. ψ_0 is used to find the characteristics value, ψ_1 for the frequent value and ψ_2 for the quasi-permanent value.

Table C.19: Partial load factors

Self-weight	Favourable	
	Yes	No
$\gamma_{G,j}$	1,3	0,9
$\xi_j \gamma_{G,j}$	1,2	0,9

Variable load	Favourable	
	Yes	No
γ_Q traffic	1,35	0
γ_Q other	1,5	0

Prestress	
γ_P	1,0

Table C.20: Combination factors (psi-values)

Load	ψ_0	ψ_1	ψ_2
Traffic group 1a	0,8	0,8	0,4
Traffic 2	0,8	0,8	0
Traffic 4	0	0,8	0
Wind	0,3	0,6	0
Thermal load	0,3	0,8	0,3

Appendix D: Existing viaducts

This appendix describes the location and layout of the existing structures. In Figure D.1 an overview of the location of the different viaducts is presented. Next for each alternative the layout of the viaducts from which the girder originate from are described.

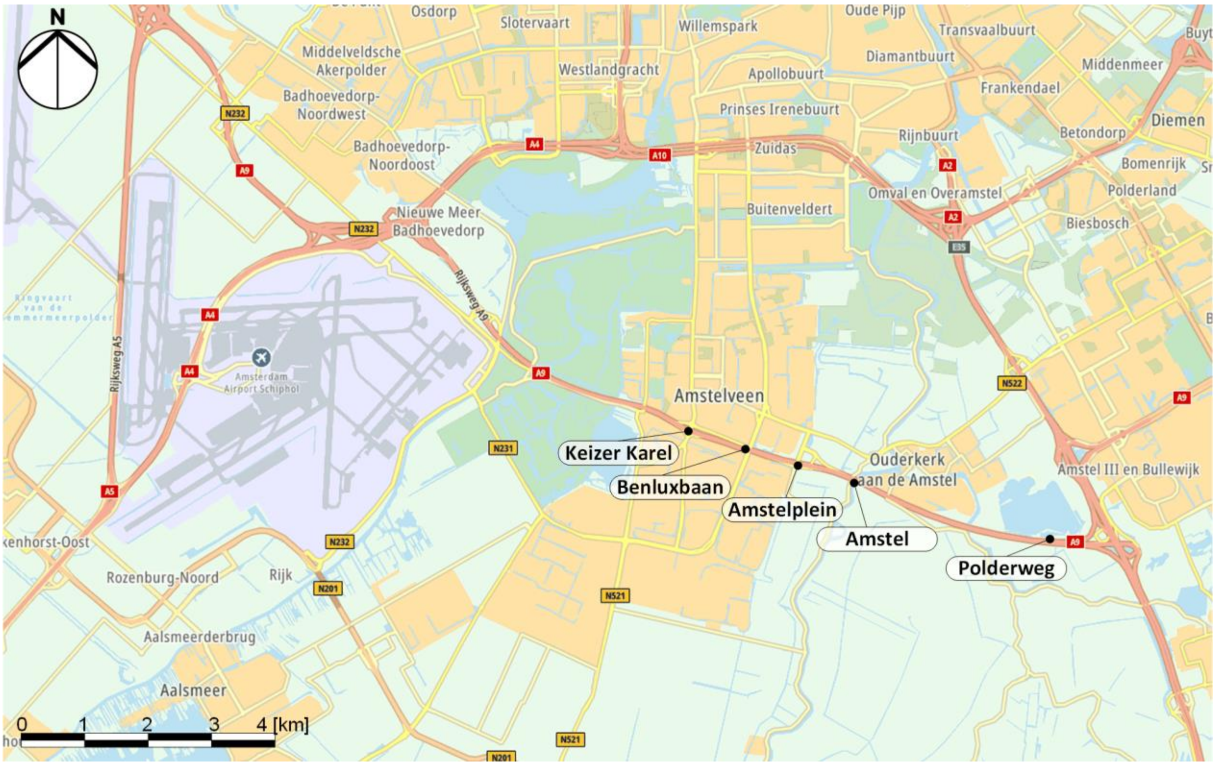


Figure D.1: Location of existing viaducts from which the girders originate [161], adapted.

D.1 <u>Alternative 1a</u>

This alternative uses girders from the civil structure ‘Polderweg’. This structure consists of three three-span viaducts next to each other. Two viaducts are made with the HIP 800 girders from 1973. The other viaduct is built in 1999 as part of a road widening. For this viaduct other girders are used, which are not of interest for the case study. In Figure D.2 provides a side view of the viaducts of interest. A short span of 18,53 [m] is situated on the sides and a longer span of 21,62 [m] in the middle. In total 31 girders per span are present, which corresponds to a total width of 37 [m]. So, each viaduct is approximately 18,5 [m] wide. In Figure D.3 a top view of the viaducts is shown. From this view it can be concluded that approximately 1 [m] of side area is present on both sides. So, 16,5 [m] is available for traffic. Currently the viaduct is divided in three lanes. To withstand the traffic load a statically indeterminate system is used.

As will be discussed further on, the viaducts are deconstructed in distinct stages. First the viaduct on the south side is deconstructed and later the viaduct on the north side. The deconstruction will probably start between the second half of 2023 and first half of 2024.

Figure D.2: Side view of viaduct 'Polderweg' [1], adapted.

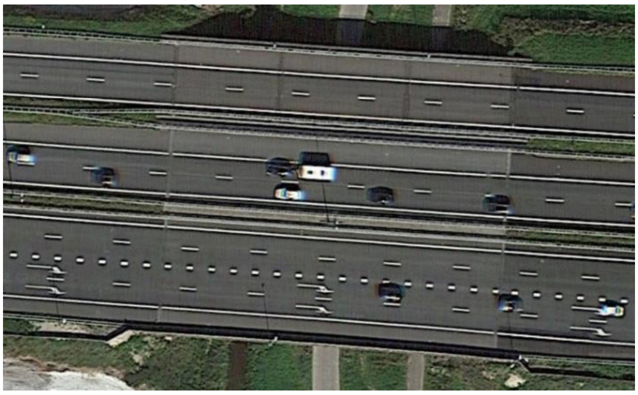


Figure D.3: Top view of viaduct 'Polderweg'. At the top side the two viaducts of interest [162].

D.2 Alternative 1b

In this alternative 23 [m] long girders from civil structure ‘Keizer Karel’ and 16,05 [m] long girders from civil structure ‘Beneluxbaan’ are used. Both viaducts are made with HNP 750 girders from 1968. To withstand the traffic loads a statically indeterminate system is used. So, the girders are connected in the longitudinal direction.

Viaduct Keizer Karel consists of three three-span viaducts next to each other. In Figure D.4 and Figure D.5 a side view and top of the viaducts is shown. In total the viaducts consist of 43 girders per span, which corresponds to a width of 43 [m]. The top two viaducts have approximately 16 [m] available for traffic. This area is divided in three lanes and an emergency lane. The bottom viaduct is approximately 10 [m] wide and has two lanes that are used as exit ramp.

Viaduct Beneluxbaan has 4 spans and is 32 [m] wide. For both directions five driving lanes are present. Of which one is an emergency lane and one is an entrance or exit ramp. A side and top view are presented in Figure D.6 and Figure D.7.

From viaduct ‘Keizer Karel’ 34 girders are released in March 2023. These girders are removed from the construction, without removing the in-situ deck in advance. Next, they are transported to a testing and storage facility where the in-situ deck layer is removed and the girders are shortened. On 6 girders destructive loadings tests are performed. The release of girders was possible, because traffic over the viaduct is redirected over a temporary viaduct build next to the original one [163]. The traffic road underneath the viaduct most likely did not have to be closed off completely as only girders from a single span at the side of the viaduct are released. The remaining girders in the viaduct are together with the rest of the viaduct demolished at the end of April 2023. For this demolition, the traffic roads underneath the viaduct are closed off during one weekend [164].

Viaduct ‘Beneluxbaan’ is deconstructed in various stages. Before deconstruction a part of the new viaduct is build on the north side. Traffic is redirected over this new viaduct and the northern part of the existing viaduct. Next the viaduct on the south side is deconstructed and the south side of the new viaduct is built. After this traffic is again redirected and the north side is deconstructed and the new viaducts are finished [165]. The girders from the north side of the viaduct are released medio June. First the asphalt layer and guiding rails are removed from the structure and then the girders are lifted out. So, also in this case the in-situ deck is probably not removed in advance. The remaining girders are not released before the end of 2023 [166].

While some of the girders are demolished during the performance of the case study. This demolishment is not considered in the case study. So, they are still assumed to be available.

Figure D.4: Side view Keizer Karel viaduct [167], adapted.

Figure D.5: Top view of Keizer Karel viaduct [168].

Figure D.6: Side view Benelux viaduct [169], adapted.

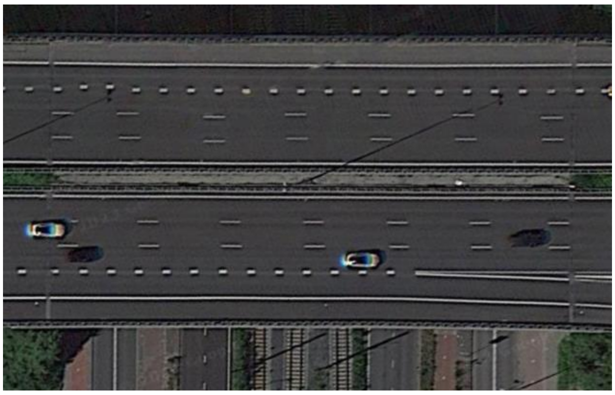


Figure D.7: Top view of Benelux viaduct [170].

D.3 Alternative 1d

In this alternative 21,5 [m] long HIP 800 girders from civil structure ‘Amstelplein’ are used. In Figure D.8 and Figure D.9 a side and top view of the structure are presented. This structure consists of two viaducts next to each other. Together they consist of 38 girders per span, which corresponds to a width of 46 [m]. Currently the deck is divided in two times four lanes. There is a side area of around 4,5 [m] present on both sides, which is not used. Though, this area is probably designed for traffic load.

The viaducts are demolished in order from west to east. Like the previous mentioned viaducts the deconstruction will be in phases. Viaduct Amstelplein is the next viaduct in line after viaduct Benelux. So, most likely the southern part of this viaduct will be deconstructed before the end of 2023.

Figure D.8: Side view Amstelplein viaduct [171], adapted.

Figure D.9: Top view viaduct Amstelplein [172].

D.4 Alternative 2a

In this alternative 28 [m] long HIP 1100 girders from civil structure ‘Amstel’ are used. A side and top view are presented in Figure D.10 and Figure D.12. This structure consists of two similar viaducts placed next to each other. In these viaducts the girders are used in a statically determinate system. So, the girders are not coupled in longitudinal direction. This is clearly visible in Figure D.11. All girders have a span of 28 [m]. Each viaduct consists of 15 girders per span, which corresponds with a total width of 36 [m]. There is a small side area of approximately 1 [m] that is not used for traffic.

Figure D.10: Side view Amstel viaduct [173].

Figure D.11: Detail of side view Amstel viaduct [173].

Figure D.12: Top view of Amstel viaduct [174].

Appendix E: Structural calculation

Table of contents

E.1 Introduction

This appendix provides the details of the structural calculation performed in the case study. This appendix is supportive material to §4.3 - §4.5, whereby only the results from the structural analysis are discussed. It should be noted that this appendix is not a self-contained document. The characteristics of the developed design alternatives are considered to be known. In addition, the layout and content of the structural roadmap, described in §3.6 is assumed to be familiar.

Although most steps are familiar, the additional or divergent steps in procedure are described in the first part of the appendix. For example, some steps differ because the findings from this structural calculation are used to improve and develop the structural roadmap of the design approach. Besides, some procedures are not prescribed in the roadmap.

Next, for three developed alternatives the described procedure is followed, which starts with deriving the sectional properties. Next the design loads on the structure are determined and verified with the computed shear and bending moment capacity. Finally, for the purpose of developing the design approach shortening possibilities are investigated. It should be noted that in line with the structural roadmap, the procedure for the second alternative is discontinued after identifying insufficient shear capacity.

To determine the load distribution SCIA-engineer is used. Therefore, in the last part of this appendix a SCIA-engineer report is included. Considering that for each alternative and scenario the same procedure is followed the SCIA results for only one scenario are included.

E.2 Calculation steps

In this part of the appendix the calculation steps are discussed. In the first section the general procedure is described. Next the method for load distribution is described. The third section provides the partial load factors. Finally, the procedure to compare the old and new loading situation is discussed.

E.2.1 General procedure

Before the structural verification can start the cross-sectional properties are determined. These are based on characteristics of the assumed concrete class and the cross-sectional dimensions of the girder type. Next the design loads for the old and new situation are determined. For the old loading situation the loads of the later version of VOSB 1963 are used, because the girders originate from structures build between 1968 and 1972. Each viaduct has more than two lanes and the span length is around 20 [m]. With the multiplication factors from Table E.1 this results in a uniformly distributed load of 3,2 [kN/m^2] and two times three axle load of 196 [kN]. More explanation can be found in §2.3.2.

Table E.1: Multiplication factors VOSB 1963.

Multiplication factors	Value	Explanation
Reduction factor	0,8	More than 2 lanes
Impact factor (S)	1,33	$1 + \frac{40}{100+L}$
Load factor (B)	0,93	$0{,}6 + \frac{40}{100+L}$

In the new situation a distinction is made between statically determinate (SD) and statically indeterminate (SI) system to carry the traffic loads. The same height of the in-situ deck is used as in the original situation. It should be noted that this height may not be able to fulfil current requirements. Consequently, the deck might need to be higher. This variation influences the load as well as the capacity of the structure. Nonetheless, the variation in height is outside the scope of this research. The traffic loads in the new situation are already discussed in Appendix C: Basis for design.

After this step all the input is present for the structural verification. First the shear capacity is verified. Both NEN-EN-1992-1-1 and RBK regulations are followed from the start to provide insight in the differences. If the structure cannot fulfil on the NEN-EN-1992-1-1 or RBK regulations shortening in width direction is investigated. Next the bending moment in serviceability limit state (SLS) and ultimate limit state (ULS) is verified for the original and adapted width in SD and SI system. Apart from verification on shear and bending moments strut and tie models for different shortening lengths are made to investigate the possibilities for shortening. At the end of each verification a conclusion is made about the structural feasibility of the alternative. In addition, a review is provided that relates to the roadmap.

E.2.2 Load distribution

The structural roadmap does not provide guidelines to determine the load distribution in the structure. The structural engineer can choose his own method based on preference and level of detail required. In this case study the theory of a Euler-Bernoulli beam is used to determine the force distribution over the length of the girder. This theory is based on the fourth order differential equation and relations shown in Equation E.1.

Equation E.1: Euler-Bernoulli beam theory.

$$EI \times \frac{d^4w}{dx^4} = q(x)$$

$$\varphi = -\frac{dw}{dx}$$
$$\kappa = \frac{d\varphi}{dx}$$
$$M = EI \times \kappa$$
$$V = \frac{dM}{dx}$$
$$q = -\frac{dV}{dx}$$

EI =bending stiffness of beam [Nmm^2]
w = displacement [mm]
x = position on beam (length direction) [mm]
q = load on beam [N/mm]
φ= rotation angle
κ= curvature
M = bending moment [Nmm]
V = shear force [N]

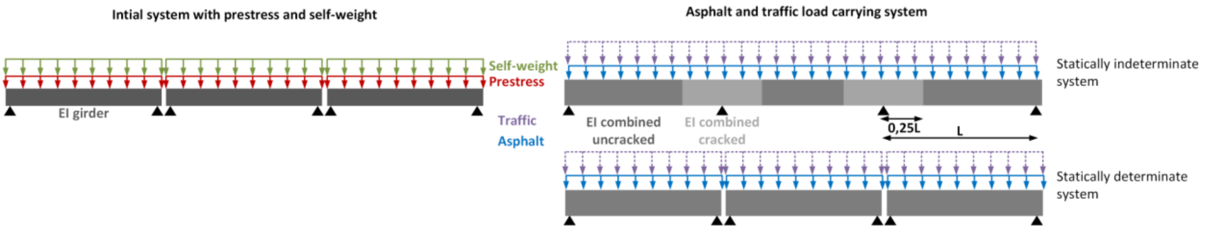


Figure E.1: Load carrying systems.

For the first two stages (self-weight and prestress) the bending stiffness of only the girder is used. For the last stage (including traffic load and weight of asphalt) the bending stiffness of the combined cross-section of girder and deck is used.

In case of a SI-system a distinction is made between the bending stiffness with a cracked and uncracked concrete deck. The bending stiffness with a cracked concrete deck is used at 25% of the span length around the intermediate supports, because around these locations the deck is subjected to tensile stresses. This is graphically shown in Figure E.1. The traffic load is a variable load. Therefore, it should be placed at the most unfavourable position. Moreover, this load is not uniformly present over the width of the bridge. So, the load should be distributed in the width direction as well. For the first indication the load is spread under a 45° angle until the neutral axis of the combined cross-section of girder and deck. Then, only the most loaded girder is modelled.

To determine the most unfavourable position for the maximum hogging and sagging bending moment of the axle traffic loads influence lines are used. For the maximum shear force the axle loads are placed at 2d from the supports, in which d is the effective height of the cross-section. The exact load configuration used can be found in the next paragraphs.

Since, the spread under 45° is conservative the traffic load is also modelled with SCIA-engineer. In this software the complete bridge deck is modelled instead of only one girder. The deck is modelled as orthotropic plate, without a reduction of bending stiffness near the supports. To determine the design load on a single girder the traffic loads are positioned at the same most unfavourable positions. At the critical locations a cut in width direction is made and the bending moments or shear forces are evaluated. To determine the design load the average is taken of several maximum values next to each other. The number of maximum values is chosen in such a way that it approximates the width of a single girder.

The asphalt load is also modelled in SCIA and can be used to verify the models. Since this load is not spread in width direction the results should be similar. Nonetheless the reduction of bending stiffness near the supports is not considered in SCIA. As a result, the sagging bending moments will be lower and the hogging bending moments higher compared to the Euler-Bernoulli theory. For a SD-system there is no difference in bending stiffness; hence the results of the SCIA calculation and Euler-Bernoulli theory should be similar.

For the characteristic loads the minimum of both methods is used. For the hogging bending moments this is reasonable since both methods overestimate this bending moment. SCIA, because of a higher stiffness and Euler-Bernoulli, because of limited load spread. For the sagging bending moment, the minimum is always found in SCIA. With a reduced stiffness at the supports this bending moment will be larger. Nonetheless, the orthotropic plate modelling in SCIA is still expected to be conservative with load spread in width direction. For the shear force the minimum is situation dependent, but the differences are less substantial compared to the bending moments.

E.2.3 Partial safety factors

The loads from the analysis are characteristics values. These values are multiplied with the partial safety factors from Table E.2 to find the design loads. These partial factors are based on the information provided in Appendix C: basis for design.

Table E.2: Partial safety factors for design situations.

Situation	Prestress	Self weight	Asphalt	Traffic
$M_{sag,ed,SLS}$	1,0	1,0	1,0	1,0
$M_{sag,ed,ULS}$	1,0	1,2	1,2	1,35
$M_{hog,ed,SLS}$	1,0	1,0	1,0	1,0
$M_{hog,ed,ULS}$	1,0	0,9	1,2	1,35
V_{ed}	1,0	1,2	1,2	1,35

E.2.4 Load comparison

The comparison between the new and old loading situation is not included in the design approach as the old loading situation is not of importance for the new situation. Nonetheless, the differences between the load conditions might help to get an indication on the structural feasibility. However due to the differences in approach of the standards this comparison is not straightforward. In current standards partial load and material factors are applied to account for uncertainties in loading conditions and material characteristics. In former standards this distinction was not made. As already discussed, these approaches were based on allowable stresses in the material. Also, sometimes a general safety factor of 1,7 was applied on the load, but this was also to deal with material factors. So, this comparison only focusses on the loads, while the capacity or resistance of the structure differed as well.

For a general comparison the bending moments in SLS are compared with the old (original) bending moments. The new bending moments in ULS are compared with the old bending moments multiplied with a factor of 1,7. Shear forces are only assessed in ULS. Therefore, these loads are compared with the old ones and the old ones multiplied with a factor of 1,7.

E.3 Design alternative 1a

First, the sectional properties are derived. In the next three sections the old and new loading situation are evaluated. The final design loads are presented in section five. Next the shear capacity and bending moment capacity are evaluated. Finally shortening possibilities are explored.

E.3.1 Sectional properties

The sectional properties in Table E.3 are based on the original dimensions of a HIP 800 girder, concrete class C55/67 for the girder and concrete class C30/37 for the deck. The cracked elastic modulus of the deck is taken as 1/3 of the original elastic modulus. If the girder is shortened in width direction to 790 [mm] the sectional properties of Table E.4 apply.

Table E.3: Sectional properties HIP 800 girder with original width of 1180 [mm].

Cross-sectional property	HIP 800
Area girder [mm^2]	362660
Area girder with deck [mm^2]	554660
Neutral axis girder [mm]	277,3
Neutral axis girder with uncracked deck [mm]	465,7
Neutral axis girder with cracked deck [mm]	356,7
EI girder [Nmm^2]	7,939 E+14
EI girder with uncracked deck [Nmm^2]	2,381 E+15
EI girder with cracked deck [Nmm^2]	1,457 E+15

Orthotropic Property	HIP 800
D11 [MNm]	1984,5
D22 [MNm]	3,7360
D12 [MNm]	0,0000
D33 [MNm]	43,053
D44 [MNm]	14511
D55 [MNm]	4378,2
d11 [MNm]	34899
d22 [MNm]	5816,4
d12 [MNm]	0,0000
d33 [MNm]	9574,8

Table E.4: Sectional properties HIP 800 girder with width of 780 [mm].

Cross-sectional property	HIP 800
Area girder [mm^2]	311765
Area girder with deck [mm^2]	438165
Neutral axis girder [mm]	311,9
Neutral axis girder with uncracked deck [mm]	458,7
Neutral axis girder with cracked deck [mm]	371,0
EI girder [Nmm^2]	6,896 E+14
EI girder with uncracked deck [Nmm^2]	1,692 E+15
EI girder with cracked deck [Nmm^2]	1,092 E+15

Orthotropic Property	HIP 800
D11 [MNm]	2141,8
D22 [MNm]	3,7360
D12 [MNm]	0,0000
D33 [MNm]	44,727
D44 [MNm]	14665
D55 [MNm]	4378,2
d11 [MNm]	35196
d22 [MNm]	5866,0
d12 [MNm]	0,0000
d33 [MNm]	9616,5

E.3.2 Prestress and self-weight

Because prestress and self-weight work in a SD-system the girders are considered separately. The bending moment and shear force line due to prestress and self weight are shown in Figure E.2 and Figure E.3.

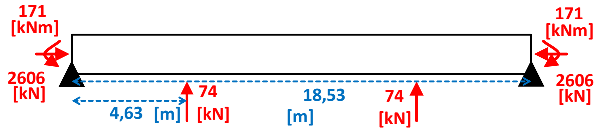


Figure E.2 a) Load distribution prestress.

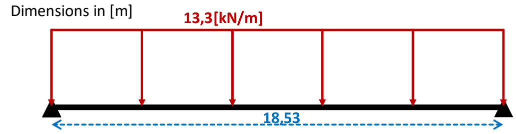


Figure E.2 b) Load distribution self-weight

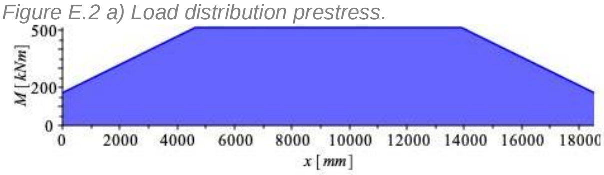


Figure E.2 c) Bending moment line due to prestress.

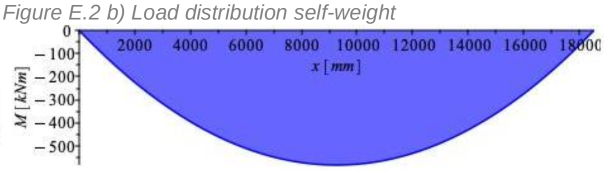


Figure E.2 d) Bending moment line due to self-weight.

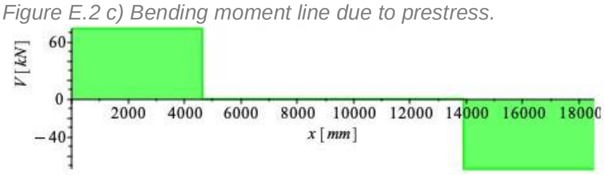


Figure E.2 e) Shear force line due to prestress.

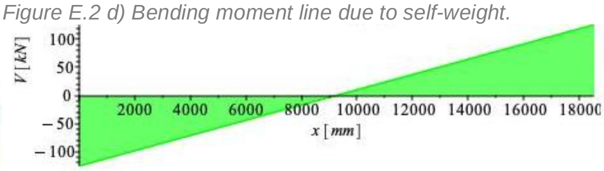


Figure E.2 f) Shear force line due to self-weight.

Figure E.2: Force distribution HIP 800 girder of 18,53 [m] due to self-weight and prestress.

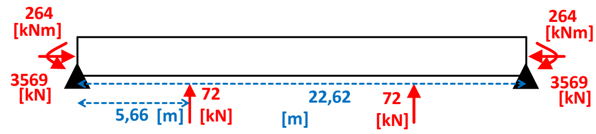


Figure E.3 a) Load distribution prestress.

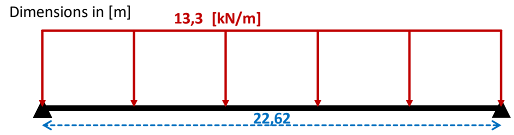


Figure E.3 b) Load distribution self-weight.

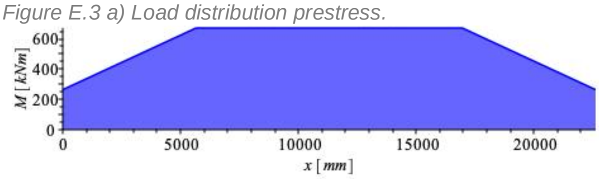


Figure E.3 c) Bending moment line due to prestress

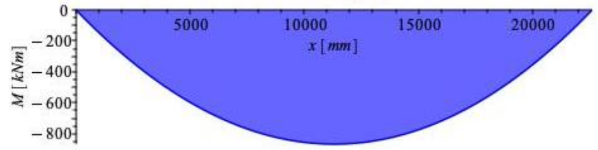


Figure E.3 d) Bending moment line due to self-weight

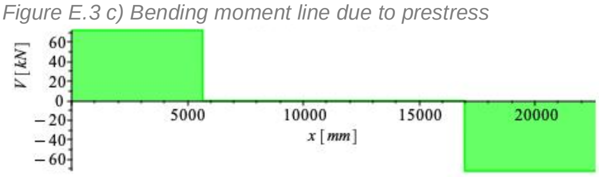


Figure E.3 e) Shear force line due to prestress

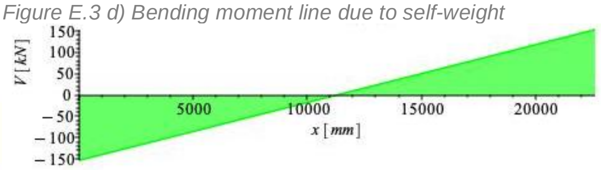


Figure E.3 f) Shear force line due to self-weight

Figure E.3: Force distribution HIP 800 girder of 22,62 [m] due to self-weight and prestress.

E.3.3 Asphalt load

To resist the load of the asphalt layer the girder and the in-situ deck work together. So, the combined cross-sectional dimensions are used. In the old situation the girders form a SI-system. In the new situation SI and SD are both considered. The bending moment and shear force lines are shown in Figure E.4.

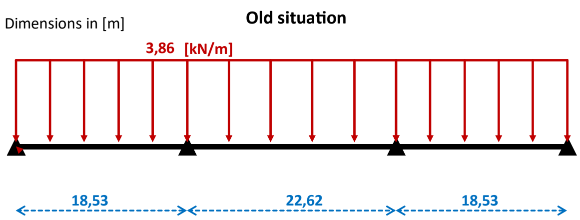


Figure E.4 a) Load distribution old situation.

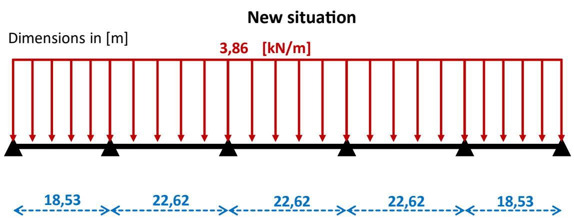


Figure E.4 b) Load distribution in new situation in SI-system.

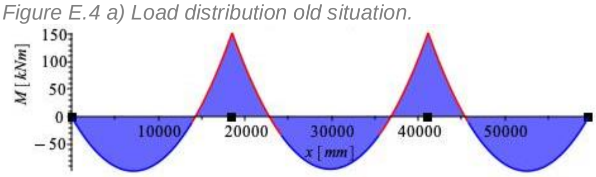


Figure E.4 c) Bending moment line old situation.

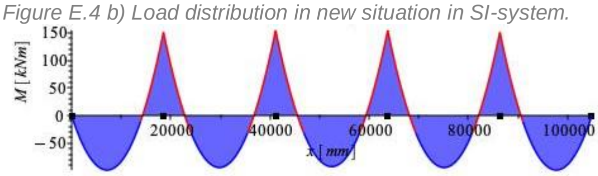


Figure E.4 d) Bending moment line new situation SI-system.

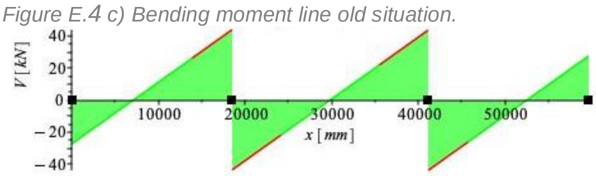


Figure E.4 e) Shear force line old situation.

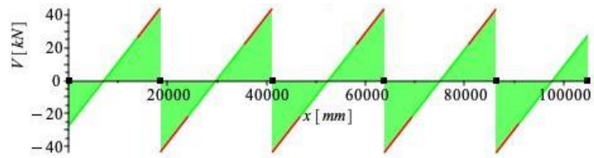


Figure E.4 f) Shear force line new situation SI-system.

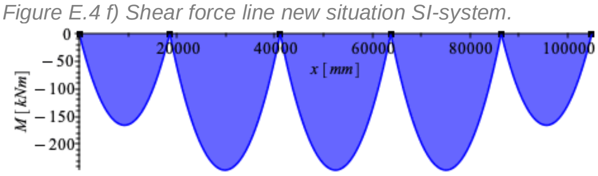


Figure E.4 g) Bending moment line new situation SD-system.

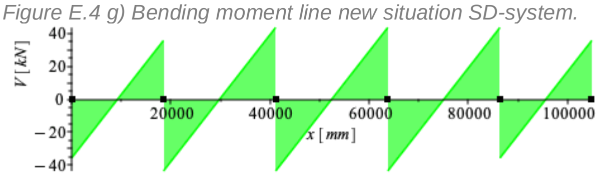


Figure E.4 f) Shear force line new situation SD-system.

Figure E.4: Force distribution in old and new situation due to asphalt load.

E.3.4 Traffic load

Based on the 45° spread the axle load in the new situation on the governing girder is 114 [kN] and the uniformly distributed load 9,07 [kN/m]. For the old situation the axle load is 78 [kN] and the uniformly distributed load 3,62 [kN/m]

In Figure E.5 to Figure E.11 the most unfavourable load configuration and corresponding bending moment and shear force diagram are shown for the old and new SI-system. For a new SD-system the results are shown in Figure E.12 and Figure E.13. Like the prestress and self-weight only one girder is considered.

In the old situation, the load configuration for the maximum hogging bending moment at the support for the 22,62 [m] girder is equivalent to the configuration for the 18,53 [m] girder. In the new situation, the load configuration for the maximum hogging bending moment at the support for the 18,53 [m] girder is equivalent to the load configuration for the maximum hogging bending moment at a distance L_{pt}.

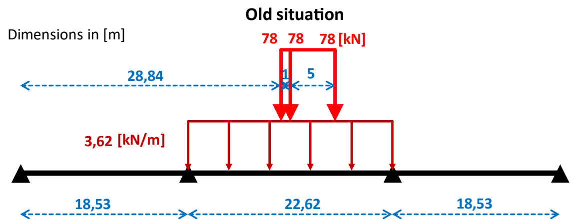


Figure E.5 a) Traffic distribution old situation.

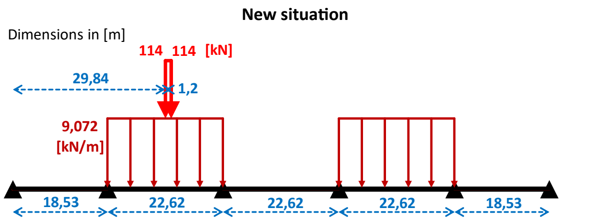


Figure E.5 b) Traffic distribution new situation.

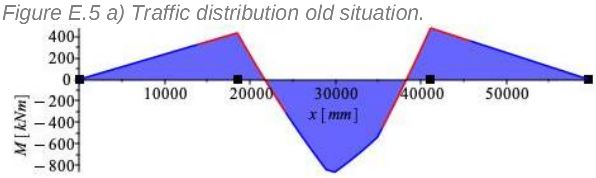


Figure E.5 c) Bending moment old situation.

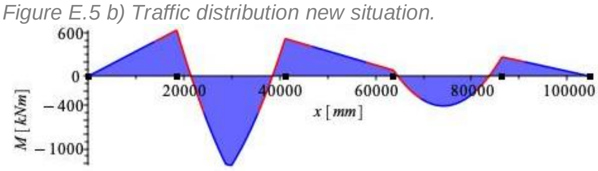


Figure E.5 d) Bending moment new situation.

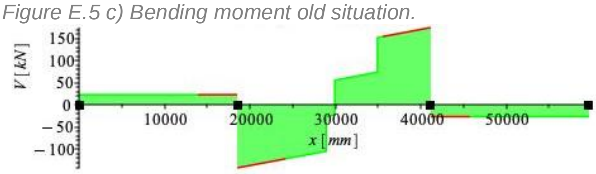


Figure E.5 e) Shear distribution old situation.

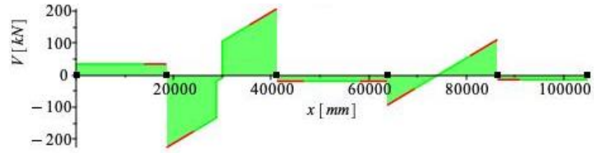


Figure E.5 f) Shear distribution new situation.

Figure E.5: Load distribution for largest sagging bending moment in 22,62 [m] girder due to traffic load.

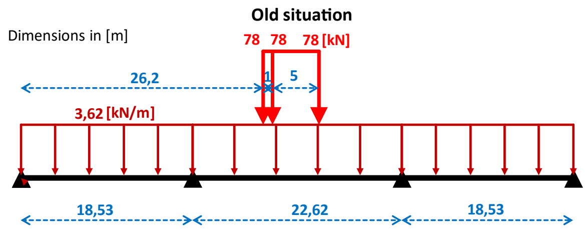


Figure E.6 a) Traffic distribution old situation.

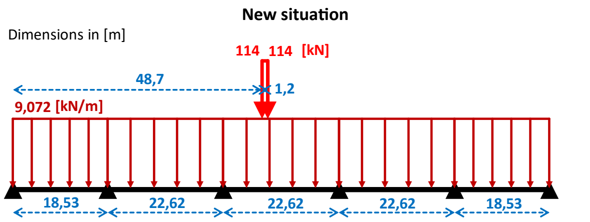


Figure E.6 b) Traffic distribution new situation.

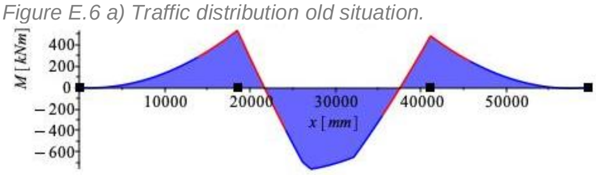


Figure E.6 c) Bending moment old situation.

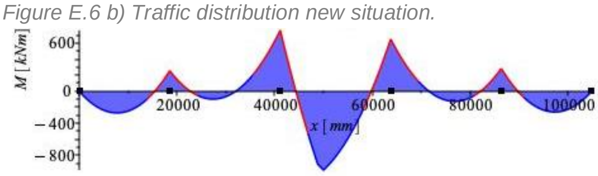


Figure E.6 d) Bending moment new situation.

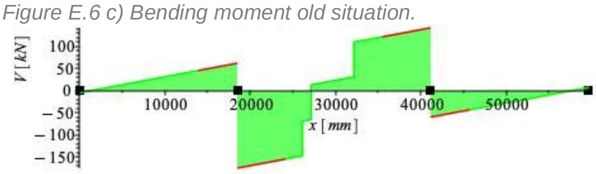


Figure E.6 e) Shear distribution old situation.

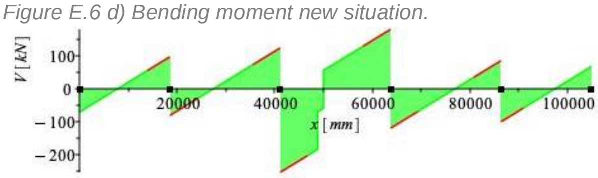


Figure E.6 f) Shear distribution new situation.

Figure E.6: Load distribution for largest hogging bending moment at support in 22,62 [m] girder due to traffic load. In the old situation this configuration corresponds with the load configuraiton for the 18,53 [m] girder.

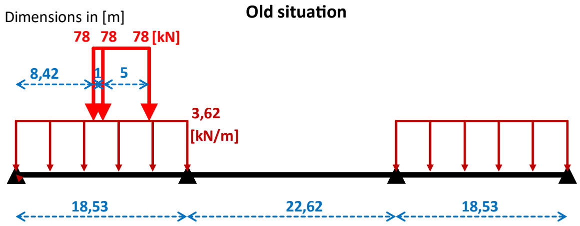


Figure E.7 a) Traffic distribution old situation.

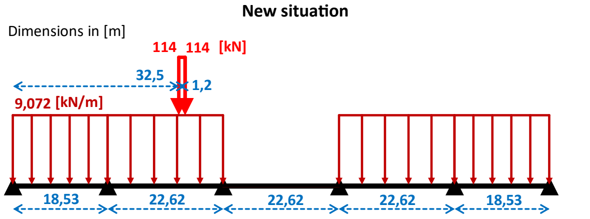


Figure E.7 b) Traffic distribution new situation.

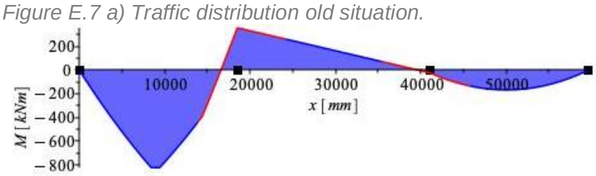


Figure E.7 c) Bending moment old situation.

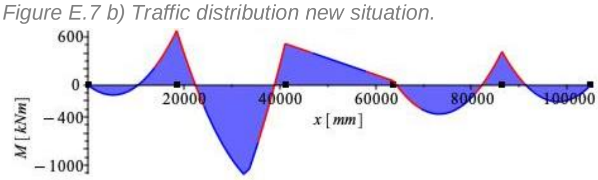


Figure E.7 d) Bending moment new situation.

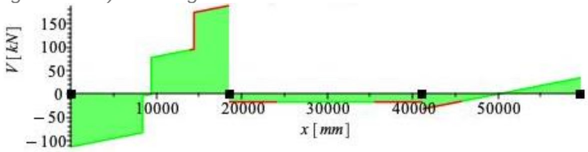


Figure E.7 e) Shear distribution old situation.

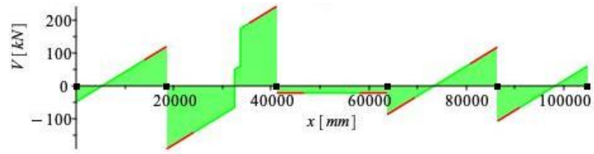


Figure E.7 f) Shear distribution new situation.

Figure E.7: Load distribution for largest hogging bending moment at distance L_{pt} from support in 22,62 [m] girder due to traffic load.

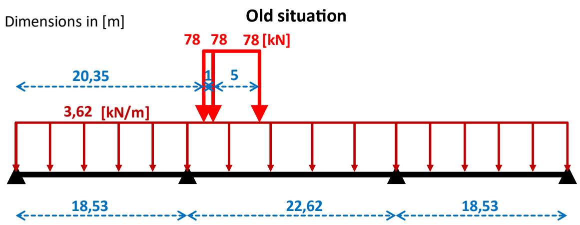


Figure E.8 a) Traffic distribution old situation.

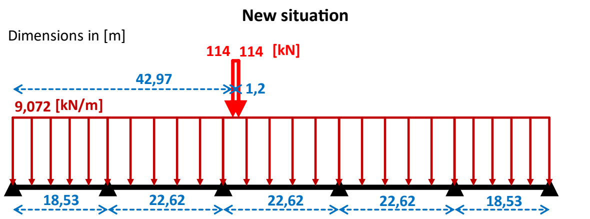


Figure E.8 b) Traffic distribution new situation.

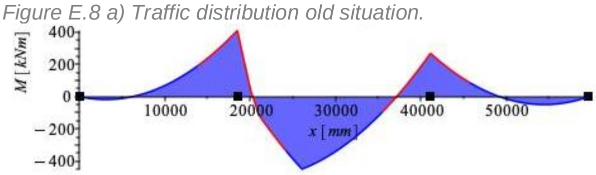


Figure E.8 c) Bending moment old situation.

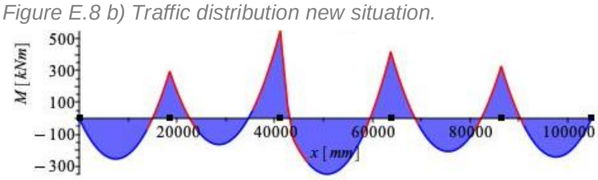


Figure E.8 d) Bending moment new situation.

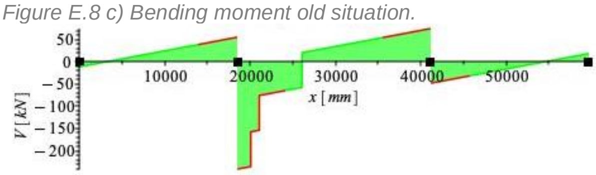


Figure E.8 e) Shear distribution old situation.

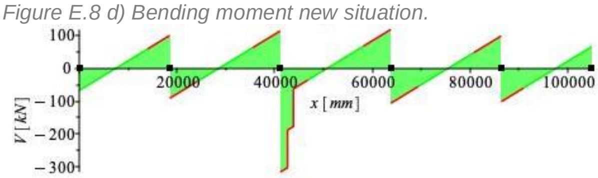


Figure E.8 f) Shear distribution new situation.

Figure E.8: Load distribution for largest shear force in 22,63 [m] girder

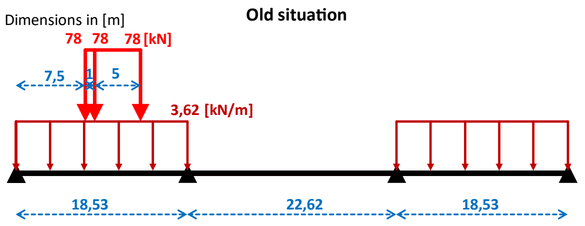


Figure E.9 a) Traffic distribution old situation.

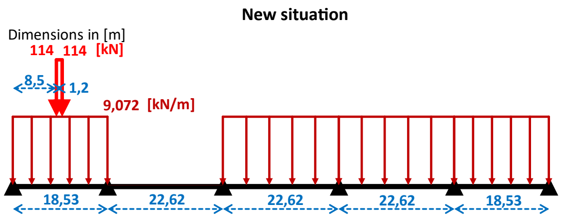


Figure E.9 b) traffic distribution new situation.

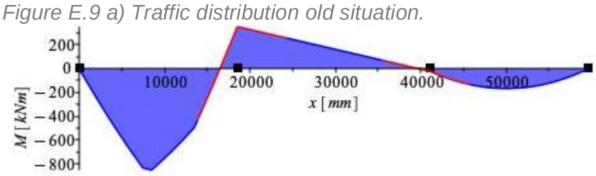


Figure E.9 c) Bending moment old situation.

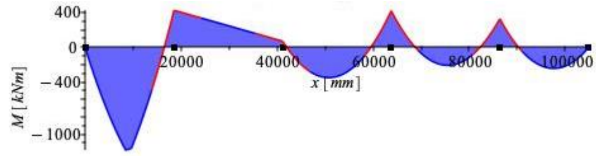


Figure E.9 d) Bending moment new situation.

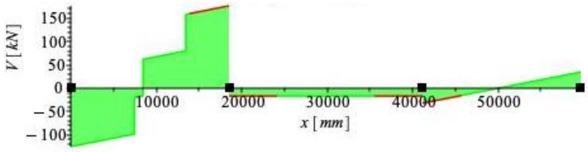


Figure E.9 e) Shear distribution old situation

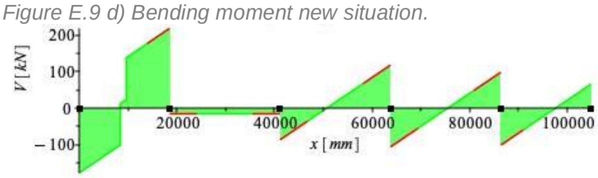


Figure E.9 f) Shear distribution new situation

Figure E.9: Load distribution for largest sagging bending moment in 18,53 [m] girder due to traffic load.

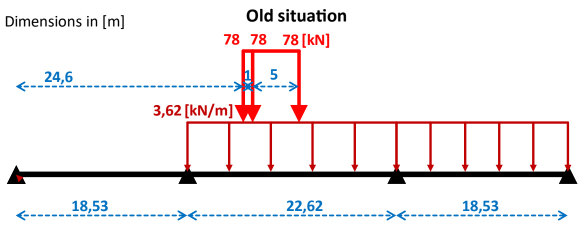


Figure E.10 a) Traffic distribution old situation.

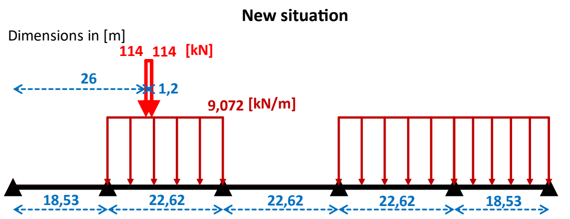


Figure E.10 b) Traffic distribution new situation.

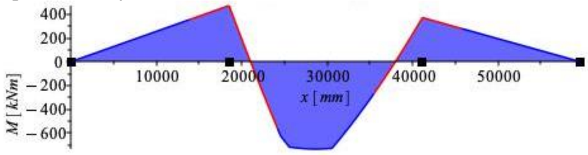


Figure E.10 c) Bending moment old situation

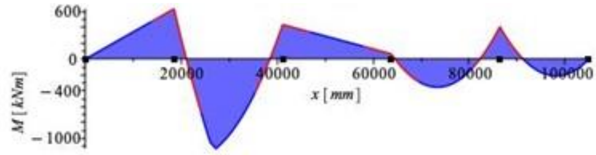


Figure E.10 d) Bending moment new situation.

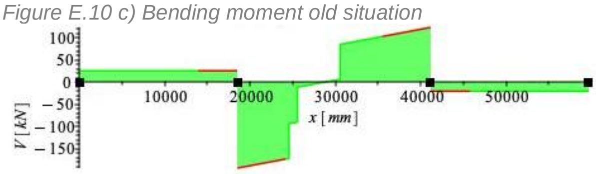


Figure E.10 e) Shear distribution old situation.

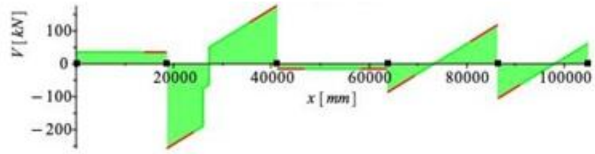


Figure E.10 f) Shear distribution new situation.

Figure E.10: Load distribution for largest hogging bending moment at distance L_{pt} of support due to traffic load in 18,53 [m] girder. In the new situation this configuration corresponds with the configuration for the largest hogging bending moment at the support.

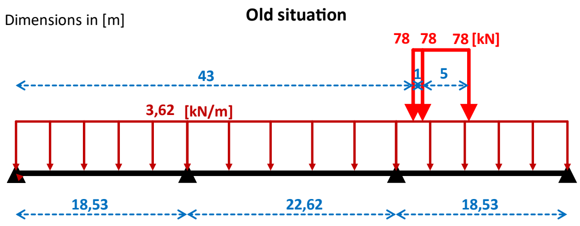


Figure E.11 a) Traffic distribution old situation.

