Thesis report

Feasibility study of lowering the Beneluxtunnel

Tomas Weeda

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Colophon

Contacts

T.R. (Tomas) Weeda
MSc. Student Hydraulic Engineering (1387596)
Pieter Vlamingstraat 226
1093 DJ Amsterdam
T: +316 4618 9857
E: tomas.weeda@gmail.com

Delft University of Technology
Faculty of Civil Engineering & Geosciences (CiTG)
Stevinweg 1
2628 CN Delft

Gemeente Rotterdam
Wilheminakade 177
3072 AP Rotterdam

Graduation committee

Chairman
Prof.dr.ir. S.N. Jonkman
Section Hydraulic Engineering
Delft University of Technology

Daily supervisor
Dr.ir. J.G. de Gijt
Section Hydraulic Engineering
Delft University of Technology /
Gemeente Rotterdam

Member of committee
Dr.ir. K.J. Bakker
Section Hydraulic Engineering
Delft University of Technology

Member of committee
Dr.ir.drs. C.R. Braam
Section structural and Building Engineering
Delft University of Technology

External member of committee
Ir. E.J. Broos
Port of Rotterdam

Preface

Dear reader,

This thesis reports the findings of my study into the possibility of lowering the Beneluxtunnel are presented. I performed this study as my graduation project to obtain a Master's degree in Hydraulic Engineering, at the Delft University of Technology. The study was facilitated by Gemeente Rotterdam and executed in collaboration with Havenbedrijf Rotterdam and Rijkswaterstaat.

First of all, I would like to express my gratitude towards my graduation committee. I would like to thank them for their valuable advice regarding this study, but even more so for their patience and guidance when personal tragedy complicated my graduation project.

Prof.dr.ir. S.N. Jonkman

Dr.ir. J.G. de Gijt

Dr.ir. K.J. Bakker

Dr.ir.drs. C.R. Braam

Ir. E.J. Broos

Furthermore, I would like to thank William van Rooijen, Jaap Brantenaar and Martijn Blom at RWS, Pieter Koman at van Oord, Marcel de Vos at Trelleborg and Bert van der Meer at Gemeente Rotterdam for their collaboration and professional advice.

I would also like to express my appreciation towards my colleagues at Gemeente Rotterdam for creating a pleasant work environment and for providing all the support I needed. A special thanks to Jaap Peters for his additional support.

On a personal note, I would like to thank my father for his confidence and support, my brother and close friends who stood me by during this period and especially my girlfriend who went through everything I went through. Most of all I would like to thank my mother, for showing me what life is.

T.R. Weeda

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Abstract

The persistent increase of container vessel dimensions requires ports to adapt their wet infrastructure in order to compete with their competitors. However, in some cases waterway tunnels are expected to restrict the maximum depth. The Beneluxtunnel in Rotterdam is an example of such a tunnel. To allow more depth, the possibility of lowering this tunnel is studied.

The Beneluxtunnel consists of two immersed tunnels constructed in 1967 and 2002. They consist of prefabricated elements that are immersed and connected to land parts on either side of the waterway.

To allow lowering, various design options have been proposed in this study:

- Regarding the land part, the options are proposed (1) to do nothing, (2) to make local adjustments utilizing the space provided by the dewatering cellar or (3) to apply general lowering which would affect the entire structure.
- Regarding the immersed part, the options are proposed (1) to adapt the river bottom, (2) to lower the elements while they remain immersed by utilizing the limited freedom of rotation in its joints or (3) to re-float the elements to be re-immersed on a lowered bed.
- Regarding traffic requirements, an option is to lower the maximum speed which would allow for steeper slopes.

These options and their combinations have been analysed to determine their feasibility and value.

To do so, first all aspects regarding the functional design are determined. The relations between traffic requirements and navigation channel dimensions have been determined, resulting in an overview of possible vertical profiles for all relevant design combinations. Apart from a first estimate of the effectivity of the design options, it also showed that the 1st Beneluxtunnel is governing because of the shorter length of its immersed part.

For the technical design, the design of the original tunnel is analysed and for some important aspects, the structural capacity has been determined. This information is used to determine the effects of lowering on the tunnel and to estimate the associated lowering capacity in terms of vertical displacement profiles. This analysis shows that for all realistic design options, the structural capacity is sufficient.

Apart from the lowering, also the effects and limitations of joint rotations have been determined. Both for the segment joints and the immersion joints, the rubber water stops are critical in determining the maximum rotation. The applied rotations must however be less as additional settlement could increase them. The vertical profiles associated with these rotations show significant depth increase, ranging up to 4.5 m.

To determine the technical feasibility, the design must also be constructible. Hence, for four of the design options, the construction methods have been determined and proposed:

- To lower the immersed part while it remains immersed, the soil must be removed below the elements by special dredging equipment. Also the supporting tiles must be removed. The large longitudinal pressure is expected to allow this without supports. However, for the lowering process, applying pre-tensioning and temporary supports is required. Special components connected to the pre-tensioning should allow to control the lowering process. Finally, the soil can be returned and the tunnel can be finished.
- To re-float the elements, the closure joint must be re-opened. Next, the weight of the elements must be reduced by removing the ballast. Instead of re-floating it is chosen to lift the elements which allows better control of the process. The elements can now be transported to a location where they can be stored and adapted. When the bed is lowered and the land parts are finished, the elements can be re-immersed.
- To locally adjust the land part, access is required which requires for the connecting element to be removed and for a watertight screen to be placed. Next, the old transition point can be demolished and the new transition point can be constructed within the existing abutment, utilizing the space provided by the dewatering cellar.
- General lowering can be achieved by wet reconstruction. First, additional anchors have to be inserted in the walls. Next the land part can be flooded and the entire abutment and underwater concrete floor must be demolished. A new floor should be constructed at a few meters below, but first additional anchors must be inserted into the soil to provide vertical stability. Finally, water could be pumped out and everything could be reconstructed.

Finally, the construction sequences are used to estimate the costs of the design options and their combinations. The results were combined with the maximum navigation depth increase to determine the most cost effective solution for different lowering ranges. Also the long and short term traffic costs (traffic jams) have been taken into account.

The results are:

Depth increase	Land part	Immersed part	Costs (millions)
0.0 - 0.9 m	Local adjustments on both sides	Remain immersed	€ 295
0.9 - 1.9 m	General lowering on one side	Remain immersed	€ 415
1.9 - 3.2 m	General lowering on both sides	Remain immersed	€ 500
3.2 - 3.6 m	General lowering on both sides	Re-float	€ 665
> 3.6 m	New immersed tunnel		€ 730

This study has proven that lowering the Beneluxtunnel is technically feasible and a cost effective method to increase the navigation draught.

Contents

Chapter 1

Introduction

Figure 1.1: This image shows the 1st Beneluxtunnel around the time when it was first introduced to the public. [16]

1.1 Problem

1.1.1 Background

The Beneluxtunnel

For almost half a century, the Beneluxtunnel has provided a valuable traffic connection between the banks of the river that connects the city of Rotterdam to the sea. Ever since the tunnel was finished in 1967, it led the A4 highway underneath the river, not disturbing any of the ships on their way to reach their destination in one of the busiest ports in the world.

At the end of the 20th century traffic numbers increased beyond expectations, leading to the construction of the second Beneluxtunnel which was completed in 2002 and created besides an increase in car capacity, also the possibility for a metro-line, cyclists and pedestrians to reach the other side of the river.

The development of the city of Rotterdam and its harbour still continues. The expansion of both will most likely continue far into the future as the port is ensuring its spot as one of the top ports in the world and Rotterdam is well on its way to become the international city such a port requires. As infrastructure is one of the key factors for success in this playing field, the importance of the Beneluxtunnel becomes evident. Especially its location makes it an important link in the interaction between the port and the city. Consequently, the tunnel must always be able to adapt to the changing demands of the future in order not to restrain the economic growth of the area.

Challenge

Just like a few decades ago, functional requirements are changing. However, this time the threat comes from the sea. Figure 1.2 gives an indication of the dimensions below the water surface of the largest container ships available at the time of construction of the Beneluxtunnels and today, relative to the depths and dimensions of the tunnels. This figure clearly shows the increase in ship size over the past decades, as well as the draught conflict that would exist if present days largest container vessels would try to pass the Beneluxtunnel.

Currently, the largest vessels do not require to reach beyond the Beneluxtunnel as they are facilitated elsewhere in the port. But if future developments of the port require for larger vessels to reach the inland port areas, the Beneluxtunnel will become an obstacle.

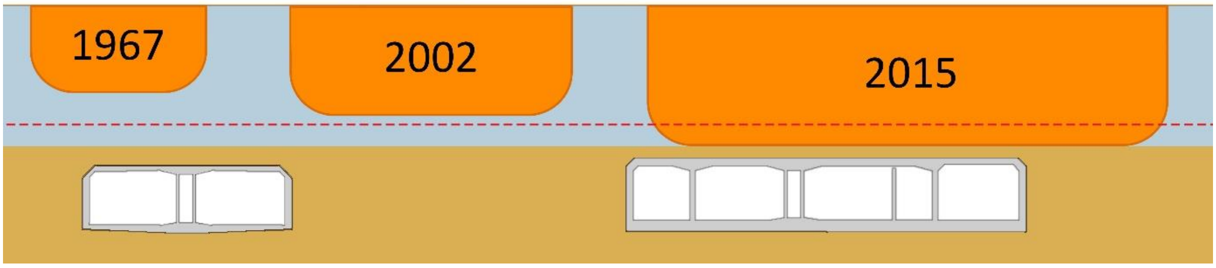


Figure 1.2: This figure gives an indication of the dimensions below the water surface (beam and draught at low tide) of the largest container ships available around the time of construction of the Beneluxtunnels and now, relative to the depths and dimensions of the tunnels. The red line indicates the current maximum depth at the edges of the navigation channel. Note: In reality, ships would never cross the tunnels like this as this would suppose they are sailing from shore to shore.

1.1.2 Relevance

In this section, the relevance of the problem scenario will be emphasised. Also the relevance of the possible solutions will be mentioned.

The Beneluxtunnel as a draught obstacle

Recent developments have increased the likelihood of the problem scenario, as the port authorities have announced to be planning to deepen the port access channel to a depth of NAP -17 m to increase the accessibility of the liquid bulk areas at Botlek and Pernis, reaching just seaward of the Beneluxtunnel. [17]

Further developments depend on the draught demands of the port areas inland of the Beneluxtunnel. But if demand grows for these city ports, the Beneluxtunnel will become the first obstacle to be overcome, both physically and economically.

Figure 1.3: This figure shows the port of Rotterdam and the part of the access channel that might be deepened.[5]

Similar situations

The possible conflict between tunnels and navigation is not exclusive to the port of Rotterdam. There are several locations where tunnels cross a port access channel. An increase in ship size could also lead to problems at these locations.

Examples of such locations are:

- The North see channel, where the Velsertunnel, the Velsertraintunnel and the Wijkertunnel cross the access channel of the port of Amsterdam. The locks providing access to this channel have just been upgraded, providing a depth larger than the provided depth at the locations of these tunnels.
- The Schelde, where the Liefkenshoektunnel and the Tijsmanstunnel cross the access channel of the port of Antwerp.
- The Elbe, where the Elbtunnel crosses the access channel of the port of Hamburg.

Relevance of the solutions

Apart from the relevance of the problem, also the relevance of the possible solutions is worth mentioning. This study will consider increasing the draught by purposely adjusting the physical position of the tunnel. This is as far as known the first time this will be examined for an existing tunnel.

The applicability of these solutions does not have to be limited solely to the problem of draught requirements. They can for instance also be used for straightening of the tunnel alignment in case of differential settlement, or for the removal or replacement of tunnel sections when they are no longer functioning properly.

1.1.3 Problem definition

In this section, the problem scenario of the Beneluxtunnel and the similar tunnels mentioned in the previous section will be defined in more general terms. The situation will be described as a dynamic equilibrium between the mechanisms responsible for required depth and available depth, and the presence of a tunnel will be held responsible for disrupting this equilibrium. This generalisation will help dividing the problem into two individually approachable sub-problems.

Required depth

The required depth of an access channel at a certain point and time is the minimum depth required for ships to be able to pass this point. Its magnitude depends mainly on the following two mechanisms:

Global maximum ship size — The first contributing aspect is the global maximum ship size. This maximum ship size has shown a significant increase throughout history, which is believed to be a result of the economy of scale which generally means that if a ship is built larger, the extra profit as a result of carrying more cargo outweighs the extra investment costs and variable costs during the lifetime of a ship. Thus, shipping costs are smaller if the ship is larger.

Inland port utilisation — Not all ships are required to pass the considered location. Only ships that are able to trade their cargo will be desiring enough depth to pass, meaning the required depth also depends on the utilisation of the inland port areas. Contributing aspects are the wet infrastructure of the inland areas which allows access for the vessel, but also the economic activity of the area and its hinterland which allows vessels to trade.

Available depth

The available depth of an access channel at a certain point and time is simply the distance between the water level and the bottom level. Its magnitude depends mainly on the following two mechanisms:

Natural depth — The natural depth can be regarded as the initial depth of the considered location. It is determined by natural processes that influence either the water level or the bottom level. In port access channels, the water level is usually determined by the sea level, including tidal fluctuations. The bottom level is determined by the morphologic processes that result from water flow and is therefore in large extent determined by the discharge of the waterway.

Human interventions — The natural depth can be adjusted by human interventions. Either the water level can be adjusted or the bottom level. Adjustments to the water level are applied often, but they are not very suitable for open waterways as they tend to block discharge and navigation. Adjustments to the bottom level can be acquired by dredging, but morphological processes tend to return bottom levels back to their natural state, requiring maintenance.

Dynamic equilibrium

Thus, the required depth is determined by global maximum ship size and inland port utilisation and the available depth is determined by natural processes influencing the water level and the bottom level, which can be adapted by human interventions but only at the expense of investment and maintenance.

If the required depth is larger than the available depth, human interventions are desired. This would however require investment and maintenance costs and would therefore only be feasible if the increased revenues following from this depth increase outweighs these costs.

Hence, the actual depth at a certain point is an equilibrium between required depth and available depth and is variable over time.

Tunnel

A tunnel would contribute to the available depth side of the equilibrium. A tunnel is similar to the river bottom providing the lower boundary of the depth, but differs greatly at one aspect: Adaptability.

A tunnel can be interpreted as a static component in the dynamic equilibrium described above. Static, not because it is made of hard concrete instead of soft sand, but because it is both fragile and valuable. Being valuable means the tunnel has certain societal importance, meaning it cannot simply be demolished. The connection provided by the tunnel would have to be reassured if one would want to remove it, requiring a new tunnel. Being fragile means that the tunnel cannot simply be repositioned to a lower position.

The investment costs of adapting the tunnel would be enormous meaning an inevitable loss of value of the port areas on the landside of the tunnel would occur if the required depth would increase beyond the available depth bounded by the tunnel. Unless of course, a solution is found which allows adapting the tunnel at feasible costs.

The likelihood of the problem scenario

Due to the distinction made above, the problem can be divided into two individually approachable sub-problems, concerning either the likelihood of the problem scenario or possible solutions to the problem scenario.

The likelihood of the problem scenario is determined by the long term expectancy of the required depth relative to the available depth. With the tunnel providing the lower boundary and the water level providing the upper boundary, the available depth is rather predictable. The required depth however, depends on economic processes that are relatively unpredictable, meaning the long term expectancy of this required depth is relatively unpredictable.

Consequently, the likelihood of the problem scenario follows solely from factors concerning the required depth as the unpredictability of the available depth is negligible relative to the unpredictability of the required depth. To determine the likelihood of the problem scenario one should therefore try to predict the development of global maximum ship size and of inland port utilisation.

Solutions to the problem scenario

Second, the solution to the problem scenario could be found in adapting the situation. The economic processes responsible for the required depth are globally and regionally influenced making the required depth practically uncontrollable and therefore unadaptable. On the other hand, the available depth is locally influenced and determined by physical and observable quantities, meaning the adaptability of the available depth is significantly larger.

Consequently, a solution to the problem scenario should be sought solely in adapting the available depth as the adaptability of the required depth is negligible relative to the adaptability of the available depth.

1.2 Research

In this section, the research methodology will be explained. First the scope of the research will be limited. Next the objectives for this study will be given, after which the approach to realise these objectives will be explained.

1.2.1 Scope

Given the limited time available and the specific scientific background of the researcher, it is chosen to limit the scope. Four decisions are made that are thought to contribute to maximise the scientific value of this study, as will be explained in this section.

Finding solutions

As discussed in section 1.1.3, the problem scenario seems to have two sides that can be interpreted separately. These sides can be translated into the following research questions:

- What is the likelihood of the problem scenario?
- What are effective solutions for the problem scenario?

In this study, the likelihood of the problem scenario will not be determined. The large scale economic processes that influence this likelihood are considered relatively unpredictable and therefore providing a satisfying answer to this question is regarded very difficult to achieve.

Instead, the problem scenario will be assumed to occur and effective solutions to this scenario will be sought.

Such a strategy is only useful when the risk of an event is regarded large enough to prepare for uncertain scenarios. Given the generally large economic activity of the port areas inland of tunnels and the increase in ship size in the past, both the probability and the consequences of the event are regarded significant, making it useful to prepare for this scenario.

Case study

The problem scenario was introduced in section 1.1.1 as a case specific problem regarding the Beneluxtunnel, while in section 1.1.2 it was mentioned that the problem scenario is not exclusive to the Beneluxtunnel case. However, it is chosen to limit the scope of this study to the Beneluxtunnel case instead of approaching the problem more generally.

Important reasons are:

- This study concerns tunnels in waterways. Tunnels are like most civil structures large in size, having much interaction with their surroundings, making them very dependent on local conditions. They also represent a large value making a unique approach economically feasible. Because of these characteristics, each case can best be approached separately.
- By only exploring a single situation, the specific knowledge of this situation will be relatively large, increasing the chance of finding innovative solutions.
- When investigating yet unexamined subjects, it is often better to first focus on a single case in more detail, so bottlenecks and risks that only appear in later stages can be located. This gained knowledge can then be used to study other cases.

Possibly, knowledge gained during the study of this case could be applied to other cases. Reflecting on this will be part of this study.

Engineering

Furthermore, it is chosen to limit the solution space to solutions in the field of civil engineering. It is expected that this type of specialisation will optimise the value of the outcome.

Examples of solutions that will not be part of this study are:

- Solutions concerning changes in law and legislation.
- Solutions concerning port planning or city planning.

Bottom of the waterway

It is also chosen to focus on solutions intended to lower the bottom of the waterway. Solutions are therefore determined to lower the vertical position of the tunnel, decrease the vertical distance between the river bottom and the top of the tunnel, or a combination of both.

Examples of solutions that will not be part of this study are:

- Manually increasing the height of the water level.
- Decreasing the draught of ships by temporarily increasing floating capacity.

1.2.2 Research methodology

In this section, the research objectives will be established. First the research question will be given, which will subsequently be translated into a research objective.

Research question

The main research question of this study is:

> *How can the Beneluxtunnel be adapted in order to effectively increase the available draught for navigation?*

Research method: Design

To find a fulfilling answer to the research question, a design will be made.

Conceptual design

Given the complexity of the subject, the problem and therefore also the expected complexity of the possible solutions, it is not realistic to expect the entire design stage to be completed. Instead, this study will remain in the conceptual design phase.

Being in the conceptual phase means being in the beginning of the design process where all solutions are still possible. Choices made in this phase have the greatest effect on the possible outcome, but are also subjected to the greatest uncertainty of all design phases.

The goal of this phase is to find and evaluate opportunities and challenges in order to give direction to following design phases. The amount of detail depends on the amount of information required for evaluation of feasibility.

Research objective

The main objective of this study is to:

> *Propose a conceptual design for adapting the Beneluxtunnel in order to effectively increase the available draught for navigation.*

An important aspect of this objective is expressed by the word effectively. It implies that not only a solution has to be found that will lower the tunnel, but also the feasibility and value of this solution has to be determined.

Purpose

The conclusions of the study are intended to:

- Provide information that will allow well founded decisions to be made regarding the future of the Beneluxtunnel.
- Provide technical insight in the subject of lowering immersed tunnels in general and would therefore be of value for other cases that have to overcome a similar problem scenario, or any other problem scenario requiring similar solutions.

1.2.3 Approach

In this study, a two phase design approach is used which is based on the processes of divergence and convergence.

Divergence In the first phase, the subject is analysed and solutions are proposed. The purpose of this phase is to gain as much knowledge regarding the problem and the possible solutions as is required to find as many creative solutions as possible. The solutions that will subsequently be proposed are solutions that are expected to have a certain level of feasibility and value, allowing them to have a chance of becoming the optimal solution.

Convergence In the second phase, the results of the first phase will be processed and evaluated. The purpose of this phase is to find the best solution amongst the solution proposed in the first phase. To do so, first a design strategy is proposed in which will be explained what subjects will need to be elaborated in order to be able to evaluate the solutions. This includes elaboration of the solutions themselves, but also of the subjects contributing to the value of the solutions. This elaboration will then be executed and finally, advice will be given regarding the value of the solutions.

This report should be a basis for later stages of design.

1.3 Report structure

The structure of the report follows the approach described in the previous section.

The analysis of the subject is presented in chapter 2, in chapter 3 design concepts are proposed which are elaborated in chapters 4, 5, 6 and 7 and evaluated in chapter 8. The structure of this report is further elaborated below:

The Beneluxtunnel	First, all essential background information regarding the Beneluxtunnel will be provided. The subject of immersed tunnels in general will be briefly treated, after which the Beneluxtunnel will be subjected to a functional and technical analysis. (Chapter 2)
Basis of design	Next, a basis of design is proposed. To do this, first the methodology used in the design process will be explained, after which the possible design options will be proposed and choices regarding further elaboration of these options will be made. Finally, design criteria and boundary conditions will be provided. (Chapter 3)
Functional design	The first part of the design to be elaborated is the functional design. In this chapter the relationship between the tunnel and its functions will be established, in order to evaluate the consequences of adjustments to the tunnel. (Chapter 4)
Structural analysis of the existing tunnel	In this chapter, the structural features of the existing tunnels will be analysed. Possible weaknesses will be located and for some aspects the structural capacity will be determined. (Chapter 5)
Effects of lowering on structure	Next, the findings of the previous chapter will be used to analyse the lowering of the tunnels. The effects of lowering on the structural aspects of the tunnel will be determined and used to estimate the maximum amount of lowering. (Chapter 6)
Construction of alternatives	Then, the constructability of the alternatives will be discussed. For the most important design alternative, the required construction sequences will be elaborated, mainly for the purpose of determining feasibility. (Chapter 7)
Evaluation	Once all design options are elaborated and their interdependence has been established, they will be evaluated and compared. (Chapter 8)
Conclusions and recommendations	Finally, the conclusions regarding the research objectives that can be made based on the findings if this study will be presented and recommendations for future investigations will be given. (Chapter 9)

At the end of each chapter, a summary is given of the most important findings within the chapter.

Apart from the main report, also a number of appendices are included, to which will be referred throughout the report.

Chapter 2

The Beneluxtunnel

Figure 2.1: This image shows both the land part and the tunnel elements in the construction dock of the 1st Beneluxtunnel [18]

2.1 Introduction

The Beneluxtunnel has been introduced in chapter 1. In this chapter, it will be further analysed. All essential background information required to understand the design process will be presented.

Three subjects are distinguished:

Immersed tunnels	As the Beneluxtunnel is an immersed tunnel, which is a very specific type of construction, first a general explanation regarding immersed tunnels will be given. (Section 2.2)
Functional analysis	Next, the two main functions of the tunnel, allowing the passage of traffic and navigation, will be analysed. (Section 2.3)
Technical analysis	In the technical analysis, relevant technical aspects of the tunnel will be treated. (Section 2.4)

Finally, a summary will be given of the most important findings within this chapter.

2.2 Immersed tunnels

In this section, a general explanation regarding immersed tunnels will be given. First the subject will be introduced by following the history of events that lead to the creation of the immersed tunnel. Second, the most important characteristics of immersed tunnels will be given.

2.2.1 History

Crossing waterways

Ever since people have been traveling the world, there have been natural borders to restrain these movements. To reach new land and to find the riches that lie beyond these borders, humans have been very resourceful, leading to various crossings. First we found natural crossings, like shallow parts of rivers and passes through mountain ranges. Later we learned to make our own crossings. We built bridges to cross rivers and gullies and we cut our way through mountains, creating tunnels. We even learned to see water as infrastructure rather than as an obstruction, using rafts or boats to move ourselves across or to travel to other parts of the world.

Doing so, a new problem occurred. The crossing of rivers by masses, conflicts with the travel along the river by individuals. When the dimensions of vessels grew in order to carry large amounts of goods or to be able to catch wind for propulsion, the demand on the height of bridges grew unfeasibly large. The request for another kind of crossing that does not interfere with navigation arose.

Digging a tunnel

This request manifested itself in the 19th century in London with the construction of a tunnel underneath the Thames. Building a tunnel underneath a low-lying waterway meant dealing with conditions that were very different from the tunnels through solid rock that were known from mountain areas and mining. The soft soil and the water had to be kept out of the shaft which demanded a strong and impermeable shell.

At the front end of this shaft, workers would excavate soil. The pressure inside the shaft was kept high enough to prevent a collapse of this vertical wall and to minimise the inflow of water. When the supporting structure moved onwards, a brick shield around the shaft prevented it from collapsing. This method is illustrated in Figure 2.2.

This tunnelling method was very slow, risky and labour intensive. Working under high pressure would expose the workers to great personal risks, including the chance to get the very painful and in many cases lethal caisson disease which was a result of rushed decompression and known from the caissons used to build bridge abutments. Also great collective risk was present, namely a blowout, meaning overpressure in the shaft would escape through irregularities in the soil, resulting in a collapse of the face of the tunnel and an uncontrolled inflow of water.

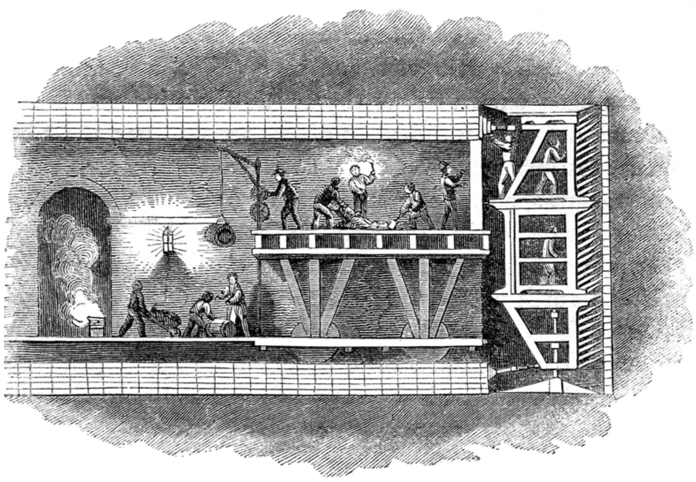


Figure 2.2: This figure shows the construction of the Thames tunnel. [11]

Tunnelling methods

Since the construction of the Thames tunnel, three methods for tunnelling waterways were developed. The bored tunnelling method – which is generally the mechanical version of the Thames method – became very popular and the in situ method in which a construction dock is made within the waterway has also been used very often.

The third method is the immersed tunnelling method, in which tubular sections were lowered onto the bottom of the waterway and connected to each other. The linked tubes would form a continuous connection between both banks of the waterway.

This immersed tunnelling method was first executed in the United States at the beginning of the 20th century where they used steel tubes to ensure the strength and impermeability required.

Figure 2.3: The Michigan Central Railway tunnel, completed in 1910 [2]

Concrete immersed tunnelling

A few decades later, the Dutch innovated the immersed tunnel design by using reinforced concrete rather than steel. Although the use of concrete dates back to the Roman Empire, the use of reinforced concrete for construction purposes started at the end of the 19th century and was therefore relatively new. Hence, it was quite a challenge to construct elements with this method. Elements that were completely watertight, able to float, able to travel, able to be immersed, able to be connected and able to be used in an environment with large water pressures acting on them.

Nevertheless, the relatively low material costs led to the first immersed reinforced concrete tunnel in 1942, the Maastunnel in Rotterdam. An event which would mark the beginning of an era of Dutch tunnelling. It would also be the first immersed tunnel with a rectangular cross-section, something that was impossible to accomplish with steel on the large scales required, but led to a cross-section that better suited its functional requirements than its steel, tubular predecessors.