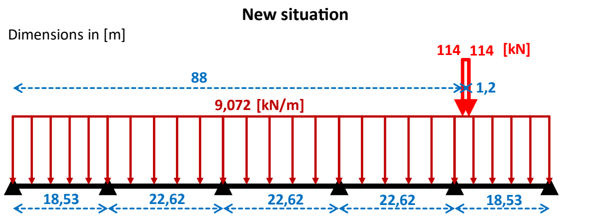


Figure E.11b) Traffic distribution new situation.

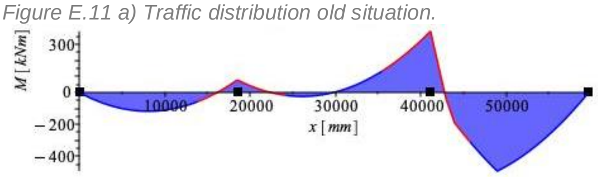


Figure E.11 c) Bending moment old situation.

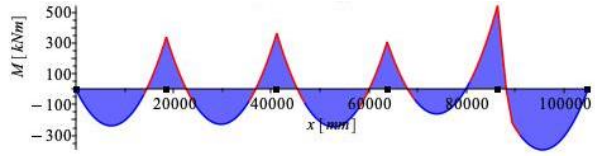


Figure E.11 d) Bending moment new situation.

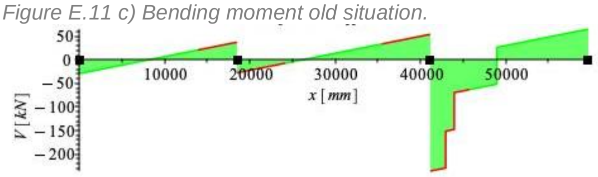


Figure E.11 e) Shear distribution old situation.

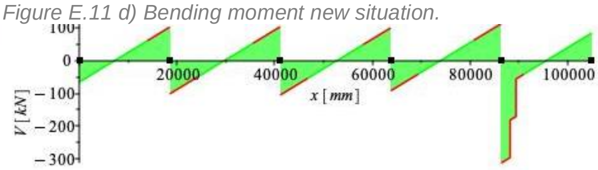


Figure E.11 f) Shear distribution new situation.

Figure E.11: Load distribution for largest shear force in 18,53 [m] girder.

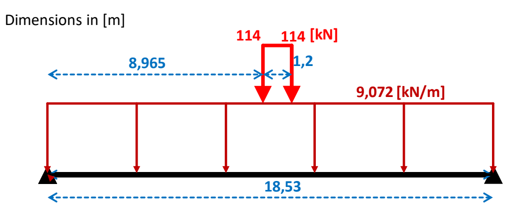


Figure E.12 a) Traffic distribution 18,53 [m] girder.

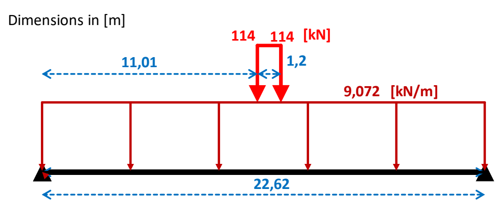


Figure E.12 b) Traffic distribution 22,62 [m] girder.

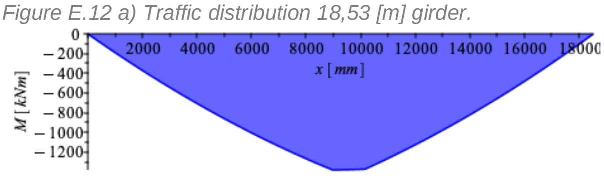


Figure E.12 c) Bending moment 18,53 [m] girder.

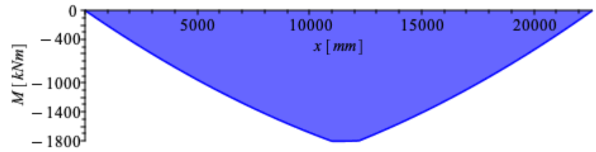


Figure E.12 d) Bending moment 22,62 [m] girder.

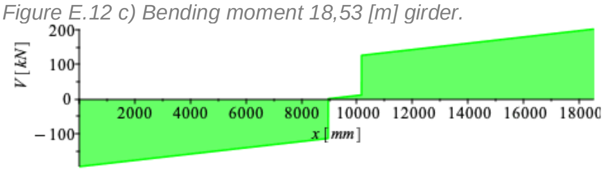


Figure E.12 e) Shear distribution 18,53 [m] girder.

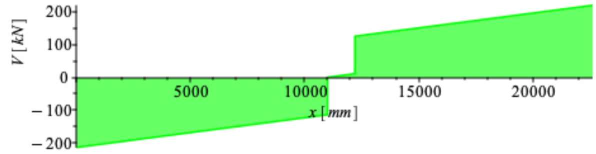


Figure E.12 f) Shear distribution 22,62 [m] girder.

Figure E.12: Load distribution for largest sagging bending moment due to traffic load in new situation with SD-system.

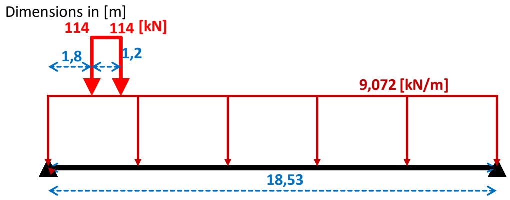


Figure E.13 a) Traffic distribution 18,53 [m] girder.

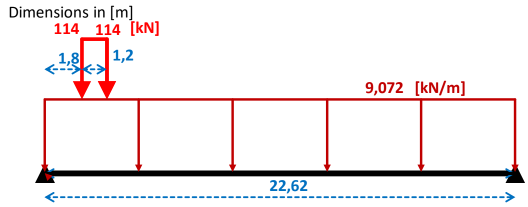


Figure E.13 b) Traffic distribution 22,62 [m] girder.

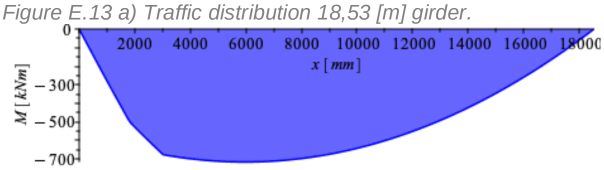


Figure E.13 c) Bending moment 18,53 [m] girder.

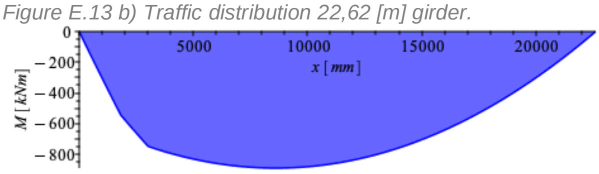


Figure E.13 d) Bending moment 22,62 [m] girder.

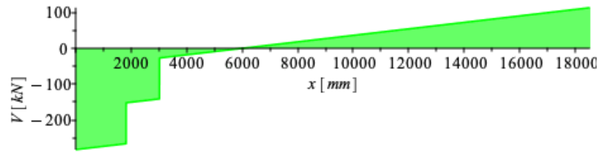


Figure E.13 e) Shear distribution 18,53 [m] girder.

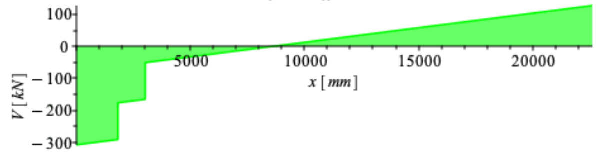


Figure E.13 f) Shear distribution 22,62 [m] girder.

Figure E.13: Load distribution for largest shear force in new situation with SD-system.

E.3.5 Loads on the structure

The results from the previous sections are summarized in Table E.5. With the partial safety factors from Table E.2 this results for the new situation in the design load shown in Table E.6.

In Table E.7 and Table E.8 the loads in the new and old situation are compared. The hogging bending moments that occur in a SD-system are more ore less comparable with safety factors. Without safety factors the load increase is around 20-30%. Only the hogging bending moment after the introduction of the prestress significantly increases for the 22,62 [m] girder in the new situation. This is due to the different span configuration. Due to more spans in the new situation, the bending moment line can be longer on the hogging side. However, this has a small effect on the total maximum hogging bending moment that occurs.

In addition, in a SI-structure additional loads have to be considered due to for example restrained deformations. In former standards these effects are often not included. Due to these effects the load difference will increase.

Only looking at traffic and asphalt load the sagging bending moment increases with 46% without safety factors. With a safety factor of 1,7 this increase is reduced to 22%. Of course, if the system is changed to SD the increase is even more.

The shear force with safety factor is approximately the same. However, it is not sure if this factor is actually applied in such manner on shear forces. For the traffic load the increase is around 25-30%. This is due to the differences in load models. In the old standards the load is divided over three axles, while in the current standard over two. In this way the space and thereby the spread in force is larger in the older situation.

Table E.5: Characteristics loads on critical cross-sections. Values in black are based on the differential equation. Values in grey come from the SCIA-model.

Situation				Location [m]	Bending moment [kNm] or shear force [kN]			
					Prestress	Self-weight	Asphalt	Traffic load
22,62 [m] girder	1. Sagging moment	Old		29,84	671	- 851	- 100 - 94	-883 -711
		New	SI	29,84	671	- 851	- 99 - 94	-1249 -899
			SD				-247 -247	-1807 -1319
	2. Hogging moment	Old		18,53	0*	0	147 166	513 547
		New		41,15	0*	0	151 166	735 725
	4. Hogging moment Lpt	Old		19,25	316	-105	122	343
		New		42,13	316	-105	121 124	540 426
	2. Shear at support	Old		18,53	-72	151	44 48	241 273
		New	SI	41,15	-72	151	44 48	313 447
			SD				44	307 305
	3. Shear next to support	Old		19,03	-72	144	42 41	237
		New	SI	41,65	-72	144	42 41	309
			SD				42 42	302 294
18,53 [m] girder	1. Sagging moment	Old		8,47	514	-567	-98	-851
		New	SI	8,46	514	-567	-97 -89	-1182 -895
			SD	9,27			-166 -166	-1381 -972
	2. Hogging moment	Old		18,53	0*	0	147	513
		New		18,53	0*	0	146	637
	4. Hogging moment Lpt	Old		17,81	224	-86	116	456
		New		17,81	224	-86	116	613
	2. Shear at support	Old		41,15	-74	123	44	236
		New	SI	86,39	-74	123	44	312
			SD				36	283
	3. Shear next to support	Old		41,65	-74	117	42	234
		New	SI	86,89	-74	117	42	308
			SD				34	279

* *Prestress is not yet introduced.*

Table E.6: Design loads in new situation.

System	Statically determinate (SD)				Statically indeterminate (SI)			
Girder	22,62		18,53 [m]		22,62 [m]		18,53 [m]	
Loads	All	Asphalt + traffic	All	Asphalt + traffic	All	Asphalt + traffic	All	Asphalt + traffic
$M_{sag,ed,SLS}$ [kNm]	-1746	-1566	-1191	-1138	-1173	-993	-1037	-984
$M_{sag,ed,ULS}$ [kNm]	-2427	-2077	-1679	-1511	-1677	-1326	-1481	-1315
$M_{hog,ed,SLS}$ [kNm]	-	-	-	-	876	876	783	783
$M_{hog,ed,ULS}$ [kNm]	-	-	-	-	1160	1160	1035	1035
$M_{hog,ed,SLS,L_{pt}}$ [kNm]	-	-	-	-	872	752	867	729
$M_{hog,ed,ULS,L_{pt}}$ [kNm]	-	-	-	-	1189	983	1113	967
$V_{ed,support}$ [kN]	576	467	499	425	585	475	548	474
$V_{ed,cross\ beam}$ [kN]	559	458	484	417	569	468	533	466

Table E.7: Comparison old and new 'design' loads for 22,62 [m] long girder.

Girder	22,62 [m]					
Loads	All	Difference [%] SD	Difference [%] SI	Asphalt + traffic	Difference [%] SD	Difference [%] SI
M_{sag} [kNm] * 1,7	-985 -1675	+77% +45%	+19% +/-0%	-805 1369	+95% +52%	+46% +22%
M_{hog} [kNm] * 1,7	660 1122		+33% +3%	660 1122		+33% +3%
$M_{hog,L_{pt}}$ [kNm] * 1,7	676 1149		+29% +3%	465 791		+62% +24%
$V_{ed,support}$ [kN] * 1,7	364 619	+58% -7%	+61% -5%	285 485	+64% -4%	+67% -2%
$V_{ed,cross\ beam}$ [kN] * 1,7	351 597	+59% -6%	+62% -5%	279 474	+64% -3%	+68% -1%

Table E.8: Comparison old and new 'design' loads for 18,53 [m] long girder.

Girder	18,53[m]					
Loads	All	Difference [%] SD	Difference [%] SI	Asphalt + traffic	Difference [%] SD	Difference [%] SI
M_{sag} [kNm] * 1,7	-1002* -1703*	+19% -1%	+3% -13%	-949* 1613*	+20% -6%	+4% -18%
M_{hog} [kNm] * 1,7	660 1122		+19% -8%	660 1122		+19% -8%
$M_{hog,L_{pt}}$ [kNm] * 1,7	710 1207		+22% -8%	660 1122		+10% -14%
$V_{ed,support}$ [kN] * 1,7	329 559	+52% -11%	+67% -2%	280 476	+52% -11%	+69% 0%
$V_{ed,cross\ beam}$ [kN] * 1,7	319 542	+52% -11%	+67% -2%	276 469	+51% -11%	+69% -1%

**Overestimation, because based on differential equation*

E.3.6 Shear capacity

From Equation E.2 follows that a stirrup spacing of 300 [mm] is insufficient to meet the minimum shear reinforcement ratio. Even with a spacing of 100 [mm] around the supports, the problem remains. The minimum reinforcement ratio ensures that the reinforcement yields before failure and thus prevents brittle failure. Therefore, the ratio should be met at all cross-sections and not just at the supports. The minimum reinforcement ratio is not a requirement in the RBK because for the assessment of existing structures it is not needed to build in extra safety. However, only if the minimum ratio is met the capacity of concrete and stirrups can be combined.

Equation E.2: Minimum shear reinforcement ratio.

$$\rho_w = \frac{A_{sw}}{s \times b_w \times \sin(\alpha)} \geq \frac{0{,}08 \times \sqrt{f_{ck}}}{f_{yk}}$$

$$\rho_{w,s=100} = 0{,}3656\ [\%] > 0{,}1483[\%]$$
$$\rho_{w,s=300} = 0{,}1117\ [\%] < 0{,}1483[\%]$$

$A_{sw} = 0{,}5 \times \pi \times 8^2 = 101\ [mm^2]$
$b_w = 300\ [mm]$
$s = 300\ [mm]$
$\alpha = 90\ [°]$
$f_{ck} = 50\ [N/mm^2]$
$f_{yk} = 400\ [N/mm^2]$

Next in Equation E.3 and Table E.9 the resistance of concrete is calculated. It is assumed that the crossbeam is 1 [m] wide. So, at the location of cross-section 2 the prestress had 500 [mm] to get partly introduced. The introduction of prestressing occurs linearly over the transmission length. So, for the 22,62 [m] girder from 0 [N/mm^2] to -9,8 [N/mm^2] over 722 [mm]. And for the 18,53 [m] girder from 0 [N/mm^2] to -7,2 [N/mm^2] over 722 [mm].

At the supports of a SI-system system tension occurs at the top. The prestress is located at the bottom, so instead of closing the cracks, the prestress widens the cracks. Therefore, it is not allowed to include the prestressing force with a SI-system.

The effective height (d) is calculated as the height minus 50 [mm], because the first prestressing tendon is located at 50 [mm] from the bottom.

Equation E.3: Shear resistance concrete.

$$V_{rd,c} = \left(0{,}035 \times k^{\frac{3}{2}} \times \sqrt{f_{ck}} + 0{,}15 \times \sigma_{cp}\right) \times b_w \times d$$

$k = 1 + \sqrt{\frac{200}{d}} = 1 + \sqrt{\frac{200}{910}} = 1{,}47\ \leq 2$

$\sigma_{cp} = \frac{F_p}{b_w \times d} < 0{,}2 \frac{f_{ck}}{\gamma_c}$

Table E.9: Shear resistance of concrete.

Location	Element		Input				Result [kN]
			f_{ck} [N/mm^2]	σ_{cp} [N/mm^2]	b_w [mm]	d [mm]	
2: At support	Girder		55	0,0	1200	750	416
	Deck		30	0,0	1200	160	66
						$V_{rd,c}$	**481**
3: Next to support SD	Girder	22,62 [m]	55	6,8	300	750	334
		18,53 [m]		5,0			272
	Deck	NEN	30	0,0	300	160	16
		RBK			375		25
						22,62 [m] $V_{rd,c}$ NEN	**350**
						18,53 [m] $V_{rd,c}$ NEN	**289**
						22,62 [m] $V_{rd,c}$ RBK	**359**
						18,53 [m] $V_{rd,c}$ RBK	**297**
3: Next to support SI	Girder		55	0,0	300	750	104
	Deck	NEN	30	0,0	300	160	16
		RBK			375		25
						$V_{rd,c}$ NEN	**120**
						$V_{rd,c}$ RBK	**129**

In Equation E.4 and Table E.10 the resistance of the shear reinforcement in calculated. The reinforcement consists of two-edged stirrups. The internal lever arm is taken as 90% of the effective height. The angle of the compressive strut is 30° according to the RBK. The NEN-EN 1992-1-1 allows a lower angle of 21,8°.

The force in the concrete compressive strut increases with a lower angle, but according to Equation E.5 this is not a critical factor. In this equation the factor α_{cw} accounts for prestressing. Because the distance of cross-section 2 and 3 is within $0{,}5 \times d \times \cot(\theta) = 1{,}14\ [m]$ from the supports the prestress is not considered. The value for v_1 is equal to 0,6 for concrete classes lower than C60/75.

Equation E.4: Shear resistance reinforcement

$$V_{rd,s} = \frac{A_{sw}}{s} \times z \times f_{yd} \times \cot(\theta)$$

$A_{sw} = 2 \times \frac{1}{4} \times \pi \times \emptyset^2 = 2 \times \frac{1}{4} \times \pi \times 8^2 = 101\ [mm^2]$
s = spacing stirrups
$z \approx 0{,}9d = 819\ [mm]$
$f_{yd} = 348\ [N/mm^2]$
θ = angle of compressive strut

Table E.10: Shear resistance reinforcement

Verification	Angle of compressive strut [°]	Spacing [mm]	$V_{rd,s}$ [kN]
NEN	21,8	100	716
		300	239
RBK	30	100	496
		300	165

Equation E.5: Resistance concrete compressive strut

$$V_{rd,c,max} = \frac{\alpha_{cw} \times b_w \times z \times v_1 \times f_{cd}}{\cot(\theta) + tan\ (\theta)} = \frac{1 \times 300 \times 819 \times 0{,}6 \times \frac{55}{1{,}5}}{\cot(21{,}8) + tan\ (21{,}8)} = 1864\ [kN]$$

From these results the shear resistance of the girder and deck is determined in Table E.11. Together with the design loads of Table E.6 this results in the Unity Checks (UC) in Table E.12.

From these UCs it can be concluded that the girders with a stirrup spacing of 100 [mm] around the supports are able to meet the regulations for new to build structures. If the spacing is 300 [mm] the capacity according to the NEN and RBK is similar and insufficient. In a SI-system the UC is even higher than 2.

It can be concluded that for in SI-structure the shear capacity of the reinforcement should be able to withstand the total shear force. Although according to the RBK the capacity of shear reinforcement and concrete can be combined, the computed capacity is smaller compared to NEN-EN 1992-1-1. This is because the RBK uses a higher angle, which activates less stirrups. In a SD-system this effect is exceed by an increase of concrete capacity. However, in a SI-system this effect is too small because there is no beneficial effect from prestress.

Table E.11: Shear resistance

Situation	Girder length [m]	System	Standard	V_{rd}	
				S=100	S=300
2. At support	22,62 + 18,53	SI + SD	NEN	716	481
			RBK	978	481
3. Next to support	22,62	SD	NEN	716	350
			RBK	932	359
	18,53	SD	NEN	716	289
			RBK	494	297
	22,62 +18,53	SI	NEN	716	239
			RBK	855	165

Table E.12: UC on shear capacity

Standard		NEN		RBK	
Spacing		S=100 [mm]	S=300 [mm]	S=100 [mm]	S=300 [mm]
Situation		$U.C = \frac{V_{ed}}{V_{rd}} \leq 1$			
Girder 22,62 [m]	2. At support SI	0,82	1,22	0,60	1,22
	2. At support SD	0,80	1,20	0,59	1,20
	3. Next to support SI	0,79	2,38	0,91	3,44
	3. Next to support SD	0,78	1,60	0,65	1,56
Girder 18,53 [m]	2. At support SI	0,76	1,14	0,56	1,14
	2. At support SD	0,70	1,04	0,51	1,04
	3. Next to support SI	0,74	2,23	0,85	3,22
	3. Next to support SD	0,68	1,68	0,61	1,63

To make the girders with a stirrup spacing of 300 [mm] in a SD-system suitable there are two options.

- Increasing the shear capacity: can be done by for example CFRP sheets. It requires detailed calculations. Moreover, it makes the alternative more expensive. For this case study, this option is not further investigated.
- Reducing the load on the girder: more simple solution, which can be achieved by putting restriction on traffic or reducing the width of the girder. This last option is a solution if only the shear capacity next to the supports is insufficient and the capacity at the support is sufficient. Since, in this case the capacity at the supports is insufficient as well, the only option is putting restriction on traffic load.

To give an indication of the load restriction. The total shear force is 576 [kN], which should be lowered to 481 [kN]. Because all other loads remain the same, the traffic load should be lowered from 307 [kN] to 237 [kN]. 103 [kN] is due to the uniformly distributed load, which leaves 134 [kN] for the axle load. This corresponds with two forces of 76 [kN] on the girder. Translating this back to the load model the wheel load should be lowered from 142,5 [kN] to 95 [kN]. In this way the unity at the support is lowered to 1,0. In the Netherlands the maximum allowed vehicle load is 50 [tons]. Assuming this load is linearly related to the vehicle load model, the maximum allowed vehicle load is 33 [tons]. This assumption is incorrect because vehicles of 50 [tons] have often more than two axles and other distances. However, it provides an indication. This might be a suitable solution because the bridge is only used to access a residential area and exceptional transport can be done by the water.

For the cross section next to the supports the shorter girder is governing and a reduction to 95 [kN] only lowers the UC to 1,35, which is still insufficient. To get the UC below 1,0 the girders can be shortened in width direction. By shortening in width direction, the same asphalt and traffic load is present on the bridge, but it is distributed over more girders. So, the load on the girder reduces in same ratio as the reduction in width. However, the self-weight also reduces because the area of the cross-section reduces. The cross-section is not rectangular; hence the self-weight does not reduce in the same proportion as the reduction in width. Therefore, to give an indication about the shortened width, the width is iteratively changed until the shear force next to the support is below 289 [kN]. If 200 [mm] is cut on both sides, the width becomes 790 [mm] and the shear force becomes 251 [kN], which gives an UC of 0,87.

E.3.7 Bending moment capacity

Figure E.14 shows the stress-distribution over the height of the girder and deck in SLS. Prestress and self-weight are resisted by the girder and asphalt and traffic loads are resisted by the combined action of girder and deck. If the girders are used in a SD-system, tensile stresses develop. The maximum tensile stress is 2,48 [N/mm^2], which is below the flexural design tensile strength of 2,8 [N/mm^2]. So, no cracking will occur. If the girders are used in a SI-system, no tensile stresses develop and no cracking occurs.

A similar situation occurs when the girders are shortened in width direction to 790 [mm]. This is shown in Figure E.15. Due to this shortening the cross-section characteristics such as area and location of neutral axis change. In the 22,62 girder 8 tendons are cut off and in the 18,53 [m] girder 6. However, the load as well as the self-weight decreases.

The next step is the ULS analysis. The bending moment resistance for the asphalt and traffic loads of the 22,62 [m] girder is 3256 [kNm]. This resistance is based on the values and graphs from Figure E.16. The assumptions made are verified with subfigure b. The ultimate strain in the girder is 2,63 [‰], which is below the rupture strain of 3,13 [‰]. The minimum strain in the deck is 1,92 [‰], which is higher than the strain at peak stress (ε_{c3}) of 1,75 [‰].

The bending moment resistance for the asphalt and traffic load of the 18,53 [m] girder is 2576 [kNm]. This resistance is based on the values and graphs from Figure E.17. For this situation the assumption that the full capacity of the deck is used is false, because the minimum strain in the deck was below the strain at peak stress of 1,75 [‰]. Therefore, new iterations are performed. In these iterations the alfa and beta of the girder are set on 0,5 and 0,33 respectively and the alfa and beta values of the deck are iteratively changed.

If the girders are shortened to 790 [mm] the ultimate sagging bending moment resistance is 2353 [kNm] and 1858 [kNm] for the 22,62 [m] and 18,53 [m] girder respectively. The values and graphs are shown in Figure E.18 and Figure E.19. In Table E.13 the results are summarized together with the UC.

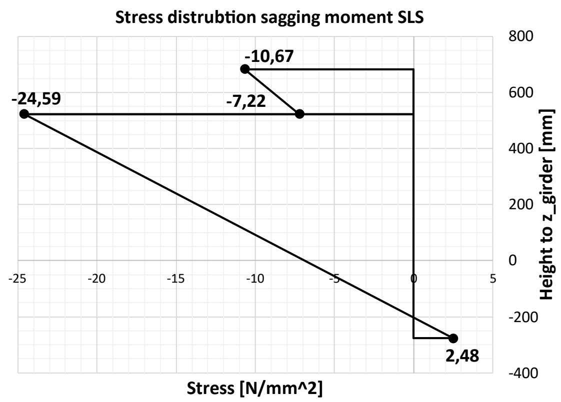


Figure E.14 a) 22,62 [m] girder in SD-system.

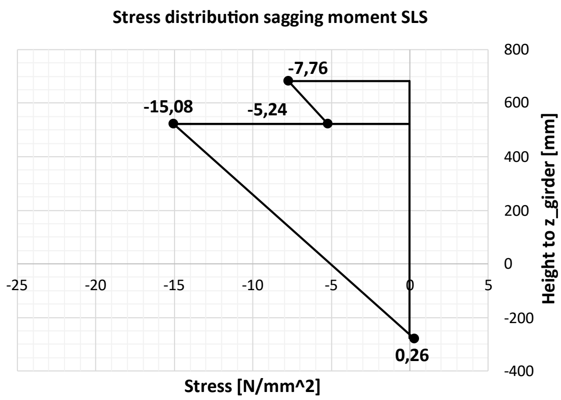


Figure E.14 b) 18,53 [m] girder in SD-system.

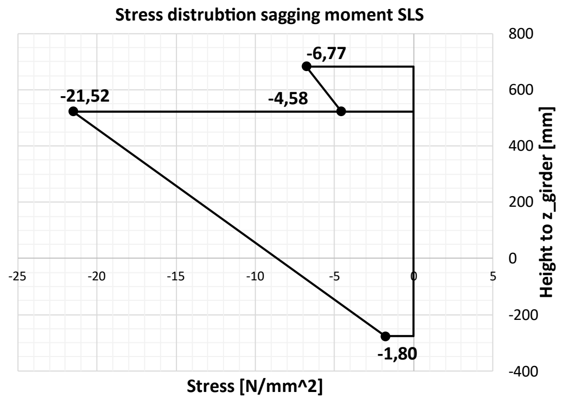


Figure E.14 c) 22,62 [m] girder in SI-system.

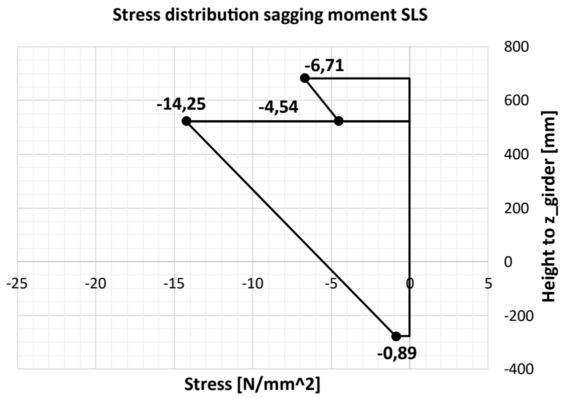


Figure E.14 d) 18,53 [m] girder in SI-system.

Figure E.14: SLS verification sagging bending moment for girders with original width of 1200 [mm].

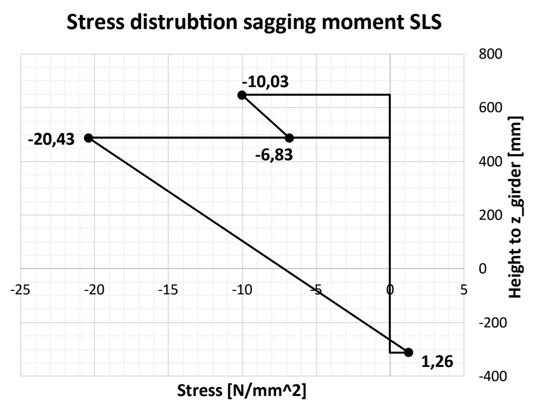


Figure E.15 a) 22,62 [m] girder in SD-system.

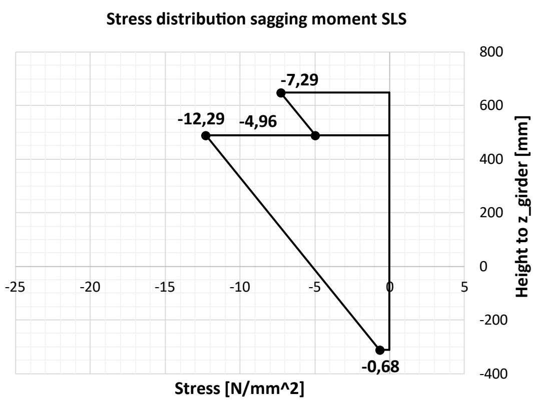


Figure E.15 b) 18,53 [m] girder in SD-system.

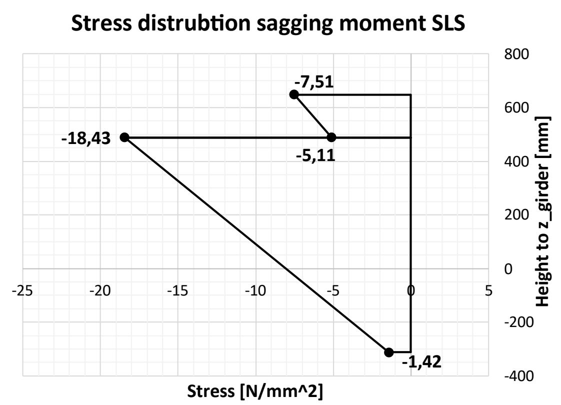


Figure E.15 c) 22,62 [m] girder in SI-system.

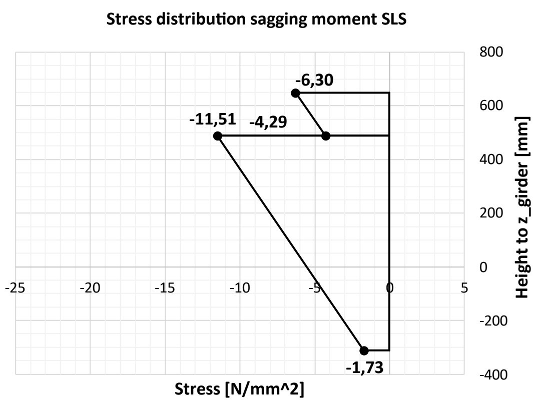


Figure E.15 d) 18,53 [m] girder in SI-system.

Figure E.15: SLS verification sagging bending moment for girders shortened in width direction to 790 [mm].

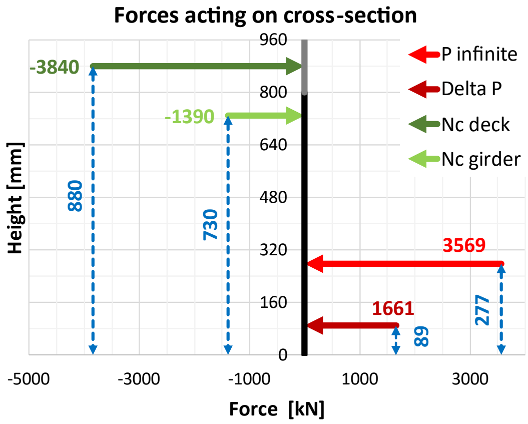


Figure E.16 a) Force distribution.

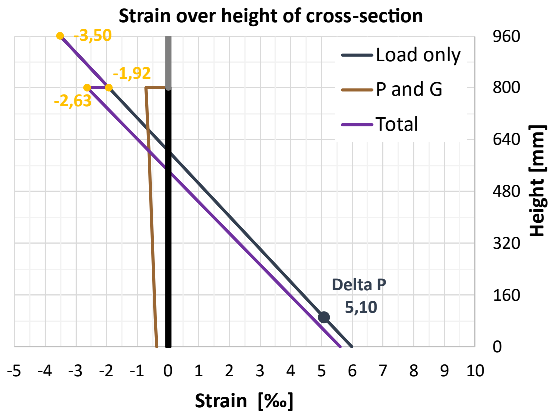


Figure E.16 b) Strain distribution.

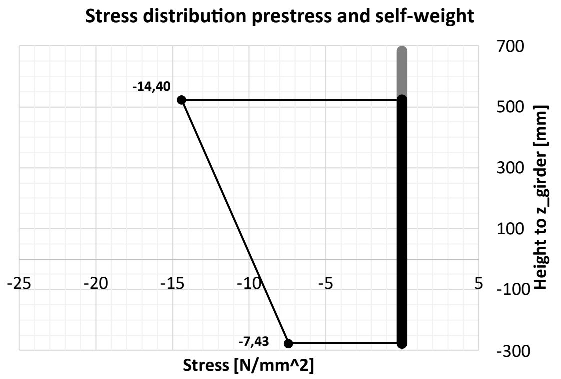


Figure E.16 c) Stress distribution for self-weight and prestress.

Assumed steel stress	1502	[N/mm^2]
Steel strain	11,26	[‰]
Initial steel strain	6,22	[‰]
Assumed delta P	5,04	[‰]
α girder	0,65	[-]
β girder	0,36	[-]
α deck	1,00	[-]
β deck	0,50	[-]
Resistance	3256	[kNm]

Figure E.16 d) Values for steel and concrete stress and strain.

Figure E.16: Results for sagging bending moment resistance of 22,62 [m] girder with original width of 1200 [mm].

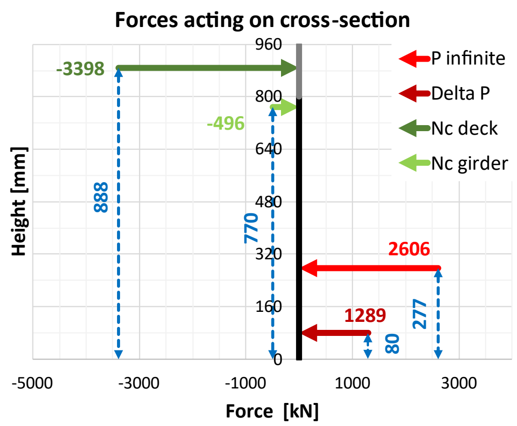


Figure E.17 a) Force distribution.

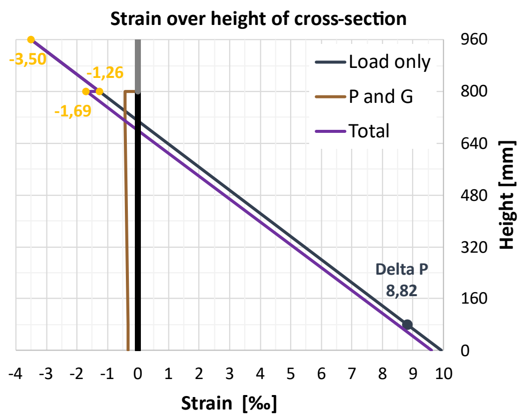


Figure E.17 b) Strain distribution.

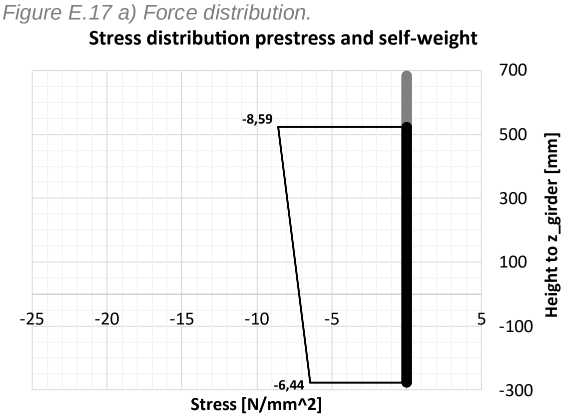


Figure E.17 c) Stress distribution for self-weight and prestress.

Assumed steel stress	1532	[N/mm^2]
Steel strain	14,97	[‰]
Initial steel strain	6,22	[‰]
Assumed delta P	8,75	[‰]
α girder	0,50	[-]
β girder	0,33	[-]
α deck	0,89	[-]
β deck	0,55	[-]
Resistance	2576	[kNm]

Figure E.17 d) Values for steel and concrete stress and strain.

Figure E.17: Results for sagging bending moment resistance of 18,53 [m] girder with original width of 1200 [mm].

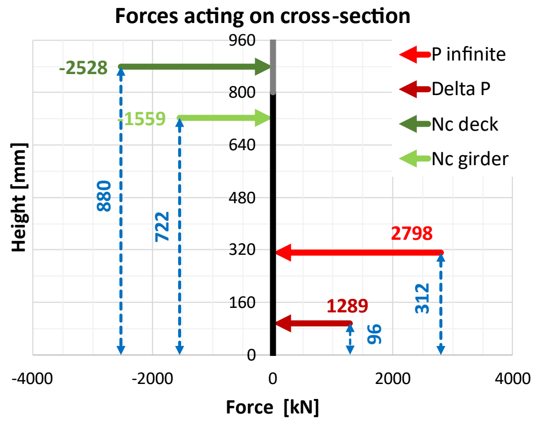


Figure E.18 a) Force distribution.

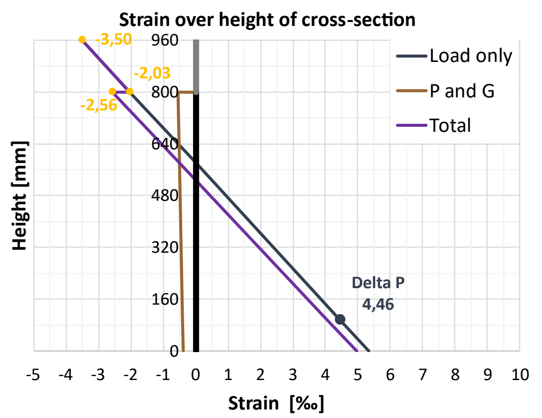


Figure E.18 b) Strain distribution.

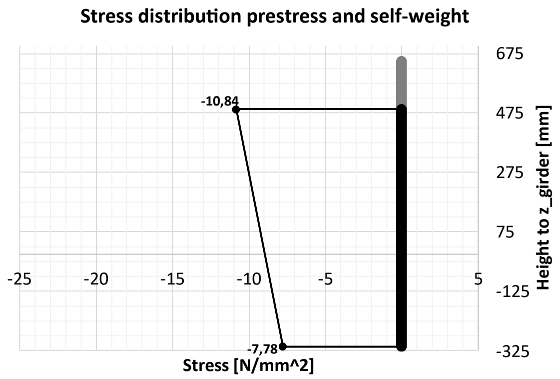


Figure E.18 c) Stress distribution for self-weight and prestress.

Assumed steel stress	1497	[N/mm^2]
Steel strain	10,64	[‰]
Initial steel strain	6,22	[‰]
Assumed delta P	4,42	[‰]
α girder	0,64	[-]
β girder	0,36	[-]
α deck	1,00	[-]
β deck	0,50	[-]
Resistance	2353	[kNm]

Figure E.18 d) Values for steel and concrete stress and strain.

Figure E.18: Results for sagging bending moment resistance of 22,62 [m] girder with a width of 790 [mm].

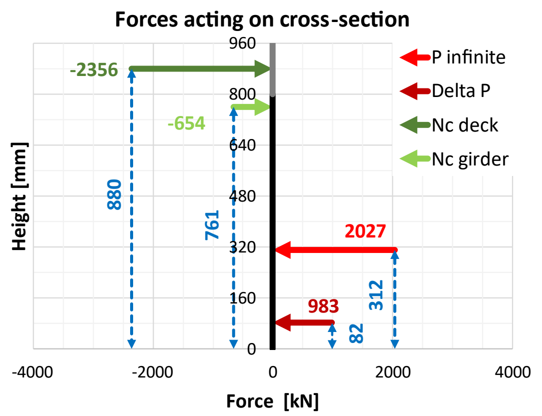


Figure E.19 a) Force distribution.

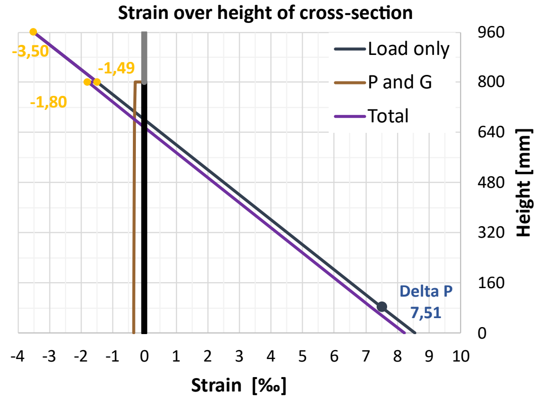


Figure E.19 b) Strain distribution.

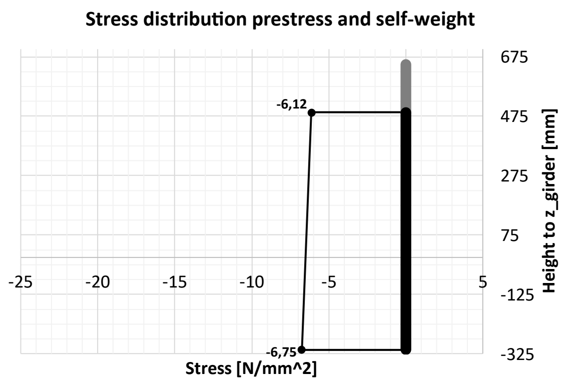


Figure E.19 c) Stress distribution for self-weight and prestress.

Assumed steel stress	1522	[N/mm^2]
Steel strain	13,73	[‰]
Initial steel strain	6,22	[‰]
Assumed delta P	7,52	[‰]
α girder	0,50	[-]
β girder	0,33	[-]
α deck	0,93	[-]
β deck	0,55	[-]
Resistance	1858	[kNm]

Figure E.19 d) Values for steel and concrete stress and strain.

Figure E.19: Results for sagging bending moment resistance of 18,53 [m] girder with a width of 790 [mm].

Table E.13: Verification sagging bending moment resistance.

Length [m]	System	Width 1200 [mm]			Width 790 [mm]		
		M_{rd} [kNm]	M_{ed} [kNm]	UC $=\frac{M_{ed}}{M_{rd}}$	M_{rd} [kNm]	M_{ed} [kNm]	UC $=\frac{M_{ed}}{M_{rd}}$
22,62	SD	3256	2077	0,64	2353	1367	0,58
	SI		1326	0,41		873	0,37
18,53	SD	2576	1511	0,59	1858	995	0,54
	SI		1315	0,51		866	0,47

In the SI-system hogging bending moments occur near the supports. This moment is resisted by a tensile force in the reinforcement in the deck and a compressive force in the bottom of the girder. The reinforcement layout in the deck will be uniform over the length of the bridge. Therefore, no distinction is made between the 22,62 [m] and the 18,53 [m] girder. The width of the girder also has no influence. From Equation E.6 follows that the reinforcement ratio of the deck should be 1,74 [%]. This can be achieved with two layers of reinforcement with bar diameter 12 [mm] and spacing 75 [mm] or diameter 16 [mm] and spacing 125 [mm].

Equation E.6: Reinforcement in deck

$$M_{ed} < M_{rd}$$

$$M_{ed} = \frac{M_{hog,ed,ULS}}{b_{girder}} = \frac{1189}{1,2} = 991 \text{ [kNm/m]}$$

$$M_{rd} = A_s \times f_{yd} \times z$$

$$z \approx 0,9 \times d \approx 819 \text{ [mm]}$$

$$f_{yd} = 435 \text{ [N/mm}^2\text{]}$$

$$A_s = \frac{0,25 \times \pi \times \phi^2}{s} > 2781 \text{ [mm}^2\text{/m]} \quad \rightarrow 2 \times \phi 12 - 75$$

$$\rho_{required} = \frac{A_s}{1000 \times h_{deck}} = 1,74 \text{ [\%]} \quad \rightarrow 2 \times \phi 16 - 125$$

From these results it can be concluded that SLS is more critical compared to ULS. In a SD-system the SLS requirement of not cracking is met, but there is not much capacity left. If the girders are shortened in width direction, the situation improves. So, the reduction in traffic load and self-weight exceeds the effect of loss of prestress. The hogging bending moment that occurs in a SI-system will not cause any problems. The reinforcement ratio is 1,75 [%], which remains well below the maximum allowed reinforcement ratio of 4 [%]. The minimum spacing between rebars is the maximum of the rebar diameter, the maximum aggregate size plus 5 [mm] or 20 [mm]. The deck height is 160 [mm]. So, with a cover of 50 [mm] the vertical space between the rebars is around 44 [mm]. The minimum horizontal space is 75 [mm], which leaves a maximum aggregate size of 39 [mm]. Thus, this will no result in any problems.

E.3.8 Shortening possibilities

Although shortening in length is not needed in this alternative 5 strut and tie models are made for the 22,62 [m] girder as it may provide valuable insights. The grey areas in the models mark the required concrete area to resist the tensile forces. The new transfer length is 495 [mm]. This is based on a σ_{pm0} of 1051 [N/mm2], which is the maximum stress that can be currently present in the prestressing steel. The design tensile strength of concrete is taken as the design strength belonging to C55/67.

First the prestressing force is distributed over the height. In Figure E.20 this is done for the force and stress distribution in the current situation, without shortening. In Figure E.21 the situation with shortening more than 25% on both sides is investigated. In this case spalling stresses develop due to the tensile zone at the end of the beam. Moreover, the tensile stress at the top of the girder is larger than the flexural tensile strength of concrete. As a result, the girder will crack at the top. This should be avoided, which makes shortening only possible until the flexural tensile strength of concrete is reached at the top.

Nonetheless, small tensile stresses still result in spalling forces at the side of the girder. There is not a lot of width available to spread these forces. So, it is preferred to avoid these stresses by avoiding tensile stresses at all. Based on this consideration the maximum shortening is determined to be 1,8 [m] on both sides, which is 16% of the girder length. The strut-and-tie model for this load configuration is shown Figure E.23. In this model still some tensile spalling forces are present in the region between the points of prestress application. However, in reality the prestress is introduced over the transfer length. So, these forces will be more distributed. The width needed to resist these forces is 57 [mm], which is only 12% of the transfer length. Therefore, shortening until this length can is considered as feasible. Due to shortening the width of the zone needed to resist tensile splitting force reduces from 155 [mm] to 140 [mm]. For shortening between 0 and 1,8 [m] linear interpolation can be used.

In Figure E.23 and Figure E.24 the prestressing force is introduced over the width. Due to shortening the width needed increases from 43 [mm] to 77 [mm], which is still less than 20% of the transfer length and smaller than the width needed to distribute the force over the height. So, it can be concluded that the force distribution over the height is more critical.

Based on these findings a Maple script and Excel file are made to generate strut-and-tie models. The maximum shortened length is divided in 20 steps and for each step the strut-and tie model is made. The result is shown in Figure E.25.

If the girder is shortened in width direction by cutting the flanges only straight tendons are cut off, because kinked tendons are located in the web of the girder. As a result, the centre of gravity of the prestress force moves up. From this point of view, it might be beneficial for the spalling stresses to reduce the width of the cross-section. So, it might be possible to shorten the girder in width direction to make larger shortening in length direction possible. Nonetheless, due to the shortening the neutral axis of the cross section will also move up and the area changes, which causes a change in the stress distribution in the Bernoulli region as well. To investigate these effects strut-and-tie models are generated for the girder shortened in width direction to 790 [mm]. These are shown in Figure E.26.

With the Maple script and Excel file strut and tie models are also generate for the 18,53 [m] girder. In Figure E.27 presents the results the original 1200 [mm] wide girder. Figure E.28 presents the results for the 790 [mm] wide girder.

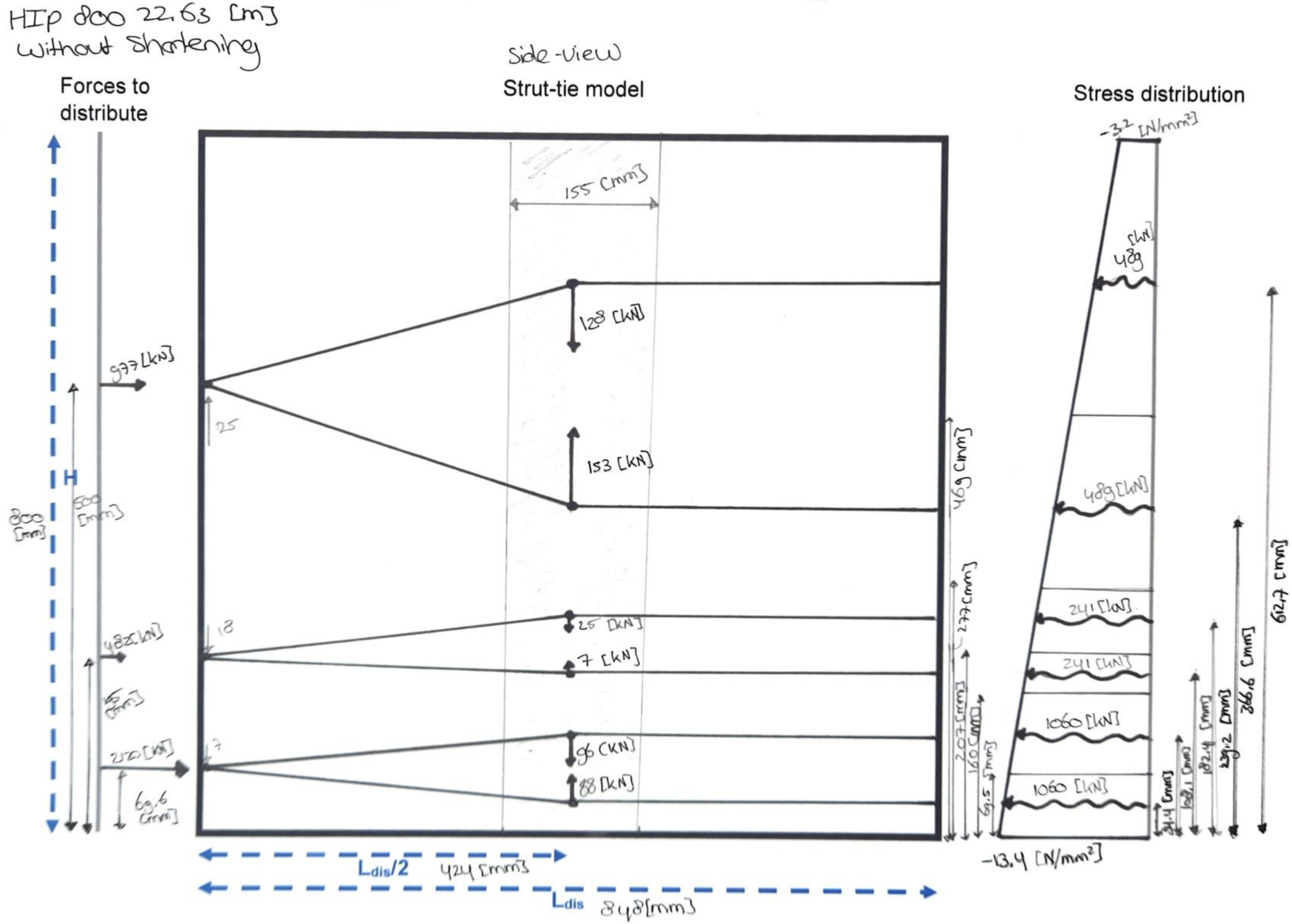


Figure E.20: Strut-tie model in side view in original situation.

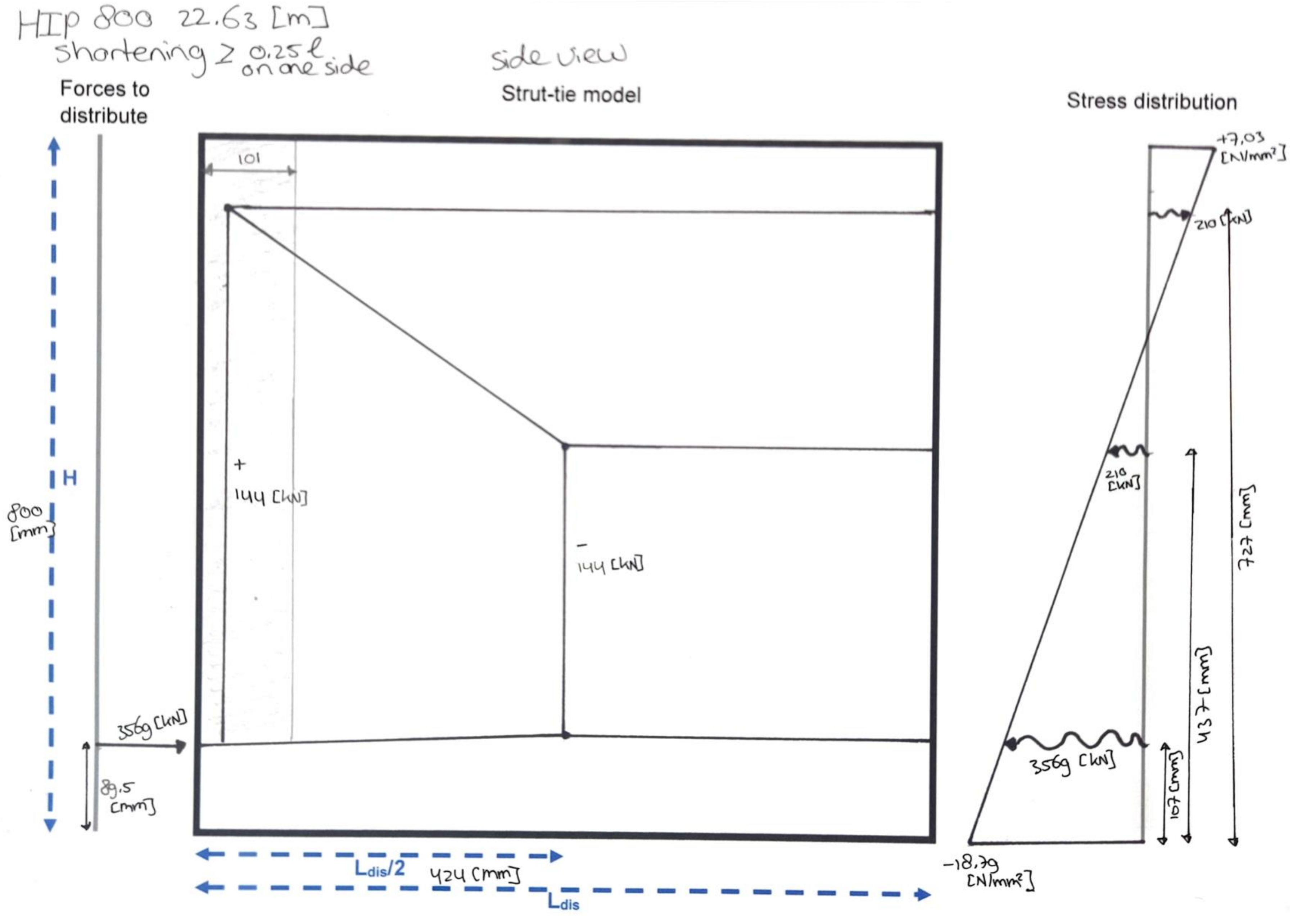


Figure E.21: Strut-tie model in side view in ultimate situation with shortening larger than 25% of length.

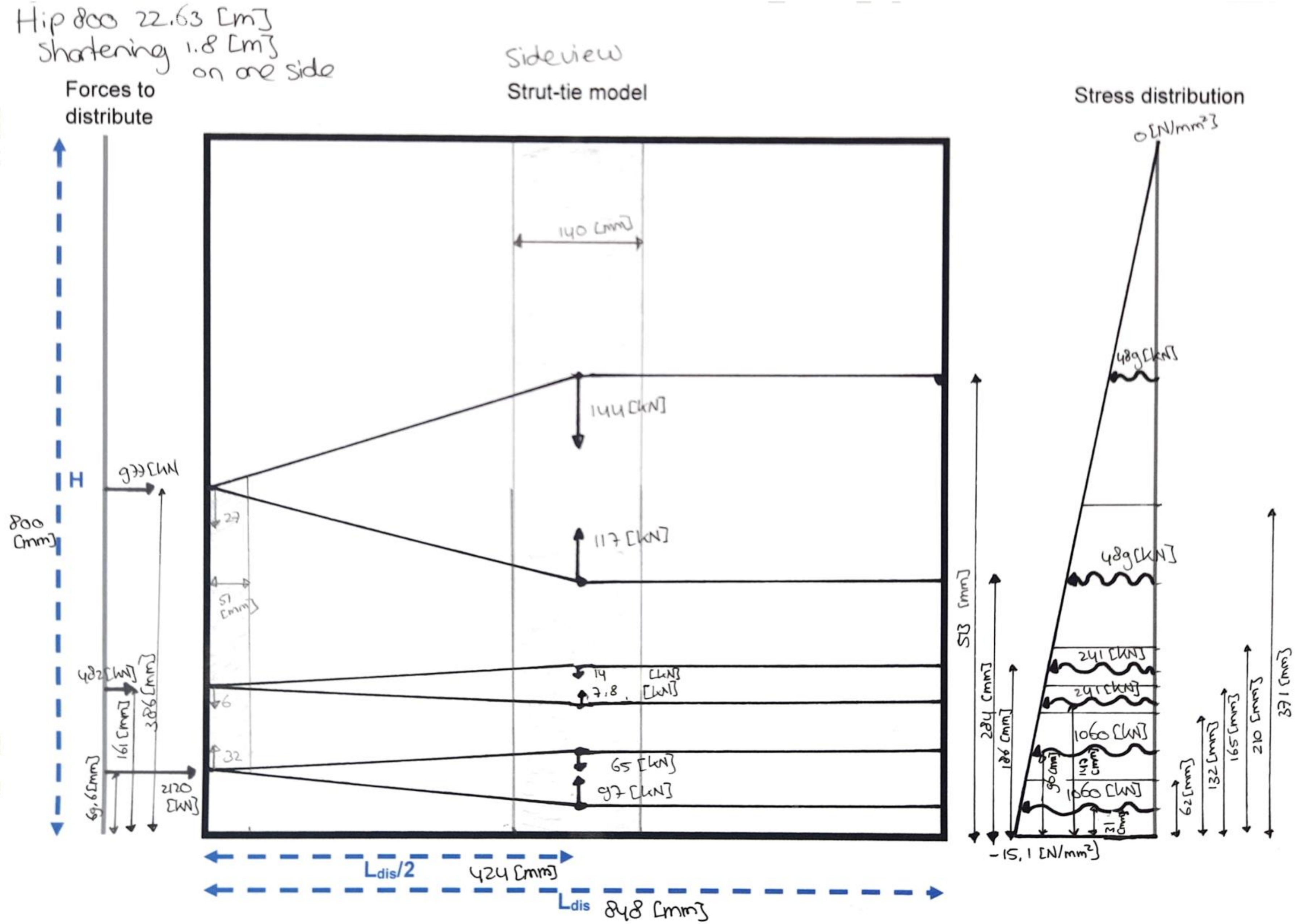


Figure E.22: Strut-tie model in side view with shortening until tensile stresses develop at the top.

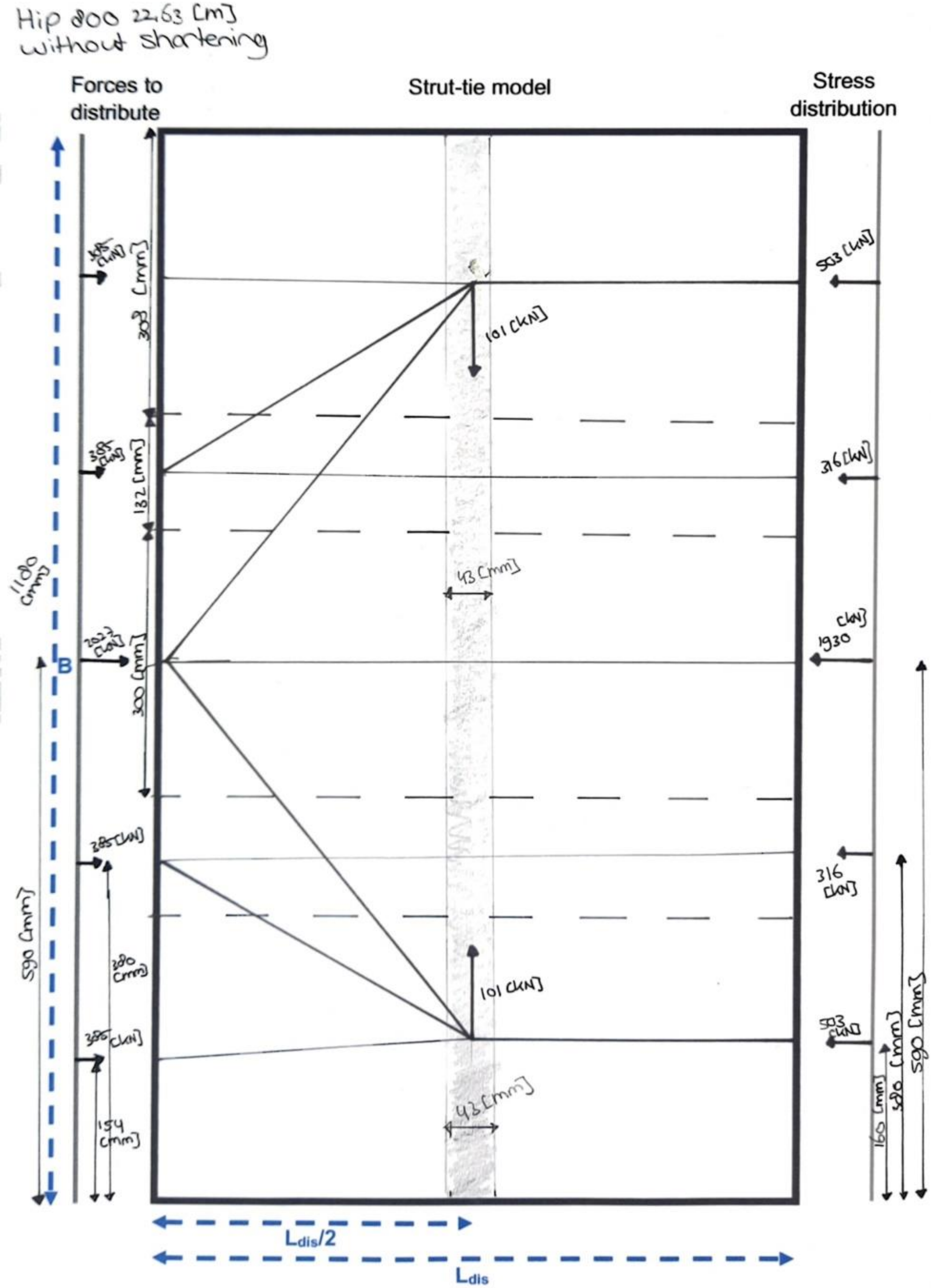


Figure E.23: Strut-tie model in top view in original situation

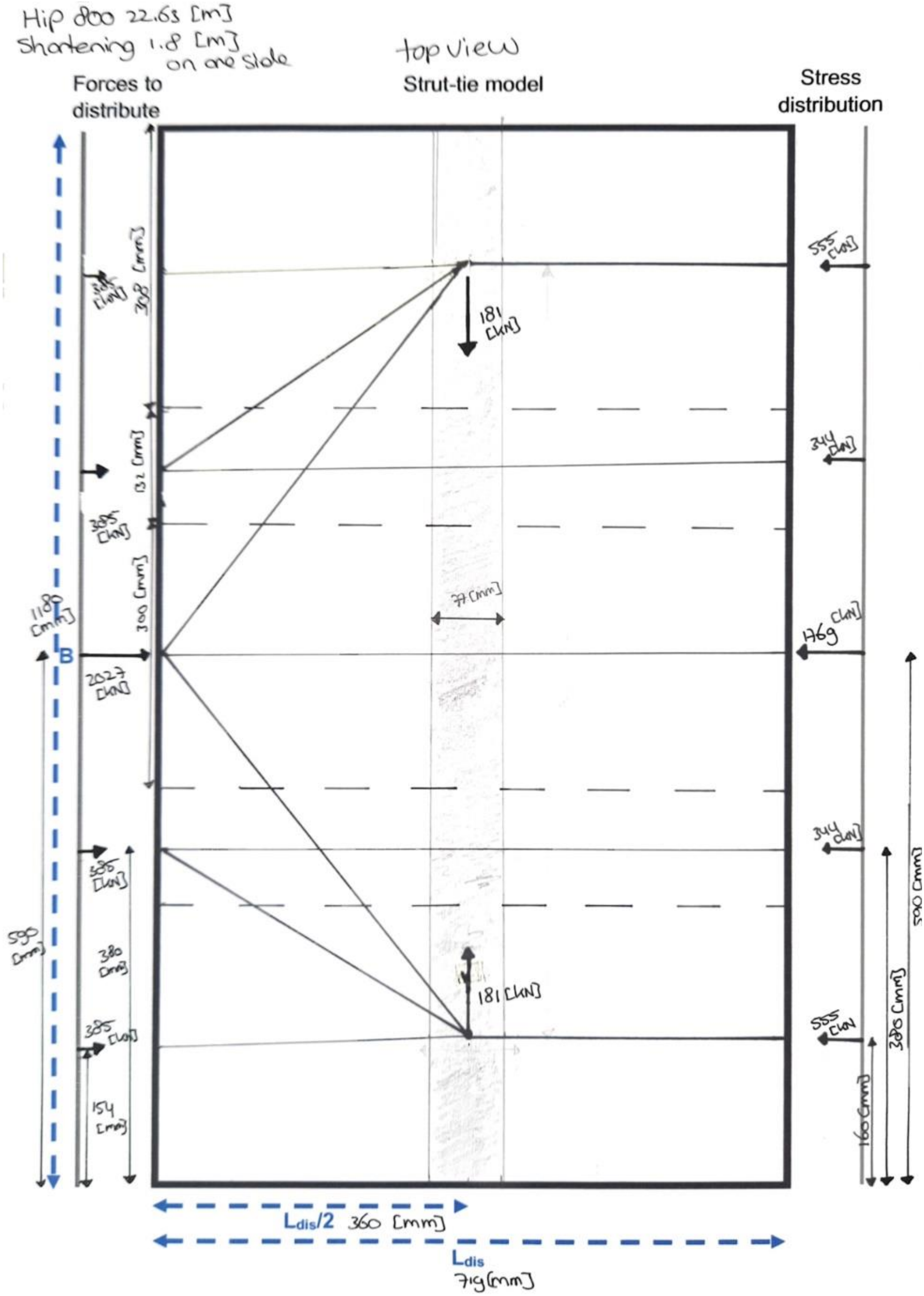


Figure E.24: Strut-tie model in top view in ultimate situation with shortening larger than 25% of the length.

Strut-and-tie models in side view

Forces in [kN], the length ranges from the orginal length of 22,62 [m] to 50% of the length Indicated shortening is shortening on one side.

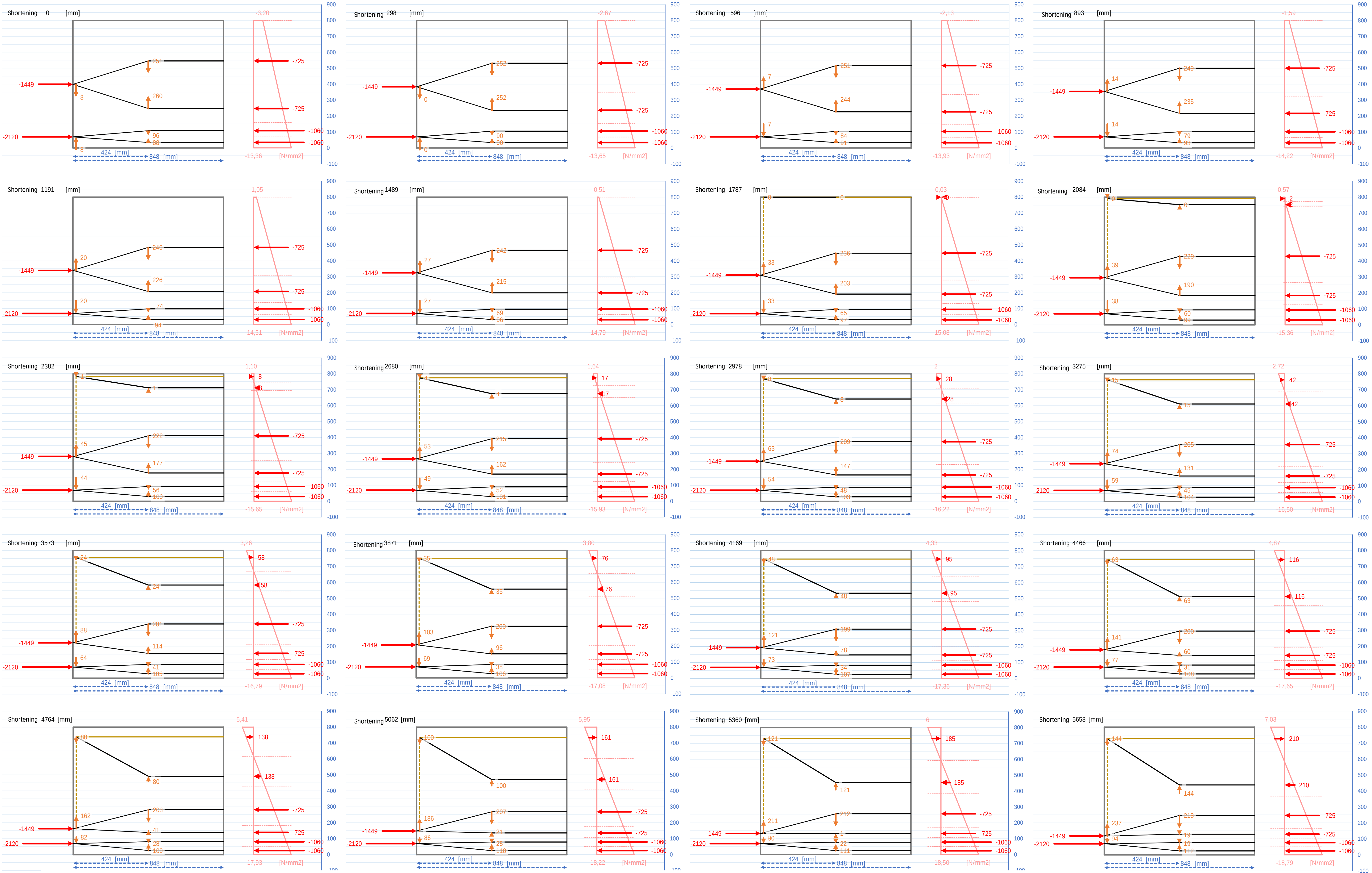


Figure E.25: Strut-tie models 22,62 [m] HIP 800 girders with a width of 1200 [mm].

Strut-and-tie models in side view

Forces in [kN], the length ranges from the orginal length of 22,62 [m] to 50% of the length. Indicated shortening is shortening on one side.

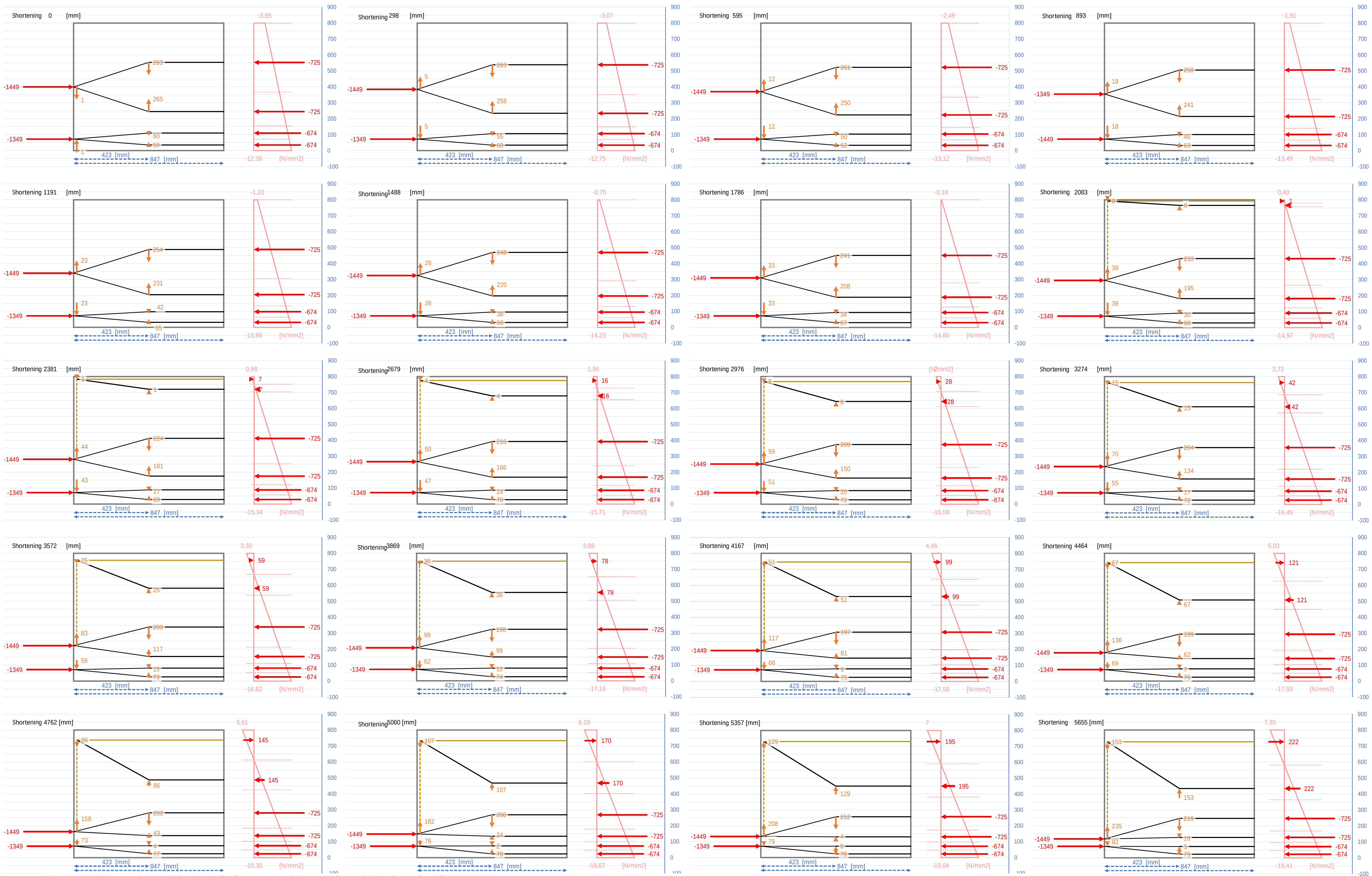


Figure E.26: Strut-tie models 22,62 [m] HIP 800 girders with a width of 790 [mm].

Strut-and-tie models in side view

Forces in [kN], the length ranges from the orginal length of 18,53 [m] to 50% of the length Indicated shortening is shortening on one side.

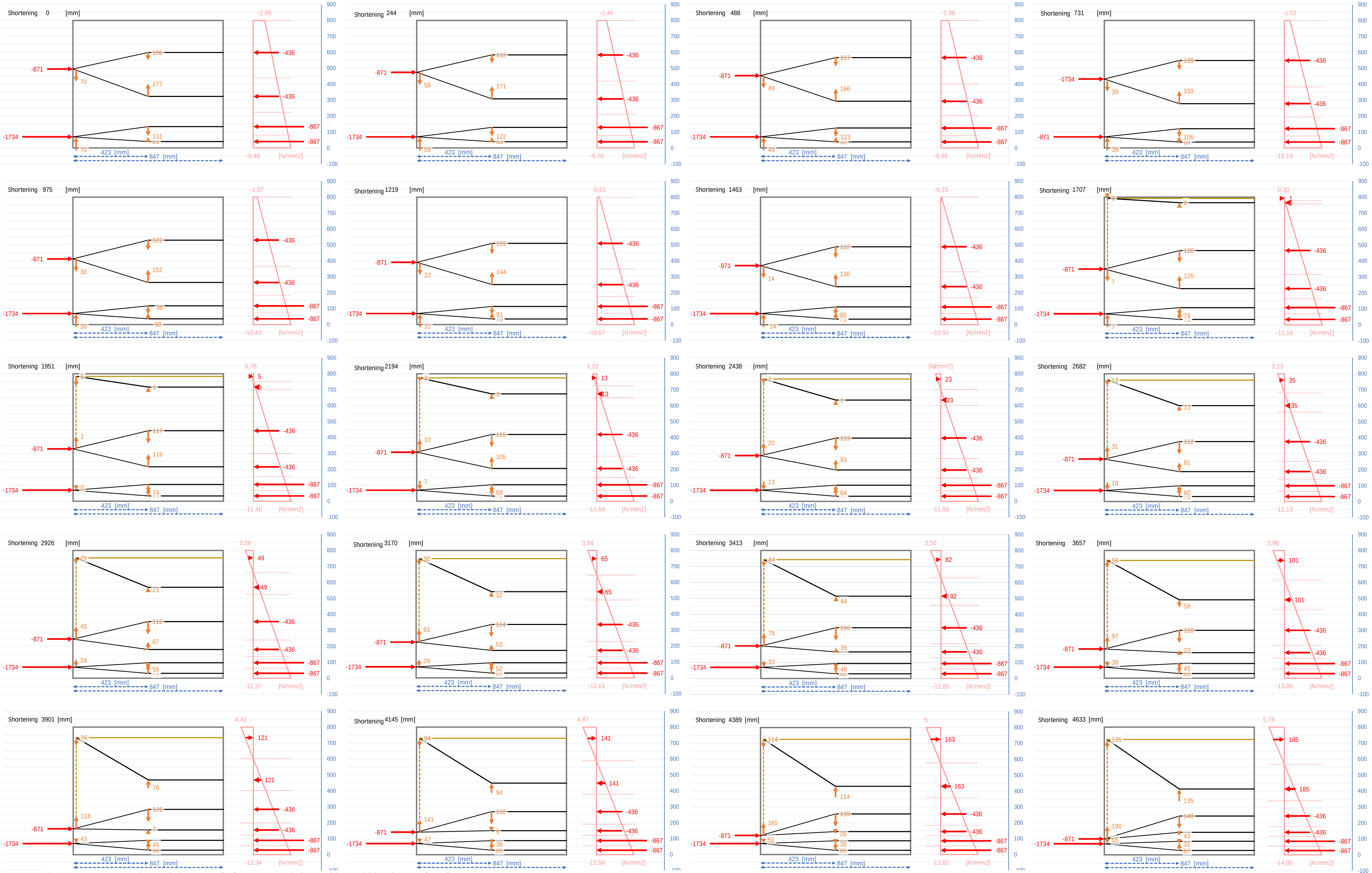


Figure E.27: Strut-tie models 18,53 [m] HIP 800 girders with a width of 1200 [mm].

Strut-and-tie models in side view

Forces in [kN], the length ranges from the orginal length of 18,53 [m] to 50% of the length Indicated shortening is shortening on one side.

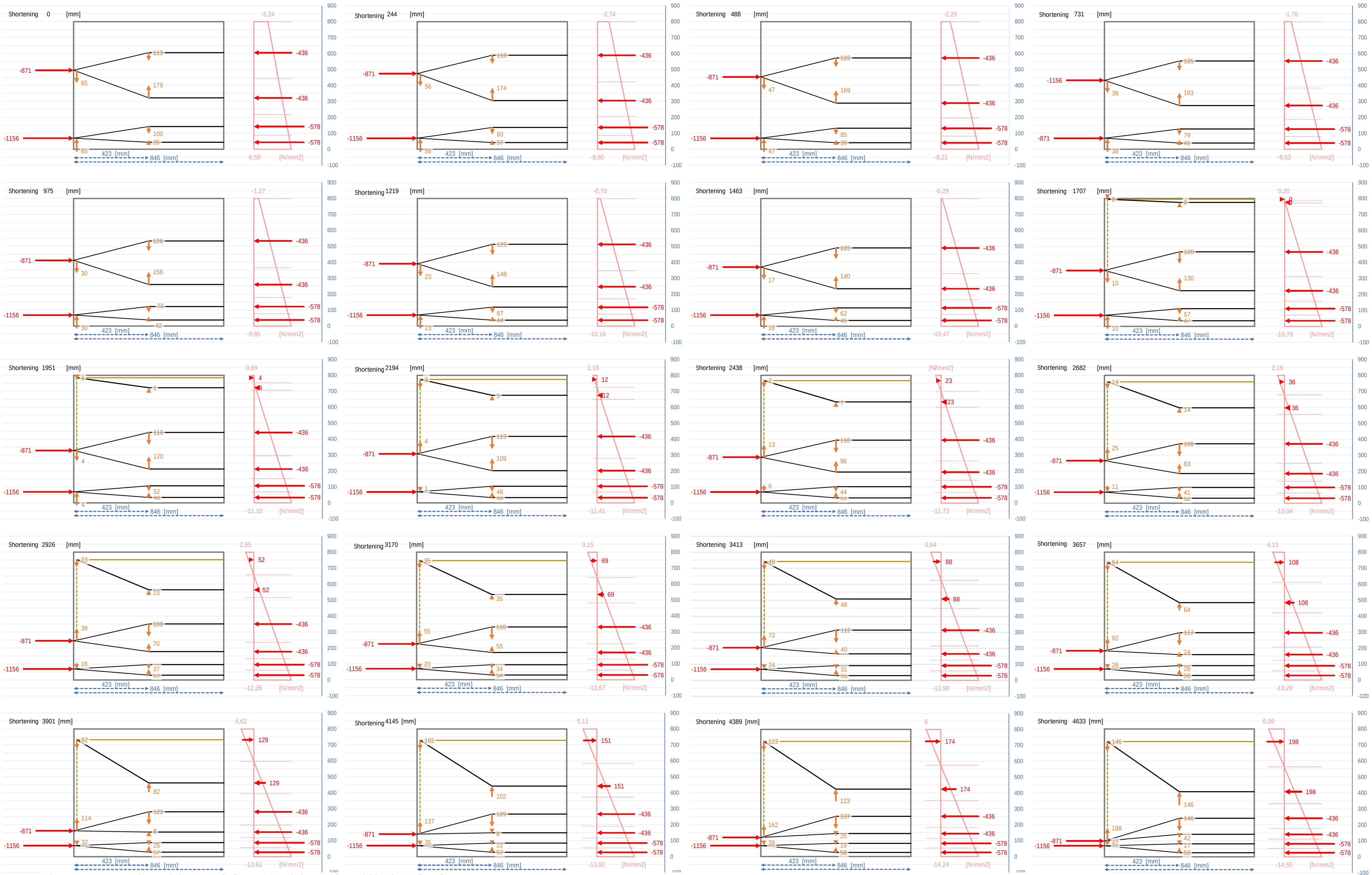


Figure E.28: Strut-tie models 18,53 [m] HIP 800 girders with a width of 790 [mm].

Based on the strut-and-tie models the needed width to resist the tensile forces is calculated. For the spalling width this is done by dividing the force in the tensile tie by the design concrete tensile strength and the height between the upper prestressing force and the tensile tie. For the splitting width the maximum splitting force is divided by the design concrete tensile strength and the height of the girder. Figure E.29 shows the results. As already indicated the spalling width increases rapidly when tensile stresses develop at the top of the girder. The splitting force require more width, however also more width is available.

Based on these results it can be concluded that shortening in width direction does not have a significant influence on the splitting and spalling stresses. The results for the girders shortened in width direction are almost identical to the results of the original girders. So, at least for this type of girder shortening in width direction is not a solution if the shortening in length direction causes too high splitting or spalling stresses.

In the search criteria the maximum shortening is set on 23%. For these girders shortening of 23%, will result in a tensile stress around 1 [N/mm^2]. This is still well below the flexural tensile strength of concrete. Moreover, the spalling forces are still almost close to 0. For the tensile splitting forces a width between 18 and 28 % of transfer length is needed. This seems reasonable. Since, in reality the prestressing force is not applied suddenly but introduced over the transfer length.

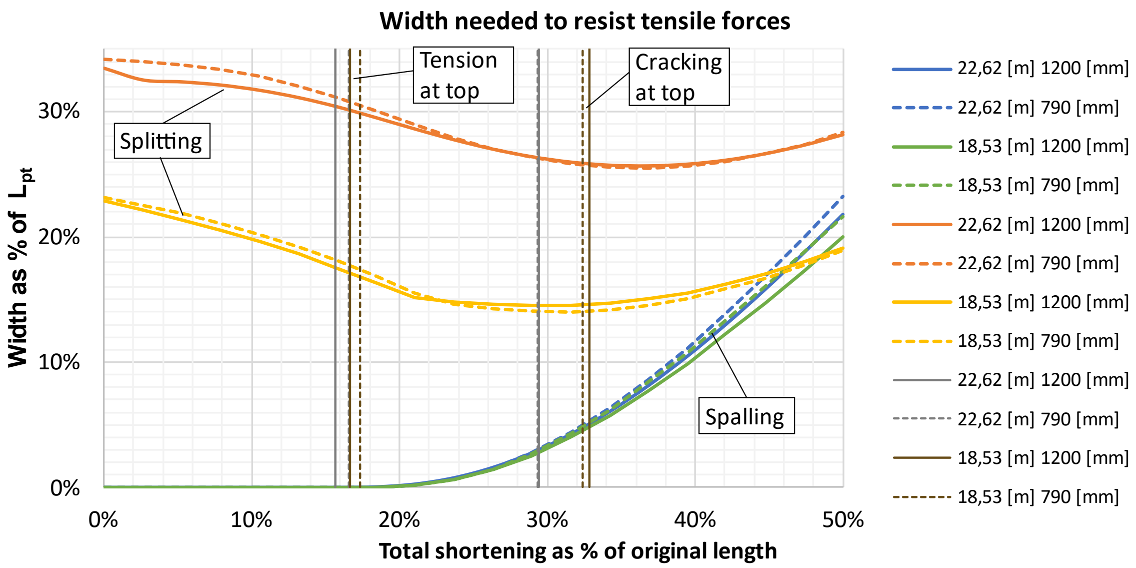


Figure E.29: Result from strut-and-tie models. Concrete width needed to resist tensile splitting and spalling forces as function of the shortened length. The results are shown for the 22,62 [m] and 18,53 [m] girder. The continuous lines show the results for the girders with the original width of 1200 [mm]. The dotted lines show the results for girders shortened in width direction to 790 [mm].

E.4 Design alternative 1b

First, the sectional properties are derived. In the next three sections the old and new loading situation are evaluated. The final design loads are presented in section five. Next the shear capacity is evaluated. Only the 23 [m] long girder is considered in this analysis since no information is available on the 16,05 [m] girder. The information on the 23 [m] is also limited and uncertain. Therefore, not all checks can be performed.

E.4.1 Sectional properties

The sectional properties in Table E.14 are based on the original dimensions of a HNP 750 girder, concrete class C55/67 for the girder and concrete class C30/37 for the deck. The cracked elastic modulus of the deck is taken as 1/3 of the original elastic modulus.

Table E.14: Sectional properties HNP 750 girder with original width of 980 [mm].

Cross-sectional property	HNP 800
Area girder [mm^2]	296420
Area girder with deck [mm^2]	496420
Neutral axis girder [mm]	312,2
Neutral axis girder with uncracked deck [mm]	509,6
Neutral axis girder with cracked deck [mm]	399,3
EI girder [Nmm^2]	7,417 E+14
EI girder with uncracked deck [Nmm^2]	1,966 E+15
EI girder with cracked deck [Nmm^2]	1,185 E+15

Orthotropic Property	HNP 750
D11 [MNm]	1966,0
D22 [MNm]	7,2970
D12 [MNm]	0,0000
D33 [MNm]	59,888
D44 [MNm]	14269
D55 [MNm]	4332,6
d11 [MNm]	34245
d22 [MNm]	7209,5
d12 [MNm]	0,0000
d33 [MNm]	9435,2

E.4.2 Prestress and self-weight

Because prestress and self-weight work in a SD-system the girders are considered separately. The bending moment and shear force lines are shown in Figure E.30. It should again be noted that the prestress distribution is uncertain and probably incorrect.

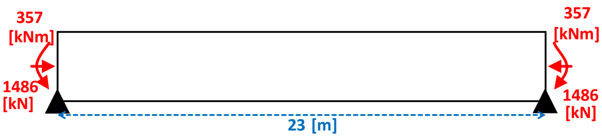


Figure E.30 a) Load distribution prestress.

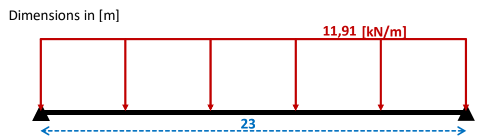


Figure E.30 b) Load distribution self-weight.

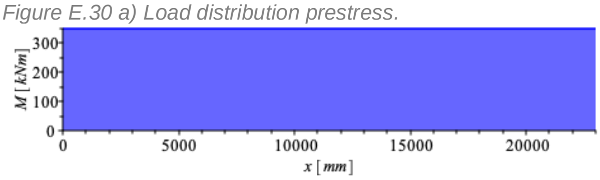


Figure E.30 c) Bending moment line due to prestress.

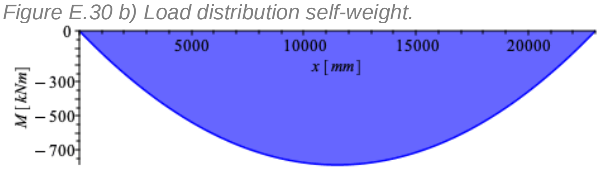


Figure E.30 d) Bending moment line due to self-weight.

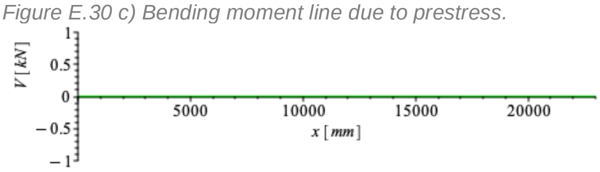


Figure E.30 e) Shear force line due to prestress.

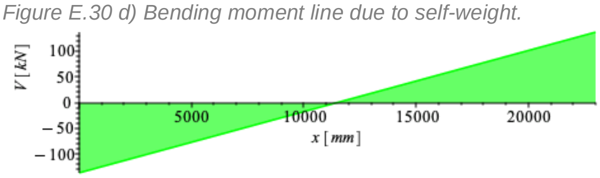


Figure E.30 f) Shear force line due to self-weight.

Figure E.30: Force distribution HNP 750 girders of 23,00 [m] due to self-weight and prestress.

E.4.3 Asphalt load

To resist the load of the asphalt layer the girder and the in-situ deck work together. So, the combined cross-sectional dimensions are used. In the old situation the girders form a SI system. In the new situation SI and SD are both considered. The bending moment and shear force lines are shown in Figure E.31.

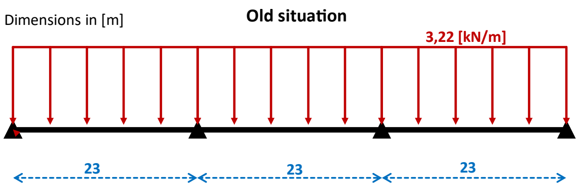


Figure E.31 a) Load distribution old situation.

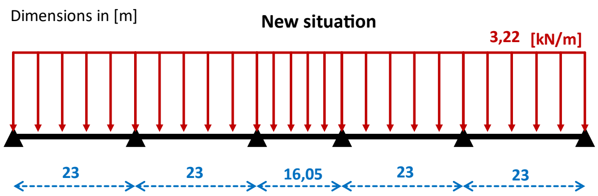


Figure E.31 b) Load distribution new situation.

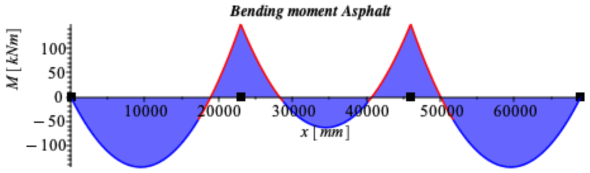


Figure E.31 c) Bending moment line old situation.

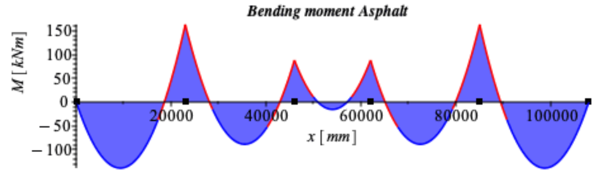


Figure E.31 d) Bending moment line new situation SI-system.

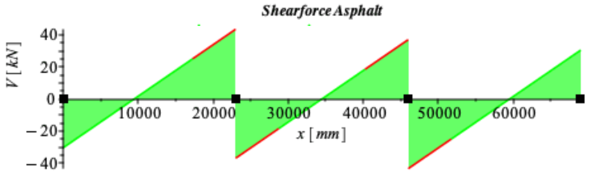


Figure E.31 e) Shear force line old situation.

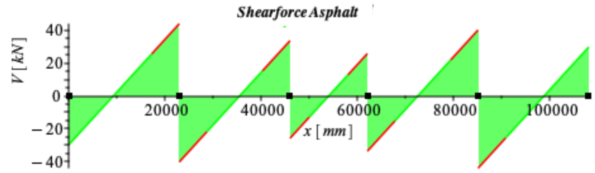


Figure E.31 f) Shear force line new situation SI-system.

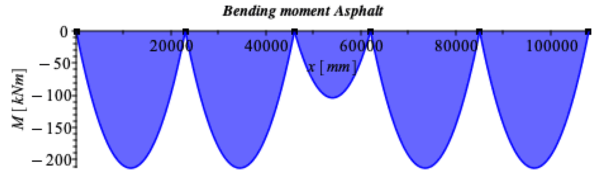


Figure E.31 g) Bending moment line new situation SD-system.

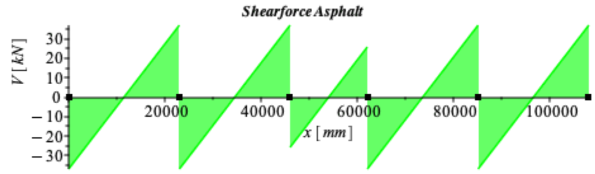


Figure E.31 f) Shear force line new situation SD-system.

Figure E.31: Force distribution in new and old situation due to asphalt load.

E.4.4 Traffic load

Based on the 45° spread the axle load in the new situation on the governing girder is 99 [kN] and the uniformly distributed load 7,76 [kN/m]. For the old situation the axle load is 67 [kN] and the uniformly distributed load 3,03 [kN/m]

In Figure E.32 to Figure E.35 the most unfavourable load configuration and corresponding bending moment and shear force diagram are shown for the old and new SI-system. For a new SD-system the results are shown in Figure E.36. Like the prestress and self-weight only one girder is considered.

In the new situation the load configuration for the maximum hogging bending moment at the support corresponds with the situation for the maximum hogging bending moment at distance L_{pt}.

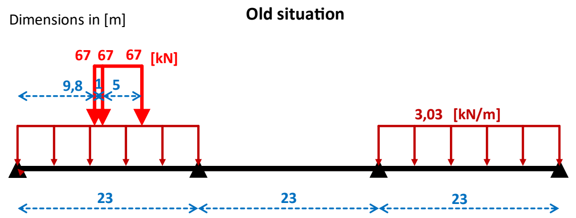


Figure E.32 a) Traffic distribution old situation.

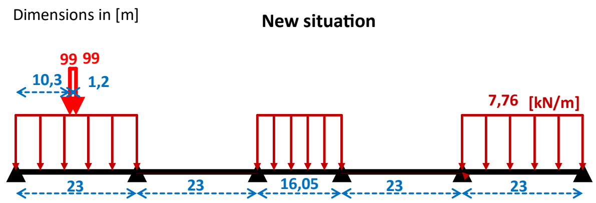


Figure E.32 b) Traffic distribution new situation.

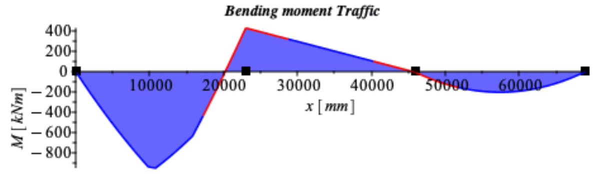


Figure E.32 c) Bending moment old situation.

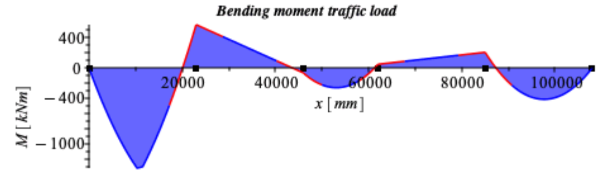


Figure E.32 d) Bending moment new situation.