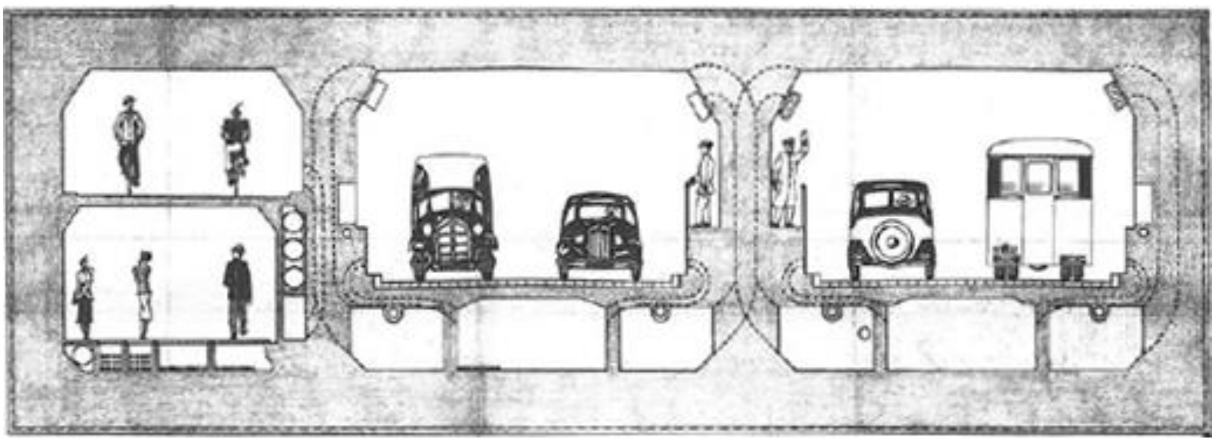


Figure 2.4: The cross-section of the Maastunnel, the first immersed tunnel having a rectangular concrete cross-section. [19]

Dutch immersed tunnels

In the following years, a large number of tunnels crossing waterways were built in the Netherlands as is indicated in figure 2.5. Most of them were immersed tunnels. This dedication, in combination with the great Dutch knowledge in hydraulic engineering, gained in centuries of living with and protecting against water, has made immersed tunnelling a typical Dutch profession.

Figure 2.5 also indicates the design lifetime of the existing Dutch tunnels. About 30% of them will have reached the 2nd half of their lifetime within the next 5 years. All of these tunnels have had – or are about to have – major renovations and minor repairs, which raises the question whether a design lifetime of 100 years is realistic. The causes of the problems vary from technical issues to changes in functional requirements.

Consequently, the focus is shifting from designing and constructing immersed tunnels to maintaining existing immersed tunnels to remain functional throughout or even beyond its design lifetime.

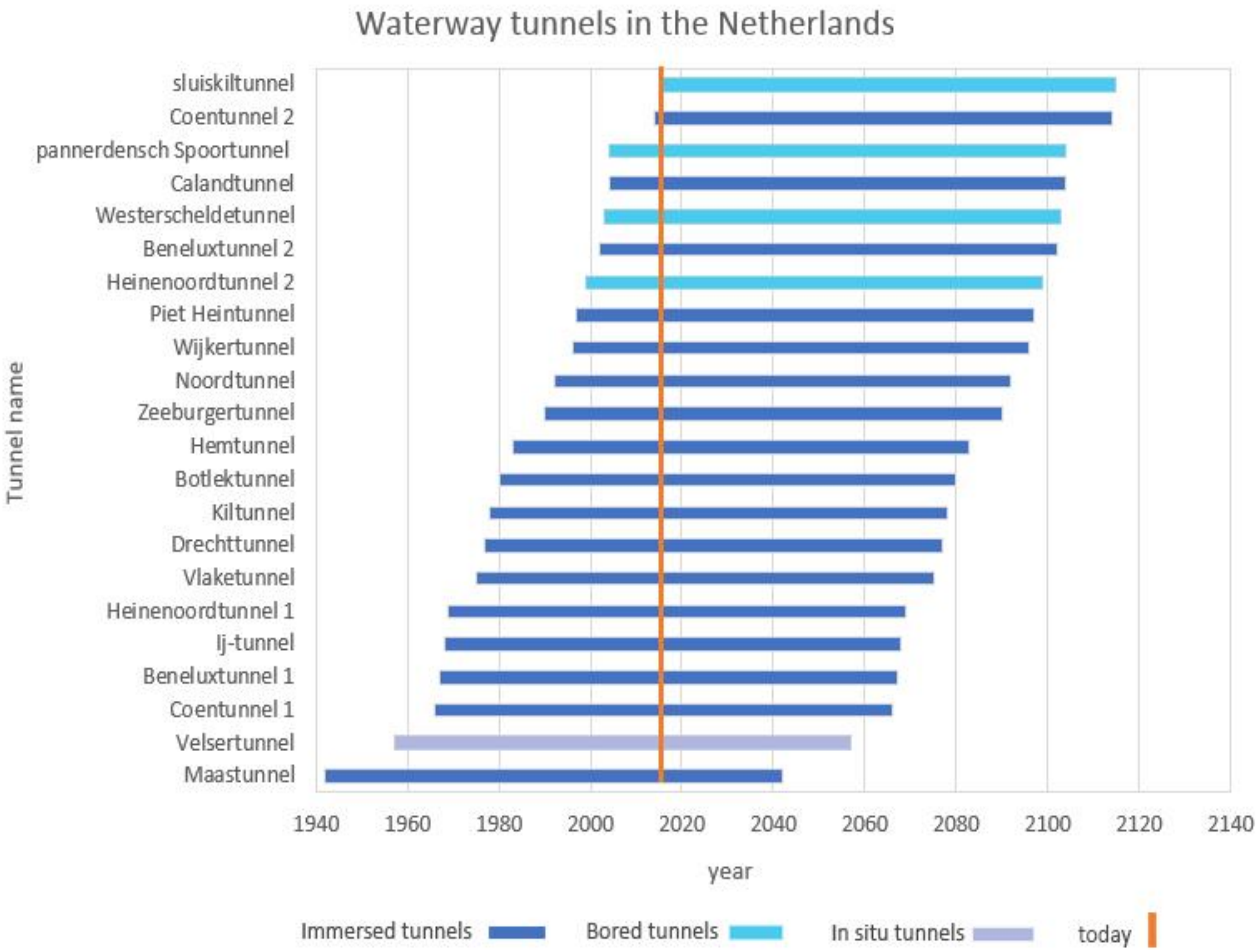


Figure 2.5: An overview of tunnels crossing waterways and their design lifetime in the Netherlands. The length of the bars indicate the 100 year design lifetime of the tunnels.

2.2.2 Characteristics

Basically, immersed tunnels are characterised by two aspects, their functions and their construction method. These aspects and their requirements combined, determine the layout of immersed tunnels.

Functions

The first defining aspect of immersed tunnels is their functions and the associated functional requirements:

Traffic The primary reason for an immersed tunnel to be built, is because a traffic connection is required. The traffic function could therefore be regarded the primary function of an immersed tunnel. Traffic types can differ, but most tunnels are designed for cars or trains.

Traffic requirements determine the layout of the tunnel's cross-section, and limits other aspects like the slope and the amount of bending.

Navigation Secondly, a tunnel is chosen over other types of connections because of the desire not to disturb navigation, which is therefore the secondary function of immersed tunnels.

Consequently, navigation requirements determine the depth of the tunnel, as the top of the tunnel must lie below the bottom of the navigation channel.

Other More functions can be ascribed to immersed tunnels that can influence their value, like aesthetics, noise hindrance, pollution, environmental impact or use of space. However, the impact of these functions is much less than the impact of traffic and navigation and will therefore not be separately discussed.

Construction method

The second defining aspect for immersed tunnels is their construction method which has already been briefly described in section 2.2.1. The general idea behind this method is that the tunnel is constructed partly on the desired location and partly prefabricated elsewhere and floated to the location.

The parts that are constructed on site are the open parts on both banks of the waterway: The land parts. They are constructed similar to other deep excavation sites, applying measures to keep water and soil from entering as is illustrated in figure 2.6. On the waterway side of these open parts, an abutment is constructed which allows the tunnel elements to be connected to the land part. Between the two abutments, crossing the waterway, a trench is dredged for the elements to be placed in.

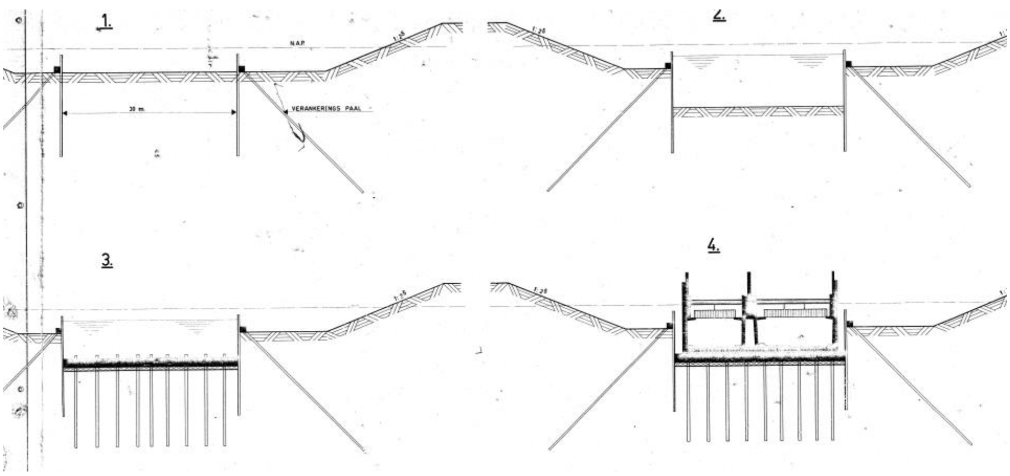


Figure 2.6: The 4 main construction stages of the 1st Beneluxtunnel. This process, or a similar process is used to construct the land parts of immersed tunnels.[6]

The tunnel elements are constructed at another location. They are constructed in relatively controlled circumstances allowing the concrete to be produced relatively watertight. When completed, the elements have to be transported towards their final position. This process is called the OTAO-process[1] in which the following four stages are distinguished:

Floating up When the elements are finished, they are fitted with ballast tanks and they are closed off with bulkheads. The construction site is then filled with water. Consequently, the water tanks are emptied and the elements start to float.

Transportation Once the elements float, they are transported towards their desired location by tugboats.

Immersing When arrived at the desired location, the elements are immersed by refilling the ballast tanks. The elements are usually lowered onto temporary supports. On one side, the newly immersed element is moved towards the previous element (or abutment) and they are connected through a rubber profile running across the circumference of one of the elements creating a sealed space between them. The water inside this space is pumped out resulting in a large pressure difference, pushing the elements tightly together. Finally, extra watertight profiles are installed, the bulkheads are removed and the ballast tanks are replaced with permanent ballast.

Underflowing In the final stage, sand is pumped in the area underneath the element which will support the elements. The temporary supports are removed and the trench is backfilled.

Finally, the road deck can be constructed and all other requirements like lighting and ventilation can be installed.

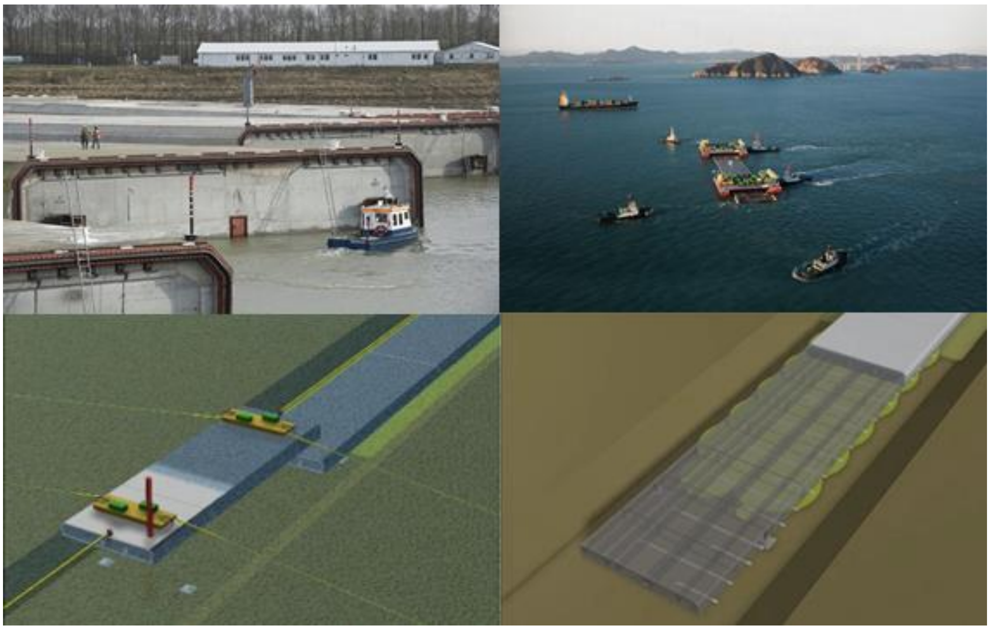


Figure 2.7: This image shows the four stages of the OTAO-process. [1-4]

[1] The abbreviation OTAO-process is made up of the Dutch translations of the stages, namely: Opdrijven, Transporter, Afzinken and Onderstromen.

2.3 Functional analysis

In this section, the functions of the Beneluxtunnel will be discussed. First the function as a traffic connection will be treaded. Subsequently, the function as a passage for navigation will be treated.

2.3.1 Traffic

The Benelux connection

In the economically prosperous 1960s, the infrastructure capacity of Rotterdam became insufficient. In an effort to lower the traffic intensity in the city, a plan was proposed to create a ring of highways around the city of Rotterdam called the "Ruit van Rotterdam". The western part of this ring would consist of a new crossing across the Nieuwe Maas, partially intended to reduce the traffic intensity in the Maastunnel.

As this crossing was planned on the seaside of major port areas, it was clear that a tunnel would be required. A 2x2 highway tunnel was built and finished in 1967.

In the 1980s, traffic intensity increased beyond expectations which resulted in an insufficient capacity of the Beneluxtunnel. In 1993, the decision was made to increase the capacity of the connection by building a second tunnel next to the existing Beneluxtunnel.

The tunnel would not only double the capacity of the connection by adding 2x2 highway traffic lanes, it would also be fitted with passages for a two way metro line, a passage for bicycles and pedestrians and a passage that was initially intended to become a reversible lane, but was later used as a lane for emergency traffic. The 2nd Beneluxtunnel was finished in 2002.

Traffic types

The Beneluxtunnel provides a connection for three traffic types:

Highway traffic	The Beneluxtunnel leads the part of the A4 highway that connects the traffic junctions Kethelplein on the north side and Beneluxster on the south side, underneath the Nieuwe Maas. It is the most seaward road traffic connection crossing the river Nieuwe Maas, making it the preferred connection for many travellers going from or to the regions near the banks of this large stretch of river.
Metro traffic	The Beneluxtunnel is used by metro line C, connecting towns on the southwest of Rotterdam with the centre parts of Schiedam and Rotterdam, and towns on the eastside of the city. It is not the only connection across the Nieuwe Maas.
Bicycles/ pedestrians	Also a line for small personal traffic like bicycles, pedestrians or small motorised vehicles is included. Unlike the other traffic types, the tunnel is accessed by a lift or escalator instead of a ramp.

The layout of the tunnel and its traffic types is shown if figure 2.8.

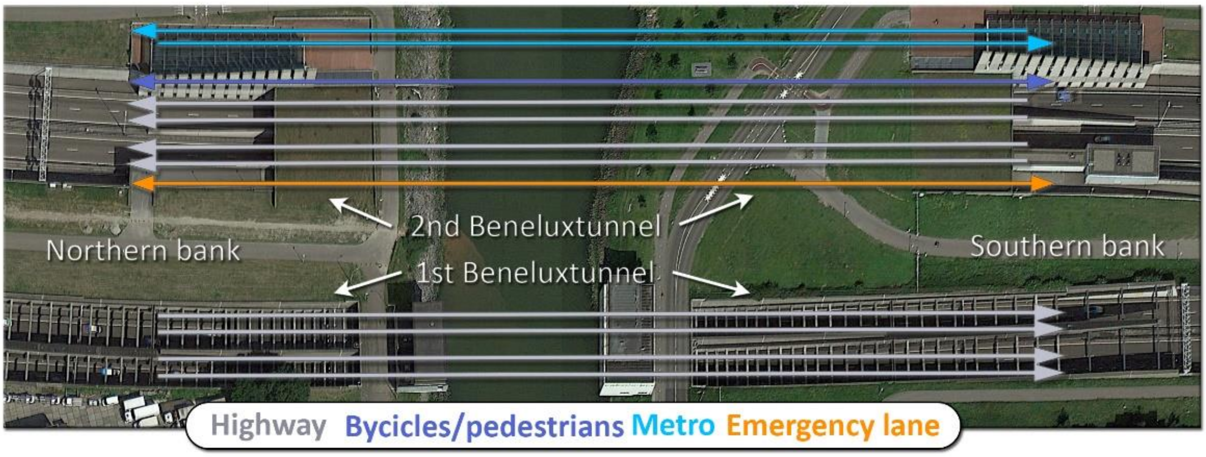


Figure 2.8: This image shows the layout of the traffic types in the Beneluxtunnel. [5]

Future

In the coming years, two major infrastructural projects are planned that will have significant impact on the traffic intensity of the Benelux connection as can be seen in figure 2.9. At the end of 2015, the stretch of the A4-highway directly north of the Beneluxconnection, which has been delayed for decades, will be finished. Also, a new tunnelled connection is planned west of the Beneluxtunnel, crossing the same waterway. This Blankenburgconnection is scheduled to be finished in 2022.

Both projects influence the traffic intensity in the Beneluxtunnel. It seems the overall effect is a decline in traffic intensity in the foreseeing future. According the traffic studies, the traffic intensity of the Beneluxtunnel will decrease from about 130,000 vehicles per day in each direction, down to about 65,000 to 95,000 vehicles per day in each direction, which is about the same level as during the realization of the 2nd Beneluxtunnel.[20, 21]

Figure 2.9: This figure shows the surroundings of the Beneluxtunnel. Existing highways are highlighted orange, while the Benelux connection is highlighted red. The two new connections are displayed as dotted lines. [5]

2.3.2 Navigation

The second function of the Beneluxtunnel is to allow navigation to pass. As explained in section 1.1.1, the desired dimensions of this passage depends not only on the available ship size, but also on the attractiveness of the inland ports. These aspects will be treated in this section.

First the port of Rotterdam will be discussed, next the access channel will be treated and finally, the types and dimensions of navigation will be discussed.

Port of Rotterdam

The port of Rotterdam lies in the delta of the rivers Rhine and Meuse and it is directly connected to sea. Figure 2.10 shows this catchment basin from an unusual perspective which clearly shows the advantageous position of the port, allowing waterway access to a large and economically active area of Europe.

This strategic position allowed the port to become the largest port in the world and to remain this position from 1962 to 2004. Nowadays, Asian ports have outgrew the port of Rotterdam, but it is still the largest port of Europe by far.

Figure 2.10: The catchments of the Rhine and Meus.[9]

The port of Rotterdam originated on the banks of the river Rotte, where currently the city centre of Rotterdam is situated. When both the city and the port grew, the port moved towards the sea. This process greatly influenced the layout of the port which now stretches from the edge of the city towards the sea, increasing in dimensions accordingly.

The Port of Rotterdam can generally be divided into four areas:

Maasvlakte: (1973, 2013) The Maasvlakte 1 and 2 are the latest addition to the port of Rotterdam. The first part was finished in 1973 and the second part in 2013. It was mostly built on new land, acquired by reclamation soil. The Maasvlakte has direct access to the sea and is the deepest part of the port giving access to the largest ships in the world. The main cargo types are containers and dry bulk.

Europoort: (1964) The Europoort lies beyond the Maasvlakte and can be accessed through the Calandkanaal, which runs just south of the navigation channel accessing Rotterdam. This area is used mainly by liquid bulk and accompanying industry.

Botlek: (1955) The Botlek is the predecessor of the Europoort. It is also being used by liquid bulk and accompanying industry, but it is accessed by the normal navigation channel that also accesses Rotterdam. The area reaches up to the Benelux tunnel. As discussed in section 1.1.2, port authorities are planning to give better access to this port by increasing the depth of the navigation channel.

City ports: The city ports are the ports that lie beyond the Beneluxtunnel and are therefore important for this study. This area will be further discussed on the next page.

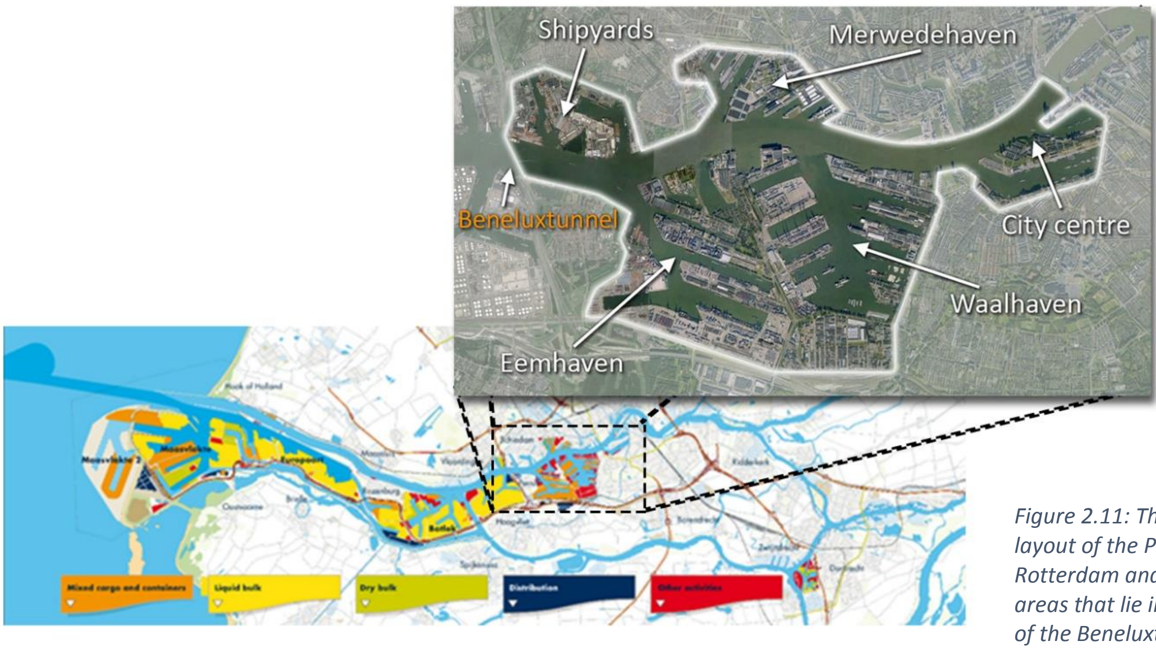


Figure 2.11: The layout of the Port of Rotterdam and ports areas that lie inland of the Beneluxtunnel. [5] [7]

Inland of the Beneluxtunnel lie the city ports. The ports within this area are shown in figure 2.11

Waalhaven and Eemhaven (1930, 1934)	The Waalhaven and Eemhaven are significant parts of the Port of Rotterdam. It houses some large container terminals for both shortsea and deepsea navigation.
City centre (1895, 1905)	The Rijnhaven and the Maashaven are situated near the city centre which is an attractive destination for cruiseships.
Merwedehaven (1930)	The Merwedehaven lies in the business area Nieuw-Mathenesse. This area is known as area for fruit transhipment.
Shipyards (1921, 1924)	The Wilheminahaven and the Wiltonhaven near the northern end of the Beneluxtunnel houses some major maritime and offshore companies that use their shipyards mostly to repair ships or to build special purpose vessels.

Whether these port areas remain functional depends on the rate of urbanisation.[2] The city centre ports are almost entirely urbanised, but the process does not seem to continue rapidly into the other areas. Only the eastern parts of the Waalhaven and the Merwedehaven are slowly changing, but the major port areas do not seem to be threatened in their functionality in the foreseeing future. [22]

Furthermore, the 2nd Maasvlakte was built most on artificial land in an area that was previously part of the North Sea. Expanding seawards means expending into deeper water which can be quite a challenge technically and financially, but can also have a major impact on the environment which makes governmental permission doubtful**.** The Port of Rotterdam authorities have indicated not to see any

[2] Inaccessibility of large ships can make existing harbours less valuable, while the expansion of the city requires housing and commercial areas, driving up the price of land until at some point, it becomes more lucrative to use the land for these urban purposes.

need for expansion of the port until at least 2030. Good utilisation of the existing port areas is expected to be sufficient to accommodate future demands.[23]

Access channel

Various parts of the ports can be accessed through various waterways. The Beneluxtunnel and all that lies inland of the tunnel can best be accessed through the access channel which starts at the sea as the Nieuwe Waterweg, then is called Scheur and at the Beneluxtunnel it is called Nieuwe Maas. From the tunnel to the sea, the distance is about 23 km.

The first 20 km, no port areas are present resulting in a constant depth. Near the city, the depth starts to differentiate. This is partly a result of the depth desires of these port area's but it is also a remnant of the so-called "trapjeslijn", which was an effort to prevent salt water from intruding inland by creating steps in the depths where the salt water would be retained. [24]

An indication of current depths is given in figure 2.12.

Figure 2.12: This figure indicates the depth ranges near Beneluxtunnel [m vs NAP]. The orange lines are crossings. [25]

The Beneluxtunnel is not the most seaward draught obstacle in the access channel. Between the Beneluxtunnel and the sea lies the Maeslantkering, a large moveable storm surge barrier designed to protect the hinterland from extreme floods. It has a sill at NAP -17 m which is being used to lower the movable doors onto. The large costs[3] of the barrier make it unlikely for it to be removed to increase the depth. [26]

Other routes that avoid the Maeslantkering exist, but they would have to be adapted much more radically, including another storm surge barrier and a tunnel, meaning it is unlikely for the access channel to change.

It should be noticed that depth is not the only requirement for a navigation channel as the width should also be sufficient. While the depth of a channel is only about 10 % more than the maximum allowed draught, the width of a two-way navigation channel has to be about 7 times the width of the largest ships. This means that not the maximum depth of the tunnel is important but the maximum depth at the sides of the navigation channel. This aspect will be treated in section 4.3.

[3] The Maeslantkering costs 830 million guilders in 1988, which would be about 715 million euros in 2015.

Navigation

Many vessels cross the Beneluxtunnel but only few are large enough to experience limitations by the presence of the Beneluxtunnel. Let us try to analyse which ship types can be governing.

- First of all, many destinations within the port lie downstream of the Beneluxtunnel, including all wet and dry bulk terminals as they desire being closer to the entrance of the port and further away from residential areas. These ships will therefore not pass the Beneluxtunnel.
- Another interesting ship type that can get very large is the cruise ship, which often ventures into Rotterdam traveling towards the city centre. However, the draught of these ships is relatively modest and therefore these ships will not be governing.
- Given the utilisation of port areas, it would be expected for container ships to be the governing type. This assumption is confirmed by a survey performed for the construction of the 2nd Beneluxtunnel which showed that of all large ships that passed the Beneluxtunnel in one week, all but one were container ships. [27]

Hence, container ships will be the governing ship type to pass the Beneluxtunnel. Currently the largest container ships to pass the Beneluxtunnel are of the Panamax class, having a maximum draught 12.0 m of and a maximum width of 32.3 m.

Figure 2.13 shows the increase in capacity of container vessels since its introduction. The drive behind this increase is the economy of size, explained in section 1.1.3. Several limiting factors can be mentioned, like for instance the design of ships, port infrastructure or the dimensions of important canals and straights. However, there is no reason to assume why these factors would stop the capacity increase. [15]

However, it is not expected for the largest existing ships to visit the city ports, as the Port of Rotterdam has other port areas that are perfectly suitable for these ships. The governing ship dimensions to pass the Beneluxtunnel therefore also depends on the distribution of ship dimensions.

It seems the governing ship dimensions for the Beneluxtunnel cannot be easily determined. This will therefore not be attempted within this study, which will be explained in section 3.3.

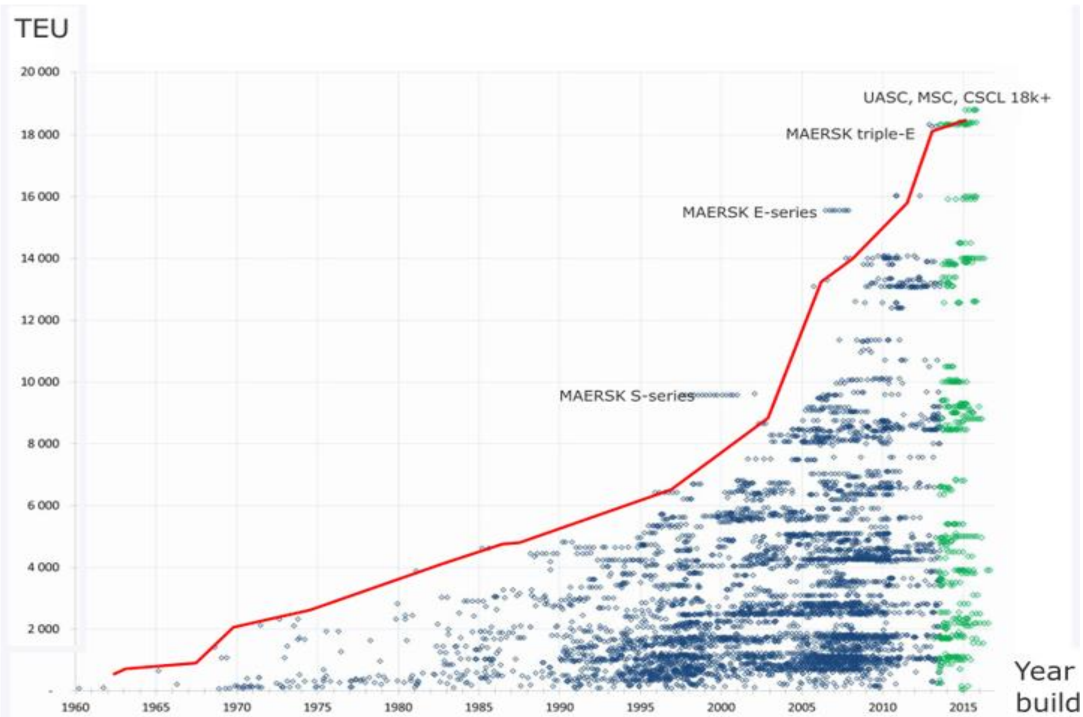


Figure 2.13: The increase of container ship capacity over the past decades[14, 15]

2.4 Technical analysis

In this section, all relevant technical aspects of the Beneluxtunnel will be treated. First, the alignment of the tunnel will be discussed. Next, the important structural components will be treated. The land part and the tunnel will be treated separately. Then, technical issues that have occurred will be briefly mentioned. And finally, some present day issues regarding immersed tunnels will be briefly mentioned.

2.4.1 Alignment

The alignment of the tunnel will be treated by means of standard perspectives, resulting in a horizontal alignment and a vertical alignment.

Horizontal alignment

The horizontal alignment of the Beneluxtunnel is shown in figure 2.14. The 2nd Beneluxtunnel lies east of the 1st one.

This figure clearly shows that the tunnel does not cross the waterway straight. The reason for this, is that the surrounding quays were also present at the time of construction of the first Beneluxtunnel, which can also be seen in figure 6.1. The tunnel had to be built around the existing quay walls, resulting in an oblique crossing with a horizontal curve in the alignment.

Figure 2.14: The horizontal alignment of the Beneluxtunnel [5]

Vertical alignment

Figure 2.15 shows the vertical alignment of the 1st and the 2nd Beneluxtunnel.

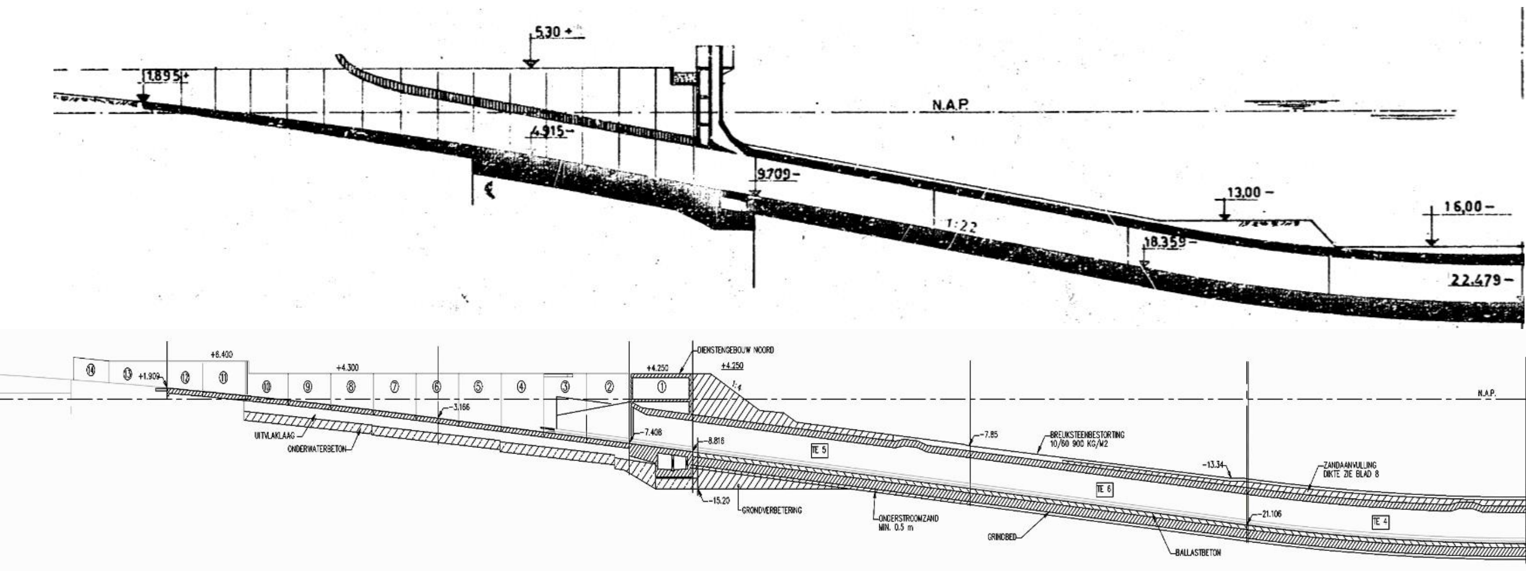


Figure 2.15: The vertical alignment of the northern halves of the 1st (above) and the 2nd (below)Beneluxtunnel [28] [29]

When approaching the Beneluxtunnel, first the road goes up to the height of the pivot dike (NAP +5.4 m) after which the road starts to descent. The transition point lies slightly lower for the 1st Beneluxtunnel (NAP -9.7 m) than for the 2nd Beneluxtunnel (NAP -8.8 m), while at the deepest point, the 1st Beneluxtunnel (NAP -22.5 m) lies slightly higher than the 2nd Beneluxtunnel (NAP -24.2 m).

The extra depth for the 2nd Beneluxtunnel does not simply result in a larger navigation channel as the resulting navigation channel depends highly on the chosen width. This will be further elaborated in section 4.3.1.

Another important aspect of the vertical alignment is the metro in the 2nd Beneluxtunnel, which has a lower maximum slope than the tunnel. To overcome this issue, two solutions are proposed. Firstly, the metro tracks are constructed at an elevated level in the centre parts of the tunnel, which is possible as metro trains need less headroom than motorway traffic. Secondly, the metro is kept tunnelled for about 300 m within the land part as the metro allows a less slope outside.

2.4.2 Land part

The Beneluxtunnel has four land parts. Two for each tunnel and two on each bank. The 2nd Beneluxtunnel is designed such that its construction would not compromise the structural functionality of the 1st Beneluxtunnel, which required for them to be built far enough apart, which depending on the location is about 25 - 35 m.

The land part of a tunnel is generally a giant box, designed to allow entrance to the tunnel part while withstanding water and soil pressures. The layout therefore depends in large extent on the solutions that help to allow this. Three defining aspects can be distinguished:

- The access ramp, which due to its slope functions both as the floor and as the landside wall of the land part.
- The abutment, which functions as the riverside wall of the land part.
- The side walls.

The access ramp

The access ramp (figure 2.16) allows the traffic to access the tunnel part. It therefore has to cover the height difference between the height of the connecting infrastructure and the entrance of the tunnel at the transitional part while crossing the pivot dikes.

The access ramp differs throughout its height. The parts that are above groundwater level are very similar to normal roads. The lower parts have to counteract the water pressure. These parts therefore contain tension piles and an underwater concrete floor. Given the relatively small slope, it is possible to pour the concrete at an angle.

The access ramp is divided over its length into multiple segments that are separated by expansion joints.

Figure 2.16: The access ramp on the Northern Bank [12]

The abutments

The abutment is the structure on the lower side of the access ramp, connecting the access ramp to the tunnel elements. It has numerous functions, namely:

- It houses the transitional part, a closed part of the ramp which creates the transition between the open part of the access ramp and the closed tunnel elements.
- It functions as an abutment for the tunnel elements, meaning a tunnel element should be able to connect to the abutment and it should be able to carry the axial force from the tunnel elements to the soil.
- It functions as a primary flood defence between the waterway and the access ramp.
- It houses a service building.
- It houses a dewatering cellar.

Figure 2.17 shows the abutments of both Beneluxtunnels. These figures however show the abutments inside a construction dock. As the Beneluxtunnels are finished tunnels, the abutments are connected to the elements and embedded in soil.

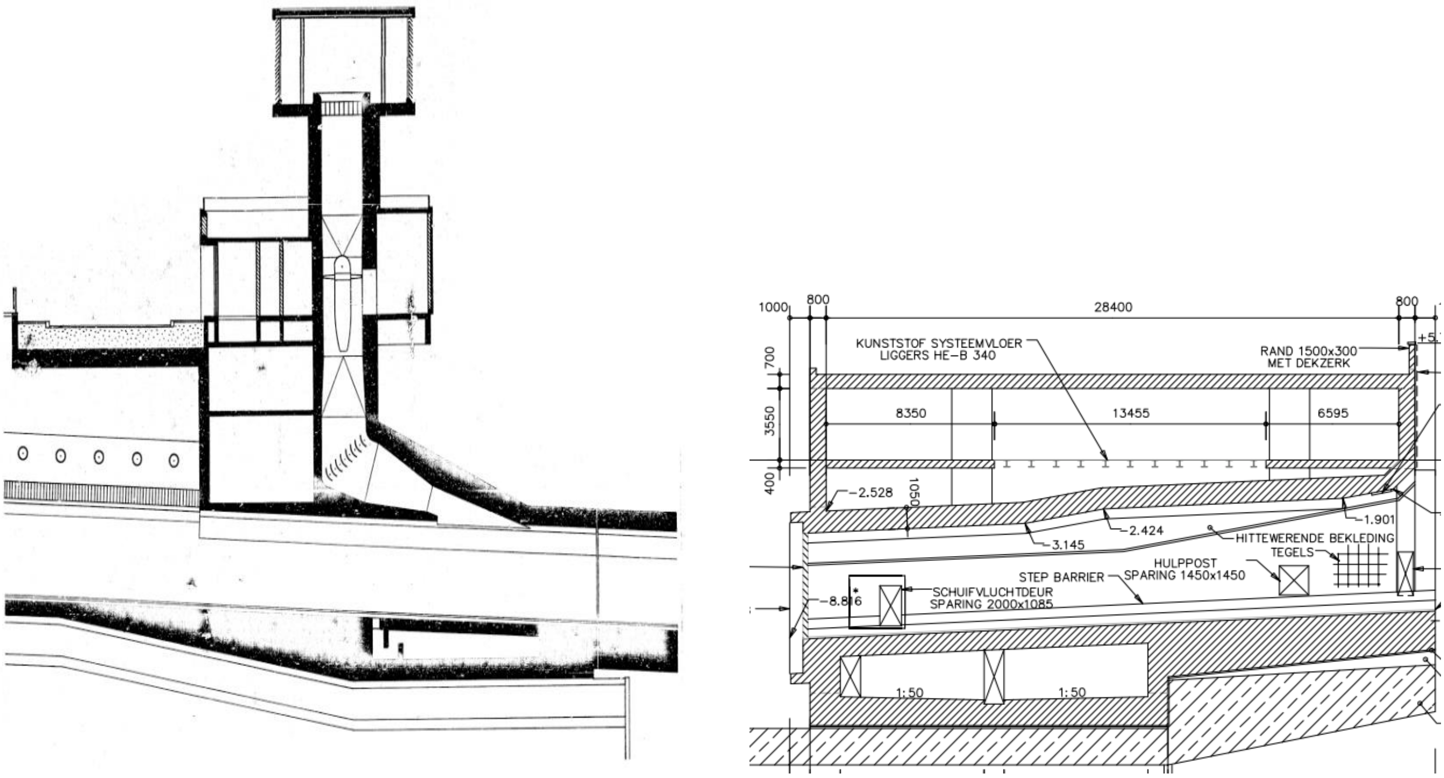


Figure 2.17: A cross-section of the western tube of the northern abutment of the 1st Beneluxtunnel (left) and of the main axis of the southern abutment of the 2nd Beneluxtunnel (right). [8] [26]

Side walls

Each tunnel contains a set of inner walls and a set of outer walls. The outer walls are the walls of the construction dock as is shown in figure 2.18. For the 1st Beneluxtunnel a regular sheet pile wall is used while for the 2nd Beneluxtunnel, a combi wall is used. The inner walls are added later to improve long term safety and water tightness. These walls are made from reinforced concrete for both tunnels.

The walls also contain some horizontal supports. For both tunnels, the outer walls contain anchors. The underwater concrete floor functions also as a horizontal support. The 1st Beneluxtunnel has an additional support using struts above the road, which can also be noticed in figures 2.1 and 2.16.

Figure 2.18: This figure shows the construction dock of the 2nd Beneluxtunnel. It clearly shows the side walls, the underwater concrete floor and the tension piles.[7, 12, 13]

2.4.3 Immersed part

The immersed part of the Beneluxtunnel consists of 2 chains of connected tunnel elements that are embedded into the river bottom. To allow functionality and constructability, many aspects are included into the design of the immersed part of which the ones that are most important for this study, will be discussed in this section.

The aspects that will be discussed are:

- Elements
- Immersion joints
- Closure joints
- Segment joints
- Foundation
- Bottom protection

Elements

The tunnel elements are box-shaped concrete structures. As they have to allow the passage of traffic, their cross-section is relatively constant throughout the alignment of the tunnel. The size and the amount of passages is determined by traffic requirements.

Tunnel elements are designed to be able to float and to be able to sink when a limited amount of ballast is added, meaning their volumetric weight must be just below the volumetric weight of the water around it. For the cross-section of this means that about 40% of the area must be concrete, which is quite a lot and therefore allows the floor, walls and roof to be relatively thick.

Figure 2.21 shows the tunnel elements of both tunnels during construction. Figures 2.19 and 2.20 show the cross-sections of the 1st and the 2nd Beneluxtunnel.

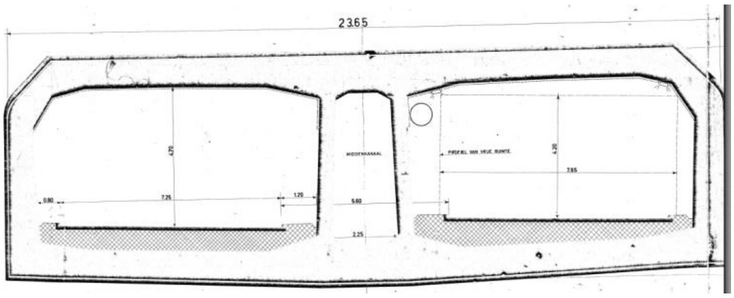


Figure 2.19: The cross-section of the 1st Beneluxtunnel [30]

Notes regarding the 1st Beneluxtunnel:

- The immersed part of the tunnel consists of 8 elements.
- The elements are about 24 m wide, 8 m high and 93 m long.
- Each element consists of 5 segments of about 18.6 m long.
- The elements are coated with a watertight layer of about 10 cm thick.
- The elements were constructed next to the land parts, in a dock which is now the Madroelhaven.

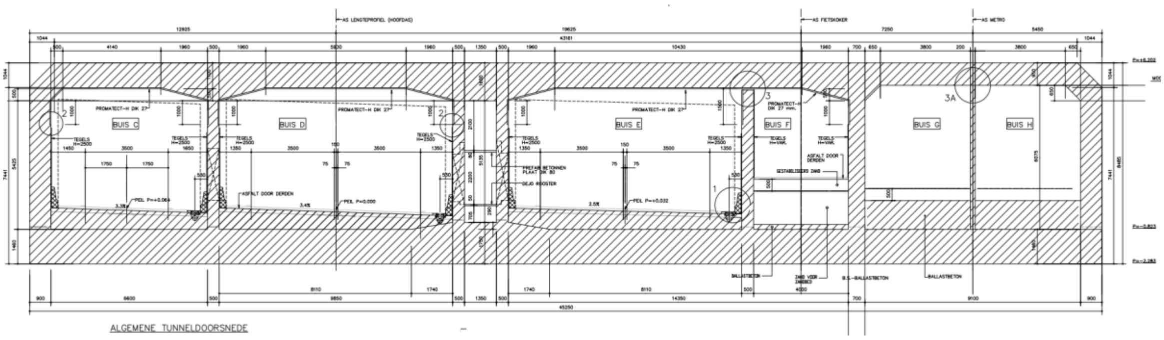


Figure 2.20: The cross-section of the 2nd Beneluxtunnel [31]

Notes regarding the 2nd Beneluxtunnel:

- The immersed part of the tunnel consists of 6 elements.
- The elements are 45 m wide, 8.5 m high and 140 m long.
- Each element consists of 7 segments of about 20 m long.
- The elements were constructed at building dock "Barendrecht" which is about 20 km sailing removed from the Benelux site. This dock has also been used for other immersed tunnels.

Figure 2.21: The elements of the 1st (left) and the 2nd (right) Beneluxtunnel during construction [26, 27]

Immersion joints

Immersion joints are the joints between the elements, at the location where two separate elements were connected during the immersion process. These joints are designed to provide a watertight space during the immersion process and to remain watertight during the lifetime of the tunnel. To allow this, the following components are used:

- GINA-profile
- OMEGA-profile
- Shear key

On one end of each element, a rubber profile called a GINA-profile is mounted. This profile follows the outer circumference of the elements, connected to the concrete through a metal profile. When the elements are connected, the GINA-profiles are compressed, resulting in a watertight connection.

The GINA-profile has to be able to resist the full axial compression force that occurs when the elements are connected and therefore have to be fairly strong.

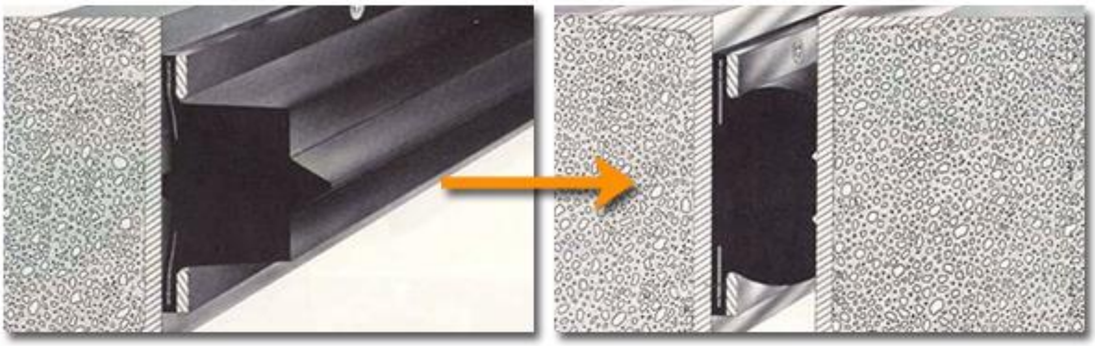


Figure 2.22: The GINA-profile in uncompressed (left) and compressed (right) state.[32]

In addition to the GINA-profile, a secondary rubber profile is installed along the inner circumference of the tunnel at the immersion joints to increase safety against leakage. This secondary seal is usually provided by the OMEGA-profile. This profile is installed after the immersion process. It consists of two metal strips that are securely bolted onto the concrete, connected by a rubber sheet.

The OMEGA-profile must be able to withstand the full force of the water in case the GINA-profile fails and must therefore be able to provide at least the same level of safety as the GINA-profile. This is usually tested after instalment by injecting water under pressure that is at least similar to the hydraulic pressure acting from outside. To maximise the pressure capacity, the profiles are reinforced with aramid fibres and have a radius.

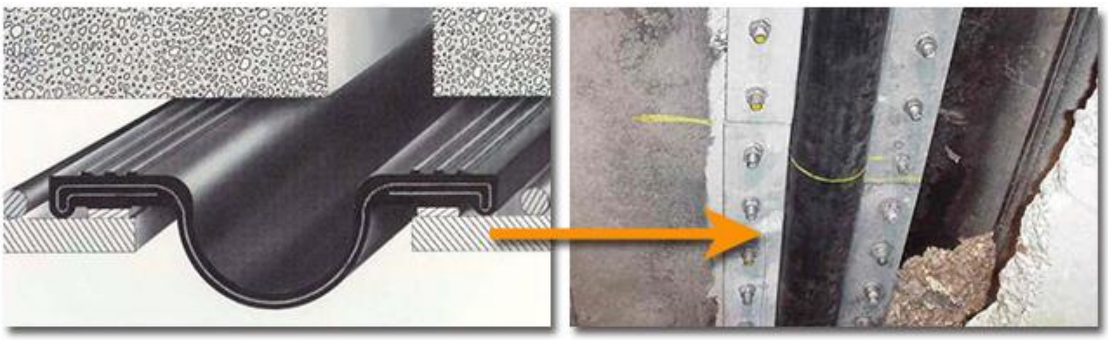


Figure 2.23: The omega profile before (left) and after (right) applying.[32]

If only the rubber profiles were used, possible complications could occur during the connection of the elements as well as during the lifetime of the tunnel regarding the alignment of the elements relative to each other. Therefore, a shear key is used.

During the immersion process, a nose/chin mechanism is used to assure the elements are perfectly aligned as is explained in section 2.2.3. However, once both profiles are installed and water tightness is secured, reinforced concrete is poured around the inner circumference of the tunnel. This concrete has multiple functions, namely:

- It functions as a shear key, preventing vertical and horizontal displacements of the elements relative to each other which could damage the tunnel as well as the rubber profiles.
- It provides a straight surface for the finishing of the tunnel interior.
- It is part of the ballast concrete.

Usually, this component is applied such that limited axial movement is still possible.

Figure 2.24 shows the immersion joint for the Beneluxtunnels

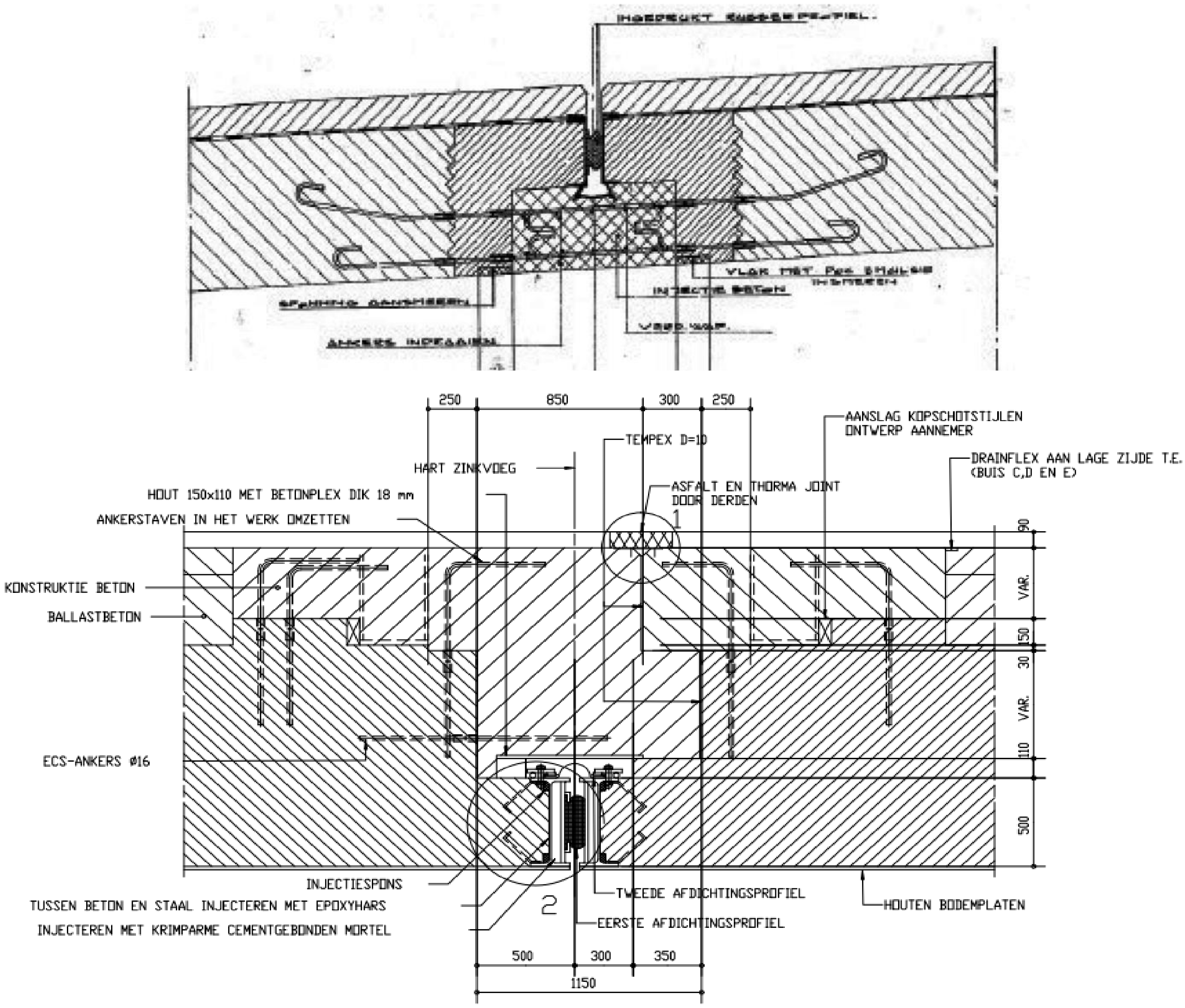


Figure 2.24: This figure shows a cross-section of the immersion joints of the 1st (above) and the 2nd (below) Beneluxtunnel.[7],[14]

Some comments regarding the 1st Beneluxtunnel:

- For the secondary watertight seal, the OMEGA-profile is not used. Instead, this seal is provided by pre-shaped 3 mm thick metal strips.
- The concrete finishing block is secured onto both elements resulting in a monolithic connection meaning all types of movements are restricted, including rotations.

Some comments regarding the 2nd Beneluxtunnel:

- Two types of GINA-profiles were used to withstand the large pressure differences between the lower and the higher situated joints
- As large deformations were expected, a special OMEGA-profile is used, containing an increased radius

Closure joint

Once all elements are immersed, a gap remains. To close this gap, another procedure is required than for the immersion joint. Triangular concrete segments are lowered into the gap creating a wedge which prevents the elements from moving towards each other when the pressure is decreased. Next, the gap is sealed off with metal plates, placed by divers. The inside can now be emptied similar to immersion joints and the joint can be finished. This process is shown if figure 2.25.

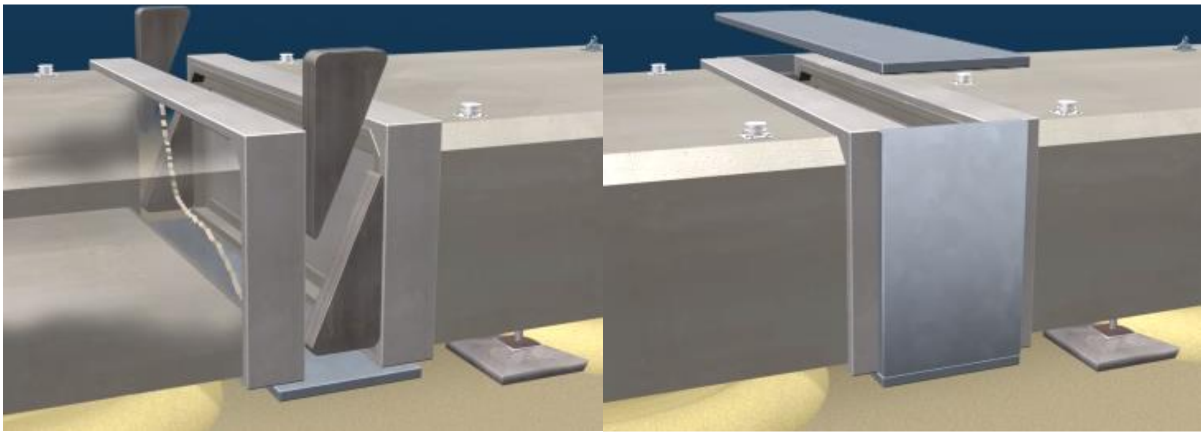


Figure 2.25: Inserting the wedge set (left) and closing of the space (right), creating a closure joint[33]

Figure 2.26 shows the closure joints of the Beneluxtunnels. The closure joint of the 1st Beneluxtunnel is constructed very differently from the example explained above. The function of the wedge was performed by gravel, to which concrete was added later.

The closure joint of the 2nd Beneluxtunnel is very similar to the example above.

The closure joint of the 1st Beneluxtunnel is located between the last element and the northern abutment. The closure joint of the 2nd Benelux-tunnel is located between elements 5 and 6, which is the second immersion joint from the northern abutment.

For both tunnels, the closure joint is a monolithic connection.