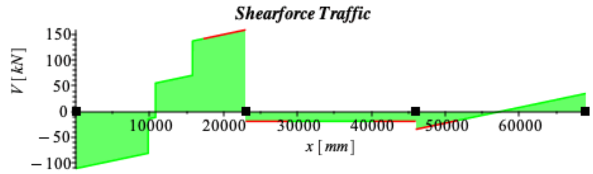


Figure E.32 e) Shear distribution old situation.

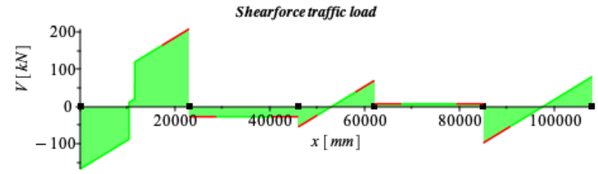


Figure E.32 f) Shear distribution new situation.

Figure E.32: Load distribution for largest sagging bending moment due to traffic load.

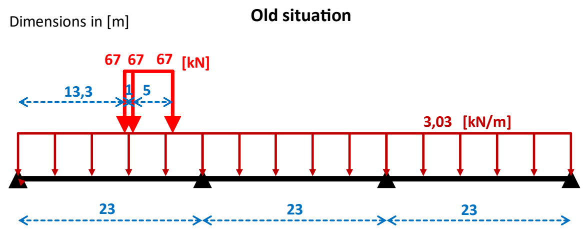


Figure E.33 a) Traffic distribution old situation.

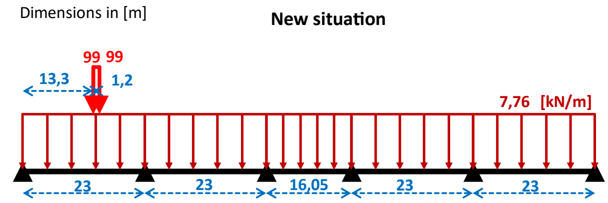


Figure E.33 b) Traffic distribution new situation.

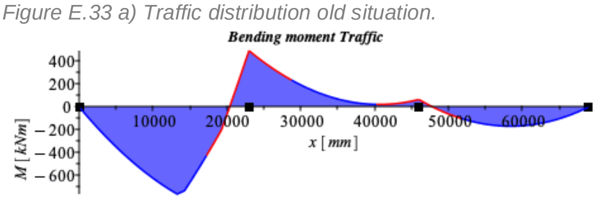


Figure E.33 c) Bending moment old situation.

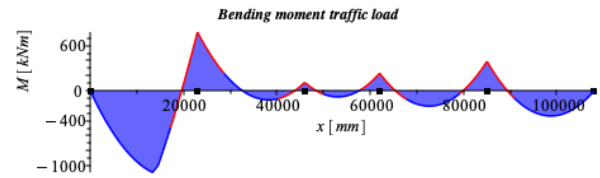


Figure E.33 d) Bending moment new situation.

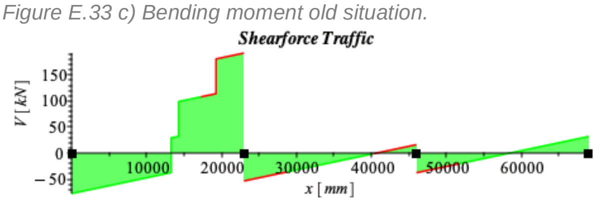


Figure E.33 e) Shear distribution old situation.

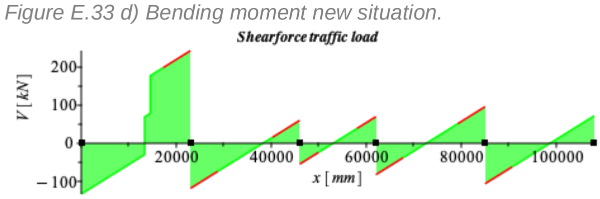


Figure E.33 f) Shear distribution new situation.

Figure E.33: Load distribution for largest hogging bending moment at support and at distance Lpt due to traffic load.

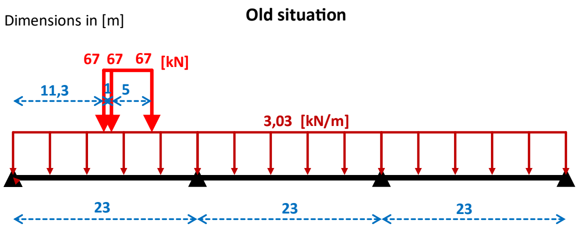


Figure E.34 a) Traffic distribution old situation

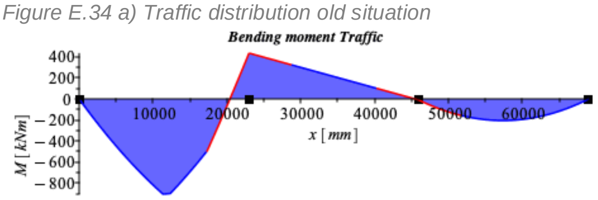


Figure E.34 b) Bending moment old situation.

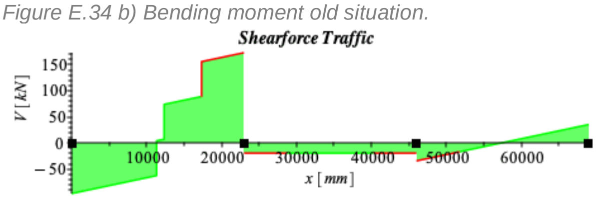


Figure E.34 c) Shear distribution old situation

Figure E.34: Load distribution for largest hogging bending moment at distance L_{pt} from due to traffic load.

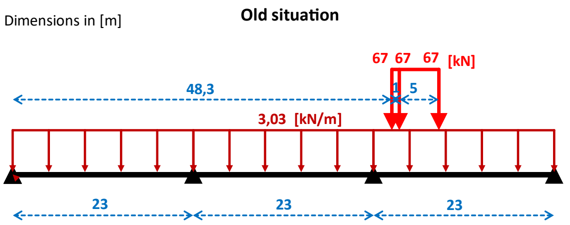


Figure E.35 a) Traffic distribution old situation.

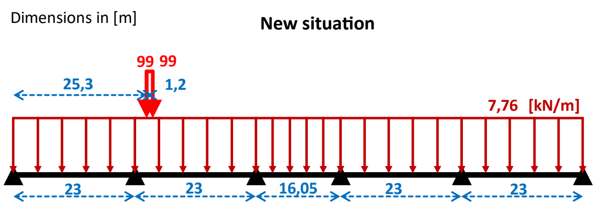


Figure E.35 b) Traffic distribution new situation.

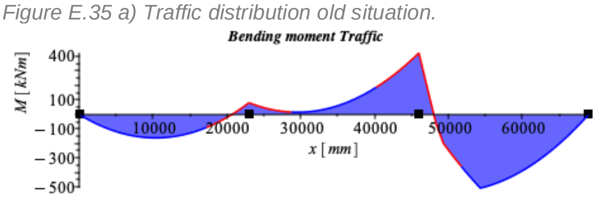


Figure E.35 c) Bending moment old situation.

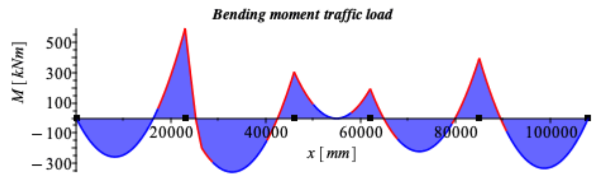


Figure E.35 d) Bending moment new situation.

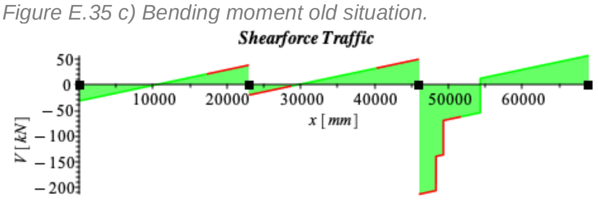


Figure E.35 e) Shear distribution old situation.

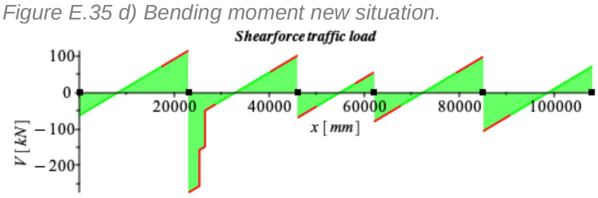


Figure E.35 f) Shear distribution new situation.

Figure E.35: Load distribution for largest shear force due to traffic load.

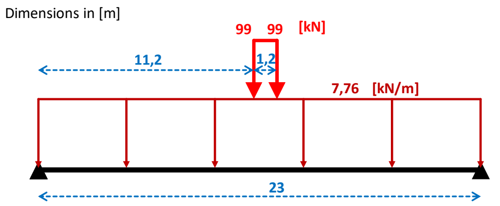


Figure E.36 a) Traffic distribution for largest sagging moment.

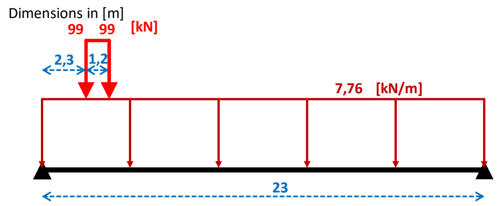


Figure E.36 b) Traffic distribution for largest shear force.

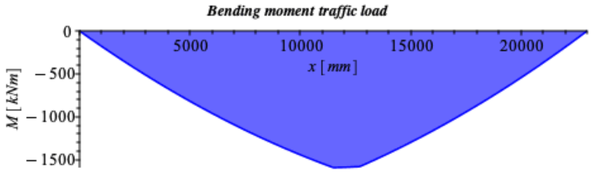


Figure E.36 c) Bending moment with traffic distribution of a).

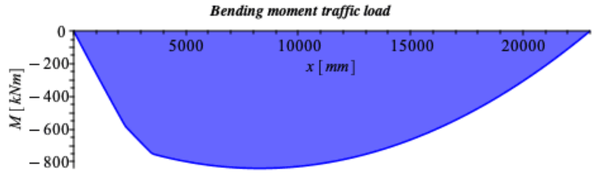


Figure E.36 d) Bending moment with traffic distribution of b).

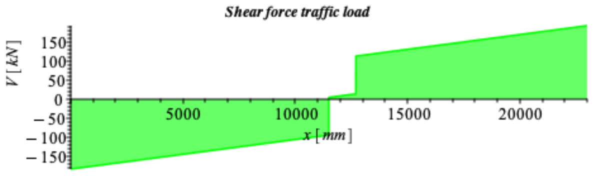


Figure E.36 e) Shear distribution with traffic distribution of a).

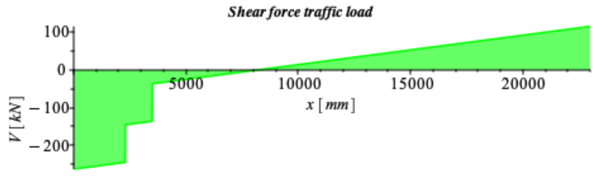


Figure E.36 f) Shear distribution with traffic distribution of b).

Figure E.36: Load distribution in new situation with SD-system for largest for largest sagging bending moment due to traffic load on the left and for largest shear force on the right.

E.4.5 Loads on the structure

The results from the previous sections are summarized in Table E.15. This results for the new situation in the design load shown in Table E.16.

In Table E.17 the loads in the new and old situation are compared. With safety factors the loads are approximately the same or even decrease in the new situation. However, this safety factor is just an assumption. Without safety factors the loads can increase up to 60%.

Table E.15: Characteristics loads on critical cross-sections. Values in black are based on the differential equation. Values in grey come from the SCIA-model.

Situation			Location [m]	Bending moment [kNm] or shear force [kN]			
				Prestress	Self-weight	Asphalt	Traffic load
1. Sagging moment	Old		10,80	350	-785	-142 -132	-924 -726
	New	SI	10,32	350	-779	-137 -126	-1324 -872
		SD	11,50		-788	-213 -213	-1592 -1080
2. Hogging moment	Old		23	0*	0	150 170	493 529
	New		23	0*	0	163 190	779 712
4. Hogging moment Lpt	Old		23,52	350	-70	131 151	480 501
	New		23,52	350	-70	143 168	719 633
2. Shear at support	Old		46	0	137	44 44	213 210
	New	SI	23	0	137	44 45	275 285
		SD				37 37	262 242
3. Shear next to support	Old		46,50	0	131	42 43	211 208
	New	SI	23,50	0	131	42 40	271 265
		SD				35 36	258 235

Table E.16: Design loads in new situation.

System	Statically determinate (SD)		Statically indeterminate (SI)	
Loads	All	Asphalt + traffic	All	Asphalt + traffic
$M_{sag,ed,SLS}$ [kNm]	-1731	-1293	-1427	-998
$M_{sag,ed,ULS}$ [kNm]	-2309	-1714	-1913	-1328
$M_{hog,ed,SLS}$ [kNm]	-	-	902	902
$M_{hog,ed,ULS}$ [kNm]	-	-	1189	1189
$M_{hog,ed,SLS,L_{pt}}$ [kNm]	-	-	1081	801
$M_{hog,ed,ULS,L_{pt}}$ [kNm]	-	-	1322	1056
$V_{ed,support}$ [kN]	536	371	588	424
$V_{ed,cross\ beam}$ [kN]	518	360	563	406

Table E.17: Comparison old and new 'design' loads.

Loads	All	Difference [%] SD	Difference [%] SI	Asphalt + traffic	Difference [%] SD	Difference [%] SI
M_{sag} [kNm] * 1,7	-1293 -2198	+34% +5%	+10% -13%	-858 -1459	+51% -17%	+16% -9%
M_{hog} [kNm] * 1,7	643 1093		+40% +9%	643 1093		+40% +9%
$M_{hog,L_{pt}}$[kNm] * 1,7	891 1515		+21% -13%	611 1039		+31% +2%
$V_{ed,support}$ [kN] * 1,7	394 670	+36% -20%	+49% -12%	257 437	+44% -15%	+65% -3%
$V_{ed,cross\ beam}$[kN] * 1,7	382 649	+36% -20%	+47% -13%	251 427	+43% -16%	+62% -5%

E.4.6 Shear capacity

Since HNP girder are not equipped with stirrups, the shear reinforcement ratio is 0% and the minimum is not met.

In Equation E.7 and Table E.18 the resistance of concrete is calculated. Again, the crossbeam is assumed to be 1 [m] wide. Consequently, at the location of cross-section 2 the prestress had 500 [mm] to get partly introduced. The introduction of prestressing occurs linearly over the transmission length. So, from 0 [N/mm^2] to -4,91 [N/mm^2] in 553 [mm].

The effective height (d) is calculated as the height minus 50 [mm], because the first prestressing tendon is located at 50 [mm] from the bottom.

Equation E.7: Shear resistance concrete.

$$V_{rd,c} = \left(0{,}035 \times k^{\frac{3}{2}} \times \sqrt{f_{ck}} + 0{,}15 \times \sigma_{cp}\right) \times b_w \times d$$

$$k = 1 + \sqrt{\frac{200}{d}} = 1 + \sqrt{\frac{200}{900}} = 1{,}47 \ \leq 2$$

$$\sigma_{cp} = \frac{F_p}{b_w \times d} < 0{,}2\frac{f_{ck}}{\gamma_c}$$

Table E.18: Shear resistance of concrete.

Location	Element		Input				Result [kN]
			f_{ck} [N/mm^2]	σ_{cp} [N/mm^2]	b_w [mm]	d [mm]	
2: At support	Girder		55	0,0	1000	700	324
	Deck		30	0,0	1000	200	68
						$V_{rd,c}$	**393**
3: Next to support SD	Girder		55	3,41	400	700	273
	Deck	NEN	30	0,0	400	200	27
		RBK			600		41
						$V_{rd,c}$ NEN	**300**
						$V_{rd,c}$ RBK	**314**
3: Next to support SI	Girder		55	0,0	400	700	130
	Deck	NEN	30	0,0	400	200	27
		RBK			600		41
						$V_{rd,c}$ NEN	**157**
						$V_{rd,c}$ RBK	**171**

Table E.19: UC on shear capacity

Standard		NEN	RBK
Situation	**System**	$U.C = \frac{V_{ed}}{V_{rd}} \leq 1$	
2. At support	SI	1,50	1,50
	SD	1,36	1,36
3. Next to support	SI	2,70	2,48
	SD	1,72	1,65

From these UCs it can be concluded that the girders are not able to meet the regulations for new to build structures nor the regulations from the RBK. Shortening the girders in width direction is not a feasible because the capacity at the supports is already insufficient. A load restriction is also not a suitable solution because the UC is much larger than 1,0.

It should be noted that the capacity of the girder is underestimated, because the prestress is most likely underestimated. Nonetheless, this does not change the situation at the supports.

According to these calculations this alternative can only be structurally feasible if the girders are strengthened. This can be done by for example CFRP sheets or shape Memory Allow. These options are not further considered in this case study. As this alternative turns out to be not directly structurally feasible the structural calculation is not further continued.

E.5 Design alternative 1d

First, the sectional properties are derived. In the next three sections the old and new loading situation are evaluated. The final design loads are presented in section five. Next the shear capacity and bending moment capacity are evaluated. Finally shortening possibilities are explored.

E.5.1 Sectional properties

The sectional properties of the of this type of girder is already presented in Table E.3 If the girder is shortened in width direction to 750 [mm] or 1000 [mm] the sectional properties of Table E.20 or Table E.21 apply.

Table E.20: Sectional properties HIP 800 girder with width of 750 [mm].

Cross-sectional property	HIP 800
Area girder [mm^2]	306545
Area girder with deck [mm^2]	426545
Neutral axis girder [mm]	316,1
Neutral axis girder with uncracked deck [mm]	458,0
Neutral axis girder with cracked deck [mm]	372,9
EI girder [Nmm^2]	6,770 E+14
EI girder with uncracked deck [Nmm^2]	1,623 E+15
EI girder with cracked deck [Nmm^2]	1,055 E+15

Orthotropic Property	HIP 800
D11 [MNm]	2164,2
D22 [MNm]	3,7361
D12 [MNm]	0,0000
D33 [MNm]	44,960
D44 [MNm]	14681
D55 [MNm]	4378,2
d11 [MNm]	35233
d22 [MNm]	5872,2
d12 [MNm]	0,0000
d33 [MNm]	9620,6

Table E.21: Sectional properties HIP 800 girder with width of 1000 [mm].

Cross-sectional property	HIP 800
Area girder [mm^2]	339170
Area girder with deck [mm^2]	499170
Neutral axis girder [mm]	292,0
Neutral axis girder with uncracked deck [mm]	461,6
Neutral axis girder with cracked deck [mm]	362,0
EI girder [Nmm^2]	7,495 E+14
EI girder with uncracked deck [Nmm^2]	2,053 E+15
EI girder with cracked deck [Nmm^2]	1,284 E+15

Orthotropic Property	HIP 800
D11 [MNm]	2053,4
D22 [MNm]	3,7360
D12 [MNm]	0,0000
D33 [MNm]	43,794
D44 [MNm]	14596
D55 [MNm]	4378,2
d11 [MNm]	35030
d22 [MNm]	5838,0
d12 [MNm]	0,0000
d33 [MNm]	9592,9

E.5.2 Prestress and self-weight

Because prestress and self-weight work in a SD-system the girders are considered separately. The bending moment and shear force line due to prestress and self-weight are shown in Figure E.37.

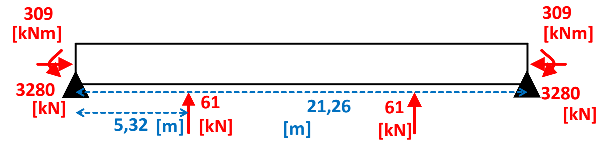


Figure E.37 a) Load distribution prestress.

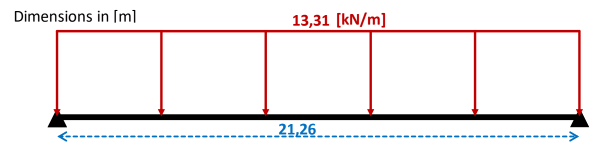


Figure E.37 b) Load distribution self-weight.

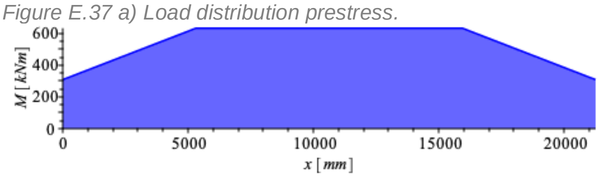


Figure E.37 c) Bending moment line due to prestress.

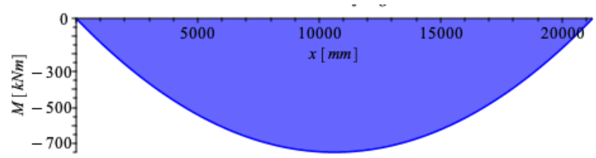


Figure E.37 d) Bending moment line due to self-weight.

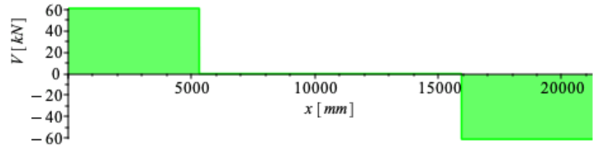


Figure E.37 e) Shear force line due to prestress.

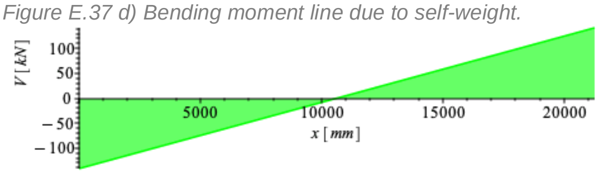


Figure E.37 f) Shear force line due to self-weight.

Figure E.37: Force distribution HIP 800 girder of 21,26 [m] due to self-weight and prestress.

E.5.3 Asphalt load

To resist the load of the asphalt layer the girder and the in-situ deck work together. So, the combined cross-sectional dimensions are used. In the old situation the girders form a SI-system. In the new situation SI and SD are both considered. The bending moment and shear force lines are shown in Figure E.38.

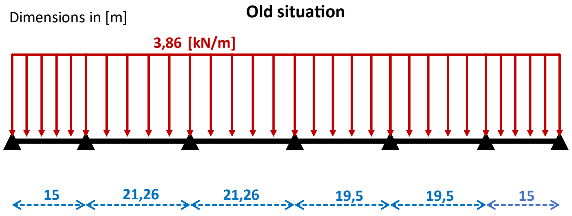


Figure E.38 a) Load distribution old situation.

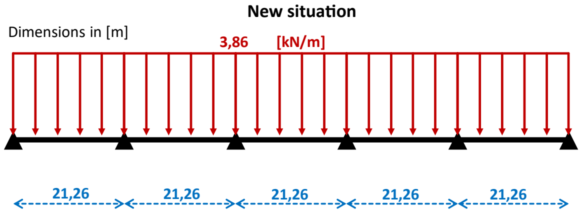


Figure E.38b) Load distribution new situation.

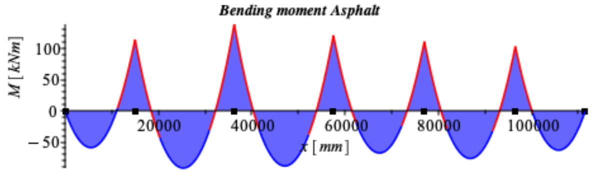


Figure E.38c) Bending moment line old situation.

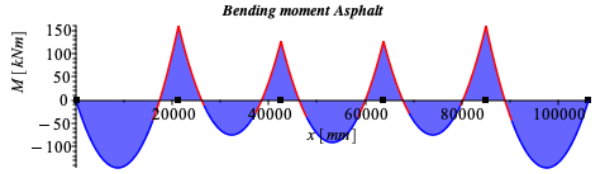


Figure E.38 d) Bending moment line new situation SI-system

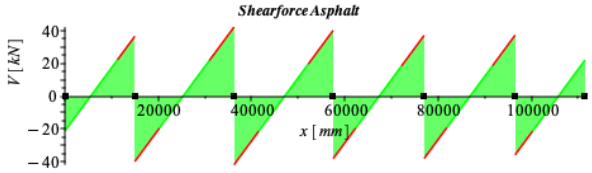


Figure E.38 e) Shear force line old situation.

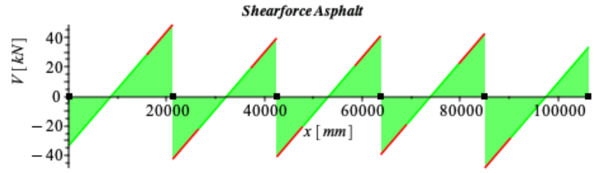


Figure E.38 f) Shear force line new situation SI-system.

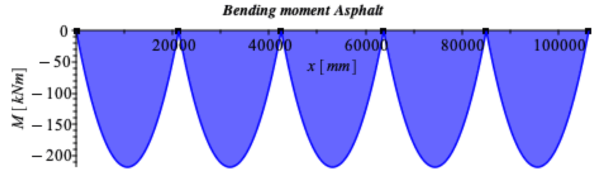


Figure E.38 g) Bending moment line new situation SD-system.

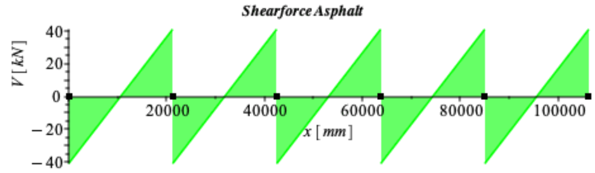


Figure E.38 f) Shear force line new situation SD-system.

Figure E.38: Force distribution in new and old situation due to asphalt load.

E.5.4 Traffic load

Based on the 45° spread the axle load in the new situation on the governing girder is 114 [kN] and the uniformly distributed load 9,07 [kN/m]. For the old situation the axle load is 79 [kN] and the uniformly distributed load 3,67 [kN/m]

In Figure E.39 to Figure E.42 the most unfavourable load configuration and corresponding bending moment and shear force diagram are shown for the old and new SI-system. For a new SD-system the most unfavourable load configuration and corresponding bending moment and shear force diagram are shown in Figure E.43. Like the prestress and self-weight only one girder is considered.

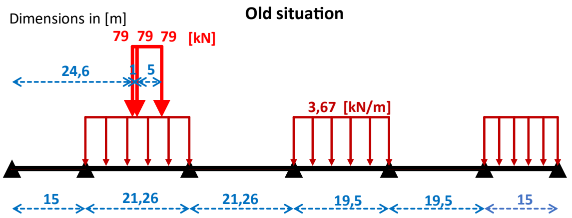


Figure E.39 a) Traffic distribution old situation.

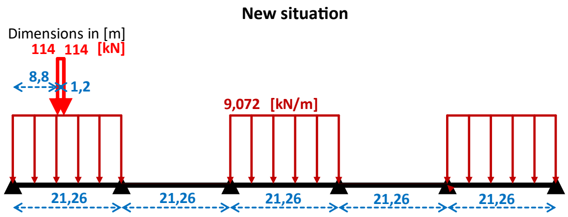


Figure E.39b) Traffic distribution new situation.

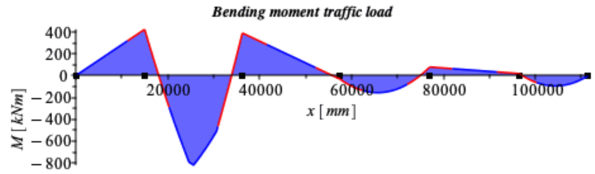


Figure E.39 c) Bending moment old situation.

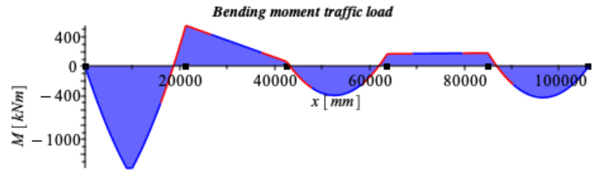


Figure E.39 d) Bending moment new situation.

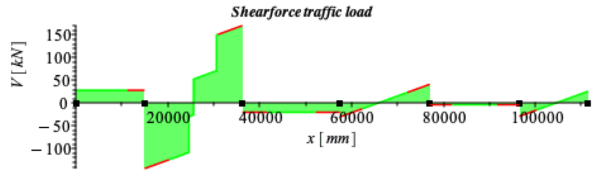


Figure E.39Figure E.5 e) Shear distribution old situation.

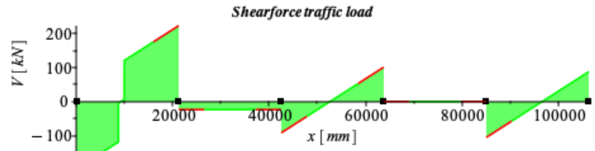


Figure E.39 f) Shear distribution new situation.

Figure E.39: Load distribution for largest sagging bending moment due to traffic load.

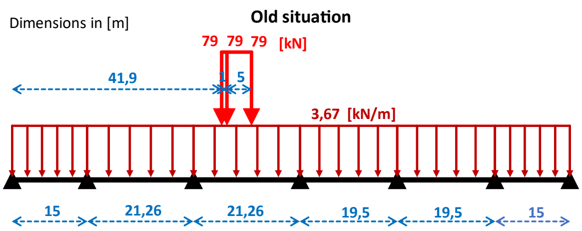


Figure E.40 a) Traffic distribution old situation.

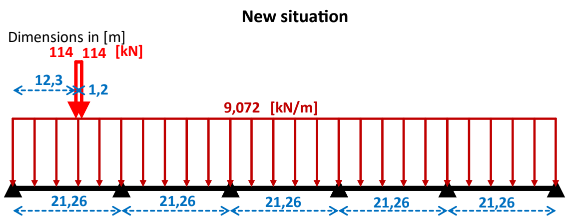


Figure E.40 b) Traffic distribution new situation.

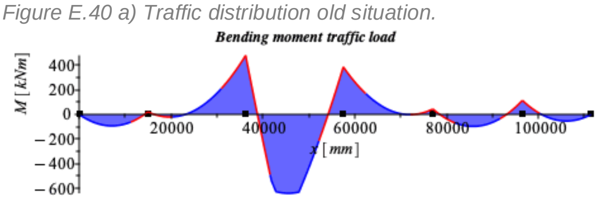


Figure E.40 c) Bending moment old situation.

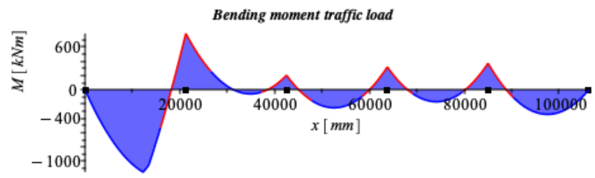


Figure E.40 d) Bending moment new situation.

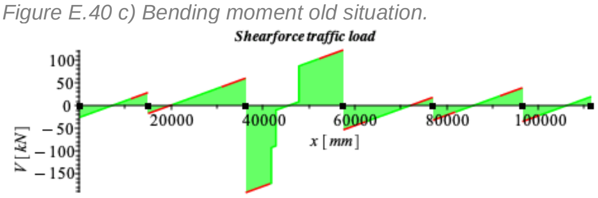


Figure E.40 e) Shear distribution old situation.

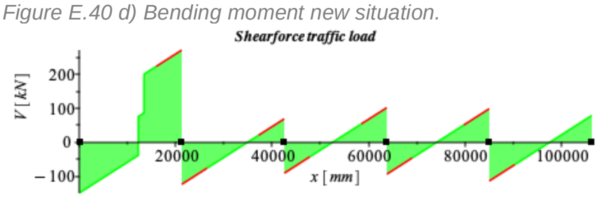


Figure E.40 f) Shear distribution new situation.

Figure E.40: Load distribution for largest hogging bending moment at support due to traffic load.

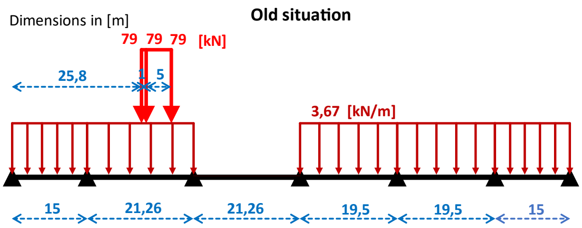


Figure E.41a) Traffic distribution old situation.

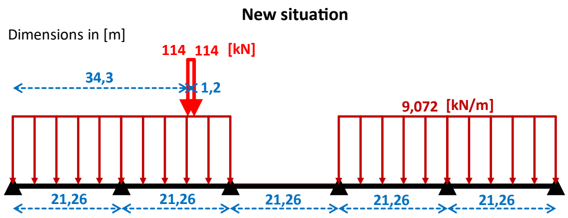


Figure E.41 b) Traffic distribution new situation.

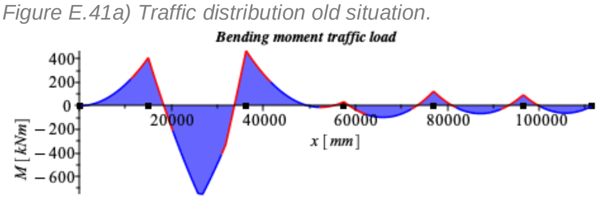


Figure E.41 c) Bending moment old situation.

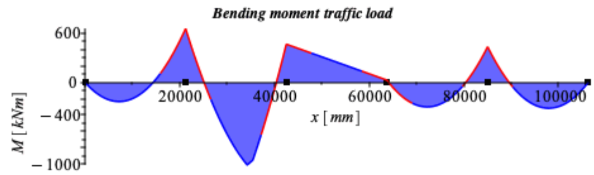


Figure E.41 d) Bending moment new situation.

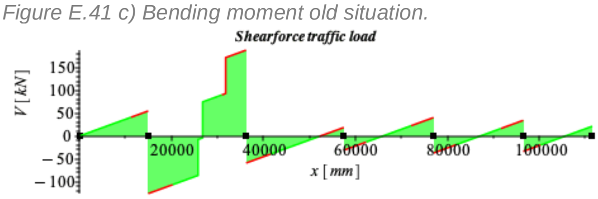


Figure E.41 e) Shear distribution old situation.

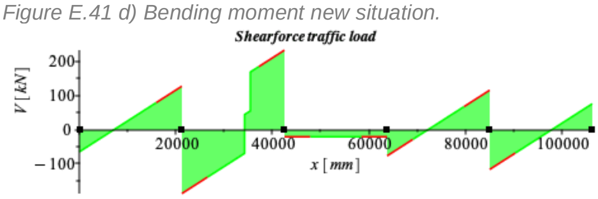


Figure E.41 f) Shear distribution new situation.

Figure E.41: Load distribution for largest hogging bending moment at distance L_{pt} from supports due to traffic load.

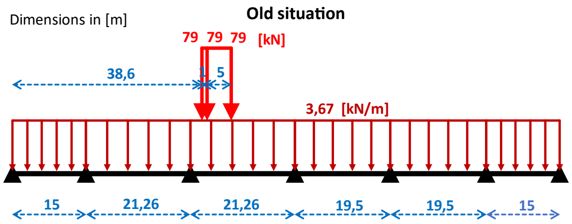


Figure E.42 a) Traffic distribution old situation.

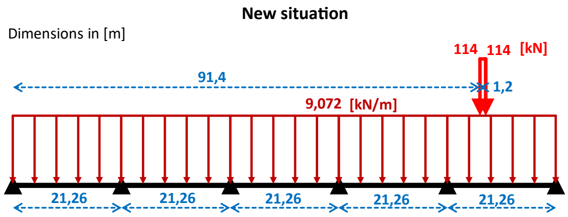


Figure E.42 b) Traffic distribution new situation.

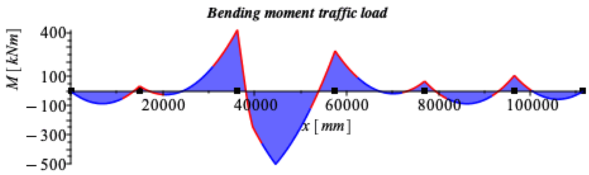


Figure E.42 c) Bending moment old situation.

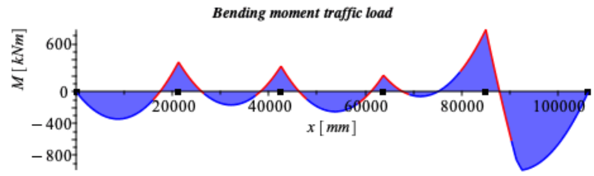


Figure E.42 d) Bending moment new situation.

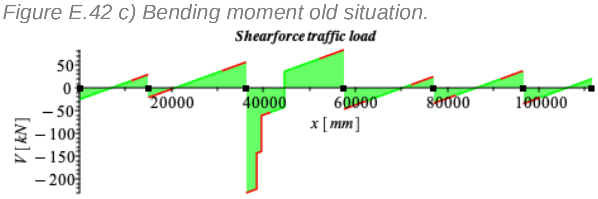


Figure E.42 e) Shear distribution old situation.

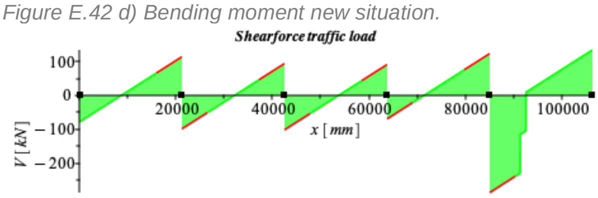


Figure E.42 f) Shear distribution new situation.

Figure E.42: Load distribution for largest shear force due to traffic load.

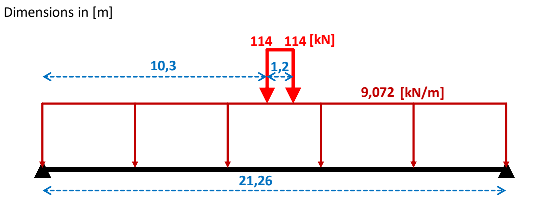


Figure E.43 a) Traffic distribution for largest sagging moment

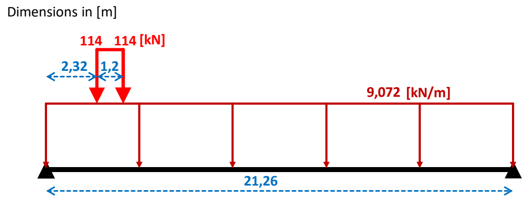


Figure E.43 b) Traffic distribution for largest shear force

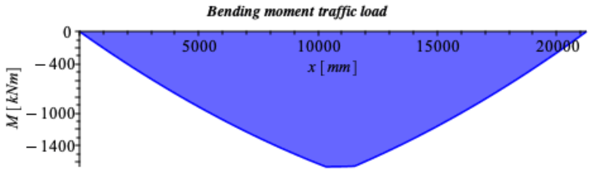


Figure E.43 c) Bending moment with traffic distribution of a).

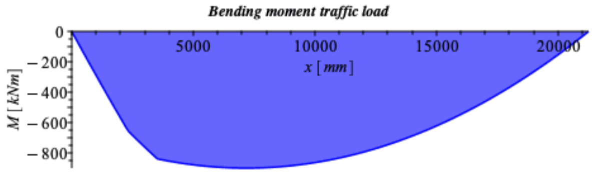


Figure E.43 d) Bending moment with traffic distribution of b).

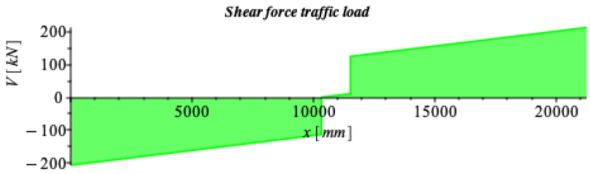


Figure E.43 e) Shear distribution with traffic distribution of a).

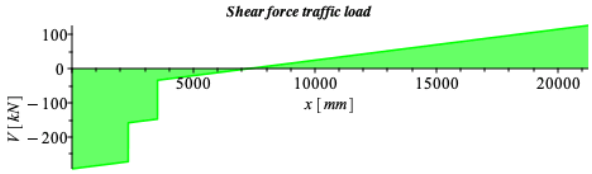


Figure E.43 f) Shear distribution with traffic distribution of b).

Figure E.43: Load distribution in new situation with SD-system for largest for largest sagging bending moment due to traffic load on the left and for largest shear force on the right.

E.5.5 Loads on the structure

The results from the previous sections are summarized in Table E.22. This results for the new situation in the design load shown in Table E.23.

In Table E.24 the comparison between the old and new situation is made. From these results it can be concluded that the ‘design’ load on the girders increases for almost all cases. Only, with safety factors the shear forces slightly decrease. Also, the hogging bending moment after the introduction of the prestressing force decreases, however the hogging bending moment at the support is governing. So, this is not of influence.

Table E.22: Characteristics loads on critical cross-sections. Values in black are based on the differential equation. Values in grey come from the SCIA-model.

Situation			Location [m]	Bending moment [kNm] or shear force [kN]			
				Prestress	Self-weight	Asphalt	Traffic load
1. Sagging moment	Old		25,64	634	-752	-92 -78	-820 -680
	New	SI	9,97	634	-749	-142 -131	-1385 -979
		SD	10,63		-752	-218 -218	-1662 -1175
2. Hogging moment	Old		36,26	0*	0	139 153	479 515
	New		21,26	0*	0	160 184	788 791
4. Hogging moment Lpt	Old		36,98	353	-99	110 124	426 400
	New		43,24	353	-99	99 110	453 406
2. Shear at support	Old		36,26	-61	142	42 42	231 232
	New	SI	85,04	-61	142	49 50	287 393
		SD				41 41	250 290
3. Shear next to support	Old		36,76	-61	132	39 40	228 230
	New	SI	85,54	-61	132	46 48	280 379
		SD				38 38	244 286

Table E.23: Design loads in new situation.

System	Statically determinate (SD)		Statically indeterminate (SI)	
Loads	All	Asphalt + traffic	All	Asphalt + traffic
$M_{sag,ed,SLS}$ [kNm]	-1511	-1393	-1236	-1121
$M_{sag,ed,ULS}$ [kNm]	-2116	-1848	-1757	-1492
$M_{hog,ed,SLS}$ [kNm]	-	-	948	948
$M_{hog,ed,ULS}$ [kNm]	-	-	1256	1256
$M_{hog,ed,SLS,L_{pt}}$ [kNm]	-	-	759	505
$M_{hog,ed,ULS,L_{pt}}$ [kNm]	-	-	901	667
$V_{ed,support}$ [kN]	496	387	556	446
$V_{ed,cross\ beam}$ [kN]	472	375	531	433

Table E.24: Comparison old and new 'design' loads.

Loads	All	Difference [%] SD	Difference [%] SI	Asphalt + traffic	Difference [%] SD	Difference [%] SI
M_{sag} [kNm] * 1,7	-876 -1489	+72% +45%	+41% +18%	-758 -1289	+84% +43%	+48% +16%
M_{hog} [kNm] * 1,7	618 1051		+53% +20%	618 1051		+53% +20%
$M_{hog,L_{pt}}$[kNm] * 1,7	778 1323		-2% -32%	524 891		-4% -25%
$V_{ed,support}$ [kN] * 1,7	354 602	+40% -18%	+57% -8%	273 464	+42% -17%	+63% -4%
$V_{ed,cross\ beam}$[kN] * 1,7	338 575	+40% -18%	+57% -8%	267 454	+40% -17%	+62% -5%

E.5.6 Shear capacity

The reinforcement ratios and the capacity at the supports are similar to design alternative 1a because the cross-sectional dimensions and the shear reinforcement are the same. Only the capacity next to the supports in a SD-system slightly differs. This is the consequence of a different compressive stress due to prestress, which is 6,3 [N/mm^2] for this case. As a result, the concrete shear resistance of the girder is 315 [kN]. The total shear resistance and UCs are shown in Table E.25 and Table E.26. For the intermediate steps a reference is made to E.3.6.

Similar to alternative 1a the minimum shear reinforcement ratio is not met. If the spacing near the supports is 100 [mm] the girders are able to meet the other NEN requirements. With a spacing of 300 [mm] the girders are not able to meet the NEN requirements. In a SD-system the UC at the supports is 1,03. Nonetheless, in a more advanced calculation model of the bridge this UC can probably be reduced to lower than 1,0.

In this case the cross-section next to the support still does not fulfil but this can be solved by reducing the width of the girder. The design shear force has to be reduced from 472 [kN] to 332 [kN]. If self-weight remains the same 235 [kN] is left for the design asphalt and traffic load. Currently this load is 375 [kN]. So, the width has to be reduced by 60%, which leaves 750 [mm]. With this width the shear force due to self-weight reduces to 109 [kN] and the capacity of the cross-section decreases to 312 [kN] due to a loss of prestress. In total the UC is lowered to 0,91.

Table E.25: Shear resistance

Situation	System	Standard	V_{rd}	
			S=100	S=300
2. At support	SI + SD	NEN	716	481
		RBK	978	481
3. Next to support	SD	NEN	716	332
		RBK	837	340
	SI	NEN	716	239
		RBK	625	165

Table E.26: UC on shear capacity

Standard	NEN		RBK	
Spacing	S=100 [mm]	S=300 [mm]	S=100 [mm]	S=300 [mm]
Situation	$U.C = \frac{V_{ed}}{V_{rd}} \leq 1$			
2. At support SI	0,78	1,16	0,57	1,16
2. At support SD	0,69	1,03	0,51	1,03
3. Next to support SI	0,74	2,22	0,85	3,21
3. Next to support SD	0,66	1,42	0,56	1,39

E.5.7 Bending moment capacity

Figure E.44 presents the stress-distribution over the height of the girder and deck in SLS. Prestress and self-weight are resisted by the girder and the traffic and asphalt load are resisted by the combined action of girder and deck. In all cases no cracking occurs. However, if no tensile stresses are allowed the girders cannot be used in a SD-system, without shortening in width direction. However, by decreasing the width from 1200 [mm] to 1000 [mm] no tensile stresses occur in a SD-system. In this case 2 prestressing tendons are cut off.

The girders of 750 [mm] wide are only considered in a SD-system because the shortening is needed for the shear capacity in a SD-system. Due to this shortening 8 tendons are cut off.

The bending moment resistance of a 1200 [mm] wide girder is 3086 [kNm]. This is based on the values and graphs from Figure E.45. It can be concluded from subfigure C that not the full capacity of the girder is used. Moreover, the ultimate strain in the girder is higher than the strain at peak stress ε_{c3}. Therefore, both for the girder and the deck the values of alfa and beta are iteratively changed.

The bending moment resistance of 750 [mm] and 1000 [mm] wide girders are 2151 [kNm] and 2764 [kNm] respectively. This follows from Figure E.46 and Figure E.47. The results and UCs are shown in Table E.27.

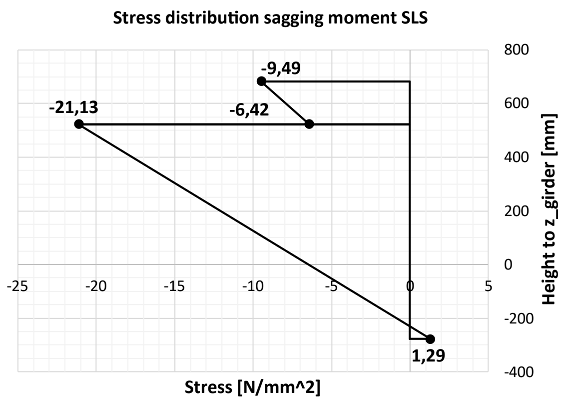


Figure E.44 a) 1200 [mm] wide girder in SD-system.

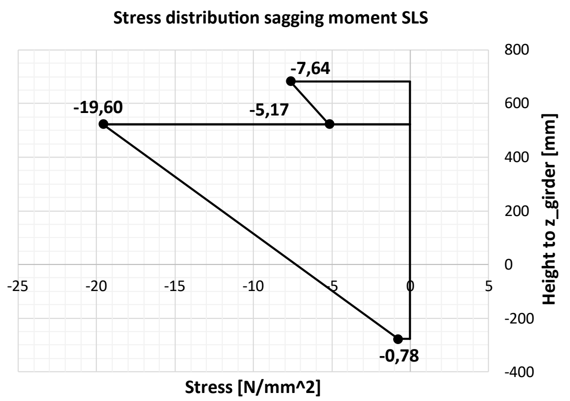


Figure E.44 b) 1200 [mm] wide girder in SI-system.

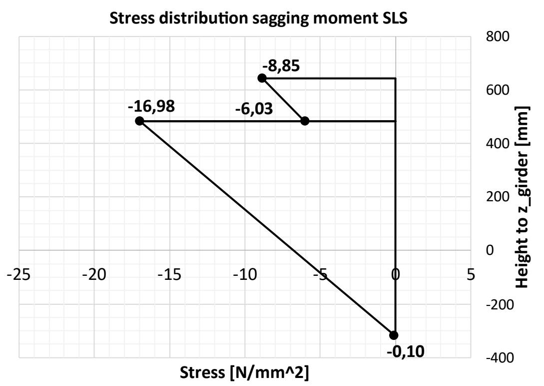


Figure E.44 c) 750 [mm] wide girder in SD-system.