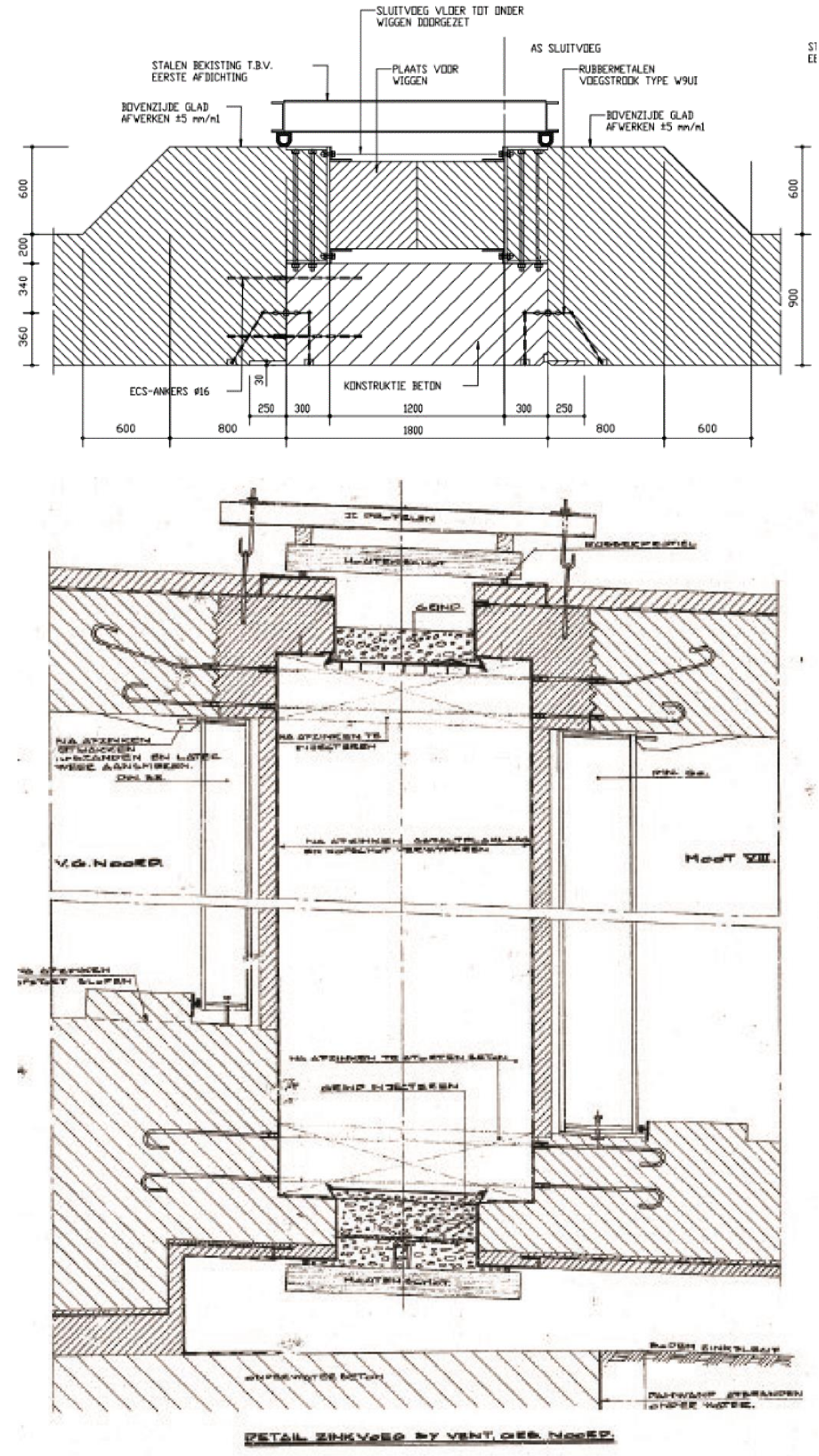


Figure 2.26: This figure shows a cross-section of one of the outer walls of the 2nd Beneluxtunnel in which clearly one of the wedges is shown (above) and it shows a cross-section of the roof and floor of the 1st Beneluxtunnel in which instead of a wedge, gravel is used to withstand the compression force (below) [13]

Segment joints

The segment joint is the type of joint that splits the elements into segments. The most important reasons to incorporate segment joints are displacements caused by either temperature or settlement, which will be discussed in section 5.4. The use of segments is also beneficial for the water tightness of the concrete.

The following components are important regarding segment joints:

- W9Ui-profile
- Shear key

To prevent leakage at these joints, a rubber profile following the outer circumference of the elements is used. The profile used in most recent tunnels is the W9Ui-profile. On both sides, the rubber connects to a metal strip which is casted into the concrete of the tunnel element resulting in a completely watertight connection.

To prevent vertical and horizontal displacement, a shear key is implemented in the design of the tunnel lining. Three types of systems are used, the dowel and socket system, the shear key system and the collar system. The Beneluxtunnels both have a collar system. Such a system is shown in figure 2.27.

With the collar system, the face of the tunnel segments contain profiles that fit into each other. An inner collar fits into the outer collar of the adjacent segment. These reinforced concrete collars provide the shear capacity of the joint.

The collars in the floor and roof provide shear capacity in the Z-direction, transferring the forces to the inner and outer walls. The collars in the outer walls provide shear capacity in the X-direction, transferring the forces to the floor and the roof of the element.

It must be mentioned that in practice, the collar system is only partly responsible for absorbing shear forces. The friction between the concrete faces of the inner and outer vertical walls that are being compressed together under large axial loading due to the hydraulic pressure, absorbs a large part of the load.[34]

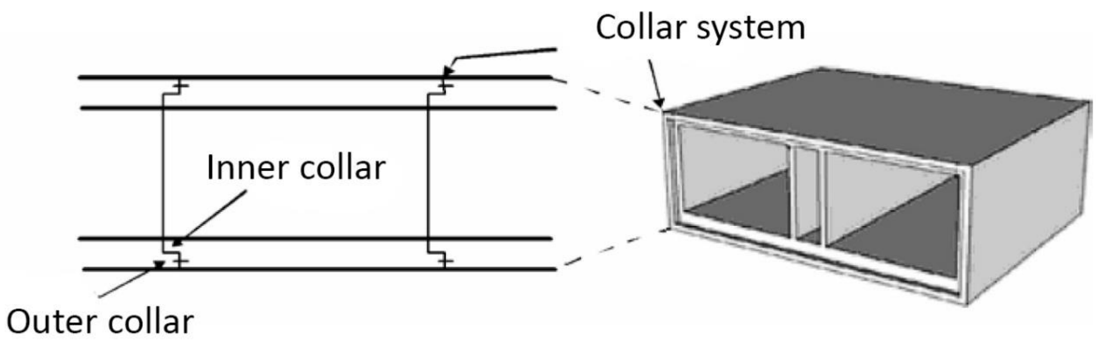


Figure 2.27: Collar system in immersed tunnel sections [38]

Figure 2.28 shows examples of segment joints of both Beneluxtunnels

Some comments

- Both tunnels contain plates on the outside to prevent soil from accumulating in the joints.
- The 1st Beneluxtunnel does not have a standard W9Ui-profile but a comparable alternative.

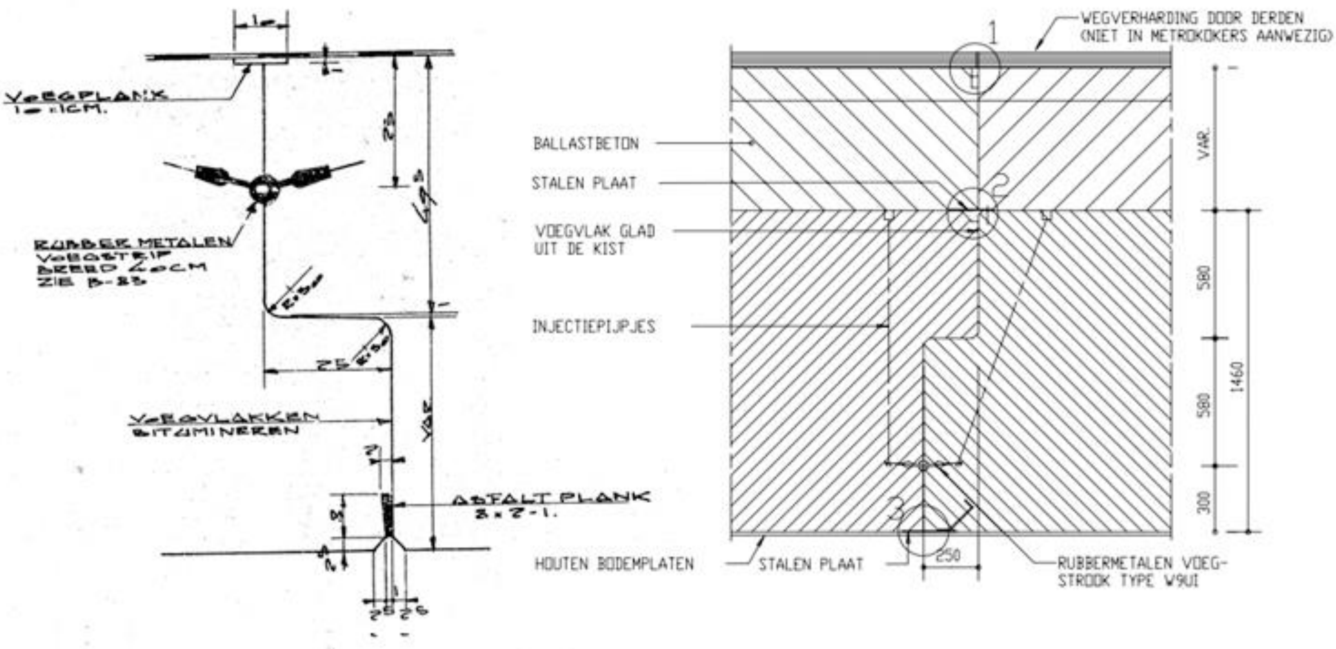


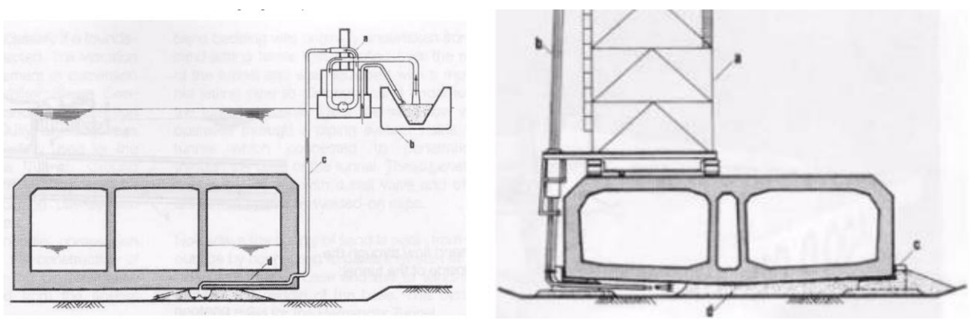
Figure 2.28: This figure shows the cross-section of a segment joint in the roof of the 1st Beneluxtunnel and a segment joint in the floor of the 2nd Beneluxtunnel.[35] [36]

Foundation

For most civil structures in Dutch soft soil, piled foundations are required to prevent differential settlements. As this would be costly and very hard to apply due to the large depth, it is usually chosen not to use piled foundations for immersed tunnel elements. This is possible because of the limited immersed weight of the tunnel elements which results in relatively low pressure on the subsoil.

Instead, the tunnels are usually immersed on temporary foundation tiles on which their position can be corrected by jacking systems. Once the elements are connected, sand is flushed or flown underneath, as is explained in section 2.2.2 and shown in figure 2.29

Figure 2.29: These figures show the flowing of soil (left) and flushing of soil (right) underneath tunnel elements.[4]



For the Beneluxtunnels, alternative methods were used. These differences are:

- For the 1st Beneluxtunnel, a special type of foundation tiles were used on which not only the vertical position could be corrected but also the horizontal position. This system is shown in figure 2.30.
- The 2nd Beneluxtunnel was initially planned to be lowered on tiles, which were all placed on the bottom of the trench, but in later stages of design it turned out that the return current caused by passing ships might be able to knock the elements of the tiles. It was therefore decided to use a different foundation method. Gravel ridges were placed on which the elements were directly placed. Initially there were problems with the accumulation of silt, but when the distance between the ridges was sufficient, there was

enough time to lower the elements. Eventually, only the outer elements were founded on tiles, the middle four elements were successfully founded on gravel ridges.

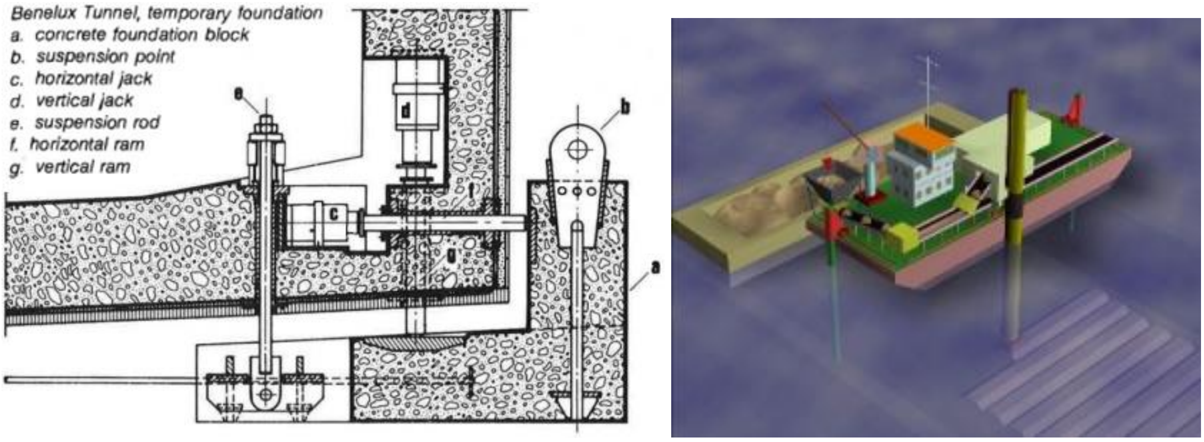


Figure 2.30: This image shows the foundation tiles of the 1st Beneluxtunnel (left) and the placement of gravel ridges for the 2nd Beneluxtunnel (right)[3, 37]

Bottom protection

Bottom protection as used in case of immersed tunnels has two functions:

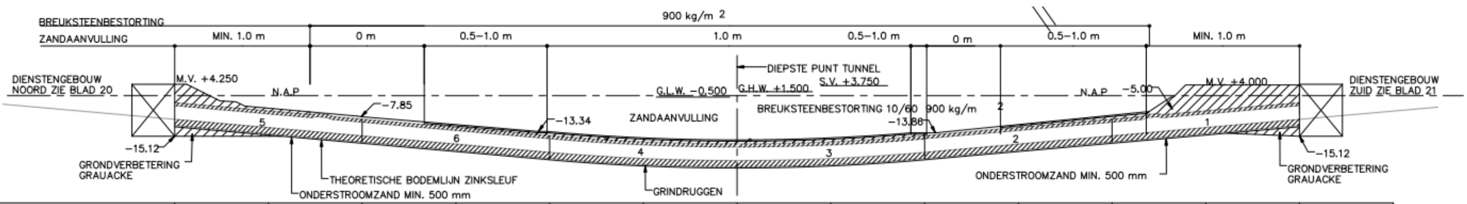
- To protect the tunnel elements from impact loads from navigation.
- To prevent soil around the elements from being flushed away.

A third function which is often ascribed to bottom protection is that it is used to keep the tunnel elements from floating up. Although the layer gives extra weight, providing additional safety, usually the ballast weight within the tunnels is designed to fully provide safety against floating up.

For the 1st Beneluxtunnel, information is missing. It is therefore assumed that the bottom protection on top of the 1st Beneluxtunnel is similar to the 2nd Beneluxtunnel.

For the 2nd Beneluxtunnel, the bottom protection differs throughout the length of the tunnel. Underneath most parts of the navigation channel, a layer of 1 m soil and 0.5 m rubble is dumped. Near the edges of the channel, the amount of soil is decreased down to 0 m to gain extra depth. The rubble stops near the land parts, where the amount of soil increases up to dike height.

Figure 2.31 shows where the bottom protection is applied.



KILOMETRERING	3.465.145	3.395.145	3.325.145	3.255.145	3.185.145	3.115.145	3.045.145	2.975.145	2.905.145	2.835.145	2.765.145	2.695.145	2.625.145
B.K. DAK TUNNEL	−2.61	−5.69	−8.77	−11.85	−14.93	−17.23	−18.00	−17.23	−14.93	−11.85	−8.77	−5.69	−2.61
B.K. ZANDAANVULLING	+4.25	−4.69	N.V.T.	−11.29	−13.93	−16.23	−17.00	−16.23	−14.35	−11.27	−7.77	+4.00	+4.00
B.K. BESTORTING MAX.	N.V.T.	N.V.T.	−7.88	−10.79	−13.43	−15.73	−16.50	−15.73	−13.86	−10.77	−7.27	N.V.T.	N.V.T.
TYPE FUNDERING T.E.	ZAND	ZAND	GRIND	GRIND	GRIND	GRIND	GRIND	GRIND	GRIND	GRIND	ZAND	ZAND	N.V.T.

LANGSDOORSNEDE OVER AS LENGTEPROFIEL (AANVULLEN)

HOR. SCHAAL 1:2000
VER. SCHAAL 1:1000

Figure 2.31: The application of bottom protection on the 2nd Beneluxtunnel [37]

2.4.4 Present day issues

For the Beneluxtunnel, the following issues were documented. Only issues regarding the 2nd Beneluxtunnel were found. For the 1st Beneluxtunnel, no issues were found.

- Three months after the 2nd Beneluxtunnel was immersed and connected, cracks were located in the outer walls near the closure joint. It is expected that the pressure on the wedges became too large for the adjacent walls, possibly resulting from temperature increase in the weeks before. [38]
- Also, near the southern abutment, the connecting element was subjected to a settlement of about 10 cm, likely to be a result from the increased surcharge. Consequently, the OMEGA-profile was significantly deformed as is shown in figure 2.32. The deformations were within tolerances and accepted. [39]
- In 2005, a leakage was discovered in the metro tube near one of the segment joints. It was repaired by injecting fluid as is shown in figure 2.32. [39]
- Also, a leak was discovered and repaired in one of the land parts. [39]

Figure 2.32: The deformed OMEGA-profile (left) and the leakage and repair in the metro tube (right). [39]

2.5 Summary

In this chapter, a general introduction has been given to the subject of immersed tunnels and the specific Beneluxtunnel case. All background information and technical details required to understand the subject well enough to make the first design decisions have been discussed.

Immersed tunnels

The immersed tunnel as a method of crossing a waterway has originated in the beginning of the 20th century in the US, but the concrete rectangular alternative was introduced in 1942 with the Maastunnel. The most important function of an immersed tunnel is to provide a continuous passage for traffic while not disturbing navigation. The land part of an immersed tunnel is constructed on site and function as an abutment for the immersed part. It is constructed similar to a large building pit. The tunnel elements are constructed elsewhere and are moved to their final location using the OTAO-process. Connected, they form the immersed part of the tunnel.

Functional analysis

The 1st Beneluxtunnel is built in 1967 as part of the ring of Rotterdam. In 2002 a second tunnel was added to prevent capacity problems. Together, the tunnels houses 8 highway traffic lanes, 2 metro lanes, 1 emergency lane and a passage for bicycles and pedestrians. In the near future, the finishing of the A4 and the construction of the Blankenburgtunnel is expected to decrease the traffic intensity in the Beneluxtunnel.

The Beneluxtunnel crosses the Nieuwe Maas which is part of the approach channel of a significant part of the port of Rotterdam. The current depth of the navigation channel near the Beneluxtunnel is about NAP -14.5 m, also the width of the navigation channel is very important is the influence of safety margins is much larger for the width. Currently, the channel is allows maximum Panamax vessels. If the size of the channel is increased, container vessels are expected to be governing. Their future size however is very uncertain and will not be determined within this study. Depth increase is however limited by the Maeslantkering at a depth of NAP -17.0 m.

Technical analysis

The alignment of the 1st Beneluxtunnel is consists of a horizontal and vertical curve. The alignment of the 2nd Beneluxtunnel is designed such that it allows the same navigation channel.

The land part of the Beneluxtunnel consists of the following components:

- Access ramp
- Abutment
- Side walls

The immersed part of the Beneluxtunnel consists of the following components:

- Elements
- Immersion joints
- Closure joints
- Segment joints
- Foundation
- Bottom protection

Chapter 3

Basis of design

Figure 3.1: These foundation piles are the literal basis of the land parts of the 1st Beneluxtunnel [40]

3.1 Introduction

In this chapter, the foundation of the design process will be laid. All aspects that are required to understand the design process will be given. This chapter can be interpreted as both the end of the diverging phase of the design and the beginning of the converging phase of the design.

Subjects

The following subjects will be discussed:

Design options	First, a general outline of the available design options will be given, which could be interpreted as the conclusions of the diverging phase of the design. (Section 3.2)
Design strategy	Next, the design strategy will be proposed, which could be interpreted as the initiation of the converging phase of the design. A design philosophy will be proposed which will be used to create a strategy for elaborating the design options. (Section 3.3)
Design specifications	In this section, certain design conditions and assumptions will be proposed. (Section 3.4)
Boundary conditions	Finally, certain boundary conditions regarding the Beneluxtunnel will be given. (Section 3.5)

3.2 Design options

In this section, a general outline of the available design options for the Beneluxtunnel will be presented. The findings within this section are based on the analysis treated in chapter 2 and could be regarded as the result of the first design phase according to the approach treated in section 1.2.3.

The design options are divided into three subjects:

- General design options
- Functional design options
- Technical design options

Finally, also the interdependencies of the design options will be discussed.

3.2.1 General design options

General design options are design options that apply to the Beneluxtunnel on system scale. On this scale, the main consideration is whether it would be more valuable to try to adapt the existing system or to create a new system that would replace the old one instead, as is indicated in figure 3.2.

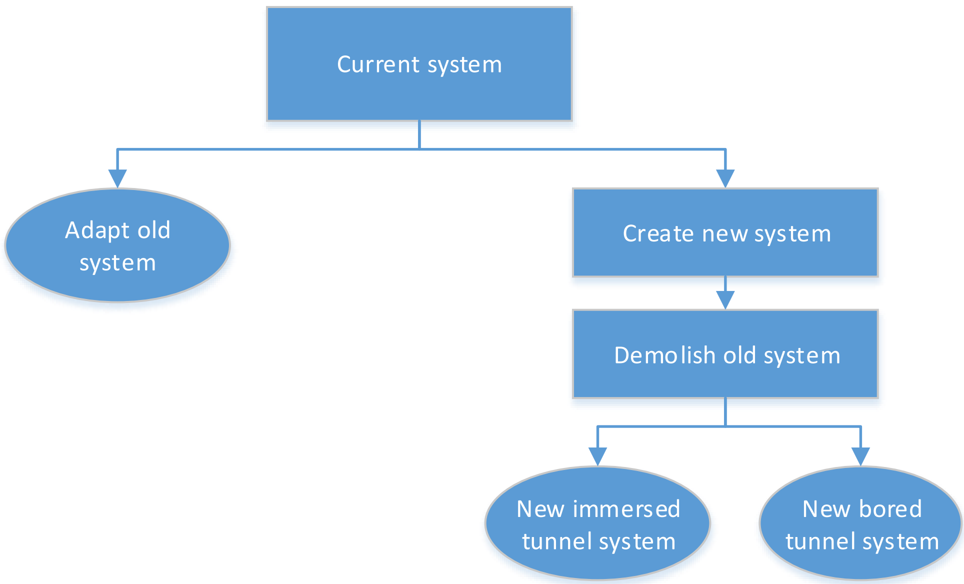


Figure 3.2: General design options

The adaptation of the old system will be discussed in the coming sections. The creation of a new connection could however be good alternative. At this point, the following statements regarding such a new connection can be made:

- One cannot simply build a new tunnel as the old tunnel would still be a draught obstacle for navigation. Therefore the old tunnel system will need to be demolished.
- For a new connection, a tunnel is still the only valid connection method as connection methods lying above the water level would be an obstacle for navigation or would significantly lower the capacity of the connection.
- The new tunnel system can be an immersed tunnel system or a bored tunnel system. The in situ tunnelling method is regarded not feasible as this method would be hard and costly to build, but more importantly, it would significantly hinder navigation.

3.2.2 Functional design options

Functional design options are the options that do not directly influence the tunnel itself, but instead influence the boundary conditions of the tunnel, which might be very beneficial for the technical part of the design. However, one must keep in mind that affecting the functional requirements of the tunnel will also influence its value. Figure 3.3 indicates the most important functional design options.

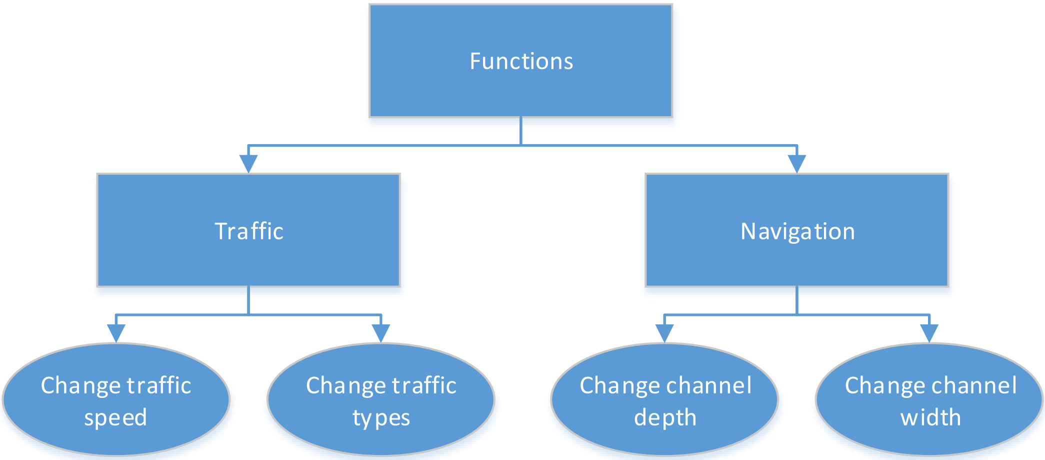


Figure 3.3: Functional design options

Change traffic speed — It is possible to change the maximum speed for the road traffic. This will allow different slopes and radii for the vertical alignment of the tunnel.

Change traffic types — It is also possible to disallow certain traffic types, which could also have beneficial effects on the traffic requirements.

Change channel depth — Increasing the depth of the navigation channel is the objective of this study so it might be strange to regard this as a design option. However, the amount of depth increase is not defined. Small or large depth increase require different lowering strategies and could both turn out beneficial.

Change channel width — Besides the depth of the channel, also the width of the channel is important. It is important for the boundaries of the technical tunnel design, but it is also important for the functionality of the navigation channel. Changing the width of the channel might therefore be beneficial for the design.

Important for all these functional design options is their effect on the boundary conditions of the technical design, but also their influence on the value of the design.

3.2.3 Technical design options

The technical design is the part of the design that concerns changing actual physical aspects of the tunnel. Similar to section 2.4, the land part and the tunnel part will be treated separately.

Land part

Regarding the land part, basically three options exists as shown in figure 3.4. These options differ in the amount of effort required to be realised and in the imposed boundary conditions for the immersed part of the tunnel.

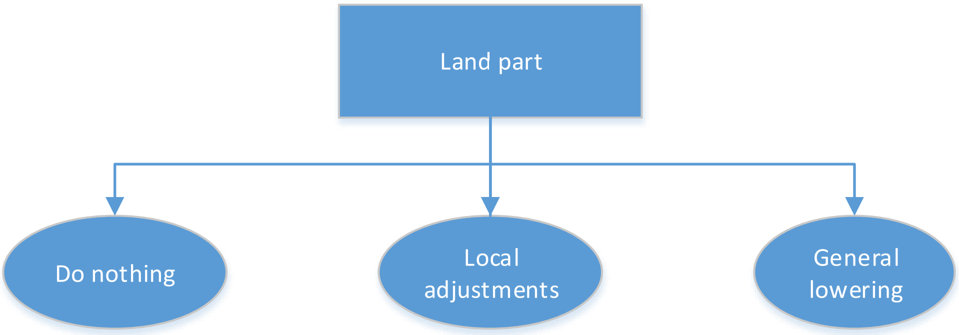


Figure 3.4: Design options land part

Do nothing Doing nothing means maintaining the land part as built. The amount of effort to accomplish this is minimal, but also the boundary conditions for the immersed part remain unchanged.

Local adjustments Local adjustment refers to locally adjusting the layout of the transition point to adjusting the connection with the immersed part. The excess space of the dewatering cellar can be used for this. This will require some effort but it will also improve the boundary condition for the immersed part, mainly by allowing an increase in slope.

General lowering General lowering refers to all types of lowering that affect the presence or position of the underwater concrete floor. This would require very much effort, as this would significantly affect the execution method. However, the boundary conditions for the immersed can be adjusted to allow relatively unlimited rotation and translation to the trajectory.

.

There are two tunnels with each two land parts that do not necessarily have to be treated similar.

Immersed part

The design options of for immersed part are shown in figure 3.5. These options are subdivided into two groups, based on the decision whether to adjust the position of the elements or not. Adjusting the positioning of the elements will require significantly more effort, but this will also allow much more lowering. If one chooses to do so, a choice in construction method remains.

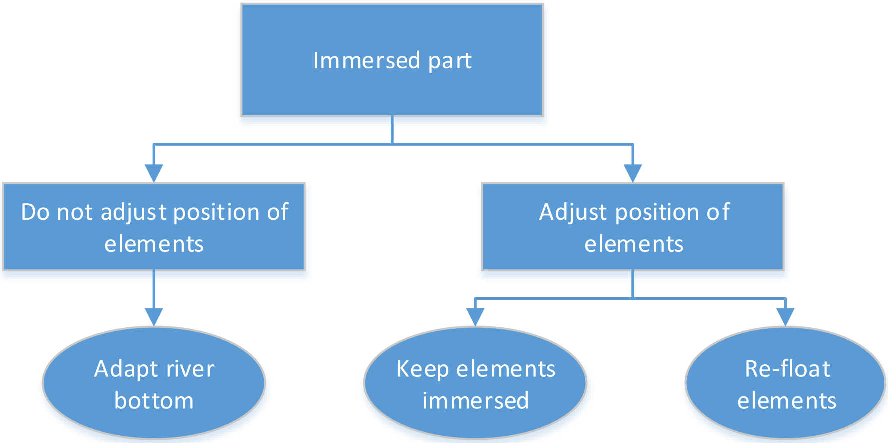


Figure 3.5: Design options immersed part

Adapt river bottom	To adapt the river bottom, the soil and bottom protection on top of the tunnel could - at some critical locations around the edges of the navigation channel - be replaced with a thinner type of bottom protection, allowing a limited increase of depth without adjusting the positions of the elements.
Keep elements immersed	To adjust the position of the elements while they remain immersed and connected to each other, one could try to utilize the limited freedom of rotation found in the joints. Combined, these rotations could produce a significant amount of lowering.
Re-float elements	The re-floatation of the elements can be interpreted as the inversed version of the immersion process. The elements therefore need to be disconnected from each other, re-floated and removed from the construction site, later to be re-immersed and reconnected at a lower position. It could also be decided to re-immerse newly constructed elements.

Note: These design options can be combined.

3.2.4 Interdependencies of the design options

In the previous two sections regarding the functional design options and the technical design options, many of the subjects were mentioned to be influencing each other. Figure 3.6 gives an indication of these interdependencies.

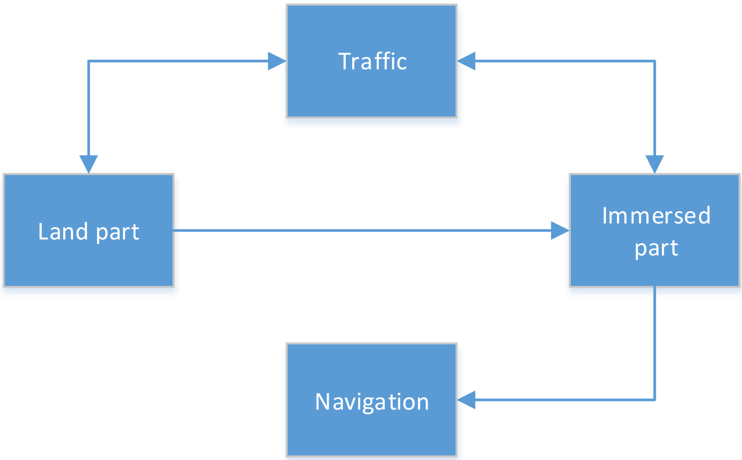


Figure 3.6: The interdependencies between the design options

The following relations can be distinguished:

- The decisions regarding traffic requirements determine the boundary conditions for the vertical profile and for the cross-section of the tunnel, which concerns both the land part and the immersed part. Subsequently, the actual slopes and radii of the land part and the immersed part determines the maximum speed and therefore influences traffic.
- The decisions regarding the land part determine the position and slope of the transition point, which is the most important boundary condition for the immersed part.
- The decisions regarding the immersed part determine the height of the river bottom along the width of the waterway, which is the most important boundary condition for the navigation channel.
- The decisions regarding the navigation channel determine the maximum size of ships, which is the objective of this study.

These relationships will be used throughout this study.

3.3 Design strategy

In this section, the design strategy will be explained. First the design philosophy will be treated which will explain why a certain approach to the design is maintained. Subsequently, the design strategy regarding technical feasibility and evaluation will be discussed.

3.3.1 Design philosophy

The definition of design

Before elaborating the design, an effort is made to determine what design actually is and what aspects are important in order to make a good design.

The following statements regarding design are proposed:

- Design can be defined as the process of optimising the value of a product by defining its properties before the product is realised.
- The value of a product can solely be determined by the functionality of the product throughout its lifetime.
- The course of the functionality of a product throughout its lifetime as well as the duration of its lifetime can only be determined afterwards, meaning that the value of a product is unknown during the design phase and can therefore only be estimated.
- Value is subjective.
- Civil goods like large scaled infrastructural objects represent common interest of society, meaning value can in large extent be democratised and therefore objectified.

This approach to design will be used throughout this study.

Design criteria

According the definition of design as presented above, a good design delivers a product with a large value. Estimating the value of the product is therefore an important aspect of design as not only a bad design can result an invaluable product, but also a good design with a bad estimation of value can result in an invaluable product.

Consequently, estimation of value will play an important role in rating the design options. Effort will be made to include all differences between the options that might be an important driver of functionality.

An important aspect regarding value is feasibility. If a product is infeasible, it will not be made. The value of a non-existing product is zero, or even negative if one would include research costs. Feasibility therefore is a very important criterium in design and therefore also the limitations of the design options will be determined.

However, it must be mentioned that only the technical feasibility will be determined. Other types of feasibility are, if required, expressed in the value of the design options in terms of risk.

Design optimisation

To make well founded design decisions, evidence is required. The better the evidence, the more certain the decision. However, gaining evidence requires energy and time and it is therefore not valuable for the design process to extensively investigate every subject in order to maximise the strength of the evidence. An optimisation should therefore be sought in which sufficient certainty can be provided with the least amount of effort.

In practice, for early stages of design, this optimisation process comes down to a decision between providing qualitative or quantitative evidence as is shown below:

Qualitative	Quantitative
Little amount of work	Large amount of work
Adaptable	Fixed
Little information required	Much information required

Thus, it is wise to start with qualitative evidence based methods and only if evidence turns out to be too weak, move onto more quantitative evidence based methods. This approach will be used throughout this study.

3.3.2 Technical feasibility

The technical feasibility of the design options will be estimated by determining the limitations for the designs. For the design options discussed in section 3.2, this has the following implications:

- The general design option of building a new tunnel can be regarded technically feasible and will therefore not be elaborated.
- The functional design options will be expressed in terms of value and will therefore not be discussed in terms of feasibility.
- The technical feasibility of the technical design options are most interesting. The two major contributing aspects are structural capacity and constructability, which will be discussed below.

Structural capacity

The structural capacity of the technical part of the design is one of the most governing aspects of this study. As the technical design covers adapting the existing tunnel system, the design options are merely deviations from the existing tunnel. This aspect will be utilized in determining the structural capacity of the design options.

First, the mechanics of the existing tunnel will be analysed and the structural capacity will be determined. Next, the impact of adjustments according to the design options will be determined. Combined, this will give an indication of the structural capacity of the tunnel in the conditions of the design options. Consequently, the design options do not have to be strictly defined in order to estimate the structural capacity.

In terms of design optimisation, the immersed part of the design will be more extensively investigated than the design part, mainly because the easy access of the land part relative to the immersed part allows more adjustments regarding to strengthening and stabilising, which makes the land part less critical in terms of technical feasibility.

Constructability

The other contributing aspect to the technical feasibility is constructability. As the construction method is one of the most defining aspects for immersed tunnels, this aspects is very important.

Regarding the land part, the only options that are interesting in terms of constructability are “local adjustments” and “general lowering” as the option “do nothing” requires no constructional effort.

Regarding the immersed part, the options that require adjustment of the position of the elements – “keep elements immersed” and “re-float elements” – are evaluated more interesting and will therefore be elaborated. The third option “adapt river bottom” requires little constructional effort and will therefore not be elaborated.

For these four remaining design options, a construction methods will be proposed. They will be elaborated up to a level that is required to, with a certain degree of certainty, prove constructability.

Report

The structural analysis of the existing tunnel will be performed in chapter 5. These findings will be used in chapter 6 to determine the general lowering capacity of the tunnel. In chapter 7, the construction methods of the four remaining design options will be presented.

3.3.3 Evaluation

To be able to rate the different outcomes of the design options, their value will be estimated.

- The value of the general design options will be estimated by comparing the situation to existing reference projects and their costs.
- The value of the functional and technical design options will be determined by expressing all significant differences between the options in money, as will be explained in this section.

Criteria

As explained in section 3.3.1, the value of a product can be dependent on functions that are hardly comparable which makes it very hard to express the value objectively. The adaptation of the Beneluxtunnel however is an example of a project in which only few functions are significantly affected. Reasons for this are:

- Adapting a project causes very limited environmental influence compared to the creation of a new project.
- As a tunnel is mostly underground, the impact of aesthetics on the value – which is very hard to objectively estimate – can be ignored.

Thus, the design options are very similar for most functions. The exceptions should therefore be evaluated in order to determine the value of the design options. These exceptions are:

- Construction costs
- Consequences for traffic
- Consequences for navigation

A proper estimation of these three functions will allow for the design options to be compared. To do so, these aspects do however need to be expressed objectively and in the same form. A form in which this is possible is to express these aspects in financial consequences for society.

Strategy

In figure 3.7, the interdependencies from figure 3.6 are displayed in combination with the value defining factors as described on the previous page.

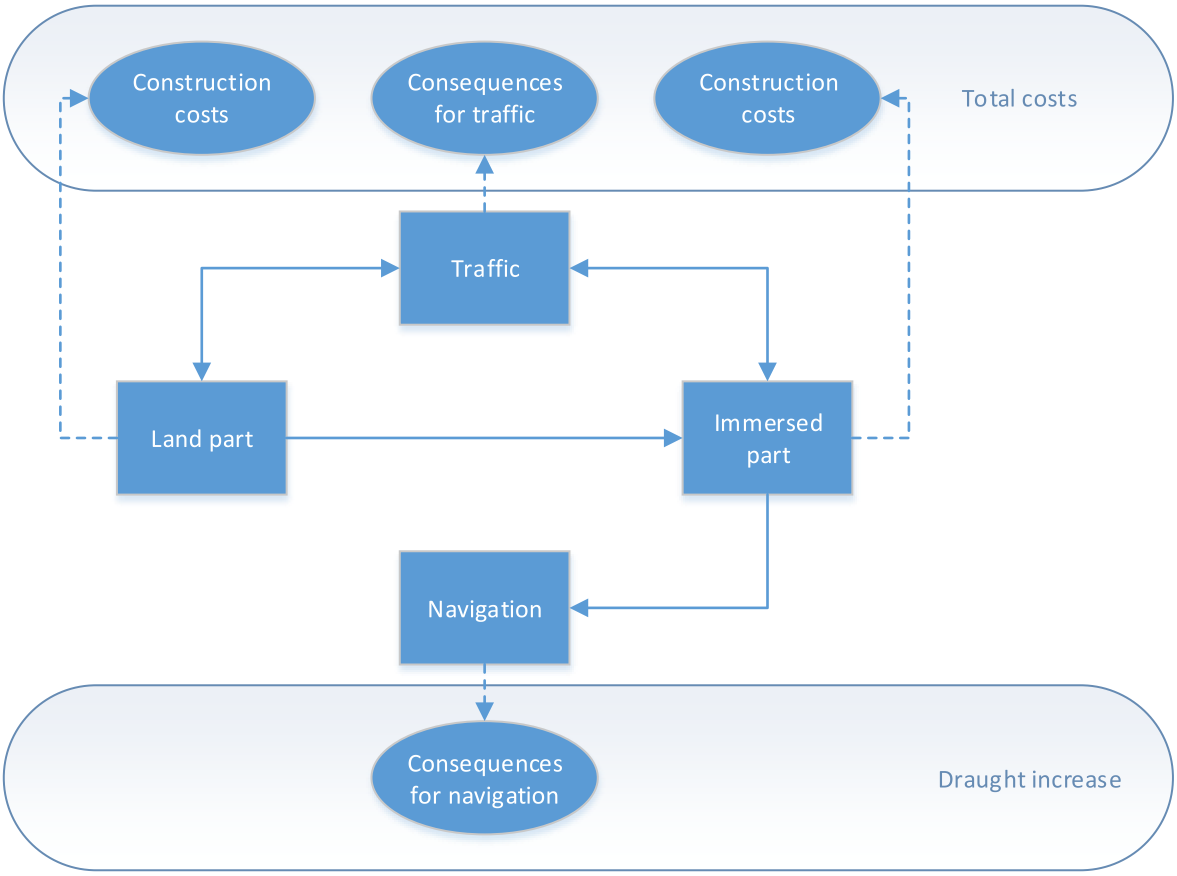


Figure 3.7: The evaluation strategy of the design options

Apart from the relations discussed in section 3.2.3, the following relations (dashed lines) are added:

- The design options regarding the traffic requirements do not only change the boundary conditions for the technical design of the tunnel, they also influence the traffic function itself which would affect the value of the outcome.
- The design options for the land part and the immersed part have certain construction costs which will also influence the value of the outcome.
- The increase of draught for navigation also affects the value of the outcome.

According to the evaluation criteria, figure 3.7 gives a complete estimate of all aspects and relations involved in the value of the outcome. The construction costs and the consequences for traffic influence the value negatively (costs) while the consequences for navigation influences the value positively (benefits). Hence, if all aspects and relations are determined, the absolute value of all design options could be estimated.

However, it is very hard to evaluate the consequences for navigation as this involves many unpredictable parameters as was also mentioned in sections 1.1.3 and 2.3.2. This will therefore not be performed. Instead, the value of the outcomes will be expressed relative to the possible draught increase. As all other value determining factors are negative and could therefore be expressed as total costs, the value of the design options will be expressed as costs relative to draught increase.

In order to predict both the total costs and the draught increase, not only the relations that directly influence their value should be determined (dashed lines), but also the relations that indirectly influence them (solid lines).

Report

In chapter 4, all relations related to the functional design options will be determined, which will conclude in an overview of vertical tunnel profiles and possible draught for all functional and technical design options. In chapter 6 the general lowering capacity will be determined which includes the effect of technical limitations on the vertical profiles presented in chapter 4. Finally, in chapter 8 the evaluation of the design options will be performed in which the total costs and draught increase of all design options will be determined and compared. Also the general design option of building a new tunnel will be discussed.

3.4 Design specifications

In this section, a number of conditions and assumptions are presented that will be used throughout the remainder of the design.

3.4.1 Conditions

The following conditions apply to the design:

Axes:

The following axes are used throughout the design.

X = transverse
Y = longitudinal
Z = vertical

Available information

Despite the efforts of RWS to provide as much information as possible regarding the Beneluxtunnel, some crucial information was missing.

The following information was not available:

- Technical drawings regarding the reinforcement of the 2nd Beneluxtunnel.
- Calculations or calculation reports regarding most design aspects of the 1st and 2nd Beneluxtunnel.

The consequences of these missing pieces of information to the design process were not crucial, as the design is performed in a conceptual phase in which detailed calculations are not yet applied.

The most important consequence of this lack of information is:

- The structural capacity analysis will only be performed on the 1st Beneluxtunnel as this was the only tunnel for which reinforcement data was available.

Value of money

All costs mentioned in this report are based on the price level of 1 January 2015 and are determined using CBS data. [41]

Lifetime

In section 3.2.1 is explained that the lifetime of tunnels is as important as it is unpredictable. However, an estimation of the lifetime is required to determine the safety and the value of the structure.

Thus, the following methodology is used:

- The lifetime of the Benelux connection will be regarded indefinite as it is expected that both the presence of the waterway and the requirement for a connection will remain far into the future.
- A reference lifetime of 100 years is used for structural safety.
- A functional lifetime of 50 years will be used for the evaluation process, so costs will be determined for this horizon.

Structural safety

The purpose of this study is to explore design options, not to prove structural safety. Design codes will therefore not strictly be applied as would be required for a detailed design.

Instead, a few basic principles will be applied, namely:

- Safety levels regarding loads will at this stage be reviewed case specifically. Load factors as defined in the design codes will not be applied.
- Safety levels regarding strength will be applied as in the codes.
- For the construction materials, deformations will only be accepted in the linear elastic phase. Plastic deformations are not allowed as the condition of the structure may not be worse than before adjustments.

3.4.2 Assumptions

The following assumptions apply to this study:

Sufficient traffic capacity

In section 2.3.1, it was mentioned that new infrastructural projects are expected to affect the future intensity of the Beneluxtunnel positively meaning less cars will use the connection. Based on this, it is assumed that the capacity of the Beneluxtunnel will be sufficient throughout its functional lifetime and no changes in the cross-section would be required.

If this assumption turns out to invalid, it would still not be of much interest to this study as insufficient tunnel capacity would require new passages, which cannot be added by adapting the tunnel.

Maeslantkering determines maximum depth

It will be assumed that the Maeslantkering will not be removed or adapted in order to gain more depth. It will also be assumed that the route passing the Maeslantkering is the only realistic route for ships to reach the Beneluxtunnel from the sea.

The resulting maximum depth of the navigation channel will therefore be NAP -17 m and the maximum width will be 360 m.

3.5 Boundary conditions

The following boundary conditions are important for the design question of lowering the Beneluxtunnel:

- Soil properties
- Hydraulic properties
- Material properties
- Sectional properties

These boundary conditions will be discussed in this section.

3.5.1 Soil properties

The soil below and around the Beneluxtunnel is typical for the western part of Holland,

- At the surface a layer of sand is found.
- Below this, many different layers of weak, cohesive soil types are found.
- From about -20m NAP, strong layers of sand are found which can be used for foundation.

The soil around the elements is backfilled. It is assumed that this soil is comparable to the soil that was removed and would therefore be silty sand, which has the following characteristics:

Submerged weight:	γ_{wet}	2000	kg/m^3
Angle of internal friction:	φ'	32.5	^{o}

[42]

The geographical profile of the 2nd Beneluxtunnel can be found in appendix C.

3.5.2 Hydraulic properties

Three types of hydraulic properties are of importance for this study:

- Water density
- Water levels
- Flow speeds

Water density

The water density is very important for immersed tunnels as the weight of the displaced water determines the floating capacity of the tunnel elements. It is also important for navigation as it partly determines the draught of ships. Water density depends mostly in salinity which can fluctuate in a tidal river. Thus, a lower and an upper boundary is used:

Lower bound	1000 kg/m^3
Upper bound	1030 kg/m^3

Water levels

The water levels that are used are important for both the floating capacity of the tunnel and the available depth. The following water levels are used for design:

	Now	**50 years**	**100 years**
Highest possible level	NAP + 3.75m	NAP + 3.75m	NAP + 3.75m
High 1/yr	NAP + 2.50m	NAP + 2.86m	NAP + 3.70m
Mean high water	NAP + 1.33m		
Mean low water	NAP - 0.44m		
Low low water spring	NAP - 0.70m	NAP - 0.54m	NAP - 0.34m
Low 1/yr	NAP - 1.10m	NAP - 0.94m	NAP - 0.74m
Lowest possible level	NAP - 1.78m	NAP - 1.78m	NAP - 1.78m

Some comments:

- These values are interpolated from the points Geulhaven and 1e Eemhaven. [43]
- The highest possible level is determined by the local height of the dikes and is protected by the Maeslantkering. It therefore will not increase in time.
- The lowest possible level is estimated as the lowest level measured minus 0.25m. Also this value remains unaltered in time. [44]
- The 1/year values are taken as the maximum and minimum values during construction. If these values are to be breached during construction, measures are ought to be taken.
- The low low water spring value is the lowest value for which the depth should be guaranteed.
- The 50 and 100 year estimations are based on KNMI scenarios. The high levels are based on the upper bound while the lower levels are based on the lower bound. The absolute maximum and minimum remains unaltered. [45]

The following groundwater levels are used for design:

Groundwater south	NAP - 0.40m
Groundwater north	NAP - 0.60m

Flow speeds

Flow speeds are important for possible displacements of the tunnel. In literature, flow speeds of just above 1m/s are found. However, near the bottom – and therefore near the tunnels – the flow speeds are less. A flow speed of 1m/s will therefore be used as maximum.[27, 46]

3.5.3 Material properties

As mentioned in section 3.4.1, only structural information regarding the 1st Beneluxtunnel has been obtained. The corresponding material properties are:

Concrete

The type of concrete used for the 1st Beneluxtunnel is not specified in any of the drawings or other documents obtained for this study. For further research and safety analysis it is therefore strongly advised to further specify the type of concrete and its characteristics, preferably by testing as this would provide information on its current state. This is assumed not to be necessary for this stage of design and thus an assumption is made.

It is assumed that concrete of the type B35 was used for the design of the tunnel, which was a commonly used type of material at the time of construction. The modern day equivalent of this concrete type is C28/35.

However, given the age of the structure, it can be assumed that the strength of the concrete has increased. According to Eurocode 2, the initial mean compressive strength can be multiplied with a certain factor β_{cc} to find the present day strength. β_{cc} depends on the age of the concrete and on the type of cement. As the type of cement is unknown, a mean value[4] will be assumed. [47]

Thus:

$$f_{cm}(t) = \beta_{cc}(t) * f_{cm}$$

With:

$$\beta_{cc}(t) = \exp\left\{s\left[1 - \left(\frac{28}{t}\right)^{\frac{1}{2}}\right]\right\}$$

In which:

f_{cm} = *Mean compressive strength after 28 days [N/mm²]*
$f_{cm}(t)$ = *Mean compressive strength after t days [N/mm²]*
$\beta_{cc}(t)$ = *Age coefficient*
t = *Age of the concrete [days]*
s = *Cement type coefficient (s=0.20 to 0.38)*

With s = 0.29 and t = 48*365 = 17520 days, the mean cube compressive strength becomes 1.32*43 = 56.8 N/mm².

However, in another study that recently has been performed on old caissons in the port of Rotterdam, the compressive strength was tested and gave an increase over time that was much larger than this. If the same increase ratio would be used on the 1st Beneluxtunnel, a strength increase factor of 1.79 would have been found. [48]

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4 A mean value is taken rather than a conservative value as the goal of this study is not to prove structural safety but to determine feasibility.

Given that the conditions of the Beneluxtunnel are relatively similar to the investigated caissons and the fact that the Eurocode is often quite conservative, it is chosen to choose the mean of the two values.

The mean cube compressive strength therefore becomes 1.55*43 = 66.7 N/mm^2. [5]

The characteristics of the concrete used in design are:

Mean cube compression strength (t=48 year):	f_{cm}	66.7	N/mm^2
Characteristic cube compression strength:	f_{ck}	58.7	N/mm^2
Material factor:		1.5	
Design compressive strength	f_{cd}	44.4	N/mm^2
Elasticity modulus:	E_{cm}	38865	N/mm^2
Elastic limit:	ε_{c3}	1.87	‰

Reinforcement

Unlike the type of concrete, the type of reinforcing steel is indicated in the drawings, but an old and unknown notation method is used. It seems two types of steel are used QR-24 and QR-40. It is assumed that the number indicates the characteristic yielding strength in KG/mm^2. For further research and safety analysis, it is strongly advised to validate these assumptions.

The characteristics of the reinforcing steel used in design are:

		QR-24	**QR-40**	
Characteristic yielding strength:	f_{yc}	24	40	kg/mm^2
Characteristic yielding strength:	f_{yc}	235	392	N/mm^2
Safety factor:		1.15	1.15	
Design yielding strength:	f_{yd}	205	341	N/mm^2
Elasticity modulus:	E_s	210,000	210,000	N/mm^2
Elastic limit:	ε	0.975	1.625	‰

The type of reinforcement used is indicated in the drawings, however, the following assumption is made after analysing various drawings:

- All longitudinal and transverse reinforcement is constructed in QR-40.
- The shear reinforcement in the segment joints is constructed in QR-24.

Furthermore, a concrete cover of 45mm is used, which might seem rather little for a structure in marine environment with a design lifetime of 100 years. However, an extra protective layer of about 100mm is applied on the outer circumference as is discussed in section 2.4.3.

[5] This analysis did not take the effect of long term loading into account. If concrete is loaded for a long time, not all of the strength of the concrete can be used. Normally this effect is counteracted by the positive effect of hardening of the concrete, but if this is taken into account, only 85% of the strength should be used. As this entire analysis is based on assumptions, it does not significantly affect the outcomes. But one should keep in mind that the strength of the concrete might be overestimated.

Pre-tensioning

Like the reinforcement, also the pre-tensioning is indicated in the drawings with an old and unknown notation method. Longitudinal pre-tensioning is of the type C-170 and transverse pre-tensioning is of the type BB-138. It is assumed that the number dictates the characteristic tensile strength in kg/mm^2, as the yielding point of pre-tensioning steel is unclear. It is also assumed that BB indicates the presence of bonding while C indicates the absence of bonding, which would correspond with the structural purposes of the types. For further research and safety analysis, it is strongly advised to validate these assumptions. The characteristics of the pre-tensioning steel used in design are:

		C-170	**BB-138**	
Characteristic tensile strength:	f_{pc}	170	138	kg/mm^2
Characteristic tensile strength:	f_{pc}	1668	1354	N/mm^2
Characteristic yielding strength:	$f_{p0.1k}$	1501	1218	N/mm^2
Material factor:		1.10	1.10	
Design yielding strength:	f_{pd}	1364	1108	N/mm^2
Elasticity modulus:	E_p	195,000	195,000	N/mm^2
Elastic limit:	ε	6.997	5.680	‰
Bond factor		0.5		

Specific weights

The following specific weights will be used in the design.

Construction concrete	2500 kg/m^3
Ballast concrete	2400 kg/m^3
Road deck	2400 kg/m^3

3.5.4 Sectional properties

The following sectional properties are derived from the cross-sections of the tunnels. A drawing of the cross-sections can be found in appendix K.

		1st Beneluxtunnel	**2nd Beneluxtunnel**	
Area:				
Outer area	A_{out}	171.9	382.9	m^2
Structural area	A_C	61.3	144.6	m^2
Moment of inertia:				
Vertical	I_{ZZ}	501.7	1583.6	m^4
Transverse	I_{XX}	3149.6	26097.0	m^4
Centroidal axis:				
from bottom	z_B	3.8	4.0	m
from top	z_T	3.7	4.5	m
from left	x_L	12.0	22.4	m
from right	x_R	12.0	22.8	m
Section modulus:				
bottom	W_B	132.1	400.7	m^3
top	W_T	135.4	349.4	m^3
left	W_L	263.6	1163.8	m^3
right	W_R	263.6	1143.3	m^3

3.6 Summary

In this chapter, the basis of the design has been presented. It concluded the diverging phase of the design by proposing design options and it initiated the converging phase of the design by proposing a strategy, specifications and boundary conditions.

Design options

The design options that were proposed are:

- The general design option of building a new bored or immersed tunnel
- The functional design options of decreasing traffic speed or type and changing the proportions of the navigation channel.
- The technical design options regarding the land part of doing nothing, applying local adjustments or general lowering.
- The technical design options regarding the immersed part of adapting the river bottom, keeping the elements immersed or re-floating the elements.

These design options also influence each other.

Design strategy

The design philosophy used in this study has appointed technical feasibility and evaluation as the defining criteria for design. The technical feasibility will be estimated by determining the structural capacity and the constructability of the design options. The design options will be evaluated by determining their costs relative to the provided depth.

Design specifications

The following conditions are important for the design:

- Not all information was available, resulting in a focus on the 1st Beneluxtunnel.
- A different lifetime will be used for the safety analysis than for the evaluation process.
- Safety factors will not be strictly applied, instead a philosophy is maintained of only allowing linear elastic deformations.

The following assumptions are important for the design:

- Sufficient traffic capacity is assumed for the design.
- The Maeslantkering determines the maximum depth.