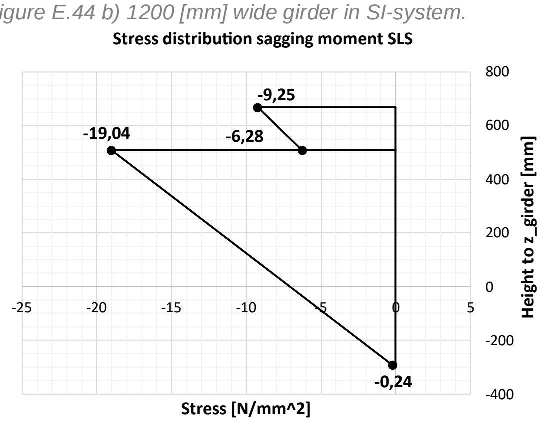


Figure E.44 d) 1000 [mm] wide girder in SD-system.

Figure E.44: SLS verification sagging bending moment.

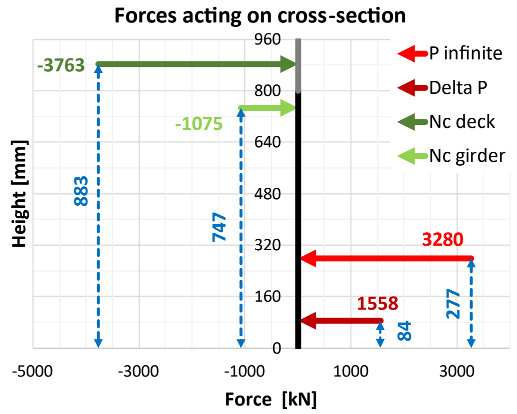


Figure E.45 a) Force distribution

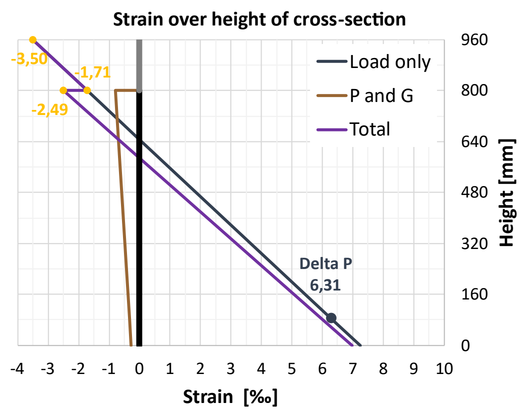


Figure E.45 b) Strain distribution

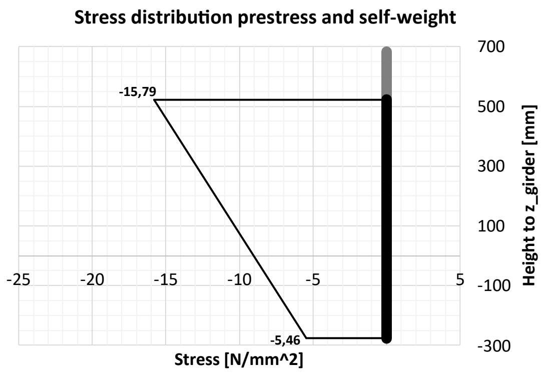


Figure E.45 c) Stress distribution for self-weight and prestress

Assumed steel stress	1512	[N/mm^2]
Steel strain	12,49	[‰]
Initial steel strain	6,22	[‰]
Assumed delta P	6,28	[‰]
α girder	0,64	[-]
β girder	0,35	[-]
α deck	0,98	[-]
β deck	0,52	[-]
Resistance	3086	[kNm]

Figure E.45 d) values for steel and concrete stress and strain

Figure E.45: Results for sagging bending moment resistance girders with a width of 1200 [mm].

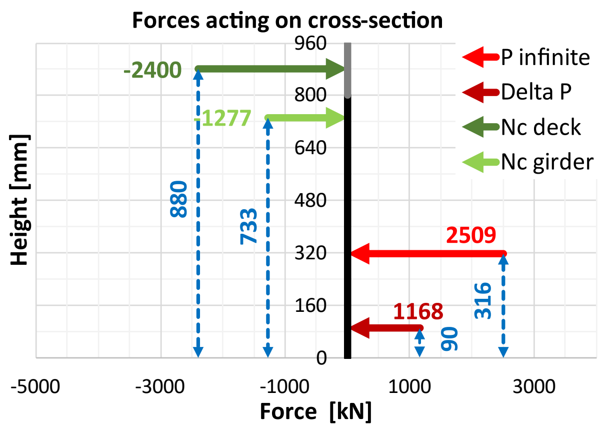


Figure E.46 a) Force distribution.

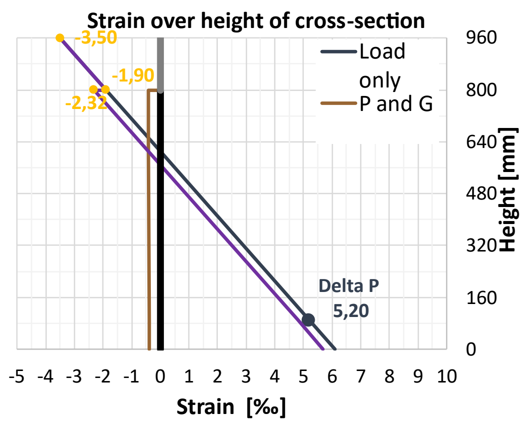


Figure E.46 b) Strain distribution.

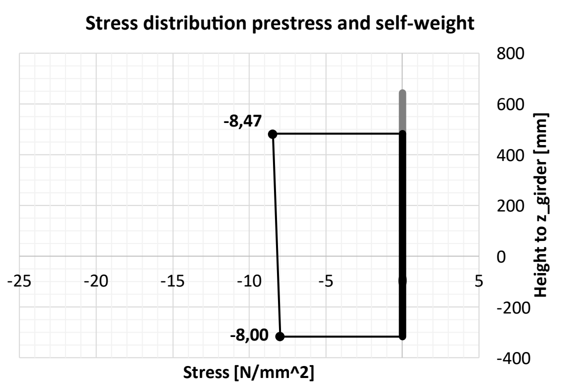


Figure E.46 c) Stress distribution for self-weight and prestress.

Assumed steel stress	1502	[N/mm^2]
Steel strain	11,26	[‰]
Initial steel strain	6,22	[‰]
Assumed delta P	5,04	[‰]
α girder	0,61	[-]
β girder	0,35	[-]
α deck	1,00	[-]
β deck	0,50	[-]
Resistance	2151	[kNm]

Figure E.46 d) values for steel and concrete stress and strain.

Figure E.46: Results for sagging bending moment resistance for girders with a width of 750 [mm].

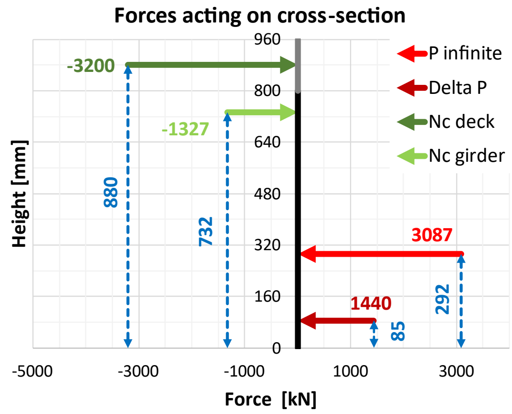


Figure E.47 a) Force distribution.

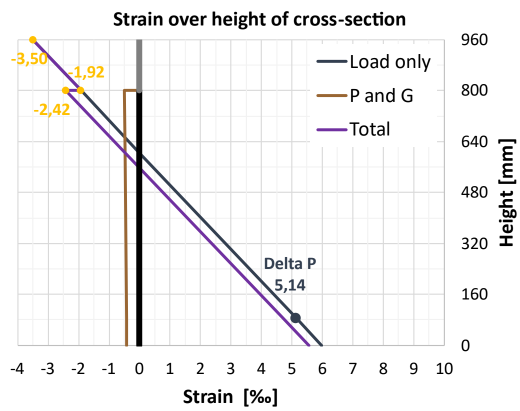
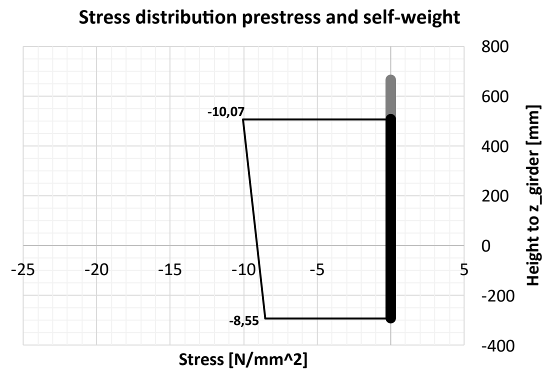


Figure E.47 b) Strain distribution.



Assumed steel stress	1503	[N/mm²]
Steel strain	11,38	[‰]
Initial steel strain	6,22	[‰]
Assumed delta P	5,17	[‰]
α girder	0,62	[-]
β girder	0,35	[-]
α deck	1,00	[-]
β deck	0,50	[-]
Resistance	2764	[kNm]

Figure E.47 c) Stress distribution for self-weight and prestress. Figure E.47 d) values for steel and concrete stress and strain.
Figure E.47: Results for sagging bending moment resistance for girders with a width of 1000 [mm].

Table E.27: Verification sagging bending moment resistance.

System	Width 1200 [mm]			Width 750 [mm]			Width 1000 [mm]		
	M_{rd} [kNm]	M_{ed} [kNm]	UC=$\frac{M_{ed}}{M_{rd}}$	M_{rd} [kNm]	M_{ed} [kNm]	UC=$\frac{M_{ed}}{M_{rd}}$	M_{rd} [kNm]	M_{ed} [kNm]	UC=$\frac{M_{ed}}{M_{rd}}$
SD	3086	1848	0,60	2151	1155	0,54	2764	1540	0,56
SI		1757	0,57		-	-		-	-

In the SI-system hogging bending moments occur near the supports. This moment is resisted by a tensile force in the reinforcement in the deck and a compressive force in the bottom of the girder. The reinforcement layout in the deck will be uniform over the length of the bridge. From Equation E.6 follows that the reinforcement ratio of the deck should be 1,8 [%]. This can be achieved with two layers of reinforcement with bar diameter 12 [mm] and spacing 75 [mm] or diameter 16 [mm] and spacing 125 [mm].

Equation E. 8: Reinforcement in deck

$$M_{ed} < M_{rd}$$
$$M_{ed} = \frac{M_{hog,ed,ULS}}{b_{girder}} = \frac{1256}{1,2} = 1047 \text{ [kNm/m]}$$
$$M_{rd} = A_s \times f_{yd} \times z$$
$$z \approx 0,9 \times d \approx 819 \text{ [mm]}$$
$$f_{yd} = 435 \text{ [N/mm}^2\text{]}$$
$$A_s = \frac{0,25 \times \pi \times \phi^2}{s} > 2938 \text{ [mm}^2\text{/m]} \quad \rightarrow 2 \times \phi 12 - 75$$
$$\rho_{required} = \frac{A_s}{1000 \times h_{deck}} = 1,84 \text{ [\%]} \quad \rightarrow 2 \times \phi 16 - 125$$

E.5.8 Shortening possibilities

Although no shortening in length is needed again side strut-and-tie are provided in Figure E.48 to Figure E.50 to give insight. Based on the strut-and-tie models the needed width to resist the tensile forces is shown in Figure D. 51. This figure confirms that shortening in width direction is not a solution for reducing splitting or spalling stresses. Compared to girders from the first alternative tension develops much sooner. It also shows that the maximum of 23% of shortening might not be feasible for all girders. In this girder a shortening of 23% results or almost results in cracking at the top, which should be avoided. Moreover at 23% of shortening spalling forces start to become more significant. Compared to the first alternative the splitting forces and widths needed are smaller.

Strut-and-tie models in side view

Forces in [kN], the length ranges from the orginal length of 21,26 [m] to 50% of the length. Indicated shortening is shortening on one side.

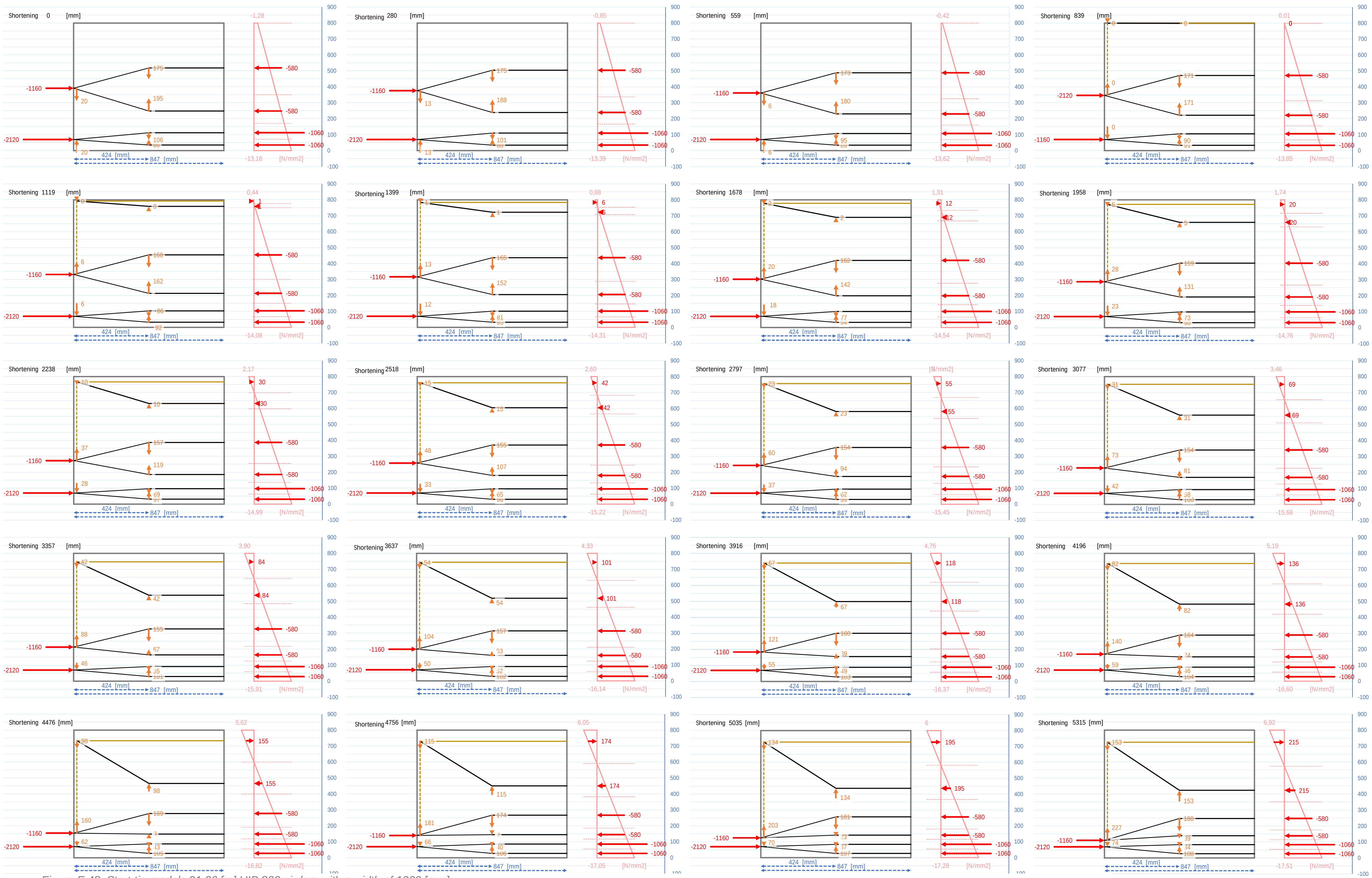


Figure E.48: Strut-tie models 21,26 [m] HIP 800 girders with a width of 1200 [mm].

Strut-and-tie models in side view

Forces in [kN], the length ranges from the orginal length of 21,26 [m] to 50% of the length   Indicated shortening is shortening on one side.

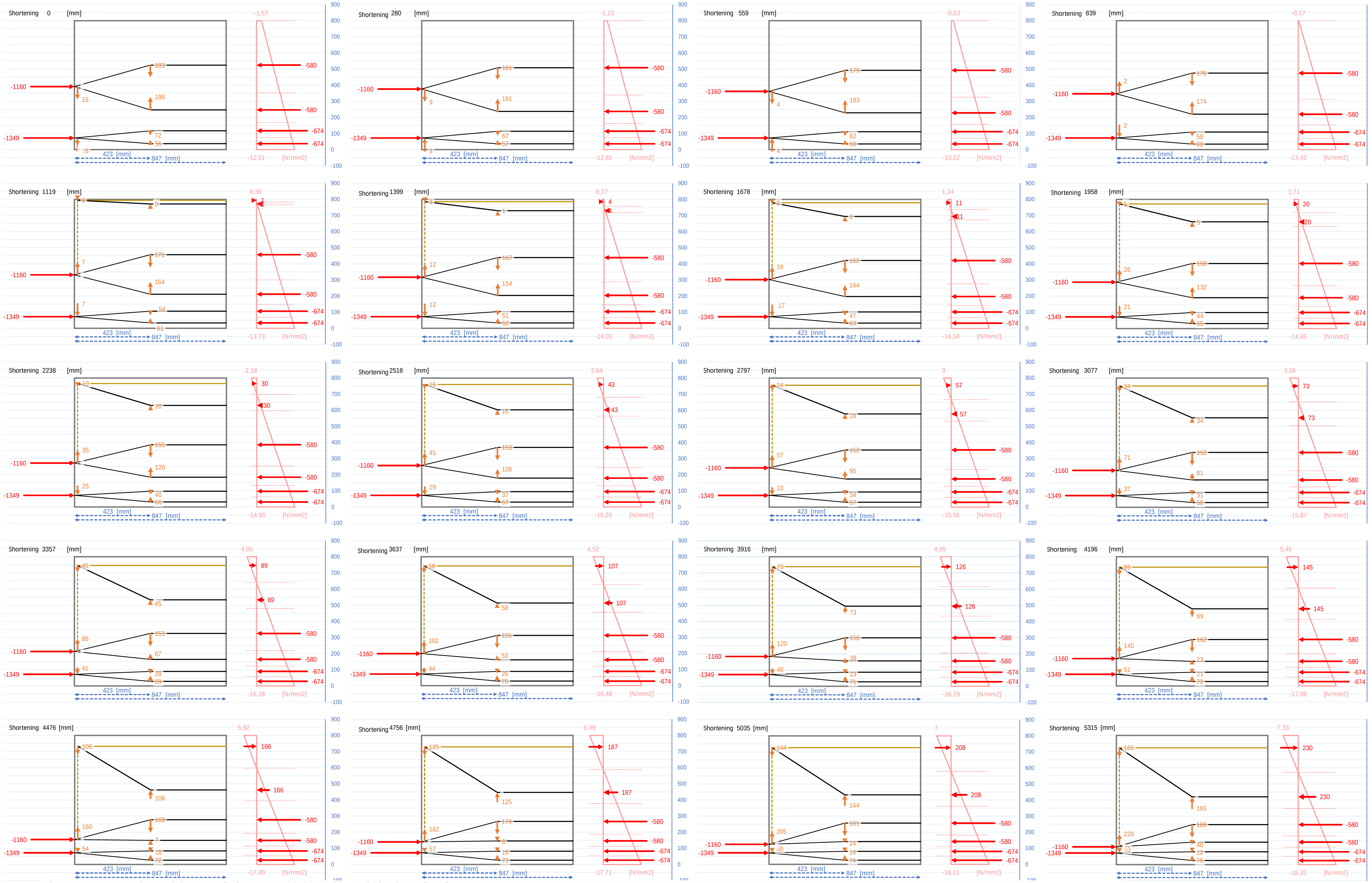


Figure E.49: Strut-tie models 21,26 [m] HIP 800 girders with a width of 750 [mm].

Strut-and-tie models in side view

Forces in [kN], the length ranges from the orginal length of 21,26 [m] to 50% of the length

Indicated shortening is shortening on one side.

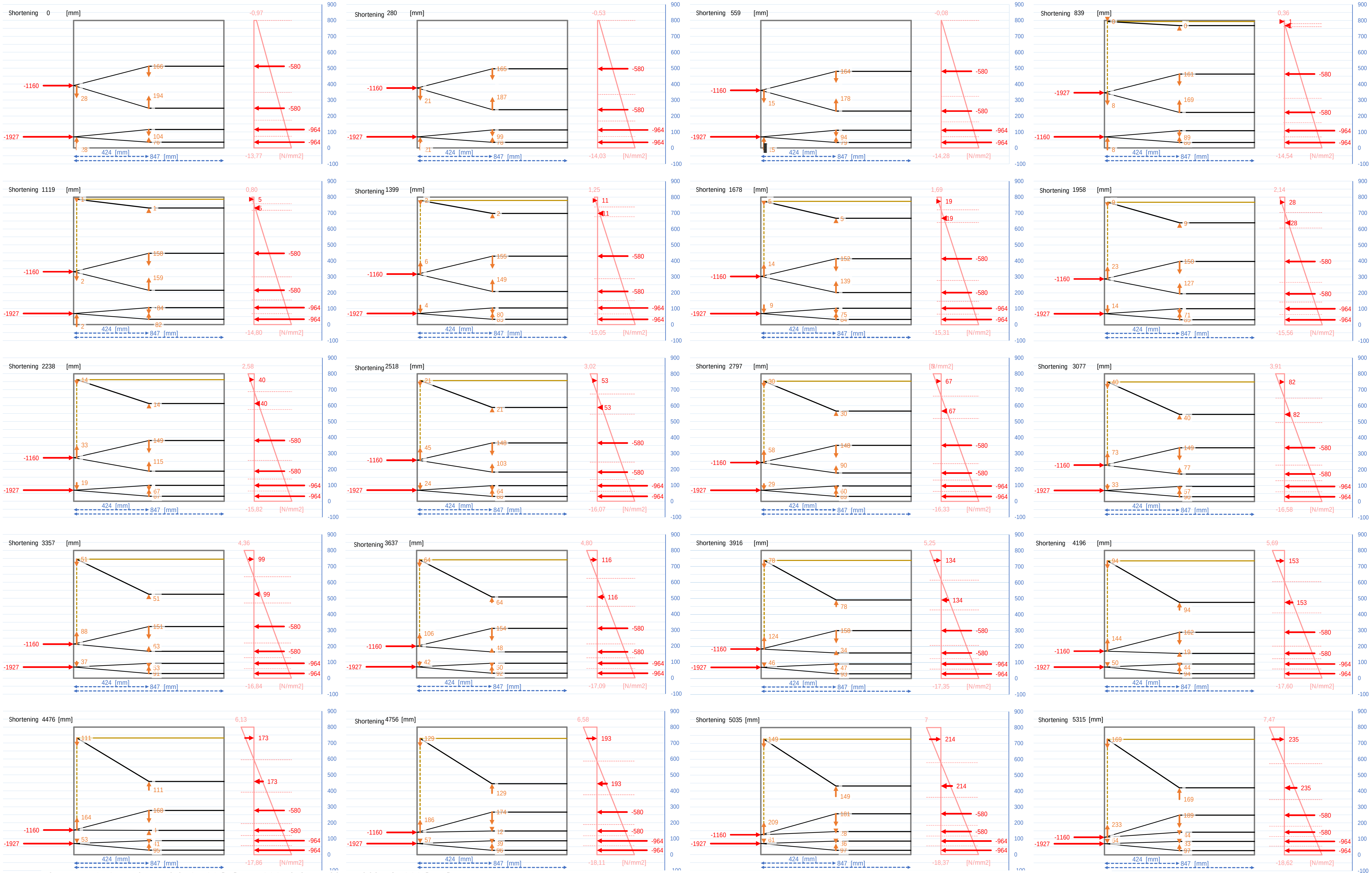


Figure E.50: Strut-tie models 21,26 [m] HIP 800 girders with a width of 1000 [mm].

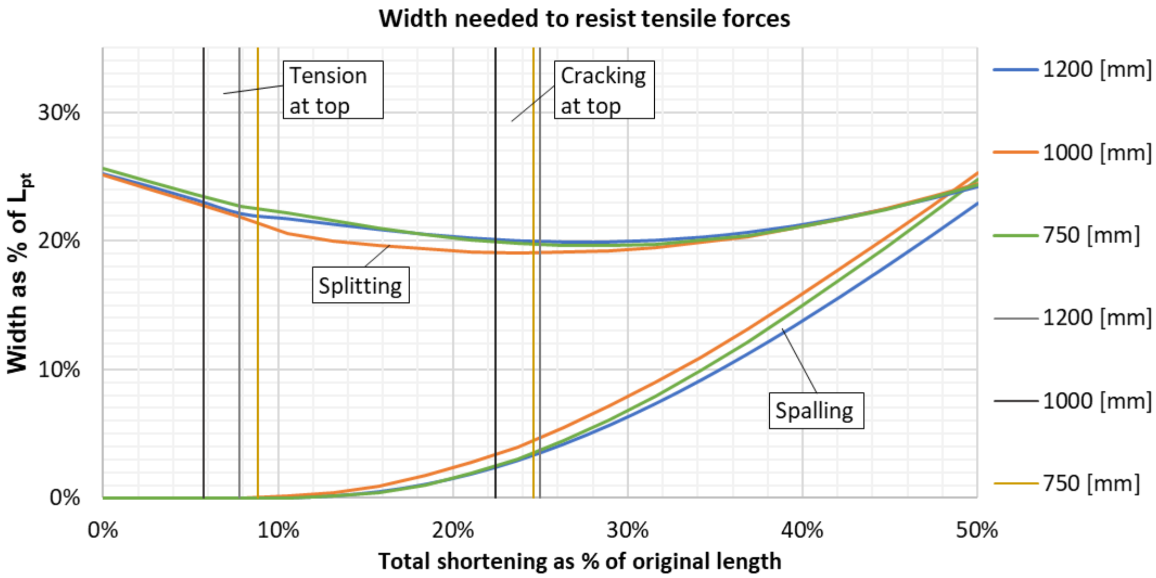


Figure D. 51: Result from strut-and-tie models. Concrete width needed to resist tensile splitting and spalling forces as function of the shortened length. The results are shown for the girders with the original width of 1200 [mm], the shortened width of 1000 [mm] and the shortened width of 750 [mm].

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E.6 SCIA-engineer report

In this final part of this appendix the results from SCIA-engineer are shown for the new loading situation of the girders in alternative 1d in a SI-system. First the geometry of bridge deck is discussed by showing the edges, the support conditions and material characteristics. Next the asphalt and traffic loads are shown for the governing situations. Finally, the bending moment and shearforce lines on the critical cross-sections are shown.

Inhoudsopgave

SCIA ENGINEER
SCIA Engineer 22.0.0019

Case study Arnhem Meinerswijk
Alternatief 1d
Nieuw situatie SI-systeem

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1. Geometrie

1.1. Knopen

Naam	Coördinaat X [m]	Coördinaat Y [m]
K1	0,000	0,000
K2	0,000	6,000
K3	21,260	0,000
K4	21,260	6,000
K5	42,520	0,000
K6	42,520	6,000
K7	63,780	0,000
K8	63,780	6,000
K9	85,040	0,000
K10	85,040	6,000
K11	106,300	0,000
K12	106,300	6,000

1.2. 2D-elementen

Naam	Type	Element type	Materiaal	D. [mm]	Orthotropie	Paneel oppervlak [m²]	Vorm	Knoop
Brugdek	vloer (90)	Standaard	C55/67	960	OT1	637,800	Vlak	K1
								K2
								K12
								K11

1.3. 2D-element interne randen

Naam	Lengte [m]	Vorm	Knoop	Rand
Tussenpunt 1	6,000	Lijn	K3	Lijn
			K4	
Tussenpunt 2	6,000	Lijn	K5	Lijn
			K6	
Tussenpunt 3	6,000	Lijn	K7	Lijn
			K8	
Tussenpunt 4	6,000	Lijn	K9	Lijn
			K10	

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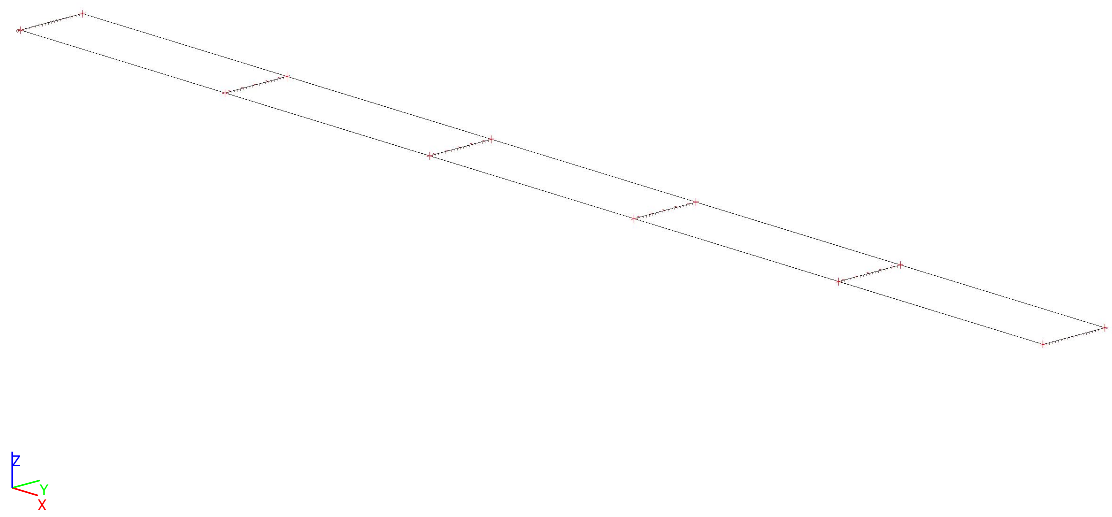
1.4. Overzicht brugdek



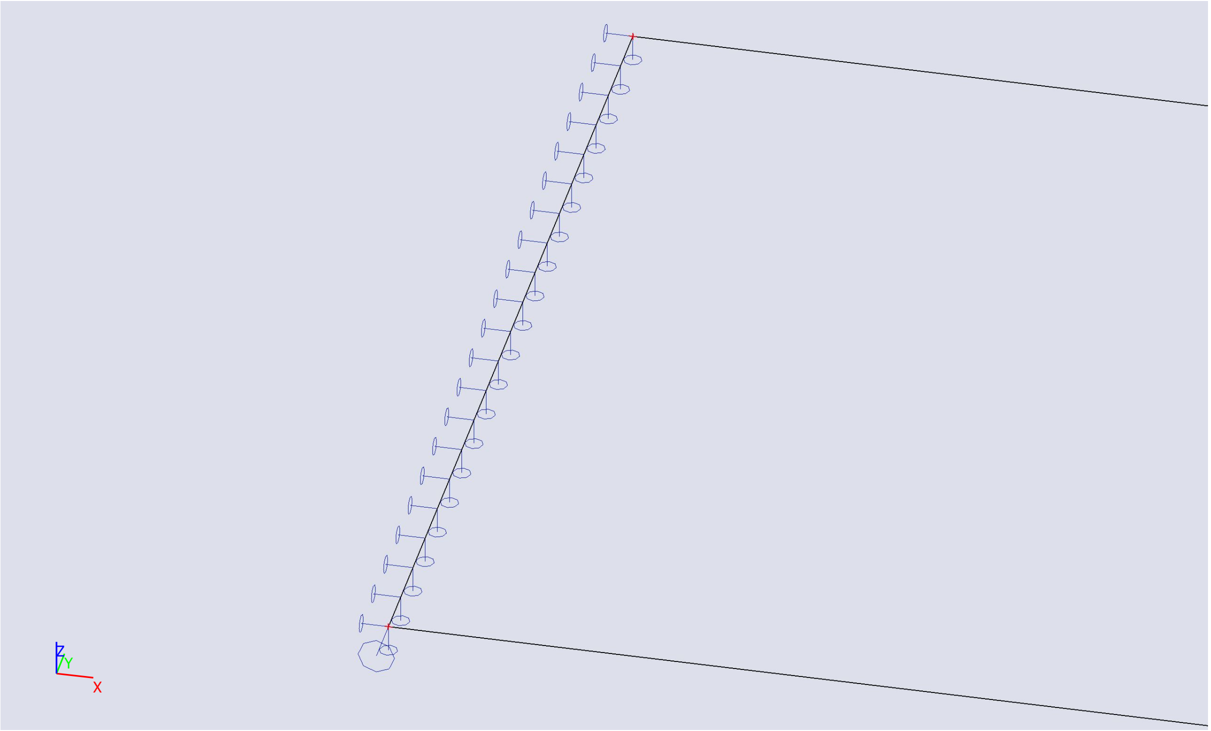
1.5. Ondersteuningen op 2D elementranden

Naam	2D-element Rand	Oors Coör	Pos x1 Pos x2	X	Y	Z	Rx	Ry	Rz
Sle1	Brugdek 1	Vanaf begin Rela	0.000 1.000	Vast	Vrij	Vast	Vrij	Vrij	Vrij
Sle4	Brugdek 3	Vanaf begin Rela	0.000 1.000	Vrij	Vrij	Vast	Vrij	Vrij	Vrij
Sle9	1	Vanaf begin Rela	0.000 1.000	Vrij	Vrij	Vast	Vrij	Vrij	Vrij
Sle10	1	Vanaf begin Rela	0.000 1.000	Vrij	Vrij	Vast	Vrij	Vrij	Vrij
Sle11	1	Vanaf begin Rela	0.000 1.000	Vrij	Vrij	Vast	Vrij	Vrij	Vrij
Sle12	1	Vanaf begin Rela	0.000 1.000	Vrij	Vrij	Vast	Vrij	Vrij	Vrij

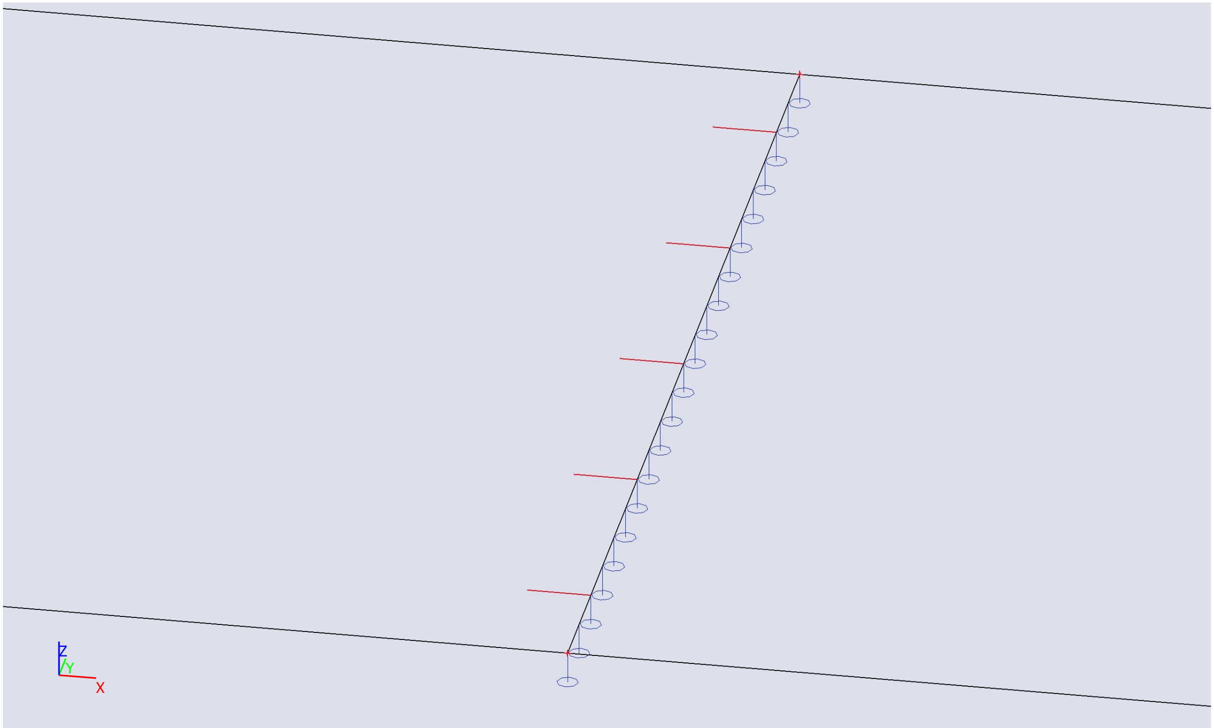
1.6. Knoopondersteuningen

Naam	Knoop	Systeem	Type	X	Y	Z	Rx	Ry	Rz
Sn1	K1	GCS	Standaard	Vrij	Vast	Vrij	Vrij	Vrij	Vrij

1.7. Detail oplegging links



1.8. Detail overige opleggingen



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1.9. Materialen

Beton NEN 6720

Type Naam	Massa eenheid [kg/m³]	E-mod [MPa]	Poisson - nu	G-mod [MPa]	Karakteristieke kubusdruksterkte (f'ck) [MPa] Rekenwaarde van de druksterkte (f'b) [MPa]
Beton C55/67	2500,00	3,8500e+04	0.2	1,6042e+04	67,00 40,20

1.10. Orthotropie

OT1	
Type van orthotropie	Standaard
Dikte van plaat/wand, h [mm]	960
Materiaal	C55/67
D11 [MNm]	2,9568e+03
D22 [MNm]	2,9568e+03
D12 [MNm]	5,9136e+02
D33 [MNm]	1,1827e+03
D44 [MN/m]	1,2833e+04
D55 [MN/m]	1,2833e+04
d11 [MN/m]	3,8500e+04
d22 [MN/m]	3,8500e+04
d12 [MN/m]	7,7000e+03
d33 [MN/m]	1,5400e+04
K xy [MN/m]	1,0000e+00
K yx [MN/m]	1,0000e+00

Verklaring van symbolen	
Coëff voor torsiestijfheid	Deze coëfficiënt vermenigvuldigt de component van torsiestijfheid D33. Standaard waarde = 1
Vormfactor voor dwarskracht	Deze factor deelt de componenten van afschuifstijfheid D44 en D55. Standaard waarde = 1.2
Coëff voor afschuifstijfheid	Deze coëfficiënt vermenigvuldigt de component van afschuifstijfheid van membraan D33. Standaard waarde = 1

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Alternatief 1d
Nieuw situatie SI-systeem

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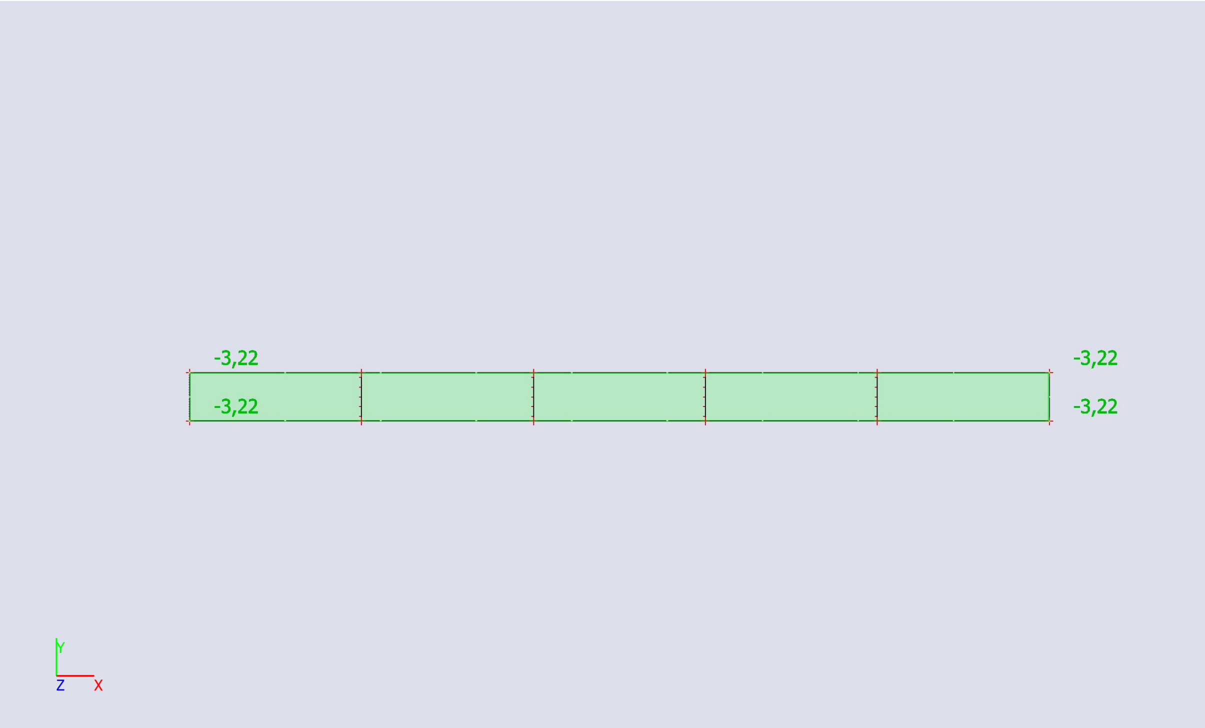
2. Belastingen

2.1. Asfalt belasting

2.1.1. Belasting

Naam	Belastingsgeval	Rich	Type	Verdeling	q [kN/m²]	Coördinaat X [m]	Coördinaat Y [m]
FF2	BG2 - Asfalt	Z	Kracht	Gelijkmatig	-3,22	0,000	0,000
						106,300	0,000
						106,300	6,000
						0,000	6,000

2.1.2. Overzicht



2.2. Verkeersbelasting

2.2.1. Maximaal positief buigend moment

2.2.1.1. Vrije oppervlakte last

Naam	Belastingsgeval	Rich	Type	Verdeling	q [kN/m²]	Coördinaat X [m]	Coördinaat Y [m]
FF15	BG7 - rijstroken sagging	Z	Kracht	Gelijkmatig	-2,00	0,000	0,500
						0,000	5,500
						21,260	5,500
						21,260	0,500
FF16	BG7 - rijstroken sagging	Z	Kracht	Gelijkmatig	-7,83	0,000	0,500
						0,000	3,500
						21,260	3,500
						21,260	0,500
FF17	BG7 - rijstroken sagging	Z	Kracht	Gelijkmatig	-7,83	42,520	0,500
						42,520	3,500

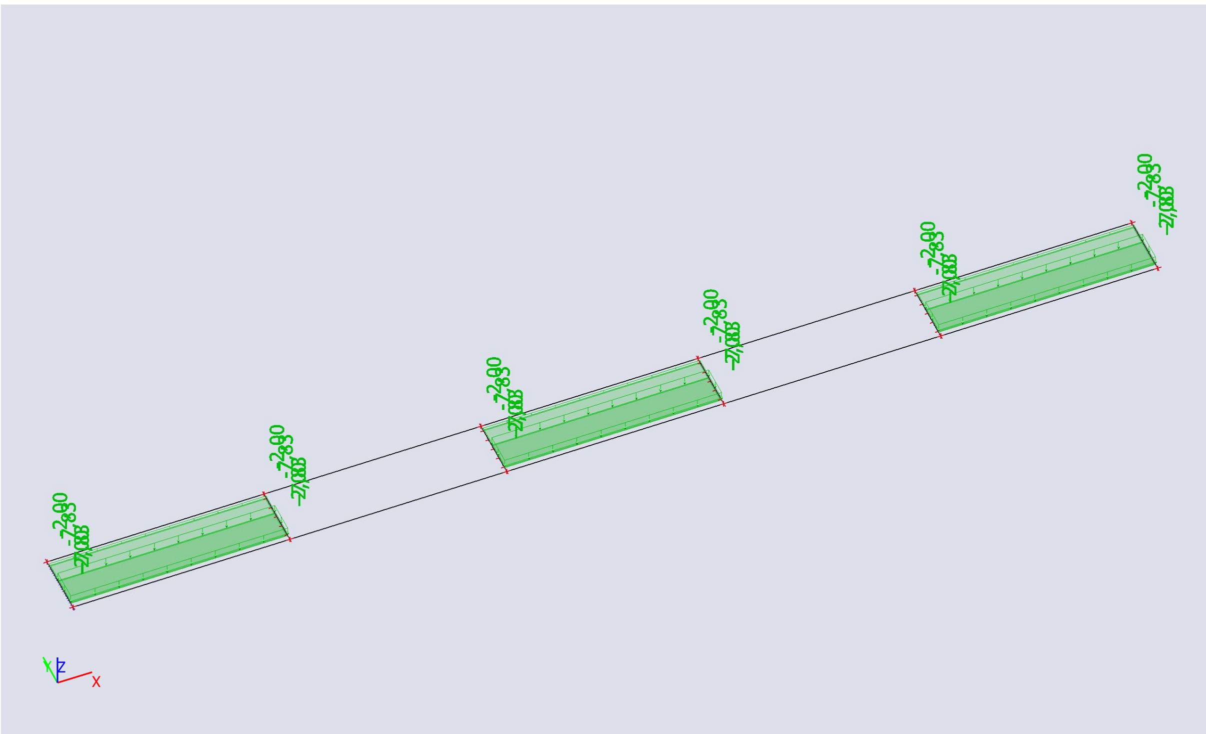
Naam	Belastingsgeval	Rich	Type	Verdeling	q [kN/m²]	Coördinaat X [m]	Coördinaat Y [m]
						63,780	3,500
						63,780	0,500
FF18	BG7 - rijstroken sagging	Z	Kracht	Gelijkmatig	-2,00	42,520	0,500
						42,520	5,500
						63,780	5,500
						63,780	0,500
FF19	BG7 - rijstroken sagging	Z	Kracht	Gelijkmatig	-7,83	85,040	0,500
						85,040	3,500
						106,300	3,500
						106,300	0,500
FF20	BG7 - rijstroken sagging	Z	Kracht	Gelijkmatig	-2,00	85,040	0,500
						85,040	5,500
						106,300	5,500
						106,300	0,500

2.2.1.2. *Vrije puntlast*

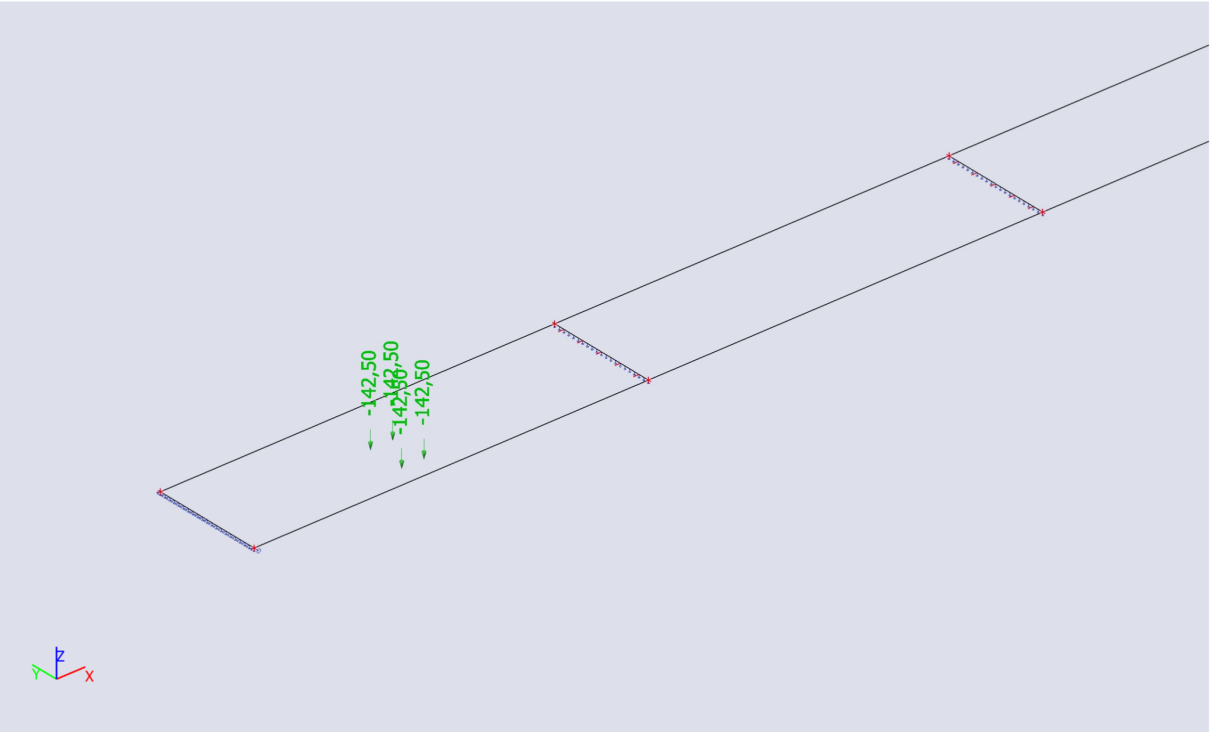
Naam	Belastingsgeval	Type	Coördinaat X [m]	Coördinaat Y [m]	Coördinaat Z [m]	Waarde - F [kN]
FF47	BG6 - assen sagging	Kracht	8,800	1,000	0,000	-142,50
FF48	BG6 - assen sagging	Kracht	8,800	3,000	0,000	-142,50
FF49	BG6 - assen sagging	Kracht	10,000	1,000	0,000	-142,50
FF50	BG6 - assen sagging	Kracht	10,000	3,000	0,000	-142,50