Boundary conditions

Finally, also the following boundary conditions are treated:

- Soil properties
- Hydraulic properties
- Material properties
- Sectional properties

Chapter 4

Functional design

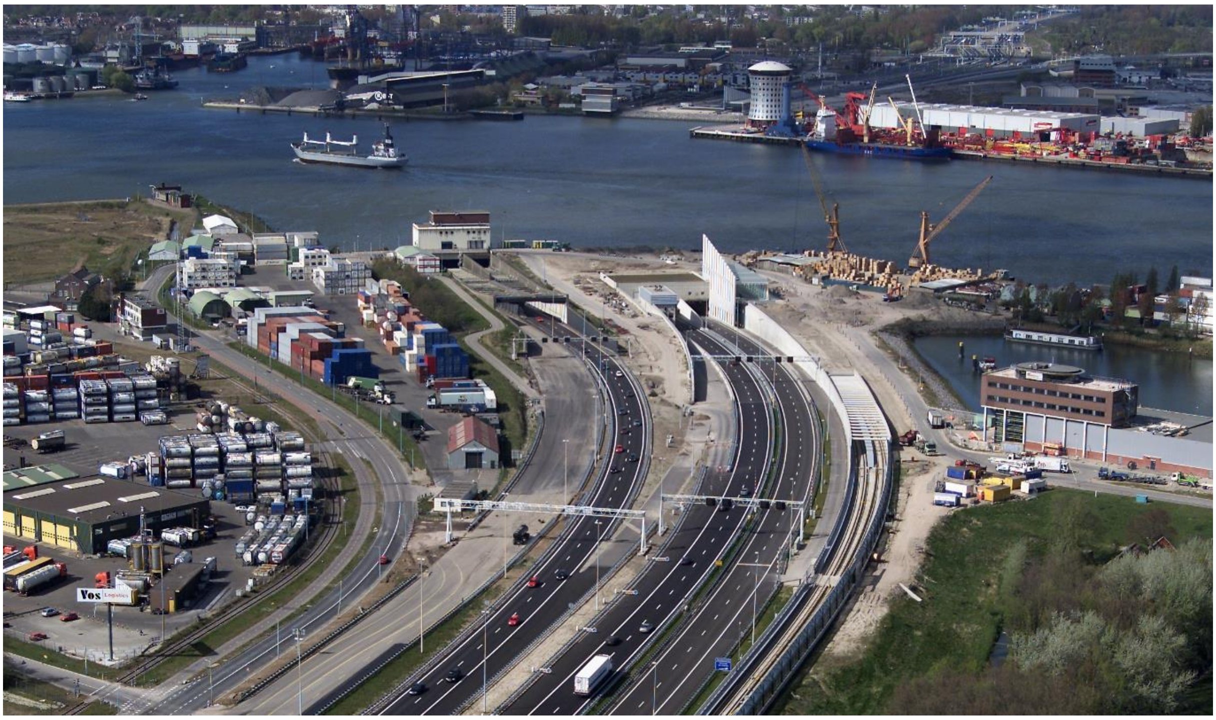


Figure 4.1: An aerial view of the southern land part of the Beneluxtunnel when the addition of the second tunnel was nearly finished. At the top of the picture, the land side of the northern bank can be seen behind the high round building on the bank of the waterway. [12]

4.1 Introduction

In this chapter, the functional part of the design will be treated. The main purpose of this chapter is to establish a relationship between the two main functions (tunnel traffic and waterway navigation) and to determine the consequences of possible adjustments to the tunnel on these two functions.

The following subjects will be discussed:

Traffic	First, the traffic function will be elaborated. The possibilities of adjusting functional requirements will be determined and the possible consequences of these adjustments will be estimated. Also the consequences of temporary traffic disturbance will be estimated. (Section 4.2)
Navigation	In this section, various aspects concerning the relationship between the vertical alignment of the tunnel and the navigable depth will be discussed. (Section 4.3)
Longitudinal profiles	Finally, the relationships established in the previous two sections will be presented as the possible longitudinal profiles and the associated navigation channels. (Section 4.4)

Finally, a summary will be given of the most important findings within this chapter.

4.2 Traffic

As discussed in section 3.3.2, it is possible to adjust traffic requirements in order to obtain design parameters that are beneficial for the technical design of this study. The downside of adjusting the traffic requirements is that the connection might become less suitable for certain traffic types. These consequences differ per traffic type, but to allow fair comparison of the design options they should be evaluated.

First the current and possible traffic requirements for both tunnels will be determined in order to provide a range of possible longitudinal alignments. Next the consequences of adjusting traffic requirements will be discussed. Finally, also the consequences of temporary traffic hindrance due to construction activities will be discussed.

4.2.1 Traffic requirements

In order to allow traffic to safely pass the tunnel at a desired speed, the tunnel is designed according to certain requirements. The most important traffic requirements are the ones that determine the cross-section and the alignment of the tunnel.

For the purpose of this study, mainly the vertical alignment of the tunnel is important as adjustments to it would influence both the traffic and the navigation functions of the tunnel.

Current design parameters

The design parameters for the vertical alignment of the 1st and the 2nd Beneluxtunnel are shown in table 4.1.

	1st Beneluxtunnel	2nd Beneluxtunnel, motorway trajectory	2nd Beneluxtunnel, metro trajectory
Slope	1/22 (≈4.55%)	4.4%	1/25.2 (≈3.97%)
Radius top arch	10000m	10000m	5000m
Radius bottom arch	4000m	3200m	3150m

Table 4.1 -Design parameters for the Beneluxtunnel as built. [29, 49]

The following comments apply to these values:

- Regarding the slopes and the top arch, maximum values allowed values were used.
- The bottom arch is not maximised as this is not required to obtain the optimal trajectory.
- The decreased slope of the metro relative to the motorway in the 2nd Beneluxtunnel is acquired by adjusting the vertical position of the metro in the cross-section of the tunnel. At the deepest point, the metro trajectory lies relatively high, while at the transition point and further inland, the metro trajectory lies relatively low.
- The bicycle and pedestrian passage is not included, but its slope is minimised similar to the metro trajectory. At the abutments, escalators and lifts help to overcome the remaining height difference.

The associated vertical profile of the motorway traffic profile of both tunnels was shown in figure 4.2. The graph stops at the height of the pivot dikes. Note that this graph shows the vertical profile from traffic requirements perspective and thus follows the main axis of the tunnels, which has a horizontal curve as discussed in section 2.4.1.

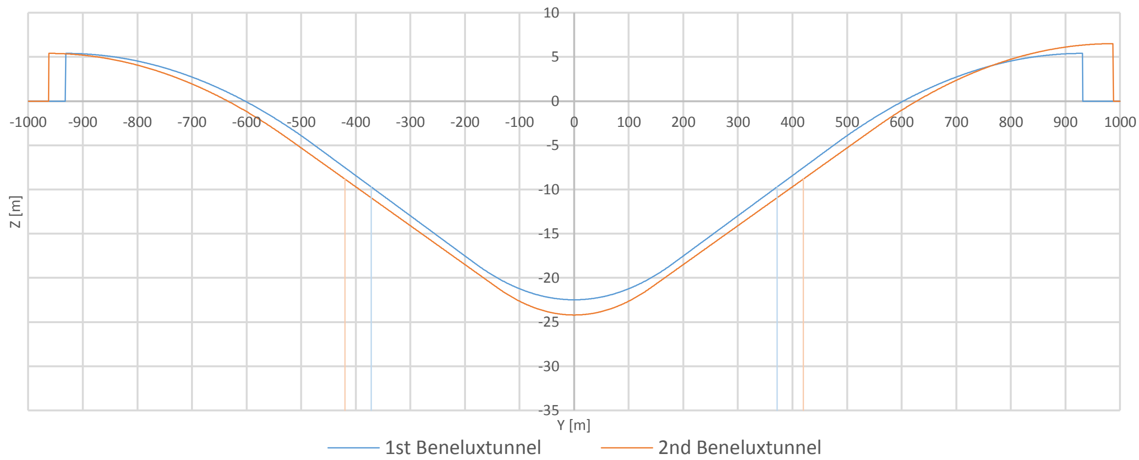


Figure 4.2: The vertical profile of the road deck of both Beneluxtunnels (0=NAP, left is south, right is north)

Theoretic traffic requirements

Table 4.2 shows the design values for the vertical profile regarding traffic requirements as indicated in the available literature and specified for the Beneluxtunnel. Calculations regarding these specifications can be found in appendix B.

Design speed	120km/h	100km/h	80km/h	50 km/h	Metro
Max slope	5%	5%	6%	7%	1/25.6 (≈3.91%)
Max length of slope	250m	250m	175m	150m	-
Min radius top arch	12375m	8284m	5011m	1095m	3100m
Min radius bottom arch	1200m	850m	500m	200m	2000m

Table 4.2 - Design values of longitudinal alignment regarding traffic requirements according to literature [50-52]

The following comments apply to these values:

- The current maximum speed is 100 km/h. All applied parameters as shown in table 4.2 regarding motorway traffic therefore satisfy the current requirements.
- The applied slope for the metro trajectory is slightly larger than the allowed slope which is probably the result of slightly stretching the limits. The applied radii however do satisfy the requirements.
- The maximum slope for 100 km/h and 120 km/h motorway traffic has increased in recent design codes to 5 % for special civil structures.
- The maximum length of the slope is a reaction to the potential unsafe situation that occurs when trucks have to climb a slope, their speed gradually drops and the difference in speed with the other traffic users becomes too large. To overcome this problem in the existing tunnel, extra lanes are available for trucks as soon as they reach the land part.

Possible vertical profiles

In section 4.4, the vertical profiles that are possible according to these theoretic traffic requirements will be presented. The 120 km/h option will not be included as this is faster than the current maximum speed and therefore very unlikely to be realised.

4.2.2 Consequences of changing traffic requirements

Adjusting the traffic requirements might be beneficial for design, it does not automatically result in larger value, as it also has consequences for the traffic itself. In this section, these consequences will be discussed.

Consequences for road traffic

A lower speed means more time is required to pass the tunnel. This time difference has consequences elsewhere in society, meaning it negatively influences the economy. These consequences will be estimated in section 8.2.2

Consequences for other traffic types

The vertical alignments proposed in section 4.2.1 are based on the requirements for motorway traffic. However, two more traffic types use the connection as it also contains a metro line and a passage for bicycles and pedestrians. The functionality of these passages can be threatened by increased slopes. It should however be pointed out that only the 2nd Beneluxtunnel contains these other connections and therefore only this tunnel may experience difficulties.

Metro passage An effective method to decrease a slope is to locally adjust the vertical position of the trajectory as discussed in section 2.4.1. However this is already fully utilised for the metro line as discussed in section 4.2.1.

The only remaining strategy therefore is to increase the allowed slope for the metro. The maximum allowed slope of metros does not depend on safety factors as is the case for motorway traffic, it is simply a technical matter determined by the engine capacity and the friction on the tracks.

A possible way of increasing the slope is therefore to increase the friction on the track using a cogwheel system like is often used in mountainous areas. As there are metro stations on both sides of the tunnel, the train could lock onto the system at these locations. Both the tracks and the metros should be adapted to this system. The consequences due to the loss of time for this metro connection would not be significantly large.

Bicycle and pedestrian passage For the bicycle and pedestrian passage, the maximum slope is less defined. However, it does not seem likely that an increased slope would be appreciated. If the slope is regarded too steep, the vertical repositioning of the alignment cannot be used as – similar to the metro – this aspect is already fully utilised. However, it might be possible to implement a second escalator at ¼ and ¾ of the track and thus create another level.

Of course also other methods of crossing could be proposed that would replace this passage. Special interest regarding this passage should be social safety.

An estimate of the economic consequences of these required adjustments to the 2nd Beneluxtunnel is given in section 8.2.2.

4.2.3 Temporary traffic disturbance

Execution hindrance is expected to be one of the main cost drivers and is therefore important to be included in the evaluation of design options. In this section the costs of execution hindrance on traffic are estimated.

Accessibility tunnel

Adapting the tunnel without traffic hindrance seems impossible. However, as the Beneluxtunnel consists of two tunnels with multiple tubes, intermediate situations with minor traffic hindrance exist.

In this stage of design there are four traffic scenarios to be distinguished:

1. Both tunnels function are fully accessible.
2. Beneluxtunnel 1 is closed while Beneluxtunnel 2 is fully accessible. 2x2 traffic lines are inaccessible.
3. Beneluxtunnel 2 is closed while Beneluxtunnel 1 is fully accessible. 2x2 traffic lines, 2x1 metro lines and 1x small scaled personal traffic line are inaccessible.
4. Both tunnels are closed. All traffic lines are inaccessible.

General planning

Planning of construction greatly influences the accessibility of the tunnel and the influence inaccessibility will have on traffic. In this stage of design, the following statements can be adopted:

- Due to the higher elevation of the 1st Beneluxtunnel relative to the 2nd Beneluxtunnel, it is most likely for the 1st Beneluxtunnel to be adapted prior to the 2nd Beneluxtunnel. It is therefore safe to assume scenario 2 will then take place prior to scenario 3.
- The adaption of the Beneluxtunnel will most likely be executed when the Blankenburgtunnel is already available for traffic. It can be assumed that the traffic intensity regarding the Beneluxtunnel will have been decreased and that traffic hindrance in case of inaccessibility of the Beneluxtunnel will have been decreased compared to the current situation, as the additional travel time will be less.

Costs

The economic consequences of temporary traffic disturbance are estimated in section 8.2.2

4.3 Navigation

Traffic requirements determine the vertical profile of the tunnel, which consequently determines the possible dimensions of the navigation channel, that consequently determine the maximum dimensions of ships. However, some complications exist within these relationships which will be discussed in this section.

First of all, the possible boundaries of the navigation channel dictated by the vertical profile of the tunnel is determined. Second, the ratio between depth and width of the navigation channel is determined, which consequently depend on the ratio between the draught and the beam of passing ships and the safety factors used in channel design.

The resulting vertical profiles including these relationships will be presented in section 4.4.

As mentioned in section 3.3.4, the value of increased depth will not be estimated. The influence of temporary navigation disturbance will be briefly discussed.

4.3.1 Channel boundaries

In this section, the channel boundary will be determined. The channel boundary is the lowest set of points above the tunnels where the bottom of the navigation channel can maximally be located. Its location depends on:

- The vertical distance between the road deck and the bottom of the navigation channel
- The relationship between the vertical profile of the tunnel and vertical profile of the navigation channel, which is distorted due difference in horizontal alignment.

Vertical distances

To finalise the relationship, we also need to determine the vertical distance between the tunnel and the bottom of the navigation channel. This distance consists of the following constituents:

1. Road deck to top tunnel.
2. Top tunnel to bottom waterway.
3. Bottom waterway to bottom navigation channel.

These distances are indicated in figure 4.3 and discussed on the next page.

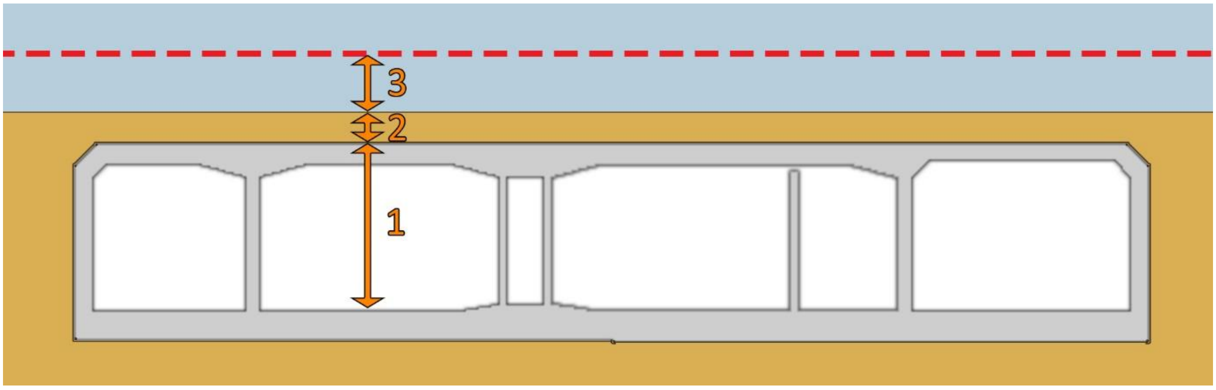


Figure 4.3: The vertical distances between the tunnel and the navigation channel

1. The distance between the vertical alignment and the top of the tunnels is found in cross-sectional drawings and is, for the main axis of the tunnels, constant[6]:

   - 1st Beneluxtunnel: 5.700m [53]
   - 2nd Beneluxtunnel: 6.202m [31]

2. The distance between the top of the tunnel and the bottom of the waterway is determined by the layer of bottom protection, which is treated in section 2.4.3. The thickness of the layer of bottom protection for the determination of the navigation channel will be assumed 0.75m for both tunnels.

3. The distance between the bottom of the waterway and the bottom of the navigation channel is actually a safety margin meant to deal with bottom related, ship related and water level related uncertainties as described in appendix C. However, this safety margin will be assumed to lie within the navigation channel and will therefore be excluded from the determination of the bottom of this navigation channel and thus equals 0.

Influence horizontal alignment

The horizontal alignment of the navigation channel is not perpendicular to the horizontal alignment of the tunnel which influences the relationship between the alignment of the tunnels and the available depth.

An overview of the alignment of the channel relative to the alignment of the tunnels is shown in figure 4.4. Calculations regarding this subject can be found in appendix D.

The following aspects influence this relationship:

- The tunnels are horizontally curved the following radii:
  - 1st Beneluxtunnel: R = 1300m
  - 2nd Beneluxtunnel: R = 1238m
- The mean direction of the horizontal alignment of the tunnels has an angle with the navigation channel of:
  - 1st Beneluxtunnel: $\varphi = 30^o$
  - 2nd Beneluxtunnel: $\varphi = 32^o$
- The influence of the width of the tunnels combined with the fact that the alignment is not perpendicular results in the highest point of the tunnel at the edge of the navigation channel being at opposite sides of the tunnel.
  - 1st Beneluxtunnel: w = 23.9m
  - 2nd Beneluxtunnel: w = 42.25m

The influence of these aspects on the relationship between the vertical alignment of the tunnel and the navigation channel is expressed as the difference in horizontal distance between the edges of the navigation channel. In the domain of the tunnel, these points can be referred to as the points of inflection as at these points, the bottom arch begins and ends.

[6] Except for the locations of the ventilation fans in the 2nd Beneluxtunnel, but they are not located near the points of inflection in the tunnel alignment.

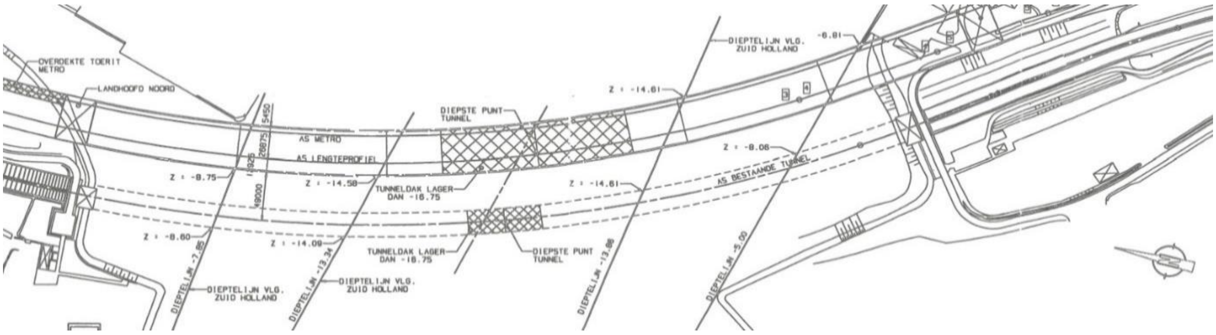


Figure 4.4: The alignment of the navigation channel and the tunnels [52]

The results are:

- The influence of the horizontal radius is negligible
- The influence of the mean angle with channel results in a decrease in horizontal distance between the points of inflection of:
    - 13% for the 1st Beneluxtunnel.
    - 15% for the 2nd Beneluxtunnel.
- The influence of the width results in an additional decrease in horizontal distance of:
    - 4% for the 1st Beneluxtunnel.
    - 10% for the 2nd Beneluxtunnel.
- The combined decrease of the horizontal distance between the vertical alignment of the tunnel and the vertical alignment navigation channel is:
    - 17% for the 1st Beneluxtunnel.
    - 24% for the 2nd Beneluxtunnel.

Given the influence of the difference in alignment on the effective size of the navigation channel, it would be possible to decrease this difference by changing the alignment of the navigation channel. This would however require very large adaptions to both the waterway and its banks which is expected to be very costly. Hence, this is not regarded feasible.

Bottom navigation channel

Using the information gained in this section, the relationship between the vertical alignment of the tunnel and the bottom profile of the navigation channel can be established. This is shown in figure 4.5.

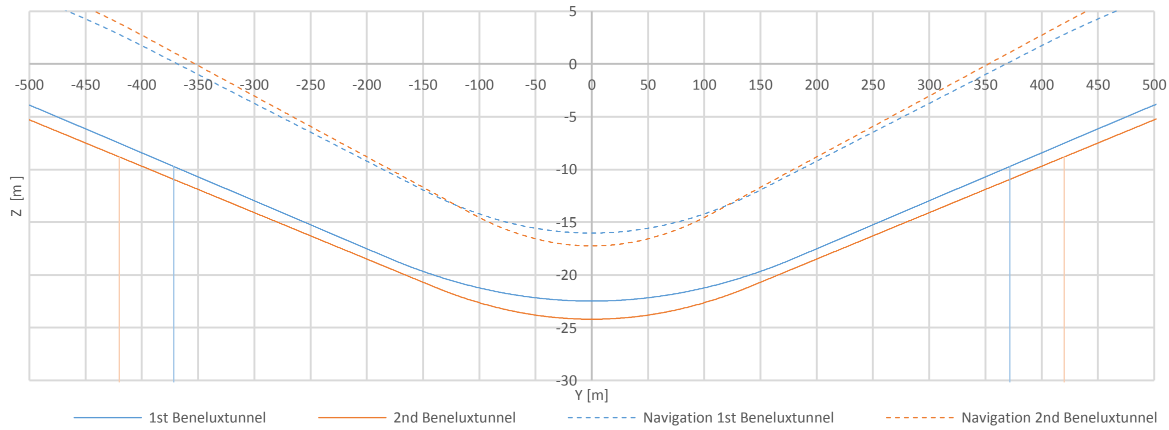


Figure 4.5: This graph shows the vertical profile of the road deck of the existing tunnels and the associated boundaries of the navigation channel. (0=NAP, left is south, right is north)

Interesting aspects regarding this relationship is that in the centre of the channel, the bottom lies deeper for the 2nd Beneluxtunnel, meaning the 1st Beneluxtunnel is governing for the maximum depth. However, closer to shore, the 2nd Beneluxtunnel becomes governing, which is mainly the result of its large width. The graphs intersect at a channel width of 240 m.

To verify this model, its outcomes will be compared to the geometric constraints of the navigation channel as provided by the drawings and reports of the Beneluxtunnels. Figure 4.6 shows the longitudinal profile the 1st and the 2nd Beneluxtunnel, including information on the navigation channel. Furthermore, in the design conditions of the 2nd Beneluxtunnel another profile for the navigation channel is maintained. [52]

The geometric constraints of the navigation channel following from these sources are:

Channel Width	Depth (vs NAP) according to literature		Depth (vs NAP) according to model	
	1st Beneluxtunnel	*2nd Beneluxtunnel*	*1st Beneluxtunnel*	*2nd Beneluxtunnel*
0m (centre)		-16.50 m	- 16.03 m	- 17.25 m
125 m	- 16.00 m		- 15.32 m	- 16.20 m
225 m	- 14.00 m	- 14.00 m	- 13.76 m	- 13.85 m
225 m		- 13.3 4m / - 13.86 m	- 13.76 m	- 13.85 m
275 m	- 13.00 m		- 12.60 m	- 12.41 m

Table 4.3: This table shows the depth of the navigation channel according to different sources for different channel widths.

The differences in depth according to literature and according to the model are not negligible which mainly results from the assumption that over the entire width of the navigation channel, the layer of bottom protection would be equal (0.75 m). In reality, the layer of protection is applied rather variable and ranges from 0 m to 1.5 m. Taking this into account, the values of the model corresponds rather well with the values found in literature. For the remainder of this study however, the thickness of the bottom protection will be assumed 0.75 m.

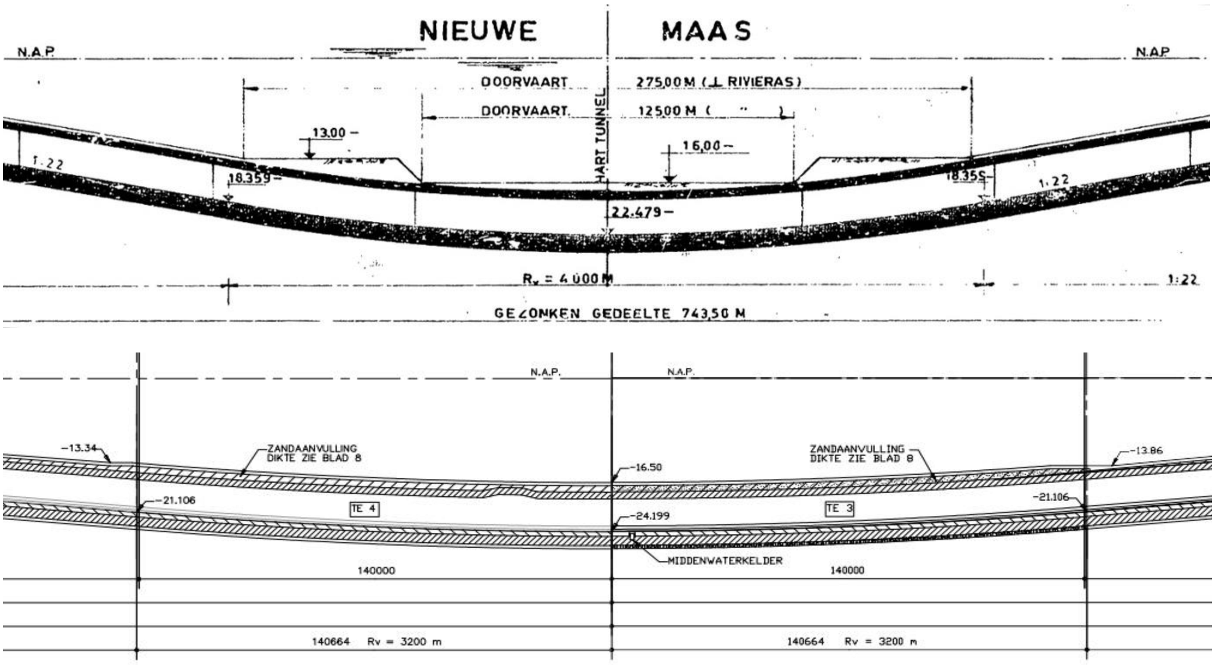


Figure 4.6 – The Longitudinal profile of the 1st Beneluxtunnel (top) and of the 2nd Beneluxtunnel (bottom), including cross-section geometry of the navigation channel. [29, 49, 54]

4.3.2 Channel proportions

Given that the depth of channel results from the design process as discussed in section 3.3.3 and is therefore variable, the accompanying width can only be determined if the channel's proportions are known.

In order to find the proportions of the navigation channel if the dimensions of the channel increases, the relationships between navigation and navigation channels is used. Furthermore, the proportions of large container vessels are estimated to find the approximate proportions of the design vessel.

Background information regarding these relationships can be found in appendix C and calculations can be found in appendix D. Furthermore, the navigation channel is assumed to be rectangular two way channel.

Depth versus draught

A first estimation for the relationship between depth and draught can be given using:

$$d = 1.1T$$

With:

d = *Available depth [m]*
T = *Maximum allowed navigation draught [m]*

[55]

The available depth is the distance from the bottom to the water level and does not include safety margins for uncertainties regarding the water level, the ship and the bottom level. These uncertainties are incorporated in the 10% difference between the depth and the draught. Not included in this 10% however, is the effect of tidal differences. Thus the upper bound of the available depth should be equal to the minimum water level at which the depth should be guaranteed.

For this area, this level is the MLLWS level is currently at NAP -0.70m. In the future this level will probably increase which would allow more draught. However, this increase will not be taken into account as it will not be determined in this study when a certain depth must be provided.

Beam versus width

The relationship between width and beam for a two-way channel can be estimated using:

$$W = \left(2W_{BM} + 2\sum W_i + W_{BR} + W_{BG} + \sum W_p\right) * B$$

In which:

W = *Channel width [m]*
W_{BM} = *Ship manoeuvrability factor*
W_i = *Additional effect factor*
W_{BR} = *Bank clearance factor (red side)*
W_{BG} = *Bank clearance factor (green side)*
W_P = *Passing distance factor*
B = *Ship's Beam [m]*

[55]

Applying all factors as described in the accompanying literature, the relationship becomes W = 7B for a two way channel and W = 3.3B for a one way channel.

If we would now apply these relationship to the widths and depths according to literature, we get:

Navigation channel dimensions		Navigation dimensions	
Width (W)	*Depth (d) (vs NAP)*	*Beam (B)*	*Draught (T)*
125m (one way)	16.0m	37.9m	13.9m
225m	14.0m	32.1m	12.1m
275m	13.0m	39.3m	11.2m

Table 4.4: This table shows associated design vessels for the three navigation channels also displayed in table 4.3.

Note that the beam associated with the 225m channel is very close to the Panamax width of 32.3m, which most likely is not a coincidence. Also the accompanying draught of 12.1m almost equals the Panamax draught of 12.04m.

Beam versus draught

The only element still missing in the relationship is an estimation of the proportions of possible design vessels. Therefore, a list of the largest container vessels is used which can be found in appendix D. The dimensions of these vessels are shown in figure 4.7.

Also, the results gained above are shown in the graph, showing the correlation between the dimensions of the estimated design vessels and the Panamax dimensions. The post-Panamax vessels are clearly too large for this navigation channel.