Verklaring van symbolen	
Belastingsgeval	assen sagging

2.2.1.3. *Overzicht q-last*



2.2.1.4. Overzicht as-last



2.2.2. Maximaal steunpunts moment

2.2.2.1. Vrije oppervlakte last

Naam	Belastingsgeval	Rich	Type	Verdeling	q [kN/m²]	Coördinaat X [m]	Coördinaat Y [m]
FF3	BG3 - rijstrook alle	Z	Kracht	Gelijkmatig	-2,00	0,000	0,500
						0,000	5,500
						106,300	5,500
						106,300	0,500
FF7	BG3 - rijstrook alle	Z	Kracht	Gelijkmatig	-7,83	0,000	0,500
						0,000	3,500
						106,300	3,500
						106,300	0,500

2.2.2.2. Vrije puntlast

Naam	Belastingsgeval	Type	Coördinaat X [m]	Coördinaat Y [m]	Coördinaat Z [m]	Waarde - F [kN]
FF55	BG9 - assen hogging	Kracht	12,300	1,000	0,000	-142,50
FF56	BG9 - assen hogging	Kracht	12,300	3,000	0,000	-142,50
FF57	BG9 - assen hogging	Kracht	13,500	1,000	0,000	-142,50
FF58	BG9 - assen hogging	Kracht	13,500	3,000	0,000	-142,50

Verklaring van symbolen	
Belastingsgeval	assen hogging

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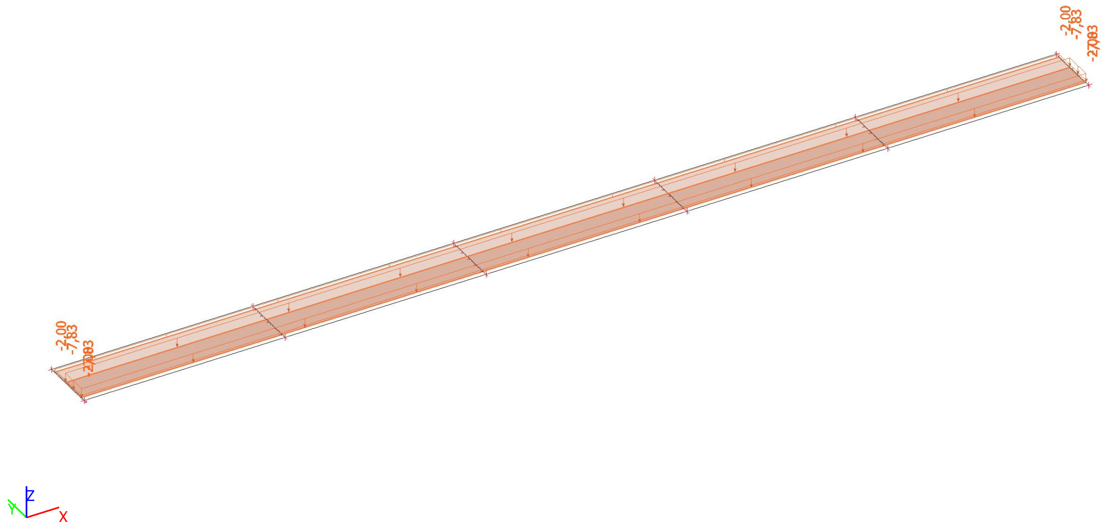
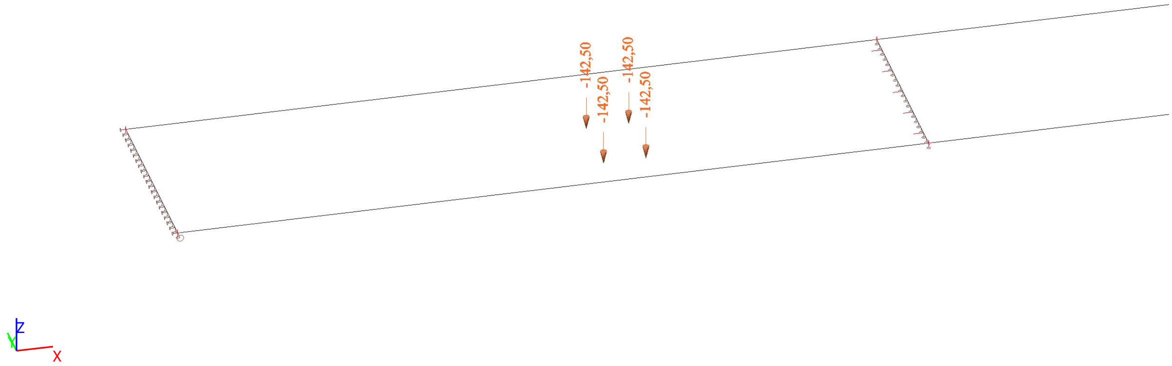
2.2.2.3. Overzicht q-last



2.2.2.4. Overzicht as-last



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2.2.3. Maximaal neerbuigend moment na introductie lengte

2.2.3.1. Vrije oppervlakte last

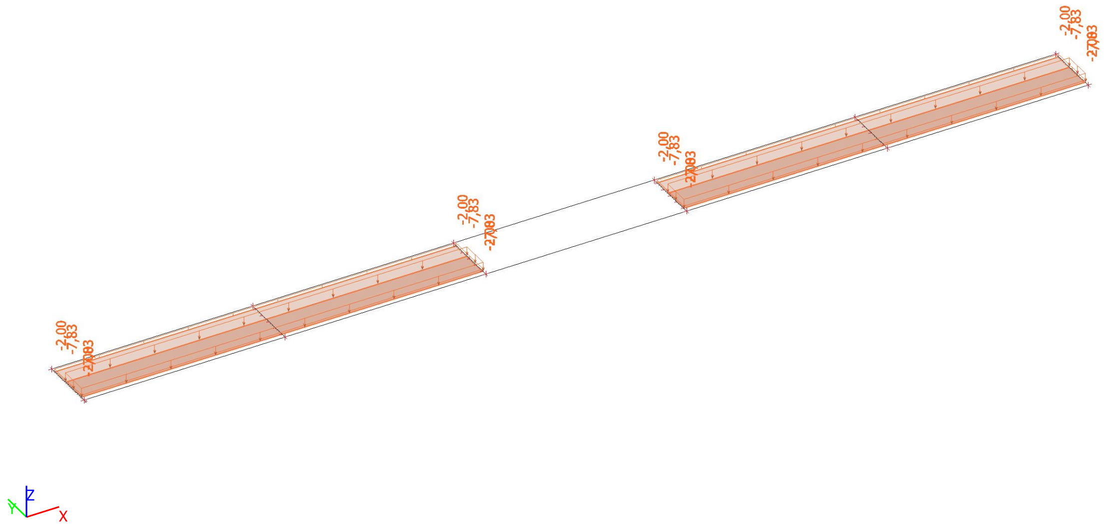
Naam	Belastingsgeval	Rich	Type	Verdeling	q [kN/m²]	Coördinaat X [m]	Coördinaat Y [m]
FF12	BG5 - rijstroken Lpt	Z	Kracht	Gelijkmatig	-2,00	0,000	0,500
						0,000	5,500
						42,520	5,500
						42,520	0,500
FF14	BG5 - rijstroken Lpt	Z	Kracht	Gelijkmatig	-7,83	0,000	0,500
						0,000	3,500
						42,520	3,500
						42,520	0,500
FF21	BG5 - rijstroken Lpt	Z	Kracht	Gelijkmatig	-7,83	63,780	0,500
						63,780	3,500
						106,300	3,500
						106,300	0,500
FF22	BG5 - rijstroken Lpt	Z	Kracht	Gelijkmatig	-2,00	63,780	0,500
						63,780	5,500
						106,300	5,500
						106,300	0,500

2.2.3.2. Vrije puntlast

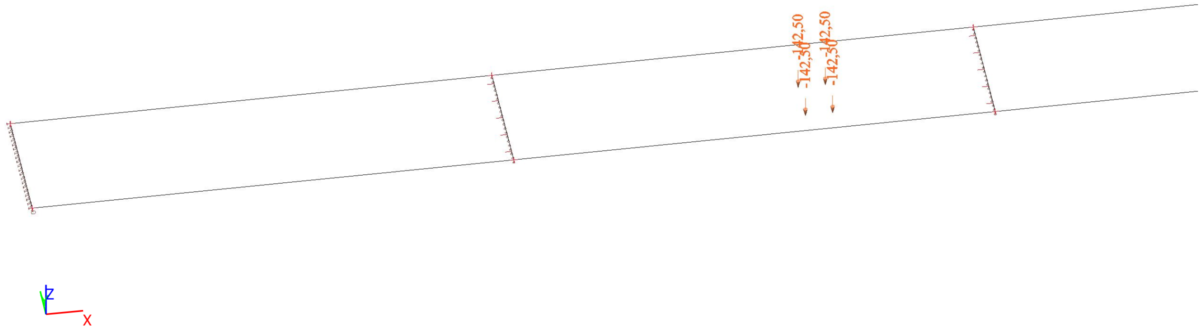
Naam	Belastingsgeval	Type	Coördinaat X [m]	Coördinaat Y [m]	Coördinaat Z [m]	Waarde - F [kN]
FF39	BG4 - assen Lpt	Kracht	34,300	1,000	0,000	-142,50
FF40	BG4 - assen Lpt	Kracht	34,300	3,000	0,000	-142,50
FF41	BG4 - assen Lpt	Kracht	35,500	1,000	0,000	-142,50
FF42	BG4 - assen Lpt	Kracht	35,500	3,000	0,000	-142,50

Verklaring van symbolen	
Belastingsgeval	assen Lpt

2.2.3.3. Overzicht q-last



2.2.3.4. Overzicht as-last



2.2.4. Maximale dwarskracht

2.2.4.1. Vrije oppervlakte last

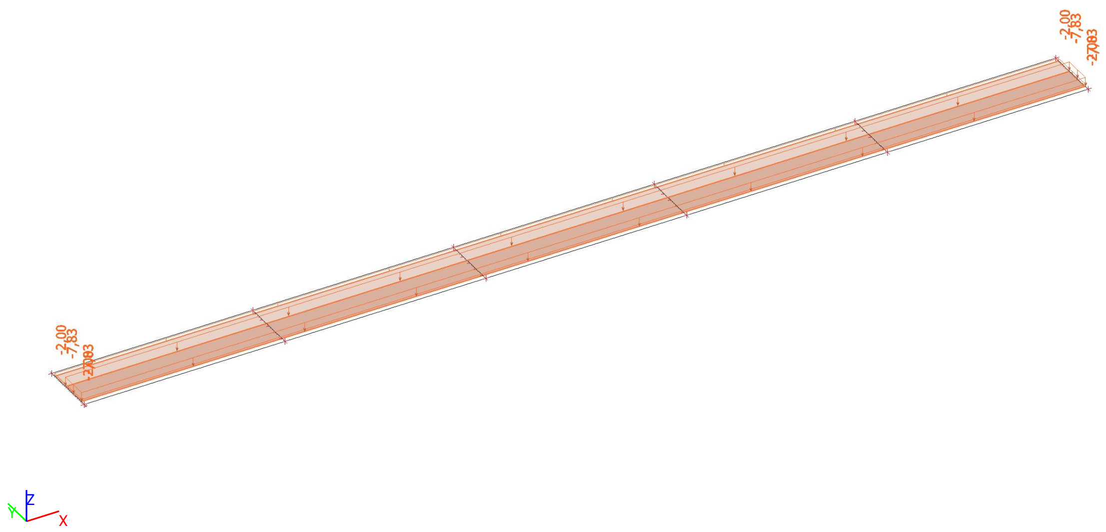
Naam	Belastingsgeval	Rich	Type	Verdeling	q [kN/m²]	Coördinaat X [m]	Coördinaat Y [m]
FF3	BG3 - rijstrook alle	Z	Kracht	Gelijkmatig	-2,00	0,000	0,500
						0,000	5,500
						106,300	5,500
						106,300	0,500
FF7	BG3 - rijstrook alle	Z	Kracht	Gelijkmatig	-7,83	0,000	0,500
						0,000	3,500
						106,300	3,500
						106,300	0,500

2.2.4.2. Vrije puntlast

Naam	Belastingsgeval	Type	Coördinaat X [m]	Coördinaat Y [m]	Coördinaat Z [m]	Waarde - F [kN]
FF51	BG8 - assen shear	Kracht	86,450	1,000	0,000	-142,50
FF52	BG8 - assen shear	Kracht	86,450	3,000	0,000	-142,50
FF53	BG8 - assen shear	Kracht	87,650	1,000	0,000	-142,50
FF54	BG8 - assen shear	Kracht	87,650	3,000	0,000	-142,50

Verklaring van symbolen	
Belastingsgeval	assen shear

2.2.4.3. Overzicht q-last



2.2.4.4. Overzicht as-last

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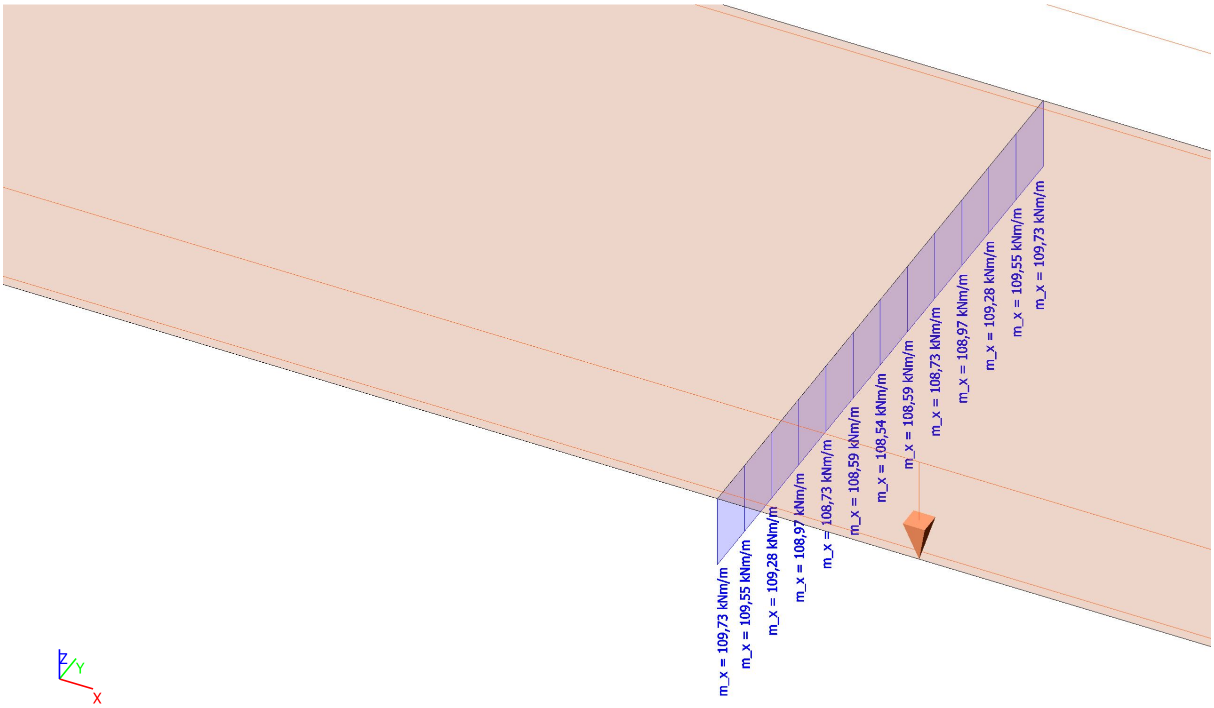
3. Resultaten

3.1. Maximale postief buigend moment

3.1.1. Kritische snede

Naam	Coördinaat X [m]	Coördinaat X [m]	Coördinaat Y [m]	Coördinaat Y [m]	Coördinaat Z [m]	Coördinaat Z [m]
Snede5	9,970	9,970	0,000	6,000	0,000	0,000

3.1.2. Moment op kritische snede door asfalt



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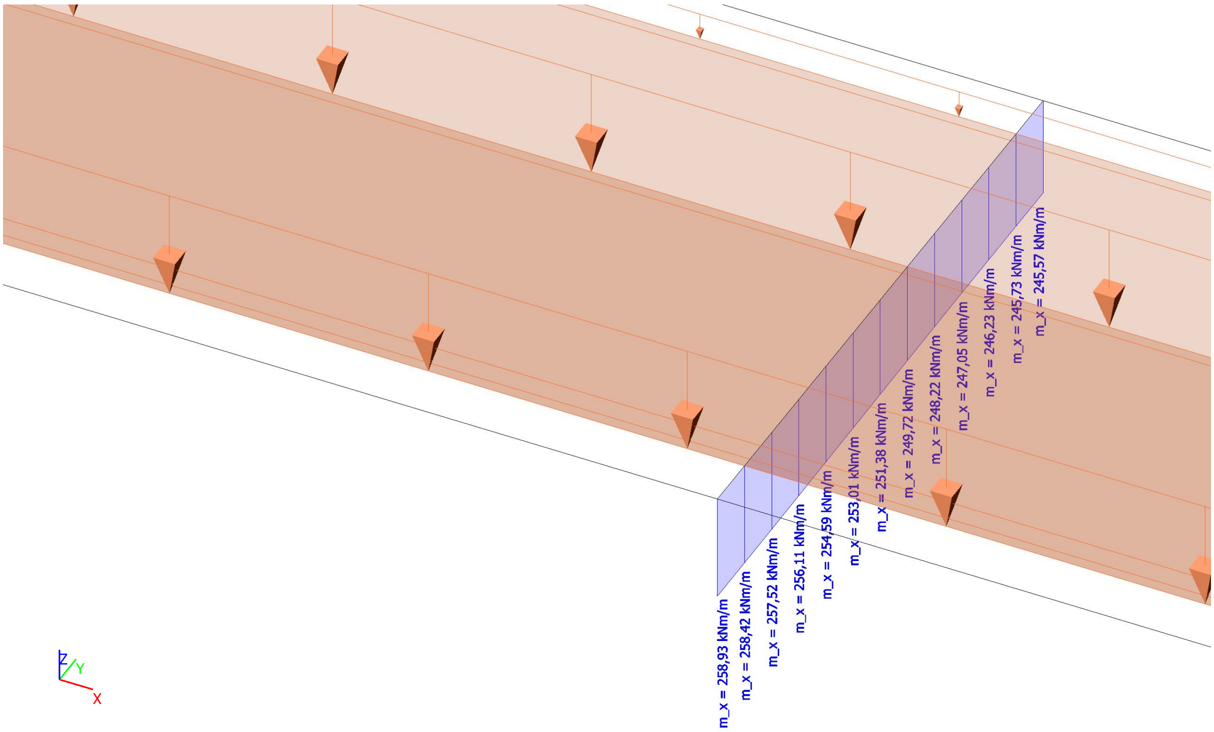
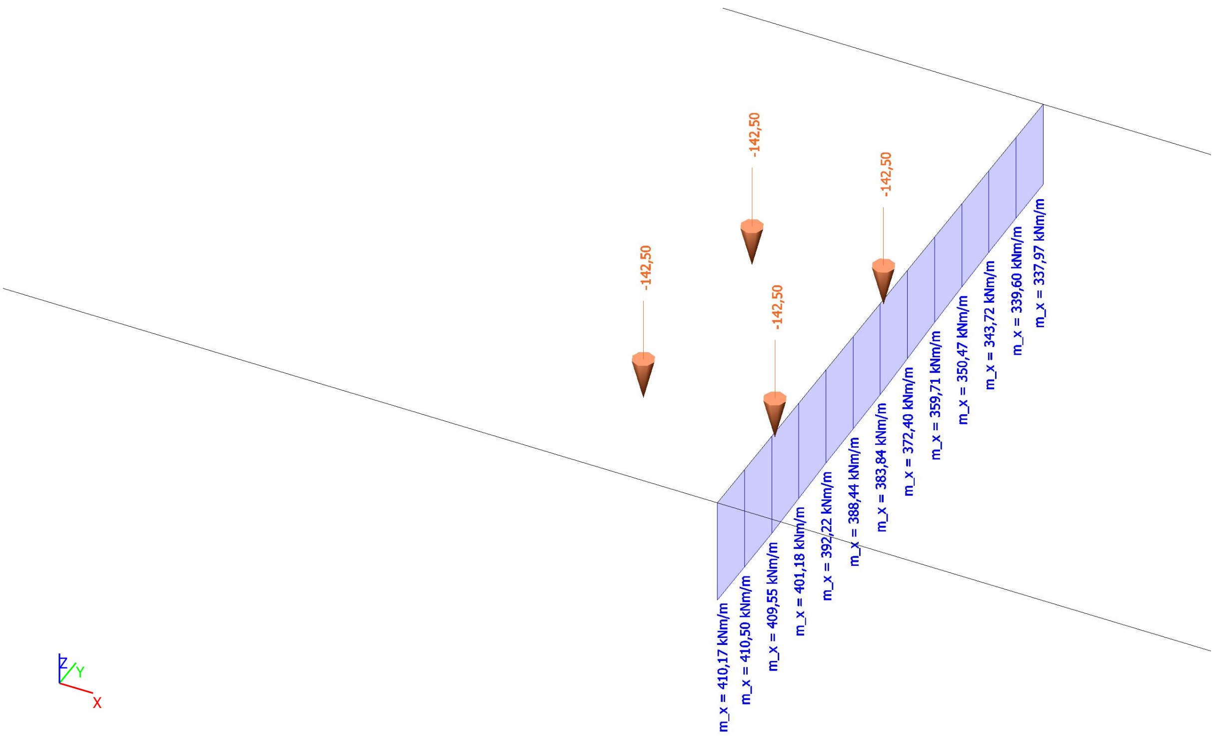
3.1.3. Moment op kritische snede door q-last



3.1.4. Moment kritische snede door assen



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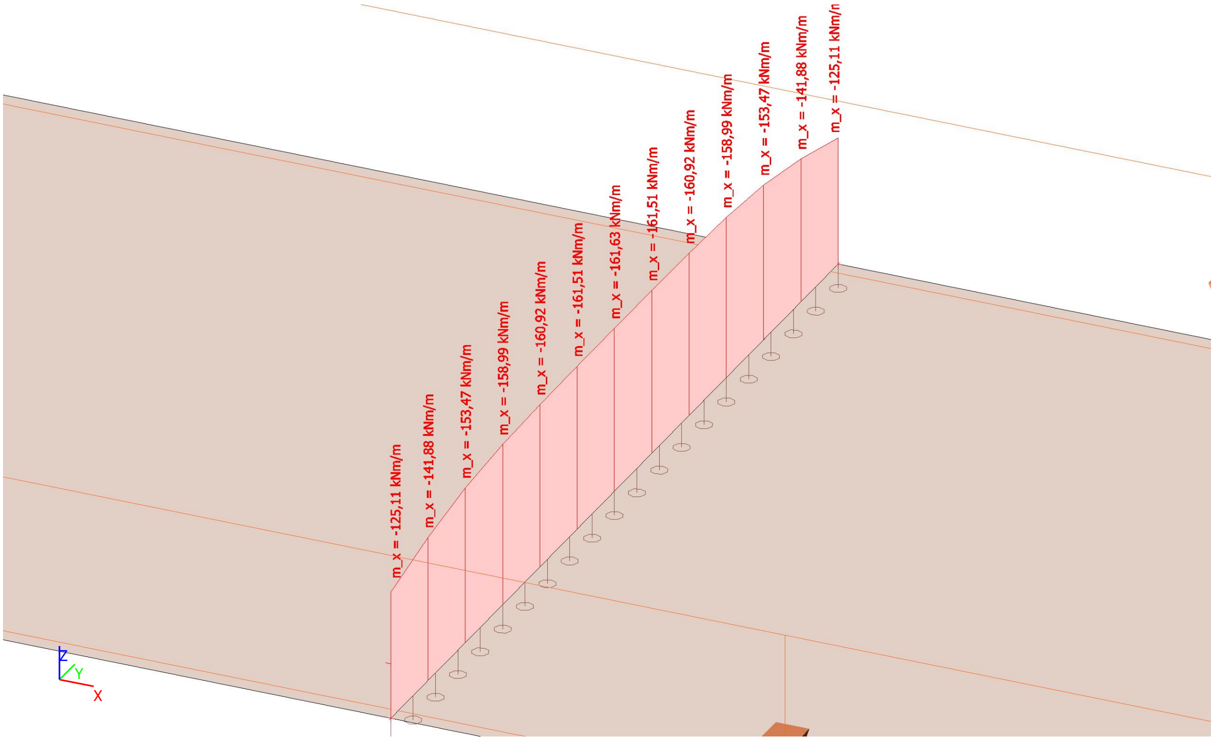
3.2. Maximaal steunpunts moment

3.2.1. Kritische snede

Naam	Coördinaat X [m]	Coördinaat X [m]	Coördinaat Y [m]	Coördinaat Y [m]	Coördinaat Z [m]	Coördinaat Z [m]
Snede14	21,260	21,260	0,000	6,000	0,000	0,000

3.2.2. Moment op kritische snede door asfalt



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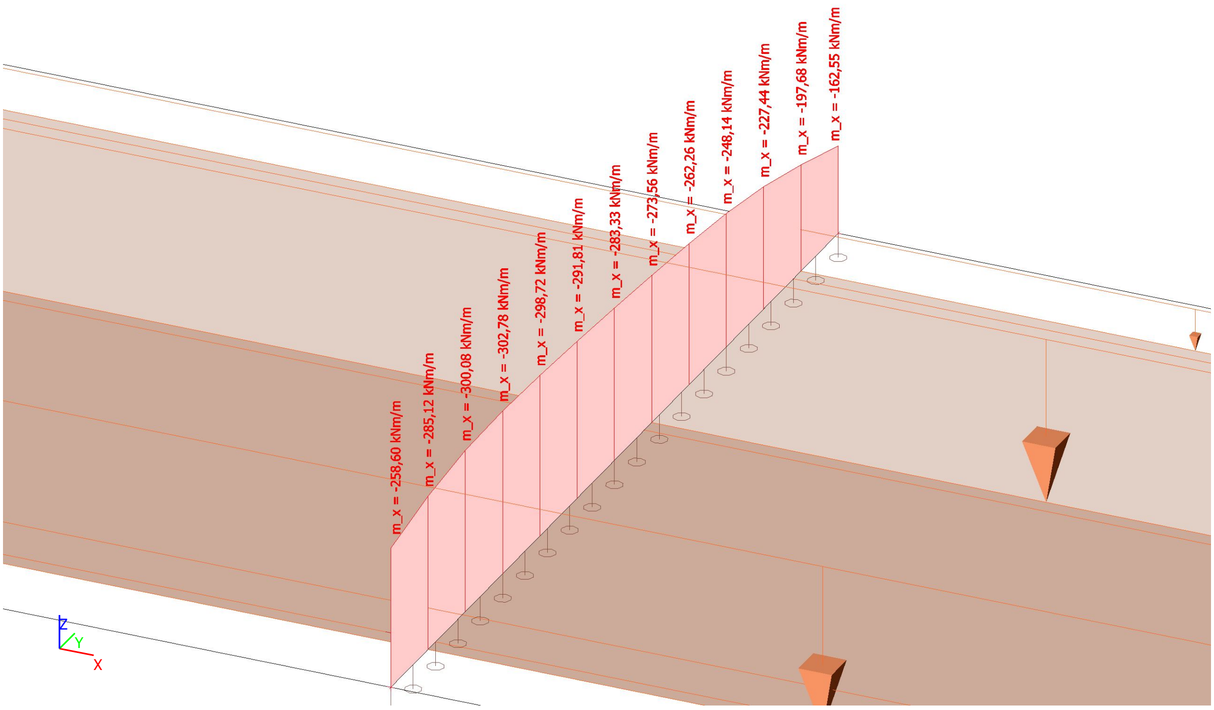
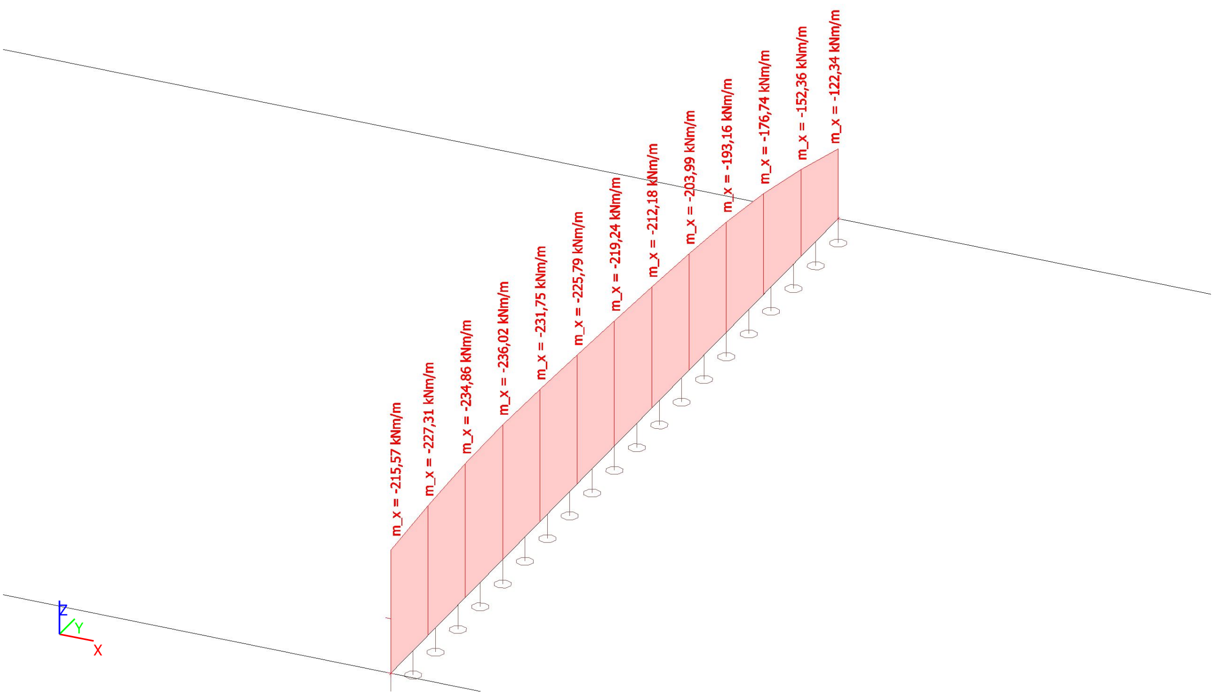
3.2.3. Moment op kritische snede door q-last



3.2.4. Moment op kritische snede door as-last

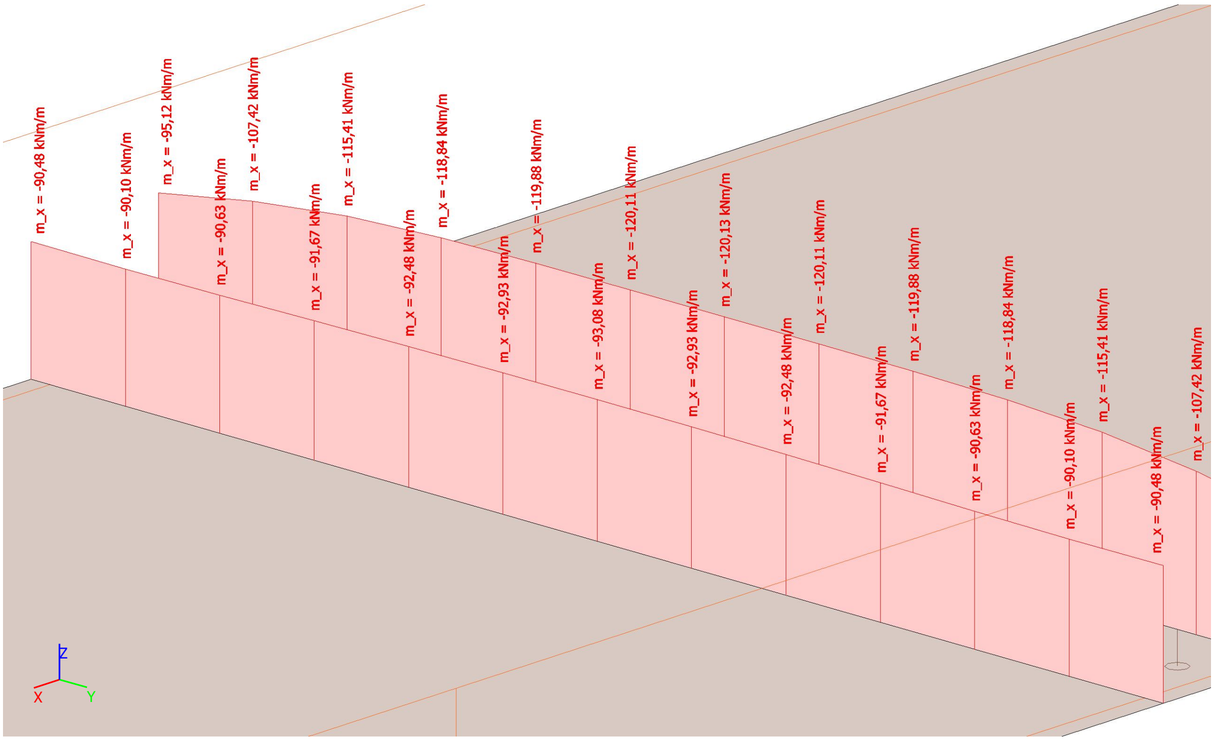


3.3. Maximaal neerbuigend moment na inleiding voorspanning

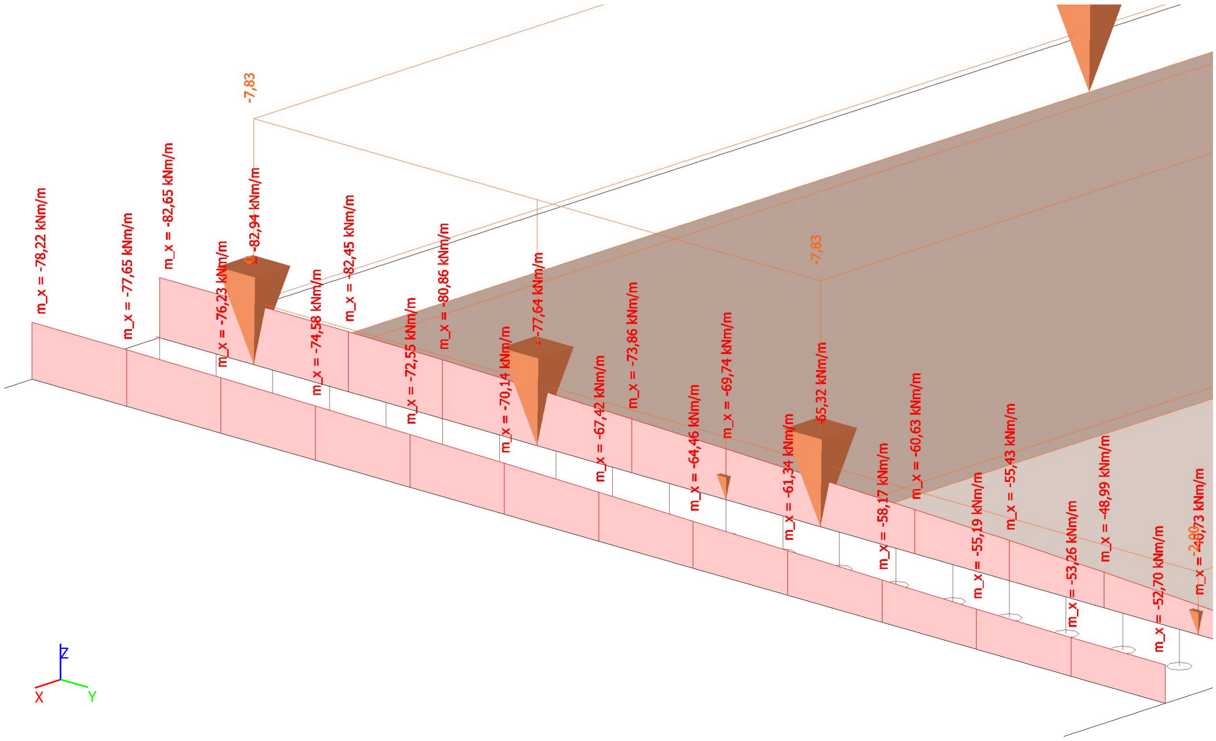
3.3.1. Kritische snede

Naam	Coördinaat X [m]	Coördinaat X [m]	Coördinaat Y [m]	Coördinaat Y [m]	Coördinaat Z [m]	Coördinaat Z [m]
Snede13	43,240	43,240	0,000	6,000	0,000	0,000

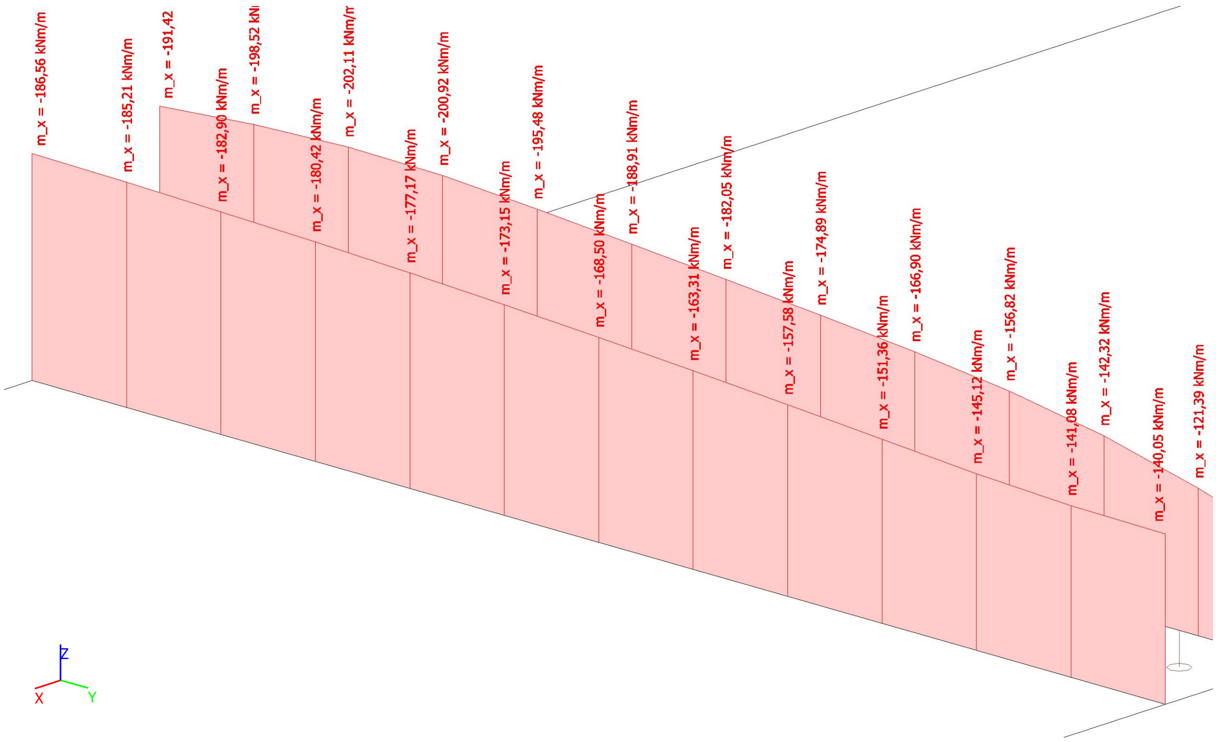
3.3.2. Moment op kritische snede (voorste) door asfalt



3.3.3. Moment op kritische snede (voorste) door q-last



3.3.4. Moment op kritische snede (voorste) door as-last

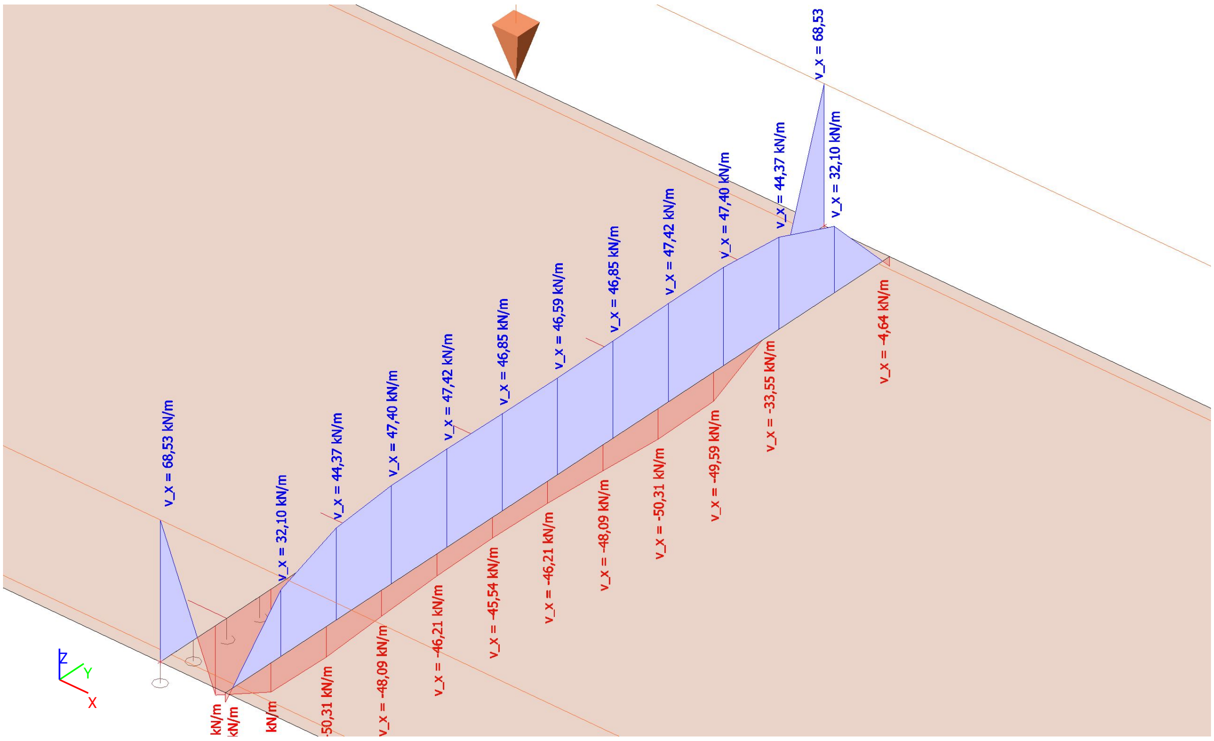


3.4. Maximale dwarskracht

3.4.1. Kritische snede

Naam	Coördinaat X [m]	Coördinaat X [m]	Coördinaat Y [m]	Coördinaat Y [m]	Coördinaat Z [m]	Coördinaat Z [m]
Snede10	85,040	85,040	0,000	6,000	0,000	0,000
Snede11	85,540	85,540	0,000	6,000	0,000	0,000

3.4.2. Moment op kritische snedes door asfalt



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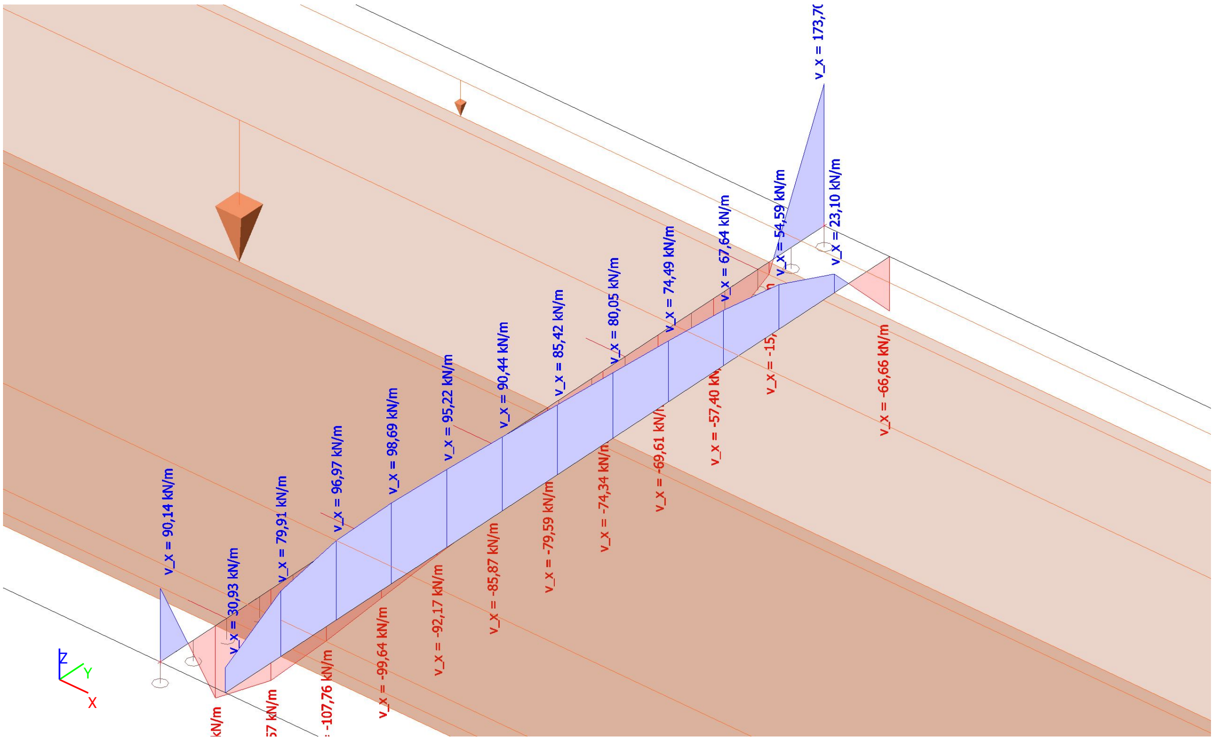
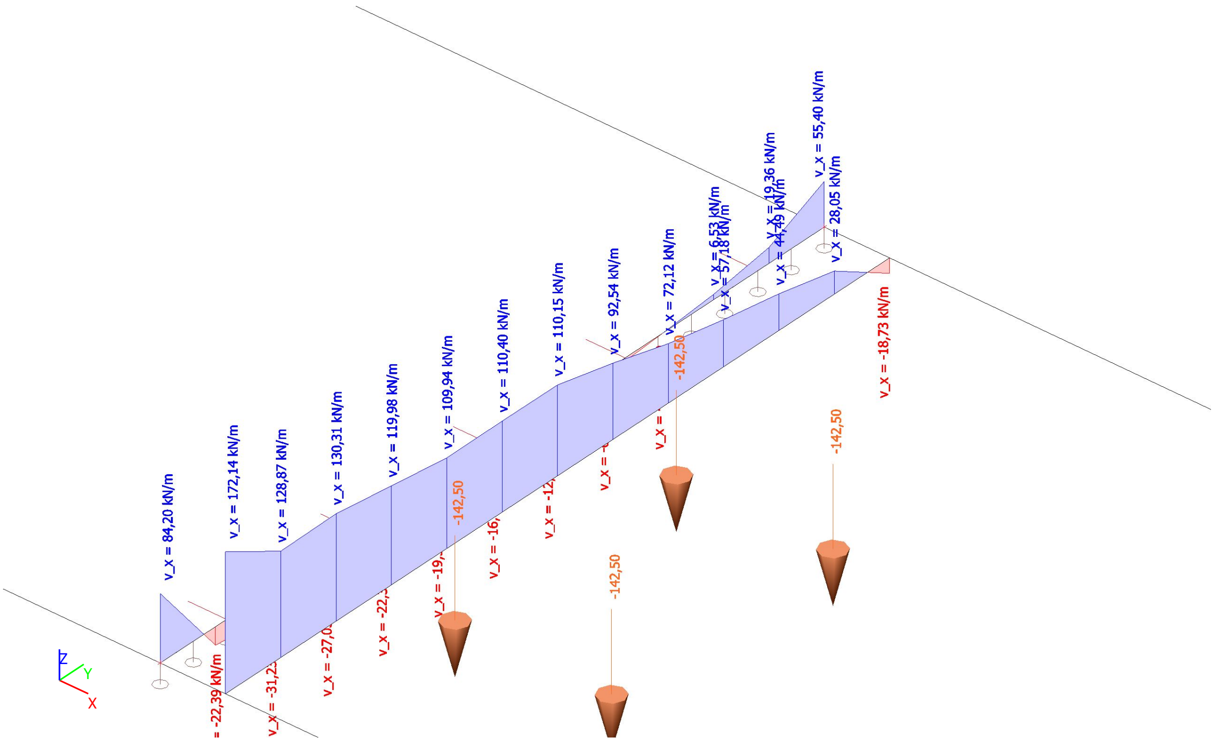
3.4.3. Moment op kritische snedes door q-last



3.4.4. Moment op kritische snedes door as-last



Appendix F: Environmental impact analysis

This appendix provides the background of the environmental impact analysis of the case study, discussed in §4.6 . First the design of the alternative with new girders is discussed. Next the reinforcement in the deck is discussed. In the third section the detailed results of ECI calculation are provided.

F.1 Alternative with new girders

In the alternative with new girders HRP 800 girders from Haitsma are used. The cross-section of this girder is modelled in Figure F.1. The cross-sectional properties used in the calculation are shown in Table F.1. At midspan it is assumed that the centre of gravity of the tendons is located around 85 [mm]. To determine the maximum prestressing force allowed the initial stage is considered, where only prestress and self weight of the girder are present. For the minimum force, the final stage is considered. In this stage prestress and self-weight of girder and deck are carried by the girder and traffic load and the weight of asphalt is carried by the combined action of girder and deck. Moreover, in this stage the prestressing force is reduced to 80% due to losses. The loads are shown in Table F.2. In Equation F.1 the minimum and maximum prestressing force is calculated. Based on this calculation the prestressing force is assumed to be 8500 [kN]. If Y1860S7 is used the maximum initial stress is 1395 [N/mm^2], which results in 6093 [mm^2] of prestressing steel. This can be multiplied by the length of the girder to find the volume of prestressing steel. The same approach is followed with the 16,11[m] girder, which results in 2024 [mm^2].

For stirrups a diameter of 10 [mm] and a spacing of 100 [mm] is assumed. This corresponds to a shear capacity of approximately 778 [kN]. If partial factors are applied on the loads of Table F.2 the shear force is approximately 658 [kN]. So, this assumption is realistic. The length of the shear reinforcement is assumed to be around 6 [m]. This is around 1,5 times the circumference of the cross-section.

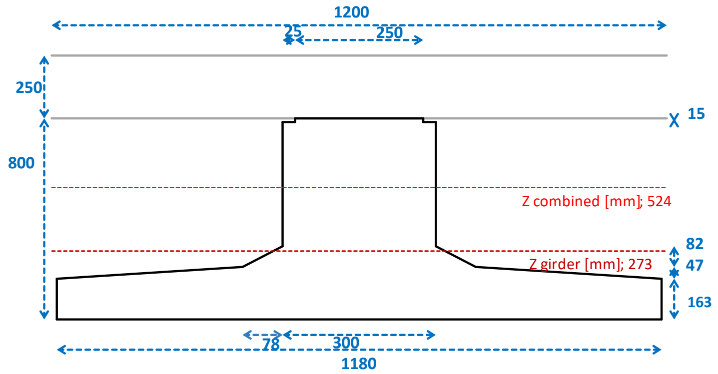


Figure F.1: Cross-section of HRP 800 girder from Haitsma.

Table F.1: Cross-sectional properties of HRP 800 from Haitsma.

Symbol	Value	Unit	Property
A_g	413	[mm^2]	Area
h_{deck}	250	[mm]	Deck height
I_g	2,23 E+10	[mm^4]	Moment of inertia girder
EI_{g+deck}	3,48 E+15	[Nmm^2]	Elastic modulus x moment of inertia girder and deck combined
z_{top}	527	[mm]	Distance neutral axis and top of girder
z_{bot}	273	[mm]	Distance neutral axis and bottom of girder
$z_{b,tot}$	524	[mm]	Distance neutral axis and bottom combined cross-section
E_g	38000	[N/mm^2]	Elastic modulus girder
e_{mid}	188	[mm]	Eccentricity tendons at midspan

Table F.2: Loads on new girders of 25 [m].

Symbol	Type	Uniformly distributed load [kN/m]	Bending moment midspan [kNm]
M_{sg}	Self weight girder	9,9	775
M_{sd}	Self weight deck	7,2	563
M_{as}	Asphalt	3,86	302
M_{tr}	Traffic	20,39	1596

Equation F.1 Minimum and maximum prestressing force allowed to prevent tensile stresses at top or bottom of girder.

Initial stage	Final stage
$\sigma = -\frac{M_{sg} \times z_{top}}{I_g} - \frac{P}{A_g} + \frac{P \times e_{mid} \times z_{top}}{I_g} \leq 0$	$\sigma = \frac{(M_{sg}+M_{sd}) \times z_{bot}}{I_g} + \frac{(M_{as}+M_{tr}) \times z_{bot} \times E_g}{EI_{g+deck}} - \frac{0{,}8 \times P}{A_g} - \frac{0{,}8 \times P \times e \times z_{bot}}{I_g} \leq 0$
$\sigma = -18{,}3 + 2{,}02 \times 10^{-6} \times P \leq 0$	$\sigma = 16{,}38 + 10{,}86 - 4{,}72 \times 10^{-6} \times 0{,}8 \times P \leq 0$
$P \leq 9{,}0 \times 10^6 [N]$	$P \geq 7{,}2 \times 10^6\ [N]$

F.2 Reinforcement in in-situ deck

The amount of reinforcement in the deck is approximated by calculating the maximum force the concrete deck is able to resist. The 95% percentile of the tensile strength of C30/37 is 3,8 [N/mm^2]. Dividing this by the yield strength of B500B reinforcement, so 435 [N/mm^2] gives the amount of reinforcement [m^3] per [m^3] of concrete. This value is multiplied by the steel density of 7850 [kg/m^3]. This value is again multiplied by two, because reinforcement is applied in a grid, so for both directions. This gives 137 [kg/m^3]. In addition, hairpins are present. Assuming a diameter of 10 [mm] a spacing of 100 [mm] and a length of 0,8 [m], this gives 16,4 [kg/m^3] The length is based on two times a height of 250 [mm] and a width of 300 [mm]. The total of 154 [kg/m^3] is multiplied by 1,1 to include additional reinforcement. So, 169 [kg/m^3] is used in the calculations.

F.3 Environmental cost indication

The build-up of the ECI-value for each alternative is shown in Table F.4 until Table F.8.

Table F.3: ECI-value alternative with new girders

Impact category	Total	A1	A2	A3	A4	A5	C1	C2	C3	C4	D
Total	**3,35E+04**	3,72E+04	8,94E+02	4,61E+02	2,65E+03	1,54E+03	1,23E+03	9,06E+02	2,02E+02	1,03E+01	-1,16E+04
Abiotic depletion, non fuel (AD)	1,31E-01	1,16E-01	3,44E-03	2,81E-04	1,02E-02	4,07E-03	5,02E-04	3,49E-03	4,12E-04	1,37E-05	-7,36E-03
Abiotic depletion, fuel (AD)	1,87E+02	2,10E+02	8,91E+00	8,47E+00	2,64E+01	1,02E+01	1,03E+01	9,06E+00	2,29E+00	1,64E-01	-9,86E+01
Global warming (GWP)	1,23E+04	1,34E+04	3,78E+02	3,23E+02	1,12E+03	5,71E+02	4,67E+02	3,83E+02	9,58E+01	3,76E+00	-4,48E+03
Ozone layer depletion (ODP)	5,24E-01	3,41E-01	4,19E-02	1,34E-02	1,24E-01	3,83E-02	5,04E-02	4,24E-02	6,70E-03	7,48E-04	-1,35E-01
Photochemical oxidation (POCP)	2,20E+02	4,33E+02	8,91E+00	2,14E+00	2,64E+01	1,48E+01	1,88E+01	9,06E+00	2,18E+00	1,60E-01	-2,96E+02
Acidification (AP)	3,47E+03	3,74E+03	1,31E+02	4,72E+01	3,88E+02	2,28E+02	2,82E+02	1,33E+02	3,67E+01	2,22E+00	-1,52E+03
Eutrophication (EP)	1,52E+03	1,30E+03	5,88E+01	2,38E+01	1,74E+02	1,08E+02	1,42E+02	5,96E+01	1,86E+01	9,39E-01	-3,67E+02
Human toxicity (HT)	1,51E+04	1,75E+04	2,72E+02	4,86E+01	8,04E+02	5,76E+02	2,97E+02	2,76E+02	4,29E+01	2,76E+00	-4,67E+03
Ecotoxicity, fresh water (FAETP)	5,19E+01	4,41E+01	2,66E+00	3,71E-01	7,89E+00	2,15E+00	1,39E+00	2,71E+00	2,34E-01	2,29E-02	-9,60E+00
Ecotoxicity, marine water (MAETP)	5,92E+02	4,92E+02	3,20E+01	6,07E+00	9,48E+01	2,44E+01	1,57E+01	3,25E+01	3,11E+00	2,60E-01	-1,09E+02
Ecotoxicity, terrestric (TETP)	8,33E+01	8,32E+01	6,44E-01	5,46E-01	1,90E+00	2,59E+00	3,32E-01	6,53E-01	3,40E-01	5,44E-03	-6,90E+00

Table F.4: ECI-value alternative 1a with girders with a width of 1200 [mm].

Impact category	Total	A1	A2	A3	A4	A5	C1	C2	C3	C4	D
Total	**1,38E+04**	1,07E+04	3,08E+02	1,64E+02	2,25E+03	1,37E+03	1,06E+03	7,81E+02	9,27E+01	3,77E+00	-2,89E+03
Abiotic depletion, non fuel (AD)	4,73E-02	3,20E-02	1,19E-03	1,00E-04	8,67E-03	3,71E-03	4,32E-04	3,00E-03	4,05E-04	5,02E-06	-2,27E-03
Abiotic depletion, fuel (AD)	9,39E+01	6,32E+01	3,07E+00	3,02E+00	2,25E+01	8,99E+00	8,89E+00	7,81E+00	9,52E-01	6,00E-02	-2,46E+01
Global warming (GWP)	5,57E+03	4,21E+03	1,30E+02	1,15E+02	9,56E+02	5,03E+02	4,02E+02	3,31E+02	4,02E+01	1,37E+00	-1,12E+03
Ozone layer depletion (ODP)	3,12E-01	1,05E-01	1,44E-02	4,78E-03	1,06E-01	3,36E-02	4,34E-02	3,66E-02	2,84E-03	2,74E-04	-3,46E-02
Photochemical oxidation (POCP)	1,11E+02	1,20E+02	3,07E+00	7,64E-01	2,25E+01	1,30E+01	1,61E+01	7,81E+00	9,59E-01	5,85E-02	-7,37E+01
Acidification (AP)	1,73E+03	1,14E+03	4,52E+01	1,68E+01	3,31E+02	1,99E+02	2,42E+02	1,14E+02	1,68E+01	8,14E-01	-3,84E+02
Eutrophication (EP)	7,56E+02	3,98E+02	2,02E+01	8,49E+00	1,48E+02	9,41E+01	1,22E+02	5,14E+01	8,52E+00	3,44E-01	-9,47E+01
Human toxicity (HT)	5,21E+03	4,53E+03	9,37E+01	1,73E+01	6,84E+02	5,25E+02	2,55E+02	2,38E+02	2,33E+01	1,01E+00	-1,16E+03
Ecotoxicity, fresh water (FAETP)	2,54E+01	1,42E+01	9,17E-01	1,32E-01	6,73E+00	1,95E+00	1,19E+00	2,34E+00	1,23E-01	8,39E-03	-2,23E+00
Ecotoxicity, marine water (MAETP)	2,89E+02	1,57E+02	1,10E+01	2,16E+00	8,08E+01	2,19E+01	1,35E+01	2,80E+01	1,75E+00	9,55E-02	-2,70E+01
Ecotoxicity, terrestric (TETP)	4,03E+01	3,43E+01	2,21E-01	1,95E-01	1,62E+00	2,54E+00	2,86E-01	5,64E-01	1,44E-01	1,98E-03	3,98E-01

Table F.5: ECI-value alternative 1a with girders with a width of 790 [mm].

Impact category	Total	A1	A2	A3	A4	A5	C1	C2	C3	C4	D
Total	**1,86E+04**	1,42E+04	3,86E+02	2,03E+02	3,25E+03	1,99E+03	1,50E+03	1,11E+03	1,02E+02	4,55E+00	-4,09E+03
Abiotic depletion, non fuel (AD)	6,52E-02	4,33E-02	1,49E-03	1,24E-04	1,25E-02	5,57E-03	6,08E-04	4,28E-03	3,42E-04	6,07E-06	-2,95E-03
Abiotic depletion, fuel (AD)	1,25E+02	8,19E+01	3,85E+00	3,74E+00	3,25E+01	1,27E+01	1,25E+01	1,11E+01	1,09E+00	7,25E-02	-3,48E+01
Global warming (GWP)	7,29E+03	5,39E+03	1,63E+02	1,43E+02	1,38E+03	7,16E+02	5,66E+02	4,71E+02	4,59E+01	1,66E+00	-1,59E+03
Ozone layer depletion (ODP)	4,27E-01	1,35E-01	1,81E-02	5,92E-03	1,53E-01	4,73E-02	6,12E-02	5,21E-02	3,23E-03	3,31E-04	-4,84E-02
Photochemical oxidation (POCP)	1,48E+02	1,62E+02	3,85E+00	9,46E-01	3,25E+01	1,85E+01	2,28E+01	1,11E+01	1,07E+00	7,07E-02	-1,04E+02
Acidification (AP)	2,30E+03	1,48E+03	5,67E+01	2,08E+01	4,77E+02	2,83E+02	3,42E+02	1,63E+02	1,84E+01	9,84E-01	-5,41E+02
Eutrophication (EP)	1,02E+03	5,14E+02	2,54E+01	1,05E+01	2,14E+02	1,34E+02	1,72E+02	7,32E+01	9,36E+00	4,16E-01	-1,32E+02
Human toxicity (HT)	7,28E+03	6,29E+03	1,17E+02	2,15E+01	9,87E+02	7,84E+02	3,61E+02	3,39E+02	2,38E+01	1,22E+00	-1,65E+03
Ecotoxicity, fresh water (FAETP)	3,34E+01	1,78E+01	1,15E+00	1,64E-01	9,70E+00	2,78E+00	1,68E+00	3,32E+00	1,28E-01	1,01E-02	-3,29E+00
Ecotoxicity, marine water (MAETP)	3,85E+02	1,98E+02	1,38E+01	2,68E+00	1,16E+02	3,13E+01	1,90E+01	3,98E+01	1,77E+00	1,15E-01	-3,84E+01
Ecotoxicity, terrestric (TETP)	4,49E+01	3,83E+01	2,78E-01	2,41E-01	2,34E+00	3,46E+00	4,03E-01	8,02E-01	1,64E-01	2,40E-03	-1,09E+00

Table F.6: ECI-value alternative 1d with girders with a width of 1200 [mm].

Impact category	Total	A1	A2	A3	A4	A5	C1	C2	C3	C4	D
Total	**1,39E+04**	1,08E+04	3,09E+02	1,65E+02	2,28E+03	1,38E+03	1,07E+03	7,89E+02	9,30E+01	3,79E+00	-2,92E+03
Abiotic depletion, non fuel (AD)	4,77E-02	3,23E-02	1,19E-03	1,01E-04	8,76E-03	3,75E-03	4,37E-04	3,04E-03	4,06E-04	5,04E-06	-2,28E-03
Abiotic depletion, fuel (AD)	9,47E+01	6,37E+01	3,09E+00	3,03E+00	2,27E+01	9,09E+00	8,98E+00	7,89E+00	9,56E-01	6,03E-02	-2,49E+01
Global warming (GWP)	5,61E+03	4,24E+03	1,31E+02	1,16E+02	9,66E+02	5,09E+02	4,06E+02	3,34E+02	4,03E+01	1,38E+00	-1,13E+03
Ozone layer depletion (ODP)	3,15E-01	1,05E-01	1,45E-02	4,80E-03	1,07E-01	3,40E-02	4,38E-02	3,70E-02	2,85E-03	2,76E-04	-3,49E-02
Photochemical oxidation (POCP)	1,12E+02	1,21E+02	3,09E+00	7,68E-01	2,27E+01	1,31E+01	1,63E+01	7,90E+00	9,62E-01	5,88E-02	-7,44E+01
Acidification (AP)	1,74E+03	1,15E+03	4,55E+01	1,69E+01	3,34E+02	2,01E+02	2,44E+02	1,16E+02	1,68E+01	8,18E-01	-3,88E+02
Eutrophication (EP)	7,63E+02	4,00E+02	2,03E+01	8,53E+00	1,50E+02	9,51E+01	1,23E+02	5,19E+01	8,55E+00	3,46E-01	-9,56E+01
Human toxicity (HT)	5,26E+03	4,57E+03	9,42E+01	1,74E+01	6,92E+02	5,31E+02	2,58E+02	2,41E+02	2,34E+01	1,02E+00	-1,17E+03
Ecotoxicity, fresh water (FAETP)	2,56E+01	1,43E+01	9,22E-01	1,33E-01	6,80E+00	1,97E+00	1,20E+00	2,36E+00	1,24E-01	8,43E-03	-2,25E+00
Ecotoxicity, marine water (MAETP)	2,92E+02	1,58E+02	1,11E+01	2,17E+00	8,16E+01	2,21E+01	1,36E+01	2,83E+01	1,75E+00	9,60E-02	-2,73E+01
Ecotoxicity, terrestric (TETP)	4,05E+01	3,45E+01	2,23E-01	1,96E-01	1,64E+00	2,57E+00	2,89E-01	5,70E-01	1,45E-01	1,99E-03	3,77E-01

Table F.7: ECI-value alternative 1d with girders with a width of 1000 [mm].

Impact category	Total	A1	A2	A3	A4	A5	C1	C2	C3	C4	D
Total	**1,64E+04**	1,25E+04	3,45E+02	1,82E+02	2,79E+03	1,70E+03	1,30E+03	9,59E+02	9,80E+01	4,17E+00	-3,52E+03
Abiotic depletion, non fuel (AD)	5,67E-02	3,79E-02	1,33E-03	1,12E-04	1,07E-02	4,71E-03	5,27E-04	3,69E-03	3,87E-04	5,55E-06	-2,61E-03
Abiotic depletion, fuel (AD)	1,10E+02	7,31E+01	3,44E+00	3,36E+00	2,78E+01	1,10E+01	1,09E+01	9,59E+00	1,03E+00	6,63E-02	-3,00E+01
Global warming (GWP)	6,47E+03	4,82E+03	1,46E+02	1,28E+02	1,18E+03	6,18E+02	4,90E+02	4,06E+02	4,32E+01	1,52E+00	-1,36E+03
Ozone layer depletion (ODP)	3,73E-01	1,20E-01	1,62E-02	5,32E-03	1,31E-01	4,10E-02	5,30E-02	4,49E-02	3,04E-03	3,03E-04	-4,18E-02
Photochemical oxidation (POCP)	1,31E+02	1,42E+02	3,44E+00	8,49E-01	2,78E+01	1,60E+01	1,97E+01	9,59E+00	1,02E+00	6,46E-02	-8,97E+01
Acidification (AP)	2,03E+03	1,32E+03	5,07E+01	1,87E+01	4,09E+02	2,45E+02	2,96E+02	1,40E+02	1,77E+01	9,00E-01	-4,66E+02
Eutrophication (EP)	8,97E+02	4,58E+02	2,27E+01	9,44E+00	1,83E+02	1,16E+02	1,49E+02	6,31E+01	9,00E+00	3,80E-01	-1,14E+02
Human toxicity (HT)	6,30E+03	5,46E+03	1,05E+02	1,93E+01	8,46E+02	6,64E+02	3,12E+02	2,92E+02	2,39E+01	1,12E+00	-1,42E+03
Ecotoxicity, fresh water (FAETP)	2,98E+01	1,62E+01	1,03E+00	1,47E-01	8,32E+00	2,40E+00	1,45E+00	2,87E+00	1,27E-01	9,27E-03	-2,77E+00
Ecotoxicity, marine water (MAETP)	3,41E+02	1,79E+02	1,24E+01	2,41E+00	9,98E+01	2,70E+01	1,65E+01	3,44E+01	1,79E+00	1,06E-01	-3,29E+01
Ecotoxicity, terrestric (TETP)	4,35E+01	3,70E+01	2,48E-01	2,16E-01	2,01E+00	3,06E+00	3,49E-01	6,92E-01	1,54E-01	2,19E-03	-2,61E-01

Table F.8: ECI-value alternative 1d with girders with a width of 750 [mm].

Impact category	Total	A1	A2	A3	A4	A5	C1	C2	C3	C4	D
Total	**1,97E+04**	1,49E+04	3,94E+02	2,06E+02	3,50E+03	2,15E+03	1,61E+03	1,19E+03	1,04E+02	4,67E+00	-4,36E+03
Abiotic depletion, non fuel (AD)	6,94E-02	4,57E-02	1,52E-03	1,26E-04	1,35E-02	6,05E-03	6,52E-04	4,60E-03	3,48E-04	6,22E-06	-3,06E-03
Abiotic depletion, fuel (AD)	1,32E+02	8,61E+01	3,92E+00	3,79E+00	3,49E+01	1,37E+01	1,34E+01	1,20E+01	1,11E+00	7,43E-02	-3,72E+01
Global warming (GWP)	7,65E+03	5,62E+03	1,67E+02	1,45E+02	1,48E+03	7,70E+02	6,08E+02	5,06E+02	4,67E+01	1,70E+00	-1,69E+03
Ozone layer depletion (ODP)	4,54E-01	1,41E-01	1,84E-02	6,01E-03	1,64E-01	5,08E-02	6,56E-02	5,59E-02	3,28E-03	3,39E-04	-5,15E-02
Photochemical oxidation (POCP)	1,57E+02	1,71E+02	3,92E+00	9,61E-01	3,49E+01	1,99E+01	2,44E+01	1,20E+01	1,09E+00	7,24E-02	-1,11E+02
Acidification (AP)	2,43E+03	1,55E+03	5,78E+01	2,11E+01	5,13E+02	3,05E+02	3,67E+02	1,75E+02	1,88E+01	1,01E+00	-5,76E+02
Eutrophication (EP)	1,08E+03	5,38E+02	2,59E+01	1,07E+01	2,30E+02	1,45E+02	1,85E+02	7,86E+01	9,53E+00	4,26E-01	-1,40E+02
Human toxicity (HT)	7,77E+03	6,70E+03	1,20E+02	2,18E+01	1,06E+03	8,52E+02	3,87E+02	3,64E+02	2,43E+01	1,25E+00	-1,76E+03
Ecotoxicity, fresh water (FAETP)	3,55E+01	1,87E+01	1,17E+00	1,66E-01	1,04E+01	3,00E+00	1,80E+00	3,57E+00	1,30E-01	1,04E-02	-3,51E+00
Ecotoxicity, marine water (MAETP)	4,08E+02	2,08E+02	1,41E+01	2,72E+00	1,25E+02	3,38E+01	2,04E+01	4,28E+01	1,80E+00	1,18E-01	-4,09E+01
Ecotoxicity, terrestric (TETP)	4,70E+01	4,00E+01	2,83E-01	2,45E-01	2,51E+00	3,74E+00	4,32E-01	8,61E-01	1,67E-01	2,46E-03	-1,25E+00

F.4 Material origin and use

The build-up of the material origin and use for each alternative is shown in Table F. 9 until Table F. 14.

Table F. 9: Material input alternative with new girders

Element	Material	Volume [m³]	Amount	Total [m³]	Mass [kg]	Function	Source
Prefab 25 [m] girder	Concrete C55/67	10,3	15 [girders]	154,50	370800	Construction	Primary
	Prestressing steel Y1860S7	0,152		2,28	5472	Construction	Primary
	Stirrups B500	0,111		1,67	3996	Construction	Primary
Prefab 16,11 [m] girder	Concrete C55/67	6,66	10 [girders]	66,60	159840	Construction	Primary
	Prestressing steel Y1860S7	0,0326		0,33	782	Construction	Primary
	Stirrups B500	0,0759		0,76	1822	Construction	Primary
Deck	Concrete C30/37	0,25	643 [m²]	160,75	385800	Construction	Primary
	Reinforcement B500	0,00538		3,46	27156	Construction	Primary
Crossbeam	Concrete C30/37	2,07	6 [crossbeams]	12,40	29767	Construction	Primary
	Reinforcement B500	0,06978		0,42	3287	Construction	Primary

Table F. 10: Material input alternative 1a with girders with a width of 1200 [mm].

Element	Material	Volume [m³]	Amount	Total [m³]	Mass [kg]	Function	Source
Prefab 22,62 [m] girder	Concrete K600	8,2	15 [girders]	123,00	295200	Construction	Secondary
	Prestressing steel QP190	0,0788		1,18	9279	Construction	Secondary
	Stirrups QRn40	0,0217		0,33	2555	Construction	Secondary
Prefab 18,53 [m] girder	Concrete K600	6,72	10 [girders]	67,20	161280	Construction	Secondary
	Prestressing steel QP190	0,0471		0,47	3697	Construction	Secondary
	Stirrups QRn40	0,0185		0,19	1452	Construction	Secondary
Deck	Concrete C30/37	0,16	643 [m²]	102,88	246912	Construction	Primary
	Reinforcement B500	0,00344		2,21	17364	Construction	Primary
Crossbeam	Concrete C30/37	2,99	6 [crossbeams]	17,92	43008	Construction	Primary
	Reinforcement B500	0,10082		0,60	4749	Construction	Primary

Table F. 11: Material input alternative 1a with girders with a width of 790 [mm].

Element	Material	Volume [m³]	Amount needed	Total [m³]	Mass [kg]	Function	Source
Prefab 22,62 [m] girder	Concrete K600	7,05	24 [girders]	169,20	406080	Construction	Secondary
	Concrete K600	1,15		27,60	66240	Lost	Secondary
	Prestressing steel QP190	0,0788		1,89	14846	Construction	Secondary
	Prestressing steel QP190	0,0171		0,41	3222	Lost	Secondary
	Stirrups QRn40	0,0217		0,52	4088	Construction	Secondary
Prefab 18,53 [m] girder	Concrete K600	5,78	16 [girders]	92,48	221952	Construction	Secondary
	Concrete K600	0,94		15,04	36096	Lost	Secondary
	Prestressing steel QP190	0,0367		0,59	4610	Construction	Secondary
	Prestressing steel QP190	0,0105		0,17	1319	Lost	Secondary
	Stirrups QRn40	0,0185		0,30	2324	Construction	Secondary
Deck	Concrete C30/37	0,16	677 [m²]	108,32	259968	Construction	Primary
	Reinforcement B500	0,00344		2,33	18282	Construction	Primary
Crossbeam	Concrete C30/37	2,31	6 [crossbeams]	13,84	33206	Construction	Primary
	Reinforcement B500	0,07785		0,47	3667	Construction	Primary

Table F. 12: Material input alternative 1d with girders with a width of 1200 [mm].

Element	Material	Volume [m³]	Amount needed	Total [m³]	Mass [kg]	Function	Source
Prefab 21,26 [m] girder	Concrete K600	7,71	25 [girders]	192,75	462600	Construction	Secondary
	Prestressing steel QP190	0,068		1,70	13345	Construction	Secondary
	Stirrups QRn40	0,0206		0,52	4043	Construction	Secondary
Deck	Concrete C30/37	0,16	643 [m²]	102,88	246912	Construction	Primary
	Reinforcement B500	0,00344		2,21	17364	Construction	Primary
Crossbeam	Concrete C30/37	2,99	6 [crossbeams]	17,92	43008	Construction	Primary
	Reinforcement B500	0,10082		0,60	4749	Construction	Primary

Table F. 13: Material input alternative 1d with girders with a width of 1000 [mm].

Element	Material	Volume [m³]	Amount needed	Total [m³]	Mass [kg]	Function	Source
Prefab 21,26 [m] girder	Concrete K600	7,211	30 [girders]	216,33	519192	Construction	Secondary
	Concrete K600	0,499		14,97	35928	Lost	Secondary
	Prestressing steel QP190	0,064		1,92	15072	Construction	Secondary
	Prestressing steel QP190	0,003997		0,12	941	Lost	Secondary
	Stirrups QRn40	0,0217		0,65	5110	Construction	Secondary
Deck	Concrete C30/37	0,16	643 [m²]	102,88	246912	Construction	Primary
	Reinforcement B500	0,00344		2,21	17364	Construction	Primary
Crossbeam	Concrete C30/37	2,76	6 [crossbeams]	16,59	39816	Construction	Primary
	Reinforcement B500	0,09334		0,56	4396	Construction	Primary

Table F. 14: Material input alternative 1d with girders with a width of 790 [mm].

Element	Material	Volume [m³]	Amount needed	Total [m³]	Mass [kg]	Function	Source
Prefab 21,26 [m] girder	Concrete K600	6,517	40 [girders]	260,68	625632	Construction	Secondary
	Concrete K600	1,1929		47,72	114518	Lost	Secondary
	Prestressing steel QP190	0,06		2,40	18840	Construction	Secondary
	Prestressing steel QP190	0,00799		0,32	2509	Lost	Secondary
	Stirrups QRn40	0,0217		0,87	6814	Construction	Secondary
Deck	Concrete C30/37	0,16	638 [m²]	102,08	244992	Construction	Primary
	Reinforcement B500	0,00344		2,19	17229	Construction	Primary
Crossbeam	Concrete C30/37	2,35	6 [crossbeams]	14,09	33806	Construction	Primary
	Reinforcement B500	0,07925		0,48	3733	Construction	Primary

Project Afstudeerproject Meinerseiland Arnhem, TOM liggers

Wijzigingsdatum: 20-6-2023
Peildatum 20-6-2023

Leverancier	S	Omschrijving	Hoevelh.	Eh	Norm U	Uren	Materiaal	Materieel	Onderaannemers	TOTAAL	Prijs/eenh	#
	*	VARIANTENRAMING PREFAB DEKKEN BRUG MEINERSEILAND ARNHEM										
	"	Uitgangspunten:										
	"	- Landhoofden, tussensteunpunten en funderingen zijn voor alle varianten gelijk										
	"	- Randafwerking van de dekken is niet meegenomen: geen separate (eventueel nieuwe) randliggers meegenomen in de oplossingen met hergebruik van bestaande liggers										
	"	- Breedte van de dekken is bij alle oplossingen 6 m1 (mogelijk moet dit wel passend gemaakt worden op liggerbreedtes vrijkomende liggers en eventuele toepassing van randliggers)										
	"	- Kraanopstelplaatsen, randbeveiligingen, bereikbaarheidsvoorzieningen etc. -> voor alle varianten identiek (bij leggen liggers)										
	"	- Buigslappe voegen en voegovergangen -> voor alle varianten identiek										
	"	- Overspanningen wijken van elkaar af, dit i.v.m. verschillende vrijkomende lengtes van de bestaande liggers										
	"	- Alleen de te oogsten liggers worden meegenomen bij de demontage/sloopwerk -> mochten wij de liggers willen hebben, dan draaien wij mogelijk op voor het slopen van álle dekken van het desbetreffende bestaande kunstwerk in de A9										
	"	- Indien we vrij zijn om de h.o.h. afstanden te kiezen, dan is het aan te raden uit kostentechnisch oogpunt om bij de oplossing met de nieuwe prefab liggers voor alle 5 de velden dezelfde overspanning te kiezen										
	,											
		VARIANTENRAMING PREFAB DEKKEN BRUG										
	*	Variant 1a: Hergebruik HIP-800 liggers uit A9 Viaduct Polderweg (over de Holendrechterzijweg)	630,00	m2								#
	"	Overspanningen: 18,53 - 22,62 - 22,62 - 22,62 - 18,53 m1										
	#	Sloopwerk	1,00	ps								

Raming varianten (hergebruikte) liggers

Leverancier	S	Omschrijving	Hoevelh.	Eh	Norm U	Uren	Mate	riaal	Mate	rieel	Onder	aannemers	TOTAAL	Prijs/eenh	#
	+	Sloopwerk	1,00	ps											
		sloopwerk schampkanten	200,00	m1							225,00	45.000	45.000	225,00	
		slopen druklaag	750,00	m2							100,00	75.000	75.000	100,00	
		doorhalen einddwarsdragers/slopen einddwarsdragers	1,00	ps							15.000,00	15.000	15.000	15.000,00	
	,														
		Sloopwerk	1,00	ps							135.000,00	135.000	135.000	135.000,00	
	+	**Meerkosten VKM bij langere wegafsluiting i.v.m. langere slooptijd**	1,00	ps											
	"	*In de A9 verder geen aanvullende maatregelen, anders dan reeds voorzien, met als uitgangspunt dat het sloopwerk nooit op de rode draad van de planning komt*													
	"	*Kosten voor wegafsluiting zouden reeds moeten zijn inbegrepen, wel langere doorlooptijd van de afsluiting*													
		langere huur bebordingen en bebakeningen voor wegafsluiting	5,00	wkn							750,00	3.750	3.750	750,00	
		langere huur materialen voor omleidingsroutes	5,00	wkn							1.000,00	5.000	5.000	1.000,00	
		inzet verkeersregelaars tijdens demontage en afvoer liggers	200,00	uur							35,00	7.000	7.000	35,00	
	,														
		Meerkosten VKM bij langere wegafsluiting i.v.m.	1,00	ps							15.750,00	15.750	15.750	15.750,00	
		Sloopwerk	1,00	ps							150.750,00	150.750	150.750	150.750,00	
	#	Demontage liggers	1,00	ps											
	"	*Maatgevend gewicht is ca. 22 ton (o.b.v. liggerinhoud conform standaard HRP-profiel Haitsma)*													
	"	*Uitgangspunt is inzet van een 500 tons telekraan*													
		/aan-/afvoer 500 tons telekraan	1,00	ps					1.200,00	1.200			1.200	1.200,00	
		inzet 500 tons telekraan, 2 velden verwijderen per dag	24,00	uur					600,00	14.400			14.400	600,00	
		assistentie personeel (5 man)	24,00	uur	5	120,0							7.080	295,00	
		maken kraanopstelplaatsen, 1 per veld	5,00	loc							2.500,00	12.500	12.500	2.500,00	
	,														
		Demontage liggers	1,00	ps	120	120,0			15.600,00	15.600	12.500,00	12.500	35.180	35.180,00	
	#	Handeling en transport liggers	1,00	ps											
	+	Transport naar opslag/onderzoeks/opwerk locatie	1,00	ps											
		transport liggers naar opslag/onderzoeks/opwerk locatie	25,00	st							3.200,00	80.000	80.000	3.200,00	
	"	*Prijs o.b.v. 1 dag een Combex plus begeleiding voor en achter*													
	,														
		Transport naar opslag/onderzoeks/opwerk locatie	1,00	ps							80.000,00	80.000	80.000	80.000,00	
	+	Lossen bij opslag/onderzoeks/opwerk locatie	1,00	ps											
	"	*Maatgevend gewicht is ca. 22 ton (o.b.v. liggerinhoud conform standaard HRP-profiel Haitsma)*													
	"	*Uitgangspunt is inzet van een 200 tons telekraan*													
		aan-/afvoer 200 tons telekraan	1,00	ps					600,00	600			600	600,00	

Raming varianten (hergebruikte) liggers

	Leverancier	S	Omschrijving	Hoevelh.	Eh	Norm U	Uren	Mate riaal		Mate rieel		Onder aannemers		TOTAAL	Prijs/eenh	#
			inzet 200 tons telekraan	16,00	uur					280,00	4.480			4.480	280,00	
			inzet rigger	16,00	uur							50,00	800	800	50,00	
			assistentie	16,00	uur	3	48,0							2.832	177,00	
		,														
			Lossen bij opslag/onderzoeks/opwerk locatie	*1,00*	*ps*	*48*	*48,0*			*5.080,00*	*5.080*	*800,00*	*800*	*8.712*	*8.712,00*	
		+	*Laden bij opslag/onderzoeks/opwerk locatie*	*1,00*	*ps*											
			aan-/afvoer 200 tons telekraan	1,00	ps					600,00	600			600	600,00	
			inzet 200 tons telekraan	16,00	uur					280,00	4.480			4.480	280,00	
			inzet rigger	16,00	uur							50,00	800	800	50,00	
			assistentie	16,00	uur	3	48,0							2.832	177,00	
		,														
			Laden bij opslag/onderzoeks/opwerk locatie	*1,00*	*ps*	*48*	*48,0*			*5.080,00*	*5.080*	*800,00*	*800*	*8.712*	*8.712,00*	
		+	*Transport naar Meinerseiland, Arnhem*	*1,00*	*ps*											
			transport liggers naar Meinerseiland, Arnhem	25,00	st							3.200,00	80.000	80.000	3.200,00	
		,														
			Transport naar Meinerseiland, Arnhem	*1,00*	*ps*							*80.000,00*	*80.000*	*80.000*	*80.000,00*	
			Handeling en transport liggers	*1,00*	*ps*	*96*	*96,0*			*10.160,00*	*10.160*	*161.600,00*	*161.600*	*177.424*	*177.424,00*	
		#	*Opwaarderen bestaande prefab liggers*	*1,00*	*ps*											
			plooien/rechtbuigen bestaande beugelwapening uit liggers	525,00	m1							45,00	23.625	23.625	45,00	#
			schoonspuiten bestaande liggers	525,00	m1	0,15	78,8							4.646	8,85	#
			bijwerken beschadigingen bestaande liggers	525,00	m1	1	525,0	25,00	13.125					44.100	84,00	#
		,														
			Opwaarderen bestaande prefab liggers	*1,00*	*ps*	*603,75*	*603,8*	*13.125,00*	*13.125*			*23.625,00*	*23.625*	*72.371*	*72.371,25*	
		#	*Leggen prefab liggers*	*630,00*	*m2*											#
		"	*Maatgevend gewicht is ca. 22 ton (o.b.v. liggerinhoud conform standaard HRP-profiel Haitsma)*													
		"	*Uitgangspunt is inzet van een 500 tons telekraan*													
			leveren nieuwe oplegblokken	50,00	st			1.250,00	62.500					62.500	1.250,00	
			aan-/afvoer 500 tons telekraan	1,00	ps					1.200,00	1.200			1.200	1.200,00	
			inzet 500 tons telekraan, 2 velden leggen per dag	24,00	uur					600,00	14.400			14.400	600,00	
			assistentie personeel (5 man)	24,00	uur	5	120,0							7.080	295,00	
		,														
			Leggen prefab liggers	*630,00*	*m2*	*0,19*	*120,0*	*99,21*	*62.500*	*24,76*	*15.600*			*85.180*	*135,21*	
		#	*Betonwerk dwarsdragers*	*28,00*	*m3*											#
		"	*Dwarsdragers 1.000 mm breed*													
		+	*Bekisting dwarsdragers*	*96,00*	*m2*											#
241			verloren kist dwarsdragers omgekeerde T-ligger (liggeroppervlak er NIET afgehaald)	96,00	m2	2,4	230,4	25,00	2.400					15.994	166,60	#
249			afwerken beton / bekisting algemeen	96,00	m2	0,15	14,4	5,00	480					1.330	13,85	#
815	kraan		kraanhulp	24,00	uur					105,00	2.520			2.520	105,00	
		,														
			Bekisting dwarsdragers	*96,00*	*m2*	*2,55*	*244,8*	*30,00*	*2.880*	*26,25*	*2.520*			*19.843*	*206,70*	
		+	*Wapening*	*7,36*	*ton*											#
		"	*Uitgangspunt: 250 kg/m3 (incl. laslengtes, stekken en beugels)*													
301	wap		leveren en verwerken betonstaal B500B (liggerinhoud	6938,00	kg							1,20	8.326	8.326	1,20	#

	Leverancier	S	Omschrijving	Hoeveelh.	Eh	Norm U	Uren	Mate	riaal	Mate	rieel	Onder	aannemers	TOTAAL	Prijs/eenh	#
			eraf gehaald)													
301	wap		hulpwapening, 6%	417,00	kg							1,20	500	500	1,20	#
249			hulpvlechter, betonblokjes, ed.	7,36	ton	0,2	1,5	5,00	37					124	16,80	#
815	kraan		kraanhulp	8,00	uur					105,00	840			840	105,00	
		,														
			Wapening	*7,36*	*ton*	*0,2*	*1,5*	*5,00*	*37*	*114,21*	*840*	*1.200,00*	*8.826*	*9.790*	*1.331,01*	
		+	*Beton*	*28,00*	*m3*											#
251	C35/45, XC4, XD3,		leveren beton C35/45, XC4, XD3, XA2, XF4 (consistentie S3), incl. 3% verlies	29,00	m3			135,00	3.915					3.915	135,00	#
251	F4		toeslag consistentie F4	29,00	m3			4,50	131					131	4,50	#
		"	*Storten beton dwarsdragers -> gaat mee met druklaag*													
		"	*Aanvoer betonpomp, 39 - 43 m1 -> gaat mee met druklaag*													
253	Nijwa (raam)		variabele kosten betonpomp (1- 101 m3), 39 - 43 m1	28,00	m3							5,05	141	141	5,05	#
		,														
			Beton	*28,00*	*m3*			*144,48*	*4.046*			*5,05*	*141*	*4.187*	*149,53*	
			Betonwerk dwarsdragers	*28,00*	*m3*	*8,80*	*246,3*	*248,65*	*6.962*	*120,00*	*3.360*	*320,26*	*8.967*	*33.820*	*1.207,85*	
		#	*Betonwerk druklaag*	*177,00*	*m3*											#
		"	*Opgave conform document "Informatie kostenindicatie": dikte 160 mm en betonkwaliteit C30/37 -> echter dikte van 280 mm aangehouden, wat een gebruikelijke dikte is bij nieuwe druklagen evenals een betonkwaliteit van C35/45*													
		+	*Bekisting druklaag*	*562,00*	*m2*											#
241			ondersteunende badding aan zijkanten liggers t.b.v. oplegging bekistingsplaten (sponning voor verloren bekistingsplaten is niet weg te slopen)	1050,00	m1	0,3	315,0	3,00	3.150			15,00	15.750	37.485	35,70	#
241			verloren kist onderzijde dek, omgekeerde T-ligger	499,00	m2	0,4	199,6	25,00	12.475					24.251	48,60	#
241			langskisten en kopkisten druklagen	222,00	m1	0,8	177,6	14,00	3.108					13.586	61,20	#
249			afwerken beton / bekisting algemeen	562,00	m2	0,15	84,3	5,00	2.810					7.784	13,85	#
815	kraan		kraanhulp	48,00	uur					105,00	5.040			5.040	105,00	
		,														
			Bekisting druklaag	*562,00*	*m2*	*1,38*	*776,5*	*38,33*	*21.543*	*8,97*	*5.040*	*28,02*	*15.750*	*88.147*	*156,84*	
		+	*Wapening druklaag dek*	*29,90*	*ton*											#
		"	*Uitgangspunt: 160 kg/m3 = 45 kg/m2 (incl. laslengtes, stekken en beugels)*													
		"	*Uitgangspunt: 315 kg/m3 = 50 kg/m2 (incl. laslengtes, stekken en beugels) -> indien druklaag van 160 mm -> nu 280 mm gerekend*													
301	wap		leveren en verwerken betonstaal B500B	28203,00	kg							1,20	33.844	33.844	1,20	#
301	wap		hulpwapening, 6%	1693,00	kg							1,20	2.032	2.032	1,20	#
249			hulpvlechter, betonblokjes, ed.	29,90	ton	0,2	6,0	5,00	149					502	16,80	#
815	kraan		kraanhulp	32,00	uur					105,00	3.360			3.360	105,00	
		,														
			Wapening druklaag dek	*29,90*	*ton*	*0,2*	*6,0*	*5,00*	*149*	*112,39*	*3.360*	*1.200,00*	*35.875*	*39.737*	*1.329,19*	
		+	*Beton*	*177,00*	*m3*											#
251	C35/45, XC4, XD3,		leveren beton C35/45, XC4, XD3, XA2, XF4	182,00	m3			135,00	24.570					24.570	135,00	#

	Leverancier	S	Omschrijving	Hoevelh.	Eh	Norm	U Uren	Mate riaal		Mate rieel		Onder aannemers		TOTAAL	Prijs/eenh	#
			(consistentie S3), incl. 3% verlies													
251	F4		toeslag consistentie F4	182,00	m3			4,50	819					819	4,50	#
252			storten beton, 1 stort, 6 man 8 uur (incl. dwarsdragers)	177,00	m3	0,35	62,0							3.655	20,65	#
253	Nijwa (raam		aanvoer betonpomp, 39 - 43 m1	1,00	kee							455,00	455	455	455,00	
253	Nijwa (raam		variabele kosten betonpomp (101- 200 m3), 39 - 43 m1	177,00	m3							5,05	894	894	5,05	#
252			vlinderen/afwerken dek	630,00	m2							3,50	2.205	2.205	3,50	#
252			nabehandelen met curing compound	630,00	m2							0,45	284	284	0,45	#
252			afdekken gestort dek	630,00	m2	0,02	12,6	0,40	252					995	1,58	#
		,														
			Beton	*177,00*	*m3*	*0,42*	*74,6*	*144,86*	*25.641*			*21,68*	*3.837*	*33.877*	*191,39*	
		+	*In te storten onderdelen*													
		"	*Niet meegenomen in deze TOM*													
		,														
			In te storten onderdelen													
			Betonwerk druklaag	*177,00*	*m3*	*4,84*	*857,0*	*267,42*	*47.333*	*47,46*	*8.400*	*313,35*	*55.463*	*161.761*	*913,90*	
		#	*Onderzoekskosten*	*1,00*	*ps*											
			onderzoekskosten/herberekeningen bestaande liggers	1,00	ps							75.000,00	75.000	75.000	75.000,00	
		,														
			Onderzoekskosten	*1,00*	*ps*							*75.000,00*	*75.000*	*75.000*	*75.000,00*	
			Variant 1a: Hergebruik HIP-800 liggers uit A9 Viaduct	*630,00*	*m2*	*3,24*	*2.043,1*	*206,22*	*129.921*	*84,32*	*53.120*	*774,45*	*487.905*	*791.486*	*1.256,33*	
		*	*Variant 1a-2: Hergebruik HIP-800 liggers uit A9 Viaduct Polderweg (over de Holendrechterzijweg), incl. doorhalen liggers*	*630,00*	*m2*											#
		?	MET TOEPASSING VAN DOORGEHAALDE LIGGERS ZULLEN WE WAARSCHIJNLIJK NIET KUNNEN VOLDOEN AAN 100-JARIGE LEVENSDUUR I.V.M. ONVOLDOENDE/GEEN DEKKING OP DE BESTAANDE WAPENING VAN DE GEOOGSTE PREFAB LIGGERS													
		"	*Overspanningen: 18,53 - 22,62 - 22,62 - 22,62 - 18,53 m1*													
		#	*Sloopwerk*	*1,00*	*ps*											
		+	*Sloopwerk*	*1,00*	*ps*											
		"	*15 liggers extra bij deze variant*													
			sloopwerk schampkanten	200,00	m1							225,00	45.000	45.000	225,00	
			slopen druklaag	1150,00	m2							100,00	115.000	115.000	100,00	
			doorhalen einddwarsdragers/slopen einddwarsdragers	1,00	ps							25.000,00	25.000	25.000	25.000,00	
		,														
			Sloopwerk	*1,00*	*ps*							*185.000,00*	*185.000*	*185.000*	*185.000,00*	
		+	***Meerkosten VKM bij langere wegafsluiting i.v.m. langere slooptijd***	*1,00*	*ps*											
		"	*In de A9 verder geen aanvullende maatregelen, anders dan reeds voorzien, met als uitgangspunt dat het sloopwerk nooit op de rode draad van de planning komt*													