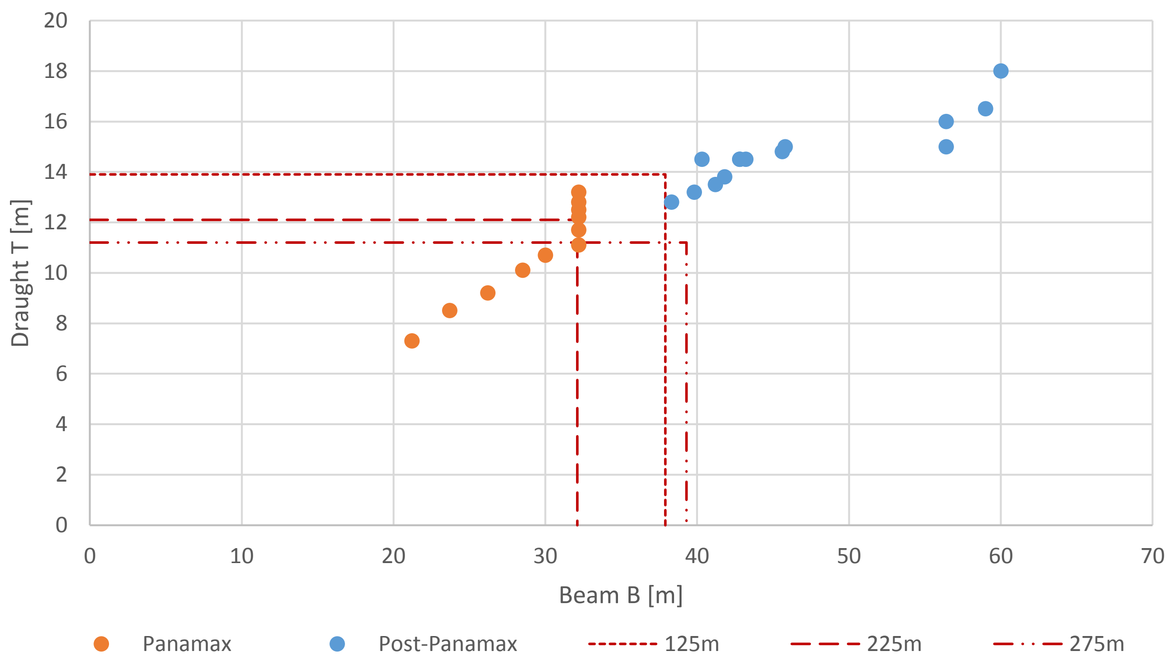


Figure 4.7: Dimensions of Panamax and Post-Panamax vessels, combined with the estimated design vessels of the Beneluxtunnel for the three channel widths as given in table 4.4.

For the navigation channel to become suitable for post-Panamax vessels, a linear trend line is fitted through the points in the graph. The two values with the largest deviation from the mean (positive and negative) are excluded from the calculation. This is shown in figure 4.8.

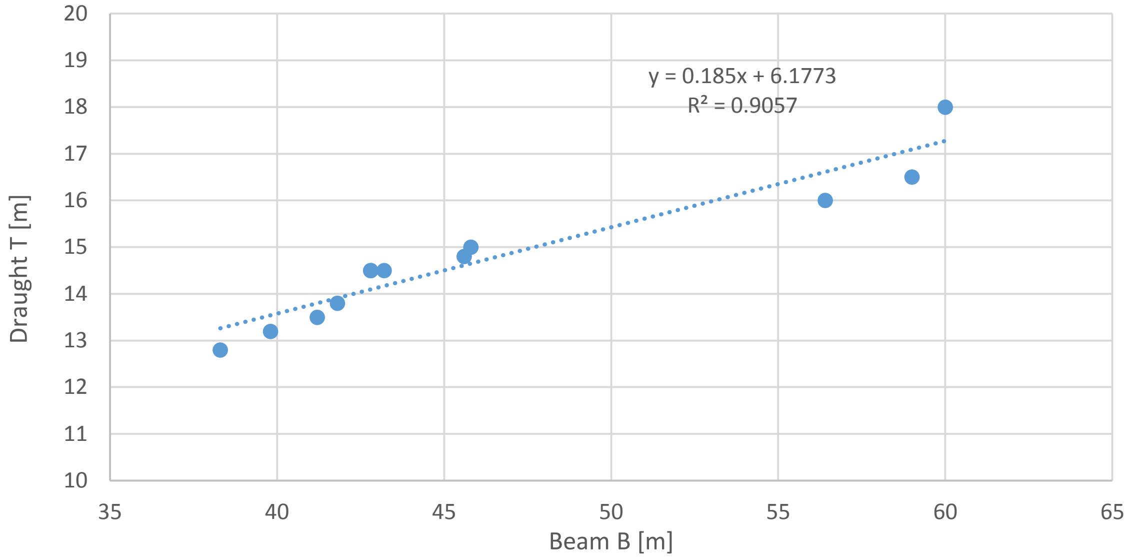


Figure 4.8: The dimensions of post-Panamax container vessels, including a linear trend line.

Relationship channel proportions

The three relationships that follow from the subjects treated in this section are:

$$d = 1.1T$$

$$W = 7B$$

$$T = 0.185B + 6.1773$$

The relationship between the depth and width of the navigation channel therefore becomes:

$$d = 0.029W + 6.795$$

Or:

$$W = 34.398 * d - 233.736$$

Using this relationship, the location of the corner points of the navigation channel can be determined. The associated points of inflection lie in the domain of the vertical tunnel profile. To find these points, the equations should be transferred to this domain, using the relations found in section 4.3.1.

4.3.3 Temporary navigation disturbance

Different construction methods affect navigation differently. As the economic value of the navigation in the Nieuwe Maas is regarded very high, it is clear that for proper comparison of design decisions, this aspect should be included in the evaluation process as proposed in section 3.3.4.

However, estimating both the influence of hindrance on navigation as well as the economic consequences resulting from this is very hard and the reliability of the results is debatable. Therefore it is chosen not to attempt such an estimation.

Instead, two preconditions are proposed to partially include the effects of construction on navigation in the design process:

- Dredging works will not disturb navigation.
- Other, disturbing works will be executed during the weekend.

4.4 Vertical profiles according to functional requirements

In this section, the possible vertical profiles of the 1st and the 2nd Beneluxtunnel, based on the relations as determined in sections 4.2 and 4.3. First a summary of the conditions will be given and consequently, the possible vertical profiles for the 1st and 2nd Beneluxtunnel will be given. Finally, the results will be discussed.

4.4.1 Conditions

The graphs presented in this section are a result of three aspects, treated previously in this report: the design choices regarding the land parts, the possible vertical profiles based on traffic requirements and the relationship between these vertical profiles and the maximum size of navigation channels.

Land parts

As discussed in section 3.3.3, each land part has three design options: do nothing, local adjustments or general lowering.

- Do nothing results in unchanged parameters at the land parts
- Local adjustments results in freedom of rotation and a vertical translations of 0.5 m, 1.0 m and 1.5 m at the transition point. This option will be treated in section 7.4.
- General lowering results in freedom of rotation at the transition points. The possible translations is relatively unlimited, therefore a vertical translation of 5 m will be provided which will give a general idea of this method. This option will be treated in section 7.5.

The following five combinations regarding these options are regarded interesting and are thus included in this study:

Primary land part (south)	**Secondary land part (north)**
Do nothing	Do nothing
Local adjustments	Local adjustments
General lowering	General lowering
Do nothing	Local adjustments
Do nothing	General lowering

Whether the north or the south land parts is primary or secondary is not relevant at this stage of design. It is only indicated for clarity.

Traffic requirements

As discussed in section 4.2.1, increased slopes and radii are possible of the maximum speed is lowered. Consequently, a maximum speed of 50 km/h, 80 km/h or 100 km/h is possible. The associated design parameters can be found in table 4.2.

Furthermore, the following conditions are applied:

- Only the road traffic is taken into account.
- The 120 km/h scenario is regarded infeasible and will not be included.
- The minimum radii of the bottom arch are regarded as being very low. As the bottom arch has little effect on the outcomes, it is chosen to use different values that give more realistic outcomes.
    - For the 1st Beneluxtunnel, a bottom arch of R = 2000 m is applied which is half of the current arch.
    - For the 2nd Beneluxtunnel, a bottom arch of R = 2000 m is applied which the minimum allowed radius for the metro connection.
- The top of the pivot dike is regarded as being the boundary of the model.
- In case of general lowering, it is possible to acquire lowering while maintaining the existing traffic requirements. This option is indicated as "current slopes".

Navigation channel

Apart from the vertical profiles of the tunnel, also the corresponding dimensions for the navigation channel are shown. Their size and position are determined according to the relations presented in sections 4.3.1 and 4.3.2.

4.4.2 Results 1st Beneluxtunnel

Figures 4.9 through 4.13 show the possible vertical profiles of the 1st Beneluxtunnel for different combinations of the input conditions. The graphs show the height of the main road axis over the length of the tunnel and the corresponding boundaries of the navigation channel.

Do nothing on both sides

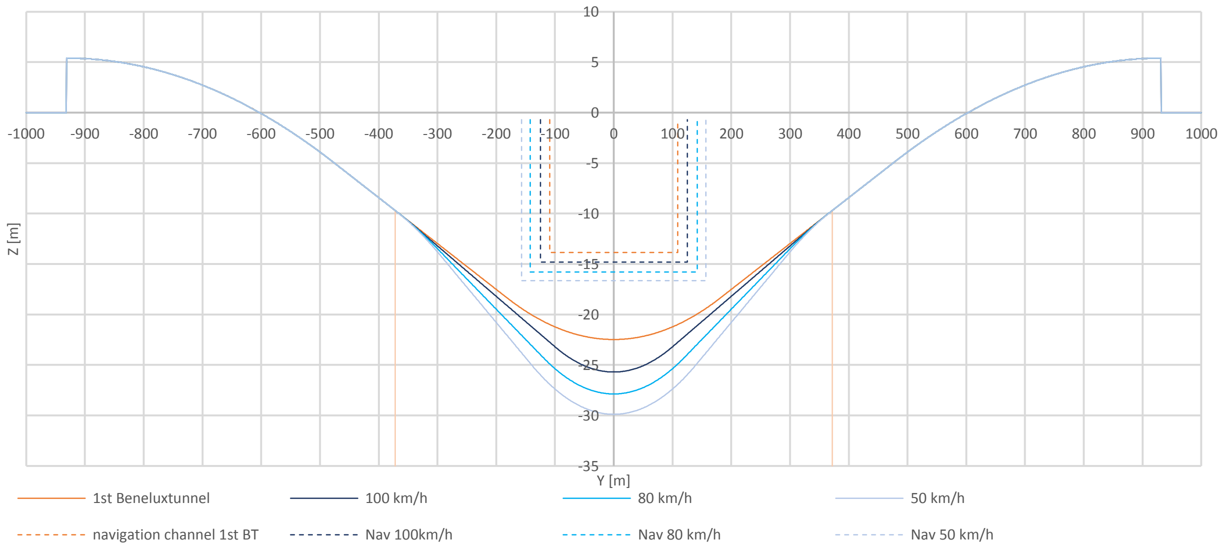


Figure 4.9: This figure shows the possible vertical profiles of the 1st Beneluxtunnel for the alternative: Do nothing on both side. (0=NAP, left is south, right is north)

Local adjustments on both sides

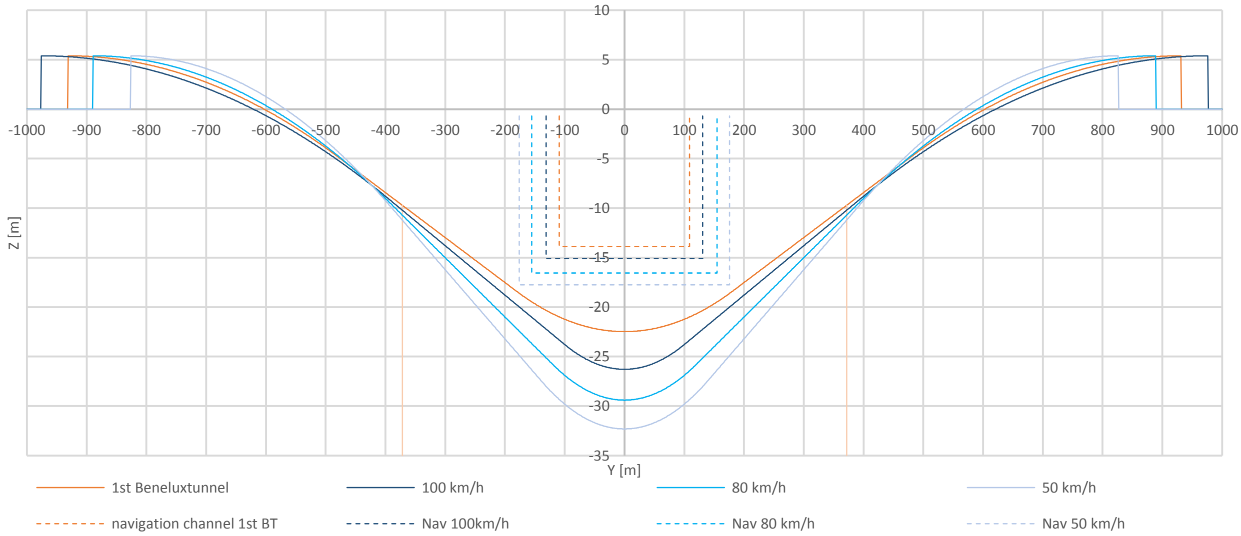


Figure 4.10: This figure shows the possible vertical profiles of the 1st Beneluxtunnel for the alternative: Local adjustments on both sides. (0=NAP, left is south, right is north)

General lowering on both sides

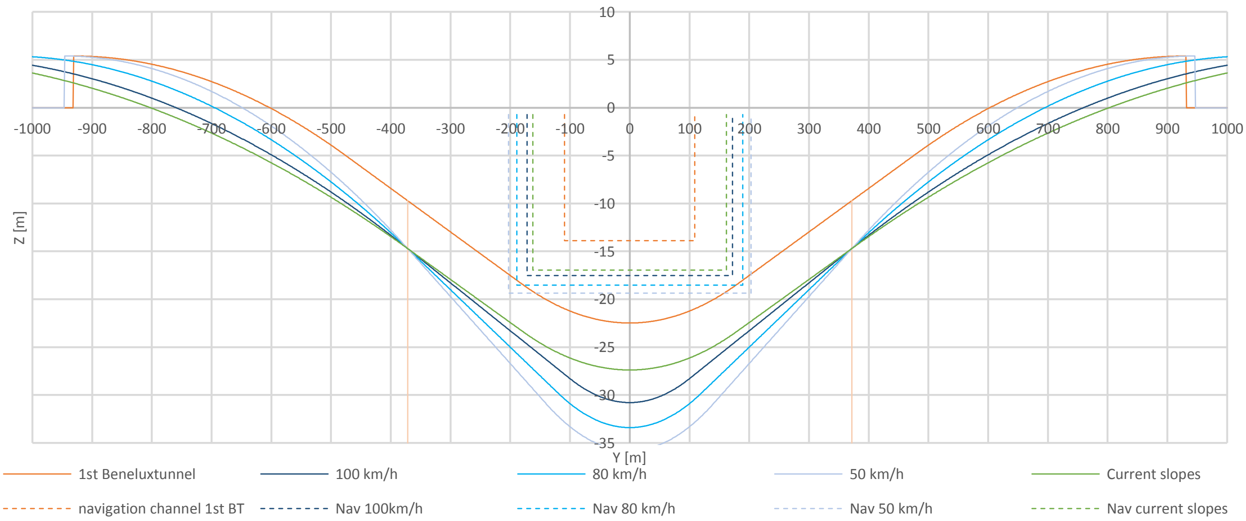


Figure 4.11: This figure shows the possible vertical profiles of the 1st Beneluxtunnel for the alternative: General lowering on both sides. (0=NAP, left is south, right is north)

Local adjustments on one side

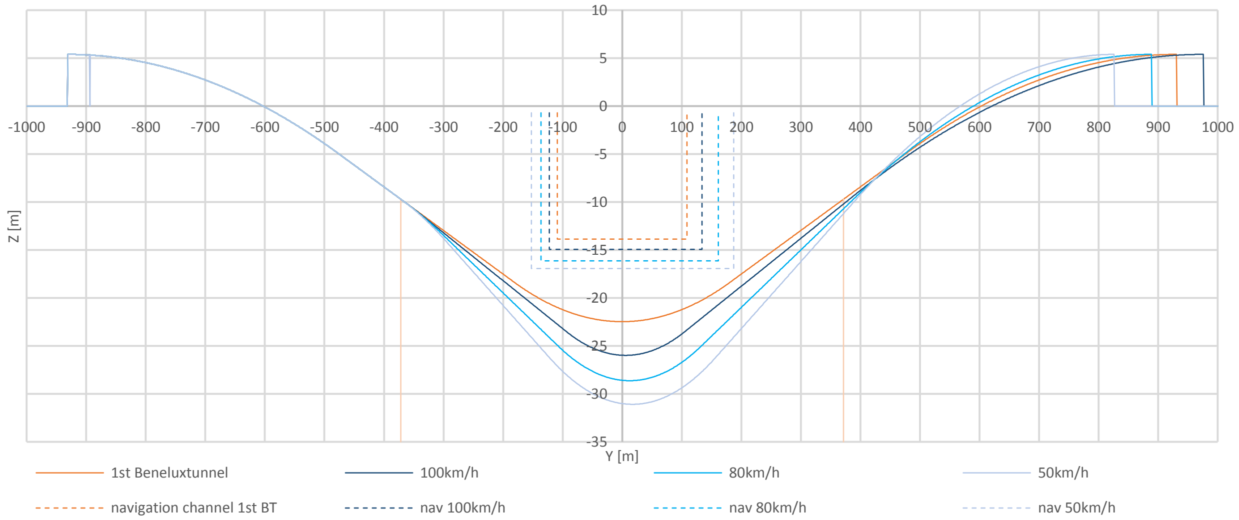


Figure 4.12: This figure shows the possible vertical profiles of the 1st Beneluxtunnel for the alternative: Local adjustments on one side. (0=NAP, left is south, right is north)

General lowering on one side

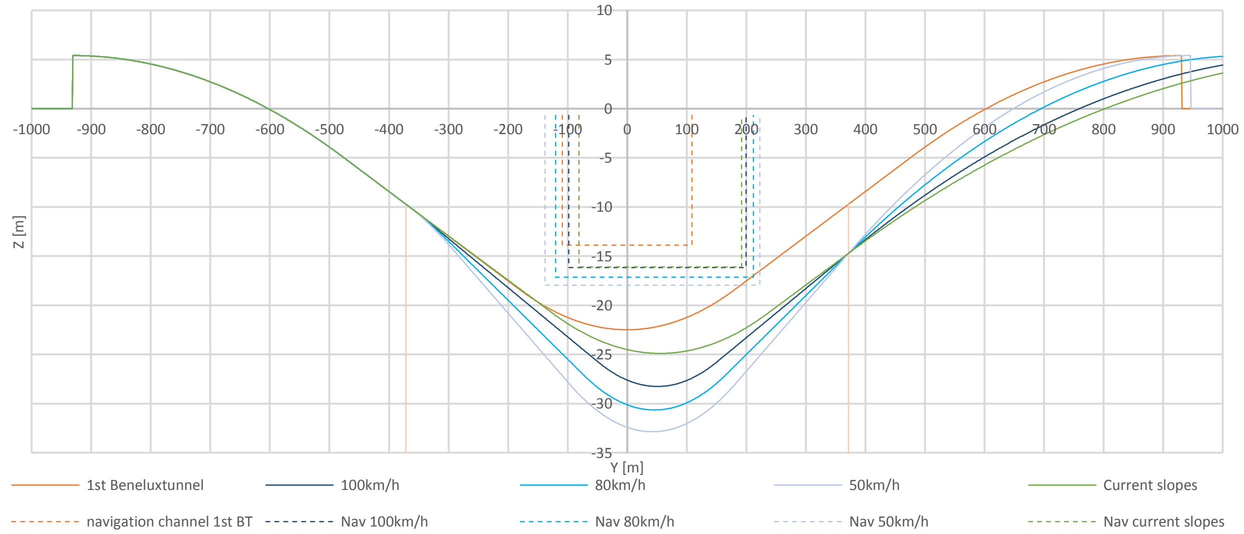


Figure 4.13: This figure shows the possible vertical profiles of the 1st Beneluxtunnel for the alternative: General lowering on one side. (0=NAP, left is south, right is north)

4.4.3 Results 2nd Beneluxtunnel

Figures 4.14 through 4.18 show the possible vertical profiles of the 2nd Beneluxtunnel for different combinations of the input conditions. The graphs show the height of the main road axis over the length of the tunnel and the corresponding boundaries of the navigation channel.

The same axes have been used as for the 1st Beneluxtunnel for clear comparison, but this also results in some parts of the graphs being out of bounds. Also the northern land part is slightly simplified for the design alternatives as it is kept at the same height is the southern land part

Do nothing on both sides

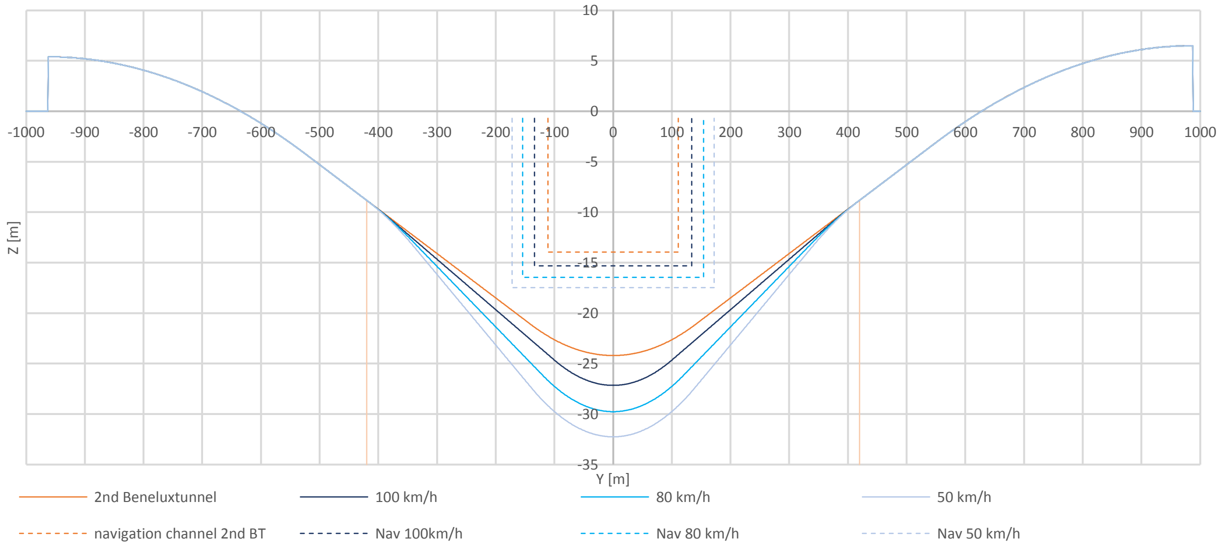


Figure 4.14: This figure shows the possible vertical profiles of the 2nd Beneluxtunnel for the alternative: Do nothing on both side. (0=NAP, left is south, right is north)

Local adjustments on both sides

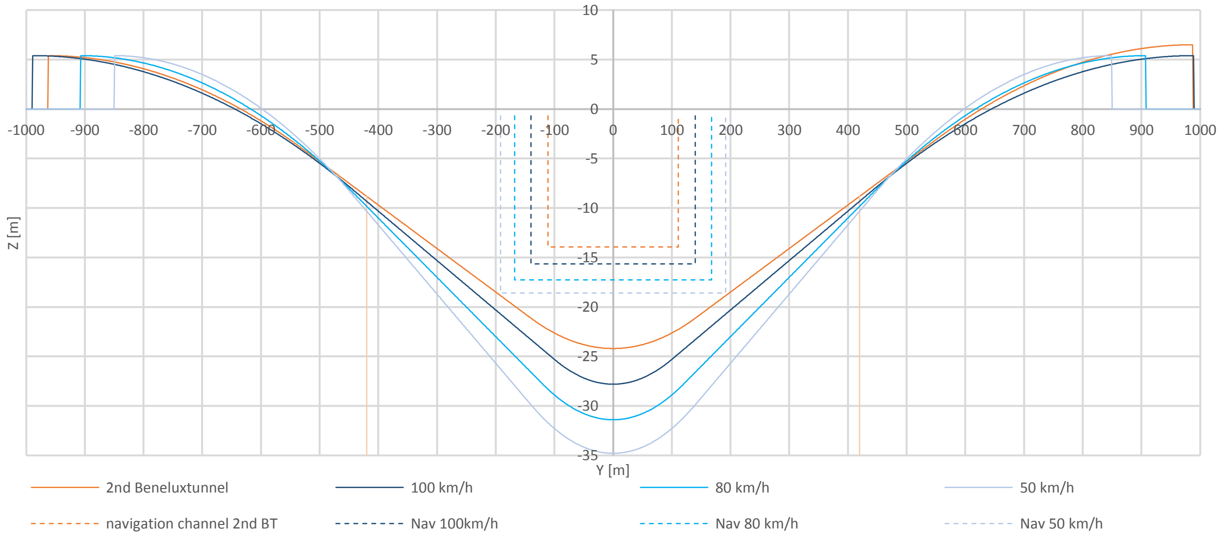


Figure 4.15: This figure shows the possible vertical profiles of the 2nd Beneluxtunnel for the alternative: Local adjustments on both sides. (0=NAP, left is south, right is north)

General lowering on both sides

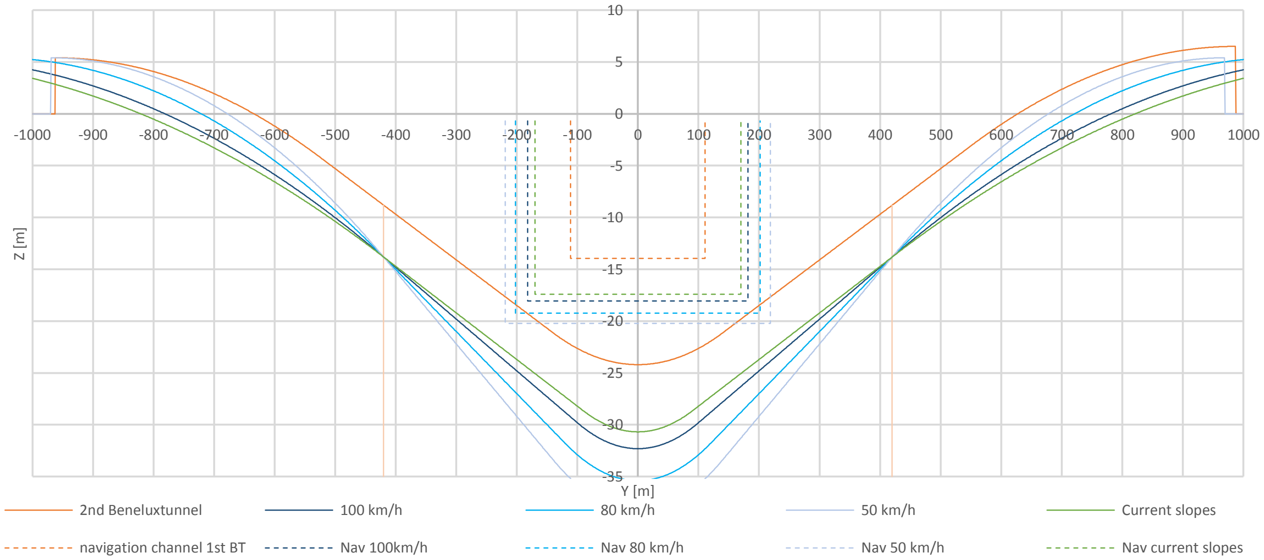


Figure 4.16: This figure shows the possible vertical profiles of the 2nd Beneluxtunnel for the alternative: General lowering on both sides. (0=NAP, left is south, right is north)

Local adjustments on one side

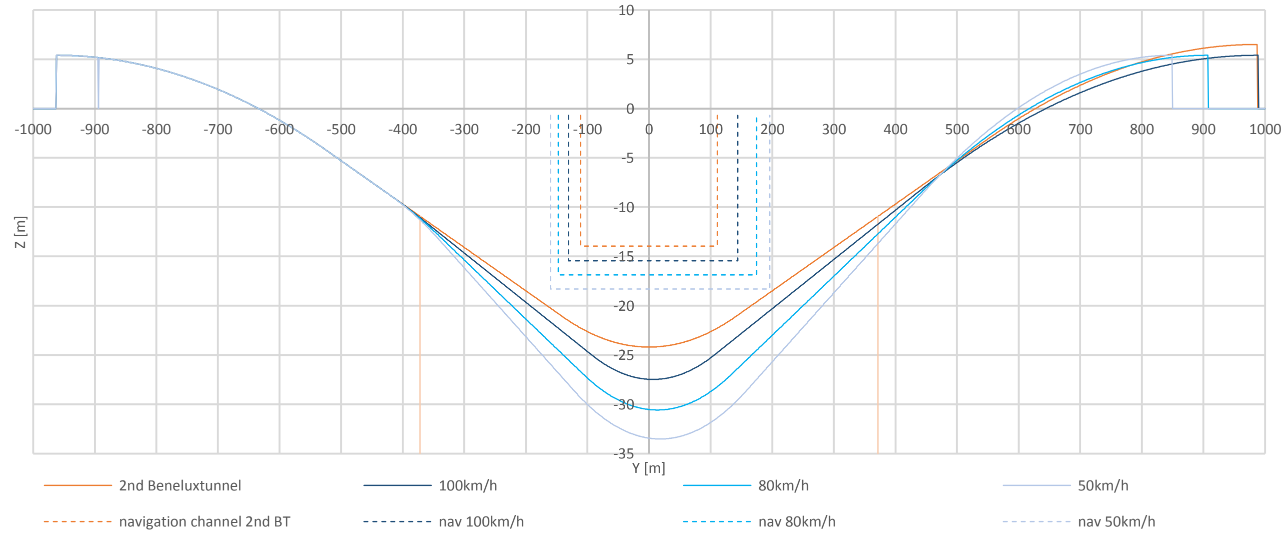


Figure 4.17: This figure shows the possible vertical profiles of the 2nd Beneluxtunnel for the alternative: Local adjustments on one side. (0=NAP, left is south, right is north)

General lowering on one side

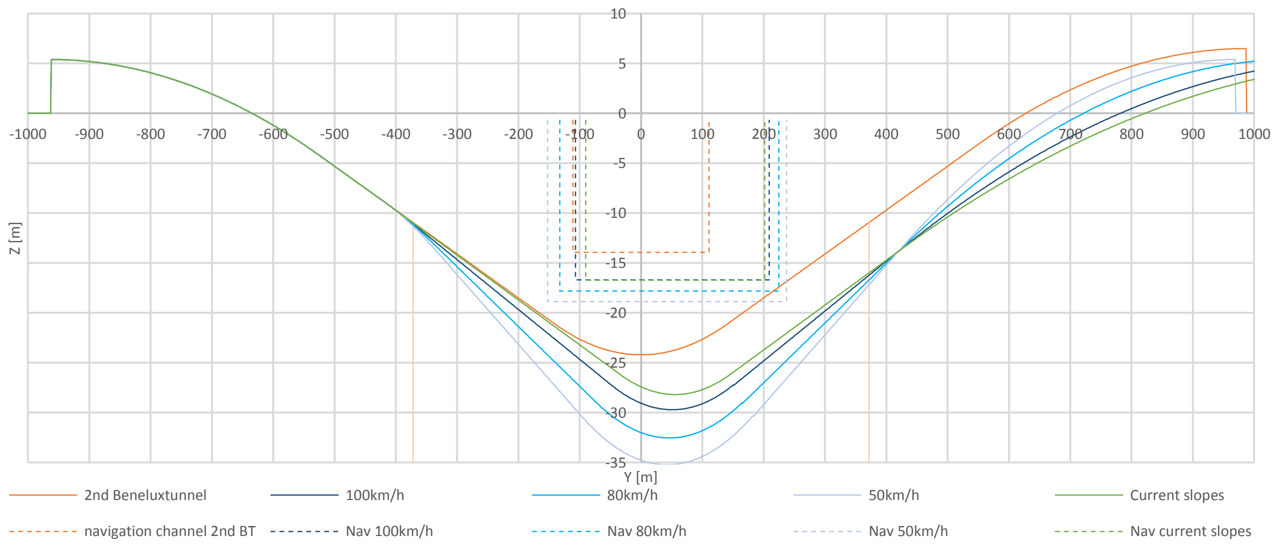


Figure 4.18: This figure shows the possible vertical profiles of the 2nd t Beneluxtunnel for the alternative: General lowering on one side. (0=NAP, left is south, right is north)

4.4.4 Conclusions

In this section, the outcomes of the vertical profiles will be discussed.