Raming varianten (hergebruikte) liggers

Leverancier	S	Omschrijving	Hoevelh.	Eh	Norm U	Uren	Mate riaal		Mate rieel		Onder aannemers		TOTAAL	Prijs/eenh	#
	"	*Kosten voor wegafsluiting zouden reeds moeten zijn inbegrepen, wel langere doorlooptijd van de afsluiting*													
		langere huur bebordingen en bebakeningen voor wegafsluiting	7,00	wkn							750,00	5.250	5.250	750,00	
		langere huur materialen voor omleidingsroutes	7,00	wkn							1.000,00	7.000	7.000	1.000,00	
		inzet verkeersregelaars tijdens demontage en afvoer liggers	280,00	uur							35,00	9.800	9.800	35,00	
	,														
		Meerkosten VKM bij langere wegafsluiting i.v.m.	1,00	ps							22.050,00	22.050	22.050	22.050,00	
		Sloopwerk	1,00	ps							207.050,00	207.050	207.050	207.050,00	
	#	*Demontage liggers*	1,00	ps											
	"	*Maatgevend gewicht is ca. 22 ton (o.b.v. liggerinhoud conform standaard HRP-profiel Haitsma)*													
	"	*Uitgangspunt is inzet van een 500 tons telekraan*													
		/aan-/afvoer 500 tons telekraan	1,00	ps					1.200,00	1.200			1.200	1.200,00	
		inzet 500 tons telekraan, 2 velden verwijderen per dag	40,00	uur					600,00	24.000			24.000	600,00	
		assistentie personeel (5 man)	40,00	uur	5	200,0							11.800	295,00	
		maken kraanopstelplaatsen, 1 per veld	5,00	loc							2.500,00	12.500	12.500	2.500,00	
	,														
		Demontage liggers	1,00	ps	200	200,0			25.200,00	25.200	12.500,00	12.500	49.500	49.500,00	
	#	*Handeling en transport liggers*	1,00	ps											
	+	*Transport naar opslag/onderzoeks/opwerk locatie*	1,00	ps											
		transport liggers naar opslag/onderzoeks/opwerk locatie	40,00	st							3.200,00	128.000	128.000	3.200,00	
	"	*Prijs o.b.v. 1 dag een Combex plus begeleiding voor en achter*													
	,														
		Transport naar opslag/onderzoeks/opwerk locatie	1,00	ps							128.000,00	128.000	128.000	128.000,00	
	+	*Lossen bij opslag/onderzoeks/opwerk locatie*	1,00	ps											
	"	*Maatgevend gewicht is ca. 22 ton (o.b.v. liggerinhoud conform standaard HRP-profiel Haitsma)*													
	"	*Uitgangspunt is inzet van een 200 tons telekraan*													
		aan-/afvoer 200 tons telekraan	1,00	ps					600,00	600			600	600,00	
		inzet 200 tons telekraan	24,00	uur					280,00	6.720			6.720	280,00	
		inzet rigger	24,00	uur							50,00	1.200	1.200	50,00	
		assistentie	24,00	uur	3	72,0							4.248	177,00	
	,														
		Lossen bij opslag/onderzoeks/opwerk locatie	1,00	ps	72	72,0			7.320,00	7.320	1.200,00	1.200	12.768	12.768,00	
	+	*Laden bij opslag/onderzoeks/opwerk locatie*	1,00	ps											
		aan-/afvoer 200 tons telekraan	1,00	ps					600,00	600			600	600,00	
		inzet 200 tons telekraan	24,00	uur					280,00	6.720			6.720	280,00	
		inzet rigger	24,00	uur							50,00	1.200	1.200	50,00	
		assistentie	24,00	uur	3	72,0							4.248	177,00	
	,														
		Laden bij opslag/onderzoeks/opwerk locatie	1,00	ps	72	72,0			7.320,00	7.320	1.200,00	1.200	12.768	12.768,00	

Raming varianten (hergebruikte) liggers

	Leverancier	S	Omschrijving	Hoevelh.	Eh	Norm U	Uren	Mate	riaal	Mate	rieel	Onder	aannemers	TOTAAL	Prijs/eenh	#
		+	Transport naar Meinerseiland, Arnhem	1,00	ps											
			transport liggers naar Meinerseiland, Arnhem	40,00	st							3.200,00	128.000	128.000	3.200,00	
		,														
			Transport naar Meinerseiland, Arnhem	1,00	ps							128.000,00	128.000	128.000	128.000,00	
			Handeling en transport liggers	1,00	ps	144	144,0			14.640,00	14.640	258.400,00	258.400	281.536	281.536,00	
		#	Opwaarderen bestaande prefab liggers	1,00	ps											
			plooien/rechtbuigen bestaande beugelwapening uit liggers	840,00	m1							45,00	37.800	37.800	45,00	#
			schoonspuiten bestaande liggers	840,00	m1	0,15	126,0							7.434	8,85	#
			bijwerken beschadigingen bestaande liggers	840,00	m1	1	840,0	25,00	21.000					70.560	84,00	#
		,														
			Opwaarderen bestaande prefab liggers	1,00	ps	966	966,0	21.000,00	21.000			37.800,00	37.800	115.794	115.794,00	
		#	Doorhalen liggers en dekkingsherstel wapening	1,00	ps											
		"	*In lengterichting worden de liggers doorgezaagd*													
			doorzagen liggers, 2 -zijdig, 40 stuks liggers	1679,00	m1							81,25	136.419	136.419	81,25	#
			opruimen betonpuin	242,00	ton							5,00	1.210	1.210	5,00	#
			afvoeren betonpuin	242,00	ton							15,00	3.630	3.630	15,00	#
			acceptatiekosten betonpuin	242,00	ton							7,00	1.694	1.694	7,00	#
			leveren en aanbrengen dekkingsherstellende mortel op vrijliggende wapening t.p.v. zaagsnede	1679,00	m1							40,00	67.160	67.160	40,00	#
		,														
			Doorhalen liggers en dekkingsherstel wapening	1,00	ps							210.112,75	210.113	210.113	210.112,75	
		#	Leggen prefab liggers	630,00	m2											#
		"	*Maatgevend gewicht is ca. 22 ton (o.b.v. liggerinhoud conform standaard HRP-profiel Haitsma)*													
		"	*Uitgangspunt is inzet van een 500 tons telekraan*													
			leveren nieuwe oplegblokken	80,00	st			1.250,00	100.000					100.000	1.250,00	
			aan-/afvoer 500 tons telekraan	1,00	ps					1.200,00	1.200			1.200	1.200,00	
			inzet 500 tons telekraan, 1 veld leggen per dag	40,00	uur					600,00	24.000			24.000	600,00	
			assistentie personeel (5 man)	40,00	uur	5	200,0							11.800	295,00	
		,														
			Leggen prefab liggers	630,00	m2	0,32	200,0	158,73	100.000	40,00	25.200			137.000	217,46	
		#	Betonwerk dwarsdragers	28,00	m3											#
		"	*Dwarsdragers 1.000 mm breed*													
		+	Bekisting dwarsdragers	96,00	m2											#
241			verloren kist dwarsdragers omgekeerde T-ligger (liggeroppervlak er NIET afgehaald)	96,00	m2	2,4	230,4	25,00	2.400					15.994	166,60	#
249			afwerken beton / bekisting algemeen	96,00	m2	0,15	14,4	5,00	480					1.330	13,85	#
815	kraan		kraanhulp	24,00	uur					105,00	2.520			2.520	105,00	
		,														
			Bekisting dwarsdragers	96,00	m2	2,55	244,8	30,00	2.880	26,25	2.520			19.843	206,70	
		+	Wapening	7,36	ton											#
		"	*Uitgangspunt: 250 kg/m3 (incl. laslengtes, stekken en beugels)*													
301	wap		leveren en verwerken betonstaal B500B (liggerinhoud eraf gehaald)	6938,00	kg							1,20	8.326	8.326	1,20	#
301	wap		hulpwapening, 6%	417,00	kg							1,20	500	500	1,20	#

	Leverancier	S	Omschrijving	Hoevelh.	Eh	Norm U	Uren	Mate	riaal	Mate	rieel	Onder	aannemers	TOTAAL	Prijs/eenh	#
249			hulpvlechter, betonblokjes, ed.	7,36	ton	0,2	1,5	5,00	37					124	16,80	#
815	kraan		kraanhulp	8,00	uur					105,00	840			840	105,00	
		,														
			Wapening	7,36	ton	0,2	1,5	5,00	37	114,21	840	1.200,00	8.826	9.790	1.331,01	
		+	Beton	28,00	m3											#
251	C35/45, XC4, XD3,		leveren beton C35/45, XC4, XD3, XA2, XF4 (consistentie S3), incl. 3% verlies	29,00	m3			135,00	3.915					3.915	135,00	#
251	F4		toeslag consistentie F4	29,00	m3			4,50	131					131	4,50	#
		"	*Storten beton dwarsdragers -> gaat mee met druklaag*													
		"	*Aanvoer betonpomp, 39 - 43 m1 -> gaat mee met druklaag*													
253	Nijwa (raam)		variabele kosten betonpomp (1- 101 m3), 39 - 43 m1	28,00	m3							5,05	141	141	5,05	#
		,														
			Beton	28,00	m3			144,48	4.046			5,05	141	4.187	149,53	
			Betonwerk dwarsdragers	28,00	m3	8,80	246,3	248,65	6.962	120,00	3.360	320,26	8.967	33.820	1.207,85	
		#	Betonwerk druklaag	177,00	m3											#
		"	*Opgave conform document "Informatie kostenindicatie": dikte 160 mm en betonkwaliteit C30/37 -> echter dikte van 280 mm aangehouden, wat een gebruikelijke dikte is bij nieuwe druklagen evenals een betonkwaliteit van C35/45*													
		+	Bekisting druklaag	483,00	m2											#
241			ondersteunende badding aan zijkanten liggers t.b.v. oplegging bekistingsplaten (sponning voor verloren bekistingsplaten is niet weg te slopen)	1679,00	m1	0,3	503,7	3,00	5.037			15,00	25.185	59.940	35,70	#
241			verloren kist onderzijde dek, omgekeerde T-ligger	420,00	m2	0,4	168,0	25,00	10.500					20.412	48,60	#
241			langskisten en kopkisten druklagen	222,00	m1	0,8	177,6	14,00	3.108					13.586	61,20	#
249			afwerken beton / bekisting algemeen	483,00	m2	0,15	72,5	5,00	2.415					6.690	13,85	#
815	kraan		kraanhulp	92,00	uur					105,00	9.660			9.660	105,00	
		,														
			Bekisting druklaag	483,00	m2	1,91	921,8	43,60	21.060	20,00	9.660	52,14	25.185	110.288	228,34	
		+	Wapening druklaag dek	29,90	ton											#
		"	*Uitgangspunt: 160 kg/m3 = 45 kg/m2 (incl. laslengtes, stekken en beugels)*													
		"	*Uitgangspunt: 315 kg/m3 = 50 kg/m2 (incl. laslengtes, stekken en beugels) -> indien druklaag van 160 mm -> nu 280 mm gerekend*													
301	wap		leveren en verwerken betonstaal B500B	28203,00	kg							1,20	33.844	33.844	1,20	#
301	wap		hulpwapening, 6%	1693,00	kg							1,20	2.032	2.032	1,20	#
249			hulpvlechter, betonblokjes, ed.	29,90	ton	0,2	6,0	5,00	149					502	16,80	#
815	kraan		kraanhulp	32,00	uur					105,00	3.360			3.360	105,00	
		,														
			Wapening druklaag dek	29,90	ton	0,2	6,0	5,00	149	112,39	3.360	1.200,00	35.875	39.737	1.329,19	
		+	Beton	177,00	m3											#
251	C35/45, XC4, XD3,		leveren beton C35/45, XC4, XD3, XA2, XF4 (consistentie S3), incl. 3% verlies	182,00	m3			135,00	24.570					24.570	135,00	#
251	F4		toeslag consistentie F4	182,00	m3			4,50	819					819	4,50	#

	Leverancier	S	Omschrijving	Hoevelh.	Eh	Norm U	Uren	Mate riaal		Mate rieel		Onder aannemers		TOTAAL	Prijs/eenh	#
252			storten beton, 1 stort, 6 man 8 uur (incl. dwarsdragers)	177,00	m3	0,35	62,0							3.655	20,65	#
253	Nijwa (raam		aanvoer betonpomp, 39 - 43 m1	1,00	kee							455,00	455	455	455,00	
253	Nijwa (raam		variabele kosten betonpomp (101- 200 m3), 39 - 43 m1	177,00	m3							5,05	894	894	5,05	#
252			vlinderen/afwerken dek	630,00	m2							3,50	2.205	2.205	3,50	#
252			nabehandelen met curing compound	630,00	m2							0,45	284	284	0,45	#
252			afdekken gestort dek	630,00	m2	0,02	12,6	0,40	252					995	1,58	#
		,														
			Beton	177,00	m3	0,42	74,6	144,86	25.641			21,68	3.837	33.877	191,39	
		+	*In te storten onderdelen*													
		"	*Niet meegenomen in deze TOM*													
		,														
			In te storten onderdelen													
			Betonwerk druklaag	177,00	m3	5,66	1.002,3	264,69	46.850	73,56	13.020	366,65	64.898	183.903	1.039,00	
		#	*Onderzoekskosten*	1,00	ps											
			onderzoekskosten/herberekeningen bestaande liggers	1,00	ps							75.000,00	75.000	75.000	75.000,00	
		,														
			Onderzoekskosten	1,00	ps							75.000,00	75.000	75.000	75.000,00	
			Variant 1a-2: Hergebruik HIP-800 liggers uit A9	630,00	m2	4,38	2.758,6	277,48	174.813	129,24	81.420	1.388,46	874.728	1.293.715	2.053,52	
		*	*Variant 1d: Hergebruik HIP-800 liggers uit A9 Viaduct Amstelplein (over de N522)*	645,00	m2											#
		"	*Overspanningen: 21,5 - 21,5 - 21,5 - 21,5 - 21,5 m1*													
		#	*Sloopwerk*	1,00	ps											
		+	*Sloopwerk*	1,00	ps											
			sloopwerk schampkanten	200,00	m1							225,00	45.000	45.000	225,00	
			slopen druklaag	750,00	m2							100,00	75.000	75.000	100,00	
			doorhalen einddwarsdragers/slopen einddwarsdragers	1,00	ps							15.000,00	15.000	15.000	15.000,00	
			nacht- en weekendtoeslagen sloopwerk incl. productieverlies bij afhankelijkheid met weekendafsluitingen	1,00	ps							50.000,00	50.000	50.000	50.000,00	
		,														
			Sloopwerk	1,00	ps							185.000,00	185.000	185.000	185.000,00	
		+	***Meerkosten VKM bij langere wegafsluiting i.v.m. langere slooptijd***	1,00	ps											
		"	*In de A9 verder geen aanvullende maatregelen, anders dan reeds voorzien, met als uitgangspunt dat het sloopwerk nooit op de rode draad van de planning komt*													
		"	*Hier zijn wel extra weekendafsluitingen nodig voor de toe- en afritten van de A9 naar de N522, daarmee ook meer sloopwerk in nachten en weekenden*													
			weekendafsluitingen, incl. omleidingsroutes bij knooppunten Holendrecht en Badhoevedorp en N522 (referentie	3,00	st							50.000,00	150.000	150.000	50.000,00	
			in-/uitschakelen VRI-installaties	3,00	kee							450,00	1.350	1.350	450,00	

Raming varianten (hergebruikte) liggers

Leverancier	S	Omschrijving	Hoevelh.	Eh	Norm	U	Uren	Mate	riaal	Mate	rieel	Onder	aannemers	TOTAAL	Prijs/eenh	#
		langere huur bebordingen en bebakeningen voor wegafsluiting	10,00	wkn								1.500,00	15.000	15.000	1.500,00	
		langere huur materialen voor omleidingsroutes	10,00	wkn								2.000,00	20.000	20.000	2.000,00	
		inzet verkeersregelaars tijdens demontage en afvoer liggers	360,00	uur								35,00	12.600	12.600	35,00	
	,															
		Meerkosten VKM bij langere wegafsluiting i.v.m.	*1,00*	*ps*								*198.950,00*	*198.950*	*198.950*	*198.950,00*	
		Sloopwerk	*1,00*	*ps*								*383.950,00*	*383.950*	*383.950*	*383.950,00*	
	#	*Demontage liggers*	*1,00*	*ps*												
	"	*Maatgeven gewicht is ca. 21 ton (o.b.v. liggerinhoud conform standaard HRP-profiel Haitsma)*														
	"	*Uitgangspunt is inzet van een 500 tons telekraan*														
	"	*Meerdere keren mobiliseren en demobiliseren i.v.m. oogsten tijdens weekendafsluitingen*														
		aan-/afvoer 500 tons telekraan	3,00	kee						1.200,00	3.600			3.600	1.200,00	
		inzet 500 tons telekraan, 1 veld verwijderen per dag (terugloop i.v.m. weekendafsluitingen)	24,00	uur						600,00	14.400			14.400	600,00	
		assistentie personeel (5 man)	24,00	uur	5		120,0							7.080	295,00	
		nacht- en weekendtoeslagen demontage personeel	144,00	uur								15,00	2.160	2.160	15,00	
		maken kraanopstelplaatsen, 1 per veld	5,00	loc								2.500,00	12.500	12.500	2.500,00	
	,															
		Demontage liggers	*1,00*	*ps*	*120*		*120,0*			*18.000,00*	*18.000*	*14.660,00*	*14.660*	*39.740*	*39.740,00*	
	#	*Handeling en transport liggers*	*1,00*	*ps*												
	+	*Transport naar opslag/onderzoeks/opwerk locatie*	*1,00*	*ps*												
		transport liggers naar opslag/onderzoeks/opwerk locatie	25,00	st								3.200,00	80.000	80.000	3.200,00	
		extra inzet combexen i.v.m. beperkte productie/meerdere weekenden	1,00	ps								40.000,00	40.000	40.000	40.000,00	
	"	*Prijs o.b.v. 1 dag een Combex plus begeleiding voor en achter*														
	,															
		Transport naar opslag/onderzoeks/opwerk locatie	*1,00*	*ps*								*120.000,00*	*120.000*	*120.000*	*120.000,00*	
	+	*Lossen bij opslag/onderzoeks/opwerk locatie*	*1,00*	*ps*												
	"	*Maatgeven gewicht is ca. 21 ton (o.b.v. liggerinhoud conform standaard HRP-profiel Haitsma)*														
	"	*Uitgangspunt is inzet van een 200 tons telekraan*														
		aan-/afvoer 200 tons telekraan	3,00	kee						600,00	1.800			1.800	600,00	
		inzet 200 tons telekraan	24,00	uur						280,00	6.720			6.720	280,00	
		inzet rigger	24,00	uur								50,00	1.200	1.200	50,00	
		assistentie	24,00	uur	3		72,0							4.248	177,00	
		nacht- en weekendtoeslagen lossen personeel	120,00	uur								15,00	1.800	1.800	15,00	
	,															
		Lossen bij opslag/onderzoeks/opwerk locatie	*1,00*	*ps*	*72*		*72,0*			*8.520,00*	*8.520*	*3.000,00*	*3.000*	*15.768*	*15.768,00*	
	+	*Laden bij opslag/onderzoeks/opwerk locatie*	*1,00*	*ps*												
		aan-/afvoer 200 tons telekraan	1,00	ps						600,00	600			600	600,00	
		inzet 200 tons telekraan	16,00	uur						280,00	4.480			4.480	280,00	
		inzet rigger	16,00	uur								50,00	800	800	50,00	
		assistentie	16,00	uur	3		48,0							2.832	177,00	

Raming varianten (hergebruikte) liggers

	Leverancier	S	Omschrijving	Hoeveelh.	Eh	Norm U	Uren	Mate	riaal	Mate	rieel	Onder	aannemers	TOTAAL	Prijs/eenh	#
			Laden bij opslag/onderzoeks/opwerk locatie	1,00	ps	48	48,0			5.080,00	5.080	800,00	800	8.712	8.712,00	
		+	Transport naar Meinerseiland, Arnhem	1,00	ps											
			transport liggers naar Meinerseiland, Arnhem	25,00	st							3.200,00	80.000	80.000	3.200,00	
			Transport naar Meinerseiland, Arnhem	1,00	ps							80.000,00	80.000	80.000	80.000,00	
			Handeling en transport liggers	1,00	ps	120	120,0			13.600,00	13.600	203.800,00	203.800	224.480	224.480,00	
		#	Opwaarderen bestaande prefab liggers	1,00	ps											
			plooien/rechtbuigen bestaande beugelwapening uit liggers	538,00	m1							45,00	24.210	24.210	45,00	#
			schoonspuiten bestaande liggers	538,00	m1	0,15	80,7							4.761	8,85	#
			bijwerken beschadigingen bestaande liggers	538,00	m1	1	538,0	25,00	13.450					45.192	84,00	#
			Opwaarderen bestaande prefab liggers	1,00	ps	618,7	618,7	13.450,00	13.450			24.210,00	24.210	74.163	74.163,30	
		#	Leggen prefab liggers	645,00	m2											#
		"	*Maatgeven gewicht is ca. 21 ton (o.b.v. liggerinhoud conform standaard HRP-profiel Haitsma)*													
		"	*Uitgangspunt is inzet van een 500 tons telekraan*													
			leveren nieuwe oplegblokken	50,00	st			1.250,00	62.500					62.500	1.250,00	
			aan-/afvoer 500 tons telekraan	1,00	ps					1.200,00	1.200			1.200	1.200,00	
			inzet 500 tons telekraan, 2 velden leggen per dag	24,00	uur					600,00	14.400			14.400	600,00	
			assistentie personeel (5 man)	24,00	uur	5	120,0							7.080	295,00	
			Leggen prefab liggers	645,00	m2	0,19	120,0	96,90	62.500	24,19	15.600			85.180	132,06	
		#	Betonwerk dwarsdragers	28,00	m3											#
		"	*Dwarsdragers 1.000 mm breed*													
		+	Bekisting dwarsdragers	96,00	m2											#
241			verloren kist dwarsdragers omgekeerde T-ligger (liggeroppervlak er NIET afgehaald)	96,00	m2	2,4	230,4	25,00	2.400					15.994	166,60	#
249			afwerken beton / bekisting algemeen	96,00	m2	0,15	14,4	5,00	480					1.330	13,85	#
815	kraan		kraanhulp	24,00	uur					105,00	2.520			2.520	105,00	
			Bekisting dwarsdragers	96,00	m2	2,55	244,8	30,00	2.880	26,25	2.520			19.843	206,70	
		+	Wapening	7,36	ton											#
		"	*Uitgangspunt: 250 kg/m3 (incl. laslengtes, stekken en beugels)*													
301	wap		leveren en verwerken betonstaal B500B (liggerinhoud eraf gehaald)	6938,00	kg							1,20	8.326	8.326	1,20	#
301	wap		hulpwapening, 6%	417,00	kg							1,20	500	500	1,20	#
249			hulpvlechter, betonblokjes, ed.	7,36	ton	0,2	1,5	5,00	37					124	16,80	#
815	kraan		kraanhulp	8,00	uur					105,00	840			840	105,00	
			Wapening	7,36	ton	0,2	1,5	5,00	37	114,21	840	1.200,00	8.826	9.790	1.331,01	
		+	Beton	28,00	m3											#
251	C35/45, XC4, XD3,		leveren beton C35/45, XC4, XD3, XA2, XF4 (consistentie S3), incl. 3% verlies	29,00	m3			135,00	3.915					3.915	135,00	#
251	F4		toeslag consistentie F4	29,00	m3			4,50	131					131	4,50	#

Raming varianten (hergebruikte) liggers

	Leverancier	S	Omschrijving	Hoevelh.	Eh	Norm U	Uren	Mate	riaal	Mate	rieel	Onder	aannemers	TOTAAL	Prijs/eenh	#
		"	Storten beton dwarsdragers -> gaat mee met druklaag													
		"	Aanvoer betonpomp, 39 - 43 m1 -> gaat mee met druklaag													
253	Nijwa (raam)		variabele kosten betonpomp (1- 101 m3), 39 - 43 m1	28,00	m3							5,05	141	141	5,05	#
		,														
			Beton	28,00	m3			144,48	4.046			5,05	141	4.187	149,53	
			Betonwerk dwarsdragers	28,00	m3	8,80	246,3	248,65	6.962	120,00	3.360	320,26	8.967	33.820	1.207,85	
		#	Betonwerk druklaag	181,00	m3											#
		"	Opgave conform document "Informatie kostenindicatie": dikte 160 mm en betonkwaliteit C30/37 -> echter dikte van 280 mm aangehouden, wat een gebruikelijke dikte is bij nieuwe druklagen evenals een betonkwaliteit van C35/45													
		+	Bekisting druklaag	575,00	m2											#
241			ondersteunende badding aan zijkanten liggers t.b.v. oplegging bekistingsplaten (sponning voor verloren bekistingsplaten is niet weg te slopen)	1075,00	m1	0,3	322,5	3,00	3.225			15,00	16.125	38.378	35,70	#
241			verloren kist onderzijde dek, omgekeerde T-ligger	511,00	m2	0,4	204,4	25,00	12.775					24.835	48,60	#
241			langskisten en kopkisten druklagen	227,00	m1	0,8	181,6	14,00	3.178					13.892	61,20	#
249			afwerken beton / bekisting algemeen	575,00	m2	0,15	86,3	5,00	2.875					7.964	13,85	#
815	kraan		kraanhulp	48,00	uur					105,00	5.040			5.040	105,00	
		,														
			Bekisting druklaag	575,00	m2	1,38	794,8	38,35	22.053	8,77	5.040	28,04	16.125	90.108	156,71	
		+	Wapening druklaag dek	30,63	ton											#
		"	Uitgangspunt: 160 kg/m3 = 45 kg/m2 (incl. laslengtes, stekken en beugels)													
		"	Uitgangspunt: 315 kg/m3 = 50 kg/m2 (incl. laslengtes, stekken en beugels) -> indien druklaag van 160 mm -> nu 280 mm gerekend													
301	wap		leveren en verwerken betonstaal B500B	28896,00	kg							1,20	34.675	34.675	1,20	#
301	wap		hulpwapening, 6%	1734,00	kg							1,20	2.081	2.081	1,20	#
249			hulpvlechter, betonblokjes, ed.	30,63	ton	0,2	6,1	5,00	153					515	16,80	#
815	kraan		kraanhulp	32,00	uur					105,00	3.360			3.360	105,00	
		,														
			Wapening druklaag dek	30,63	ton	0,2	6,1	5,00	153	109,70	3.360	1.200,00	36.756	40.631	1.326,50	
		+	Beton	181,00	m3											#
251	C35/45, XC4, XD3,		leveren beton C35/45, XC4, XD3, XA2, XF4 (consistentie S3), incl. 3% verlies	187,00	m3			135,00	25.245					25.245	135,00	#
251	F4		toeslag consistentie F4	187,00	m3			4,50	842					842	4,50	#
252			storten beton, 1 stort, 6 man 8 uur (incl. dwarsdragers)	181,00	m3	0,35	63,4							3.738	20,65	#
253	Nijwa (raam		aanvoer betonpomp, 39 - 43 m1	1,00	kee							455,00	455	455	455,00	
253	Nijwa (raam		variabele kosten betonpomp (101- 200 m3), 39 - 43 m1	181,00	m3							5,05	914	914	5,05	#
252			vlinderen/afwerken dek	645,00	m2							3,50	2.258	2.258	3,50	#
252			nabehandelen met curing compound	645,00	m2							0,45	290	290	0,45	#
252			afdekken gestort dek	645,00	m2	0,02	12,9	0,40	258					1.019	1,58	#

Leverancier	S	Omschrijving	Hoevelh.	Eh	Norm U	Uren	Mate	riaal	Mate	rieel	Onder	aannemers	TOTAAL	Prijs/eenh	#
	,														
		Beton	181,00	m3	0,42	76,3	145,55	26.345			21,64	3.917	34.760	192,04	
	+	In te storten onderdelen													
	"	Niet meegenomen in deze TOM													
	,														
		In te storten onderdelen													
		Betonwerk druklaag	181,00	m3	4,85	877,1	268,24	48.551	46,41	8.400	313,80	56.798	165.499	914,36	
	#	Onderzoekskosten	1,00	ps											
		onderzoekskosten/herberekeningen bestaande liggers	1,00	ps							75.000,00	75.000	75.000	75.000,00	
	,														
		Onderzoekskosten	1,00	ps							75.000,00	75.000	75.000	75.000,00	
		Variant 1d: Hergebruik HIP-800 liggers uit A9 Viaduct	645,00	m2	3,26	2.102,1	203,82	131.463	91,41	58.960	1.189,74	767.385	1.081.832	1.677,26	
	*	Variant 1d: Hergebruik HIP-800 liggers uit A9 Viaduct Amstelplein (over de N522), incl. doorhalen liggers (tot 1 m1 breedte)	645,00	m2											#
	"	Overspanningen: 21,5 - 21,5 - 21,5 - 21,5 - 21,5 m1													
	#	Sloopwerk	1,00	ps											
	+	Sloopwerk	1,00	ps											
		sloopwerk schampkanten	200,00	m1							225,00	45.000	45.000	225,00	
		slopen druklaag	900,00	m2							100,00	90.000	90.000	100,00	
		doorhalen einddwarsdragers/slopen einddwarsdragers	1,00	ps							25.000,00	25.000	25.000	25.000,00	
		nacht- en weekendtoeslagen sloopwerk incl. productieverlies bij afhankelijkheid met weekendafsluitingen	1,00	ps							65.000,00	65.000	65.000	65.000,00	
	,														
		Sloopwerk	1,00	ps							225.000,00	225.000	225.000	225.000,00	
	+	**Meerkosten VKM bij langere wegafsluiting i.v.m. langere slooptijd**	1,00	ps											
	"	In de A9 verder geen aanvullende maatregelen, anders dan reeds voorzien, met als uitgangspunt dat het sloopwerk nooit op de rode draad van de planning komt													
	"	Hier zijn wel extra weekendafsluitingen nodig voor de toe- en afritten van de A9 naar de N522, daarmee ook meer sloopwerk in nachten en weekenden													
		weekendafsluitingen, incl. omleidingsroutes bij knooppunten Holendrecht en Badhoevedorp en N522 (referentie	4,00	st							50.000,00	200.000	200.000	50.000,00	
		in-/uitschakelen VRI-installaties	4,00	kee							450,00	1.800	1.800	450,00	
		langere huur bebordingen en bebakeningen voor wegafsluiting	14,00	wkn							1.500,00	21.000	21.000	1.500,00	
		langere huur materialen voor omleidingsroutes	14,00	wkn							2.000,00	28.000	28.000	2.000,00	
		inzet verkeersregelaars tijdens demontage en afvoer liggers	400,00	uur							35,00	14.000	14.000	35,00	
	,														
		Meerkosten VKM bij langere wegafsluiting i.v.m.	1,00	ps							264.800,00	264.800	264.800	264.800,00	

Raming varianten (hergebruikte) liggers

Leverancier	S	Omschrijving	Hoevelh.	Eh	Norm U	Uren	Materiaal	Materieel		Onderaannemers		TOTAAL	Prijs/eenh	#
		Sloopwerk	1,00	ps						489.800,00	489.800	489.800	489.800,00	
	#	Demontage liggers	1,00	ps										
	"	Maatgeven gewicht is ca. 21 ton (o.b.v. liggerinhoud conform standaard HRP-profiel Haitsma)												
	"	Uitgangspunt is inzet van een 500 tons telekraan												
	"	Meerdere keren mobiliseren en demobiliseren i.v.m. oogsten tijdens weekendafsluitingen												
		aan-/afvoer 500 tons telekraan	4,00	kee				1.200,00	4.800			4.800	1.200,00	
		inzet 500 tons telekraan, 1 veld verwijderen per dag (terugloop i.v.m. weekendafsluitingen)	32,00	uur				600,00	19.200			19.200	600,00	
		assistentie personeel (5 man)	32,00	uur	5	160,0						9.440	295,00	
		nacht- en weekendtoeslagen demontage personeel	192,00	uur						15,00	2.880	2.880	15,00	
		maken kraanopstelplaatsen, 1 per veld	5,00	loc						2.500,00	12.500	12.500	2.500,00	
	,													
		Demontage liggers	1,00	ps	160	160,0		24.000,00	24.000	15.380,00	15.380	48.820	48.820,00	
	#	Handeling en transport liggers	1,00	ps										
	+	Transport naar opslag/onderzoeks/opwerk locatie	1,00	ps										
		transport liggers naar opslag/onderzoeks/opwerk locatie	30,00	st						3.200,00	96.000	96.000	3.200,00	
		extra inzet combexen i.v.m. beperkte productie/meerdere weekenden	1,00	ps						50.000,00	50.000	50.000	50.000,00	
	"	Prijs o.b.v. 1 dag een Combex plus begeleiding voor en achter												
	,													
		Transport naar opslag/onderzoeks/opwerk locatie	1,00	ps						146.000,00	146.000	146.000	146.000,00	
	+	Lossen bij opslag/onderzoeks/opwerk locatie	1,00	ps										
	"	Maatgeven gewicht is ca. 21 ton (o.b.v. liggerinhoud conform standaard HRP-profiel Haitsma)												
	"	Uitgangspunt is inzet van een 200 tons telekraan												
		aan-/afvoer 200 tons telekraan	4,00	kee				600,00	2.400			2.400	600,00	
		inzet 200 tons telekraan	32,00	uur				280,00	8.960			8.960	280,00	
		inzet rigger	32,00	uur						50,00	1.600	1.600	50,00	
		assistentie	32,00	uur	3	96,0						5.664	177,00	
		nacht- en weekendtoeslagen lossen personeel	160,00	uur						15,00	2.400	2.400	15,00	
	,													
		Lossen bij opslag/onderzoeks/opwerk locatie	1,00	ps	96	96,0		11.360,00	11.360	4.000,00	4.000	21.024	21.024,00	
	+	Laden bij opslag/onderzoeks/opwerk locatie	1,00	ps										
		aan-/afvoer 200 tons telekraan	1,00	ps				600,00	600			600	600,00	
		inzet 200 tons telekraan	16,00	uur				280,00	4.480			4.480	280,00	
		inzet rigger	16,00	uur						50,00	800	800	50,00	
		assistentie	16,00	uur	3	48,0						2.832	177,00	
	,													
		Laden bij opslag/onderzoeks/opwerk locatie	1,00	ps	48	48,0		5.080,00	5.080	800,00	800	8.712	8.712,00	
	+	Transport naar Meinerseiland, Arnhem	1,00	ps										
		transport liggers naar Meinerseiland, Arnhem	30,00	st						3.200,00	96.000	96.000	3.200,00	
	,													
		Transport naar Meinerseiland, Arnhem	1,00	ps						96.000,00	96.000	96.000	96.000,00	

Raming varianten (hergebruikte) liggers

	Leverancier	S	Omschrijving	Hoevelh.	Eh	Norm U	Uren	Mate riaal		Mate rieel		Onder aannemers		TOTAAL	Prijs/eenh	#
			Handeling en transport liggers	1,00	ps	144	144,0			16.440,00	16.440	246.800,00	246.800	271.736	271.736,00	
		#	Opwaarderen bestaande prefab liggers	1,00	ps											
			plooien/rechtbuigen bestaande beugelwapening uit liggers	645,00	m1							45,00	29.025	29.025	45,00	#
			schoonspuiten bestaande liggers	645,00	m1	0,15	96,8							5.708	8,85	#
			bijwerken beschadigingen bestaande liggers	645,00	m1	1	645,0	25,00	16.125					54.180	84,00	#
		,														
			Opwaarderen bestaande prefab liggers	1,00	ps	741,75	741,8	16.125,00	16.125			29.025,00	29.025	88.913	88.913,25	
		#	Doorhalen liggers en dekkingsherstel wapening	1,00	ps											
		"	*In lengterichting worden de liggers doorgezaagd*													
			doorzagen liggers, 2 -zijdig, 40 stuks liggers	1290,00	m1							81,25	104.813	104.813	81,25	#
			opruimen betonpuin	93,00	ton							5,00	465	465	5,00	#
			afvoeren betonpuin	93,00	ton							15,00	1.395	1.395	15,00	#
			acceptatiekosten betonpuin	93,00	ton							7,00	651	651	7,00	#
			leveren en aanbrengen dekkingsherstellende mortel op vrijliggende wapening t.p.v. zaagsnede	1290,00	m1							40,00	51.600	51.600	40,00	#
		,														
			Doorhalen liggers en dekkingsherstel wapening	1,00	ps							158.923,50	158.924	158.924	158.923,50	
		#	Leggen prefab liggers	645,00	m2											#
		"	*Maatgeven gewicht is ca. 21 ton (o.b.v. liggerinhoud conform standaard HRP-profiel Haitsma)*													
		"	*Uitgangspunt is inzet van een 500 tons telekraan*													
			leveren nieuwe oplegblokken	60,00	st			1.250,00	75.000					75.000	1.250,00	
			aan-/afvoer 500 tons telekraan	1,00	ps					1.200,00	1.200			1.200	1.200,00	
			inzet 500 tons telekraan, 1,5 velden leggen per dag	32,00	uur					600,00	19.200			19.200	600,00	
			assistentie personeel (5 man)	32,00	uur	5	160,0							9.440	295,00	
		,														
			Leggen prefab liggers	645,00	m2	0,25	160,0	116,28	75.000	31,63	20.400			104.840	162,54	
		#	Betonwerk dwarsdragers	28,00	m3											#
		"	*Dwarsdragers 1.000 mm breed*													
		+	Bekisting dwarsdragers	96,00	m2											#
241			verloren kist dwarsdragers omgekeerde T-ligger (liggeroppervlak er NIET afgehaald)	96,00	m2	2,4	230,4	25,00	2.400					15.994	166,60	#
249			afwerken beton / bekisting algemeen	96,00	m2	0,15	14,4	5,00	480					1.330	13,85	#
815	kraan		kraanhulp	24,00	uur					105,00	2.520			2.520	105,00	
		,														
			Bekisting dwarsdragers	96,00	m2	2,55	244,8	30,00	2.880	26,25	2.520			19.843	206,70	
		+	Wapening	7,36	ton											#
		"	*Uitgangspunt: 250 kg/m3 (incl. laslengtes, stekken en beugels)*													
301	wap		leveren en verwerken betonstaal B500B (liggerinhoud eraf gehaald)	6938,00	kg							1,20	8.326	8.326	1,20	#
301	wap		hulpwapening, 6%	417,00	kg							1,20	500	500	1,20	#
249			hulpvlechter, betonblokjes, ed.	7,36	ton	0,2	1,5	5,00	37					124	16,80	#
815	kraan		kraanhulp	8,00	uur					105,00	840			840	105,00	
		,														
			Wapening	7,36	ton	0,2	1,5	5,00	37	114,21	840	1.200,00	8.826	9.790	1.331,01	

	Leverancier	S	Omschrijving	Hoeveelh.	Eh	Norm U	Uren	Mate riaal		Mate rieel		Onder aannemers		TOTAAL	Prijs/eenh	#
		+	Beton	28,00	m3											#
251	C35/45, XC4, XD3,		leveren beton C35/45, XC4, XD3, XA2, XF4 (consistentie S3), incl. 3% verlies	29,00	m3			135,00	3.915					3.915	135,00	#
251	F4		toeslag consistentie F4	29,00	m3			4,50	131					131	4,50	#
		"	*Storten beton dwarsdragers -> gaat mee met druklaag*													
		"	*Aanvoer betonpomp, 39 - 43 m1 -> gaat mee met druklaag*													
253	Nijwa (raam)		variabele kosten betonpomp (1- 101 m3), 39 - 43 m1	28,00	m3							5,05	141	141	5,05	#
		,														
			Beton	28,00	m3			144,48	4.046			5,05	141	4.187	149,53	
			Betonwerk dwarsdragers	28,00	m3	8,80	246,3	248,65	6.962	120,00	3.360	320,26	8.967	33.820	1.207,85	
		#	Betonwerk druklaag	181,00	m3											#
		"	*Opgave conform document "Informatie kostenindicatie": dikte 160 mm en betonkwaliteit C30/37 -> echter dikte van 280 mm aangehouden, wat een gebruikelijke dikte is bij nieuwe druklagen evenals een betonkwaliteit van C35/45*													
		+	Bekisting druklaag	580,00	m2											#
241			ondersteunende badding aan zijkanten liggers t.b.v. oplegging bekistingsplaten (sponning voor verloren bekistingsplaten is niet weg te slopen)	1290,00	m1	0,3	387,0	3,00	3.870			15,00	19.350	46.053	35,70	#
241			verloren kist onderzijde dek, omgekeerde T-ligger	516,00	m2	0,4	206,4	25,00	12.900					25.078	48,60	#
241			langskisten en kopkisten druklagen	227,00	m1	0,8	181,6	14,00	3.178					13.892	61,20	#
249			afwerken beton / bekisting algemeen	580,00	m2	0,15	87,0	5,00	2.900					8.033	13,85	#
815	kraan		kraanhulp	80,00	uur					105,00	8.400			8.400	105,00	
		,														
			Bekisting druklaag	580,00	m2	1,49	862,0	39,39	22.848	14,48	8.400	33,36	19.350	101.456	174,92	
		+	Wapening druklaag dek	30,63	ton											#
		"	*Uitgangspunt: 160 kg/m3 = 45 kg/m2 (incl. laslengtes, stekken en beugels)*													
		"	*Uitgangspunt: 315 kg/m3 = 50 kg/m2 (incl. laslengtes, stekken en beugels) -> indien druklaag van 160 mm -> nu 280 mm gerekend*													
301	wap		leveren en verwerken betonstaal B500B	28896,00	kg							1,20	34.675	34.675	1,20	#
301	wap		hulpwapening, 6%	1734,00	kg							1,20	2.081	2.081	1,20	#
249			hulpvlechter, betonblokjes, ed.	30,63	ton	0,2	6,1	5,00	153					515	16,80	#
815	kraan		kraanhulp	32,00	uur					105,00	3.360			3.360	105,00	
		,														
			Wapening druklaag dek	30,63	ton	0,2	6,1	5,00	153	109,70	3.360	1.200,00	36.756	40.631	1.326,50	
		+	Beton	181,00	m3											#
251	C35/45, XC4, XD3,		leveren beton C35/45, XC4, XD3, XA2, XF4 (consistentie S3), incl. 3% verlies	187,00	m3			135,00	25.245					25.245	135,00	#
251	F4		toeslag consistentie F4	187,00	m3			4,50	842					842	4,50	#
252			storten beton, 1 stort, 6 man 8 uur (incl. dwarsdragers)	181,00	m3	0,35	63,4							3.738	20,65	#
253	Nijwa (raam		aanvoer betonpomp, 39 - 43 m1	1,00	kee							455,00	455	455	455,00	
253	Nijwa (raam		variabele kosten betonpomp (101- 200 m3), 39 - 43 m1	181,00	m3							5,05	914	914	5,05	#

Raming varianten (hergebruikte) liggers

	Leverancier	S	Omschrijving	Hoevelh.	Eh	Norm	U Uren	Mate	riaal	Mate	rieel	Onder	aannemers	TOTAAL	Prijs/eenh	#
252			vlinderen/afwerken dek	645,00	m2							3,50	2.258	2.258	3,50	#
252			nabehandelen met curing compound	645,00	m2							0,45	290	290	0,45	#
252			afdekken gestort dek	645,00	m2	0,02	12,9	0,40	258					1.019	1,58	#
		,														
			Beton	181,00	m3	0,42	76,3	145,55	26.345			21,64	3.917	34.760	192,04	
		+	*In te storten onderdelen*													
		"	*Niet meegenomen in deze TOM*													
		,														
			In te storten onderdelen													
			Betonwerk druklaag	181,00	m3	5,22	944,4	272,63	49.346	64,97	11.760	331,62	60.023	176.847	977,05	
		#	*Onderzoekskosten*	1,00	ps											
			onderzoekskosten/herberekeningen bestaande liggers	1,00	ps							75.000,00	75.000	75.000	75.000,00	
		,														
			Onderzoekskosten	1,00	ps							75.000,00	75.000	75.000	75.000,00	
			Variant 1d: Hergebruik HIP-800 liggers uit A9 Viaduct	645,00	m2	3,72	2.396,4	228,58	147.433	117,77	75.960	1.680,49	1.083.919	1.448.699	2.246,05	
		*	*Variant 2: Nieuwe liggers*	644,00	m2											#
		?	OPGAVE IS HRP-700, bij overspanning van 25 m1 zitten we eerder op minimaal HRP-800 liggers													
		"	*Overspanningen: 16,11 - 25 - 25 - 25 - 16,11 m1 -> 5 identieke overspanningen maken, indien toegestaan*													
		#	*Leveren en leggen prefab liggers*	644,00	m2											#
			leveren en leggen HRP-700 liggers	644,00	m2							460,00	296.240	296.240	460,00	#
			inzet begeleidend personeel, 3 man, 4 uur per veld	60,00	uur	1	60,0							3.540	59,00	
		,														
			Leveren en leggen prefab liggers	644,00	m2	0,09	60,0					460,00	296.240	299.780	465,50	
		#	*Betonwerk dwarsdragers*	24,00	m3											#
		"	*Dwarsdragers 1.000 mm breed*													
		+	*Bekisting dwarsdragers*	84,00	m2											#
241			verloren kist dwarsdragers omgekeerde T-ligger (liggeroppervlak er NIET afgehaald)	84,00	m2	2,4	201,6	25,00	2.100					13.994	166,60	#
249			afwerken beton / bekisting algemeen	84,00	m2	0,15	12,6	5,00	420					1.163	13,85	#
815	kraan		kraanhulp	24,00	uur					105,00	2.520			2.520	105,00	
		,														
			Bekisting dwarsdragers	84,00	m2	2,55	214,2	30,00	2.520	30,00	2.520			17.678	210,45	
		+	*Wapening*	6,16	ton											#
		"	*Uitgangspunt: 250 kg/m3 (incl. laslengtes, stekken en beugels)*													
301	wap		leveren en verwerken betonstaal B500B (liggerinhoud eraf gehaald)	5813,00	kg							1,20	6.976	6.976	1,20	#
301	wap		hulpwapening, 6%	349,00	kg							1,20	419	419	1,20	#
249			hulpvlechter, betonblokjes, ed.	6,16	ton	0,2	1,2	5,00	31					104	16,80	#
815	kraan		kraanhulp	8,00	uur					105,00	840			840	105,00	
		,														
			Wapening	6,16	ton	0,2	1,2	5,00	31	136,32	840	1.200,00	7.394	8.338	1.353,12	
		+	*Beton*	24,00	m3											#
251	C35/45, XC4, XD3,		leveren beton C35/45, XC4, XD3, XA2, XF4 (consistentie S3), incl. 3% verlies	24,00	m3			135,00	3.240					3.240	135,00	#

	Leverancier	S	Omschrijving	Hoevelh.	Eh	Norm U	Uren	Mate riaal		Mate rieel		Onder aannemers		TOTAAL	Prijs/eenh	#
251	F4		toeslag consistentie F4	24,00	m3			4,50	108					108	4,50	#
		"	*Storten beton dwarsdragers -> gaat mee met druklaag*													
		"	*Aanvoer betonpomp, 39 - 43 m1 -> gaat mee met druklaag*													
253	Nijwa (raam)		variabele kosten betonpomp (1- 101 m3), 39 - 43 m1	24,00	m3							5,05	121	121	5,05	#
		,														
			Beton	24,00	m3			139,50	3.348			5,05	121	3.469	144,55	
			Betonwerk dwarsdragers	24,00	m3	8,98	215,4	245,78	5.899	140,00	3.360	313,15	7.516	29.485	1.228,54	
		#	Betonwerk druklaag	181,00	m3											#
		+	Bekisting druklaag	574,00	m2											#
241			verloren kist onderzijde dek, omgekeerde T-ligger	510,00	m2	0,4	204,0	25,00	12.750					24.786	48,60	#
241			langskisten en kopkisten druklagen	227,00	m1	0,8	181,6	14,00	3.178					13.892	61,20	#
249			afwerken beton / bekisting algemeen	574,00	m2	0,15	86,1	5,00	2.870					7.950	13,85	#
815	kraan		kraanhulp	48,00	uur					105,00	5.040			5.040	105,00	
		,														
			Bekisting druklaag	574,00	m2	0,82	471,7	32,75	18.798	8,78	5.040			51.668	90,01	
		+	Wapening druklaag dek	30,55	ton											#
		"	*Uitgangspunt: 160 kg/m3 = 45 kg/m2 (incl. laslengtes, stekken en beugels)*													
301	wap		leveren en verwerken betonstaal B500B	28821,00	kg							1,20	34.585	34.585	1,20	#
301	wap		hulpwapening, 6%	1730,00	kg							1,20	2.076	2.076	1,20	#
249			hulpvlechter, betonblokjes, ed.	30,55	ton	0,2	6,1	5,00	153					513	16,80	#
815	kraan		kraanhulp	32,00	uur					105,00	3.360			3.360	105,00	
		,														
			Wapening druklaag dek	30,55	ton	0,2	6,1	5,00	153	109,98	3.360	1.200,00	36.661	40.534	1.326,78	
		+	Beton	181,00	m3											#
251	C35/45, XC4, XD3,		leveren beton C35/45, XC4, XD3, XA2, XF4 (consistentie S3), incl. 3% verlies	186,00	m3			135,00	25.110					25.110	135,00	#
251	F4		toeslag consistentie F4	186,00	m3			4,50	837					837	4,50	#
252			storten beton, 1 stort, 6 man 8 uur (incl. dwarsdragers)	181,00	m3	0,35	63,4							3.738	20,65	#
253	Nijwa (raam		aanvoer betonpomp, 39 - 43 m1	1,00	kee							455,00	455	455	455,00	
253	Nijwa (raam		variabele kosten betonpomp (101- 200 m3), 39 - 43 m1	181,00	m3							5,05	914	914	5,05	#
252			vlinderen/afwerken dek	644,00	m2							3,50	2.254	2.254	3,50	#
252			nabehandelen met curing compound	644,00	m2							0,45	290	290	0,45	#
252			afdekken gestort dek	644,00	m2	0,02	12,9	0,40	258					1.018	1,58	#
		,														
			Beton	181,00	m3	0,42	76,2	144,78	26.205			21,62	3.913	34.615	191,24	
		+	In te storten onderdelen													
		"	*Niet meegenomen in deze TOM*													
		,														
			In te storten onderdelen													
			Betonwerk druklaag	181,00	m3	3,06	554,0	249,48	45.155	46,41	8.400	224,17	40.574	126.818	700,65	
			Variant 2: Nieuwe liggers	644,00	m2	1,29	829,5	79,28	51.054	18,26	11.760	534,67	344.330	456.083	708,20	