Figure 4.19 shows the possible depth increase of the navigation channel for the scenarios displayed in figures 4.9 through 4.18. Also the depth of the Maeslantkering which could be regarded as being the maximum possible depth (section 3.4.2) is shown.

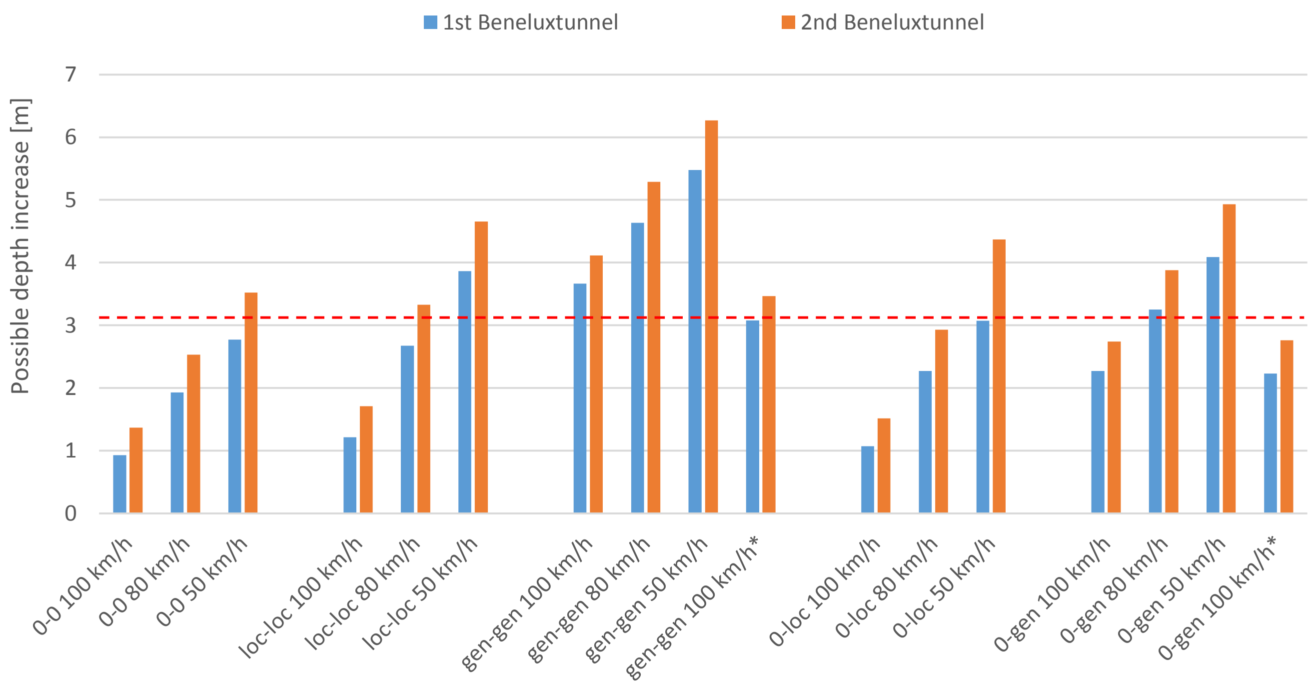


Figure 4.19: This chart shows the possible depth increase for the 1st and the 2nd Beneluxtunnel for different design options. The dashed line shows the depth of the Maeslantkering, which could be regarded as being the maximum possible depth.

Legend

The abbreviations on the horizontal axis in figure 4.19 represent the following design options:

0-0	Do nothing on both sides.
loc-loc	Local adjustments on both sides.
gen-gen	General lowering on both sides.
0-loc	Local adjustments on one side.
0-gen	General lowering on one side.

They are combined with different traffic requirements, indicated by their design speed. The star indicates the traffic requirements associated with the existing traffic requirements, which is indicated in figures 4.9 through 4.18 as “current slopes”.

Comments

Before discussing the conclusions, it must be stressed that the scenarios presented in this section are the maximum possible scenarios according to functional requirements. Whether they are also technically possible will be discussed in chapter 6. Consequently, vertical profiles of the road may lie higher, but never lower, and the provided depth of the navigation channel may be smaller but never larger.

It must also be noted that the options regarding general lowering are based on 5 m lowering of the land parts at the transition point. Whether this is technically feasible will also be discussed in chapter 6. Consequently, also less lowering of the land part is possible.

The following conclusions can be made regarding these vertical profiles and their associated navigation channel dimensions:

- The lowering capacity from a functional point of view seems to be larger for the 2nd Beneluxtunnel than for the 1st Beneluxtunnel. The most important reason seems to be the fact that the horizontal distance (along the axis) between the land parts is larger, which increases the effect of a larger slope.
- With every step the speed is decreased, the provided depth is increased with 0.8 to 1.6 m. The influence of going from 100 km/h to 80 km/h is about 20% larger than from 80 km/h to 50 km/h.
- According to functional requirements, many scenarios provide significant lowering. Both scenarios in which the functional requirements are changed as scenarios in which technical solutions are used are able to provide lowering within the required range.

4.5 Summary

In this chapter, the most important aspects regarding the functional side of the design have been discussed. The relationship between traffic and navigation has been determined and has resulted in an overview of the depth increase for different configurations of scenarios and functional requirements.

Traffic

Traffic requirements determine the vertical profile of the tunnel. The existing tunnels have a slope of about 4.5%, which could be increased if the speed is lowered.

- The slope can be increased to 5% without lowering the speed due to changed design codes.
- The slope can be increased to 6% or 7% if the speed is decreased to respectively 80 km/h or 50 km/h.

The economic consequences of such speed reduction will be incorporated in the evaluation process, as will the consequences of temporary traffic disturbance.

Navigation

The relationship between the channel boundaries and the vertical profile of the tunnel is determined by the vertical distance between the road and the channel and the differences in horizontal alignment. The proportions of the navigation channel depend on the proportions of the design vessels it is intended for. Combined, these two aspects are used to determine the size of the navigation channel if the depth is increased.

Vertical profiles

With three different options for each land part, five interesting scenarios are proposed. Combined with the changes in traffic requirements, a large range of vertical profiles is possible. Combined with the size of navigation, the functional possibilities and limitations are determined. It turns out that the 1st Beneluxtunnel is governing because of the smaller distance between the land parts of this tunnel.

Chapter 5

Structural analysis of the existing tunnel

Figure 5.1: The tunnel elements of the 1st Beneluxtunnel [12]

5.1 Introduction

In this chapter, the existing tunnels and their important attributes will be structurally analysed. The loads will be estimated, the mechanics will be elaborated and for a few critical subjects, the capacity will be determined.

The following subjects will be discussed in this chapter:

Loads	First the loads acting on the immersed part of the tunnels will be determined. (Section 5.2)
Cross-section mechanics	Next, the mechanics of the cross-section of the immersed part of the tunnel will be discussed, which includes determining the structural capacity of the 1st Beneluxtunnel. (Section 5.3)
Longitudinal mechanics	Then, the mechanic of the immersed part from a longitudinal perspective will is treated, which includes the functioning of the segmented tunnel design and estimating the shear capacity of the tunnel. (Section 5.4)
Land part	Finally, also the loads and mechanics of the land part will be discussed. This will be performed only qualitatively. (Section 5.5)

Conditions

For this chapter, the following two conditions apply:

- As discussed in section 3.3.2, the focus in this chapter will be on the immersed part rather than on the land part.
- As discussed in section 3.4.1, not all required technical information was available for this study. Consequently, structural capacity will only be determined for the 1st Beneluxtunnel.

5.2 Loads acting on the immersed part

In this section, the loads acting on the structure will be discussed. The following loads are treated:

- Hydrostatic pressure
- Self-weight
- Soil pressure
- pre-tensioning
- Flows and currents
- Temperature
- Settlement
- Other

Load factors

As mentioned in section 3.4.1, no safety factors regarding the loads will be applied.

One could argue that this would give an overly optimistic representation of structural safety, but this is not expected to be the case. Safety levels are used to express uncertainty. However, as the loads on immersed tunnels are dominated by hydrostatic pressure, which depends on the water level and therefore has a physical maximum due to the height of dikes, only to be exceeded by local waves.

It is interesting to see what the effect of the use of safety levels is. If a safety factor of 1.5 would be applied, a section of the tunnel with a depth of NAP -20m would be designed for a maximum water level of nearly NAP +16m, which is about 12m above the height of the dikes and therefore very unrealistic.

What safety factors are used for the 1st and 2nd Beneluxtunnel during their design could not be determined for this study.

5.2.1 Hydrostatic pressure

The largest and therefore most important load is the hydrostatic pressure, caused by the body of water around the elements. The magnitude of this load depends on the weight of the water above the point of interest and thus depends on the depth and on the density of the water.

Figure 5.2 shows the hydrostatic pressure for the lower and upper hydrostatic conditions as defined in section 3.5.2. Also the corresponding values for the current deepest point of the tunnel is displayed.

Figure 5.3 shows the course of the maximum hydrostatic pressure over the length of the tunnel for both the 1st and the 2nd Beneluxtunnel.

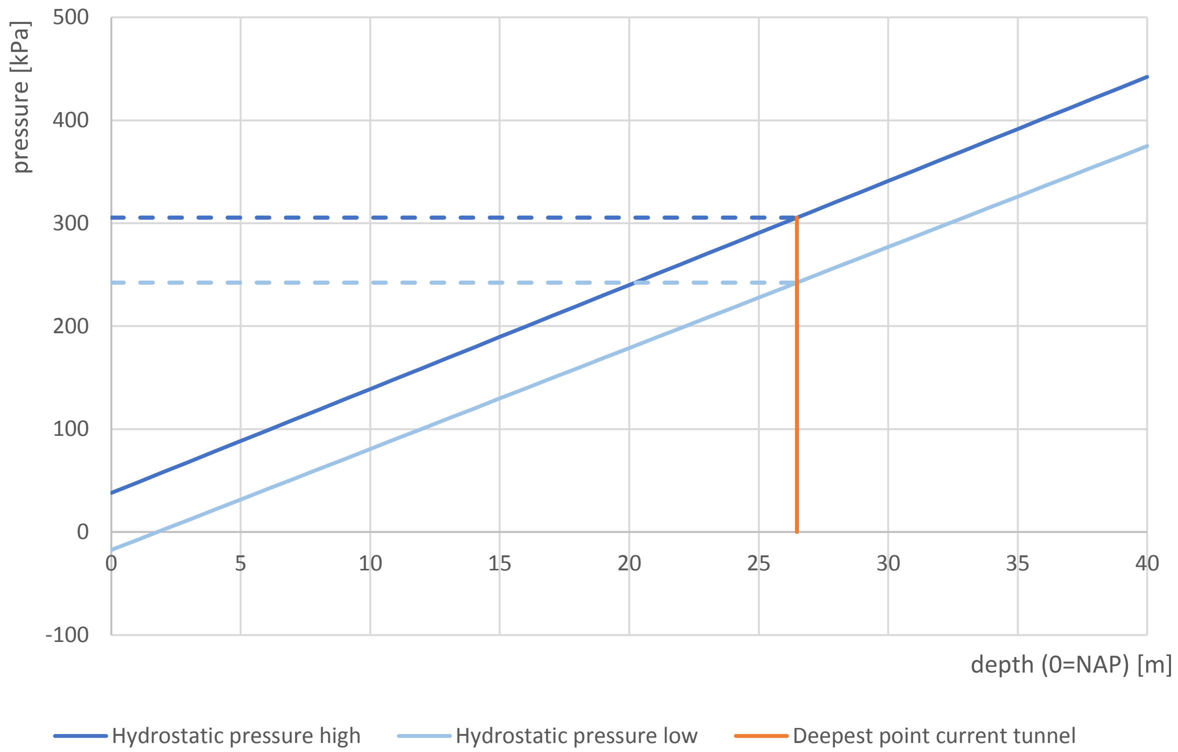


Figure 5.2: Hydrostatic pressure for different depths for the circumstances of the Beneluxtunnel. The deepest point of the tunnel is the bottom of the 2nd Beneluxtunnel in the centre of the waterway.

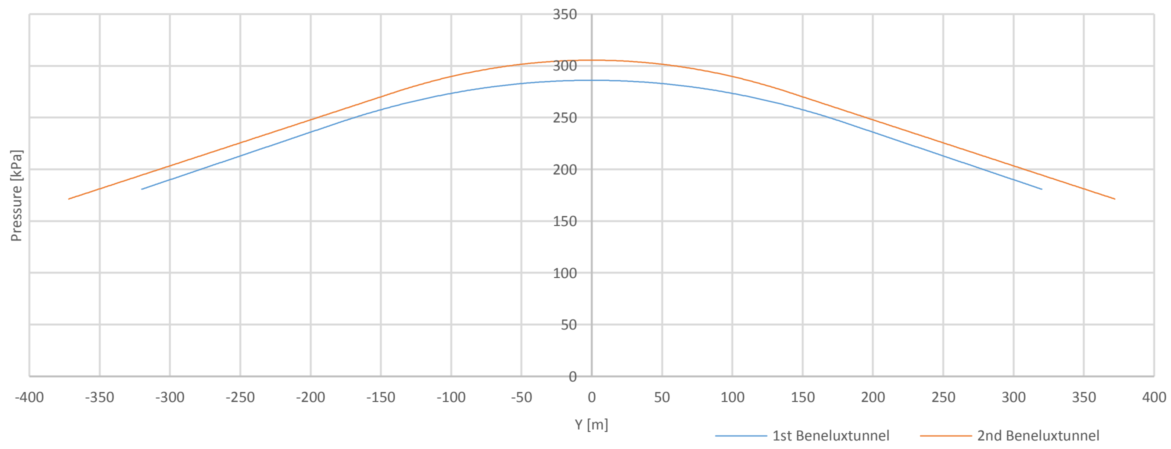


Figure 5.3: Maximum hydrostatic pressure over the longitudinal alignment of the immersed parts of the tunnel

Looking at the tunnel's cross-section, the hydrostatic pressure can be interpreted as a set of distributed loads, as is shown in figure 5.4. The characteristics of these loads are:

Floor An upward directed uniform distributed load with a magnitude as shown in figure 5.3.

Roof A downward directed uniform distributed load with a magnitude equal to the floor, minus the pressure difference over the height of the tunnel, which depending on the water density, is:

- ΔP = 76.9 - 79.2 kPa for the 1st Beneluxtunnel
- ΔP = 83.2 - 85.7 kPa for the 2nd Beneluxtunnel

Walls A horizontal directed trapezoidal distributed load with a magnitude that equals to the floor at the lower side and to the roof at the top side. Because of load this difference over height, the horizontal hydrostatic load also imposes an external moment on the tunnel.

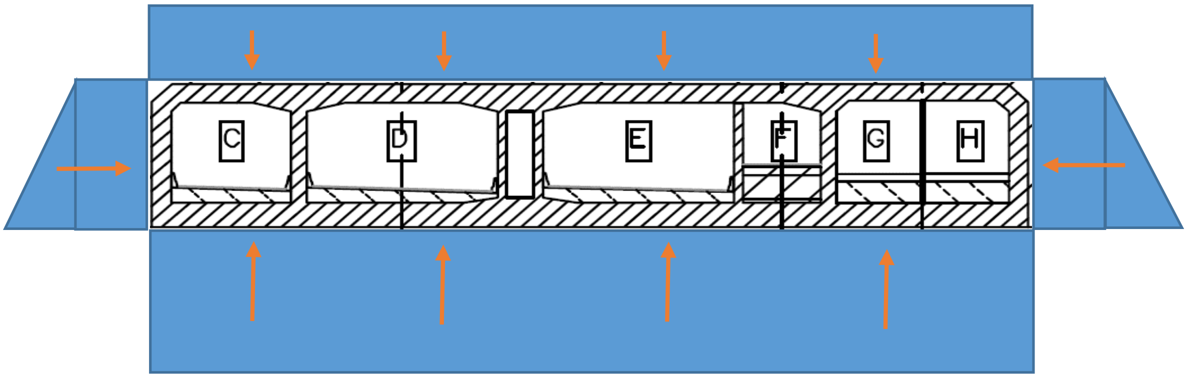


Figure 5.4: The hydrostatic load acting on the cross-section of the 2nd Beneluxtunnel.

The difference between the loads on the roof and on the floor causes an upward directed resultant load which is the buoyancy force of the elements. This force depends on the weight of the displaced water and thus not on the depth, except if the density of the water differs over depth, which in tidal areas can be the case, as is discussed in section 3.5.2.

Furthermore, the water level can fluctuate due to the presence of waves which means the hydrostatic load has a dynamic component. This dynamic loading is in thought to be a plausible reason for settlement, as it can cause cyclic compaction of the loosely packed foundation soil. However, it will not be included in this structural analysis. [56]

5.2.2 Self-weight

The self-weight of the tunnel is used to counteract the buoyancy force caused by the hydrostatic pressure. Two phases exist in the self-weight loading:

- Before immersion, the self-weight has to be slightly smaller than the buoyancy force to allow floatation of the elements, meaning the structural components of the tunnel elements weight just below the weight of the displaced water.
- After immersion, the self-weight has to be sufficiently larger than the buoyancy force. This extra weight is provided by ballast concrete and the road deck.

1st Beneluxtunnel

To estimate the cross-sectional self-weight for the 1st Beneluxtunnel, three components are included as is shown in table 5.1.

	Cross-sectional area [53]	Specific weight	Cross-sectional weight
Structural concrete	61.310 m^2	2500 kg/m^3	1504 kN/m
Ballast + road deck	17.089 m^2	2400 kg/m^3	402 kN/m
Water tight shell	8.233 m^2	2400 kg/m^3	194 kN/m
Displaced water	180.131 m^2	1000 - 1030 kg/m^3	1767 – 1820 kN/m

Table 5.1: Cross-sectional self-weights for the 1st Beneluxtunnel

The cross-sectional weight of an element before immersion would be 1697 kN/m which is 93% to 96% of the weight of the displaced water. The cross-sectional weight of an element after immersion would be 2100 kN/m which is 115% to 119% of the weight of the displaced water.

2nd Beneluxtunnel

For the 2nd Beneluxtunnel larger values are obtained as is shown in table 5.2.

	Cross-sectional area [57]	Specific weight	Cross-sectional weight
Structural concrete	144.6 m^2	2500 kg/m^3	3546 kN/m
Ballast + road deck	38.9 m^2	2400 kg/m^3	917 kN/m
Displaced water	382.9 m^2	1000 - 1030 kg/m^3	3756 - 3868 kN/m

Table 5.2: Cross-sectional self-weights for the 2nd Beneluxtunnel

The cross-sectional weight of an element before immersion would be 3546 kN/m which is 92% to 94% of the weight of the displaced water. The cross-sectional weight of an element after immersion would be 4463 kN/m which is 115% to 119% of the weight of the displaced water. The relative weights are therefore very similar for both tunnels.

5.2.3 Soil pressure

As the tunnel elements are embedded underneath the waterway, the surrounding soil acts as a load on the elements. The cross-sectional loading scheme will look similar to the hydrostatic loading scheme as shown in figure 5.4.

Roof The weight of the soil above the element, including possible bottom protection, acts as a downward directed uniform distributed load on the roof of the elements, similar to the hydrostatic pressure. The magnitude of this load differs throughout the length of the tunnel as is discussed in section 2.4.3.

For lowering purposes, we are looking at the deeper parts of the tunnel, where the maximum layers of soil are 1.0 m of sand and 0.5 m of rubble, giving a pressure of 13.7 kN/m^2 which is 328 kN/m over the width of the 1st Beneluxtunnel and 620 kN/m over the width of the 2nd Beneluxtunnel.

Walls The walls are loaded with a horizontal pressure soil pressure, which is a function of the vertical soil pressure. Due to the large width of the structure, it is assumed that yearly temperature changes result in displacements that increase the soil pressure, and thus an earth pressure coefficient of K = 1 is used. [58]

Consequently, the horizontal loads resulting from the soil pressure will be:

- 13.7 kN/m^2 for the top side of the tunnel
- 90.6 kN/m^2 for the bottom side of the tunnel of the 1st Beneluxtunnel
- 97.0 kN/m^2 for the bottom side of the tunnel of the 2nd Beneluxtunnel

Floor The upward soil pressure on the floor counteracts the vertical downward load resultant of the tunnel and is therefore case and location specific.

5.2.4 Pre-tensioning

Immersed tunnels can have longitudinal pre-tensioning and transverse pre-tensioning, which will both be discussed in this section.

Longitudinal pre-tensioning

Longitudinal pre- tensioning is very important for the design of immersed tunnels, as it keeps the segments within an element connected during the OTAO-process. However, after immersion, the longitudinal pre-tensioning is cut, allowing the segment joints to become functional. Their effect is therefore not included.

Transverse pre-tensioning

The 1st Beneluxtunnel is one of few tunnels also containing pre-tensioning in the transverse direction, which is shown in figure 5.5.

The type of pressing is BB.138-44Ø6. As indicated in section 3.5.3, this steel type corresponds with a tensile strength of 1354 N/mm^2, from which the initial stress can be derived. Furthermore, 44 is assumed to be the number of wires and Ø6 is the diameter in mm.

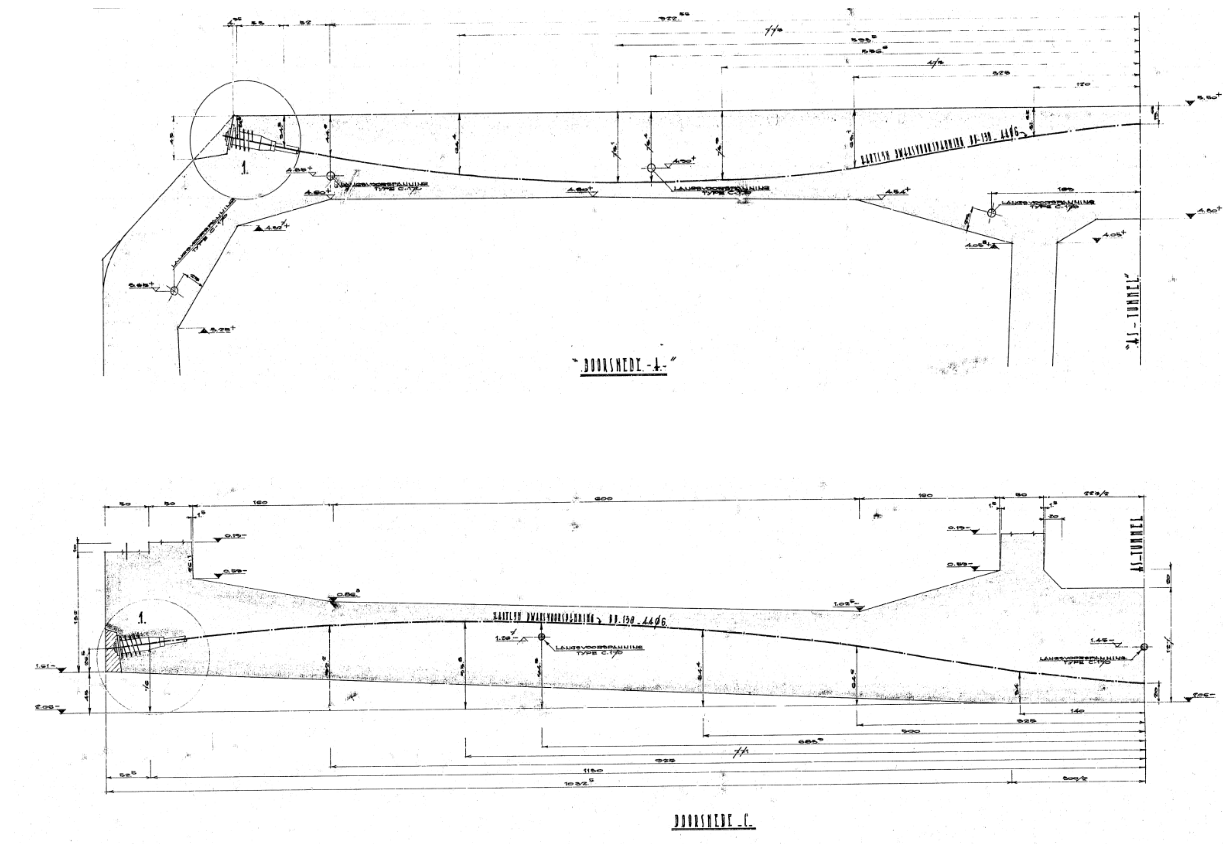


Figure 5.5: The pre-tensioning of the 1st Beneluxtunnel in the roof and floor. [59, 60]

Also, a tendon spacing of 1100 mm is found and a time dependent loss of 15 % is assumed, leading to the following design values:

		BB-138	
Characteristic tensile strength:	f_{pc}	1354	N/mm^2
Initial pre-tensioning:	σ_{Pm0}	1015	N/mm^2
Actual pre-tensioning:	$\sigma_{Pm\infty}$	863	N/mm^2
Area:	A_p	1244	mm^2/tendon
	A_p	1121	mm^2/m
Working pre-tensioning force:	$P_{m\infty}$	1074	kN/tendon
	$P_{m\infty}$	967	kN/m

To determine the equivalent loads caused by the pre-tensioning, the positioning of the tendons is slightly simplified. The lengths, drapes and the equivalent loading becomes:

	length [m]	**drape [mm]**	**q [kN/m/m]**
Roof-big	9.079	500	48
Roof-small	2.392	100	139
Floor-big	9.95	500	40
Floor-small	2.75	100	105

5.2.5 Flows and currents

As mentioned in section 2.4.3, the elements of the 2nd Beneluxtunnel were lowered onto gravel ridges instead of onto the tiles, because flow speeds caused by passing of ships would induce loads that might have been able to knock the elements of the supporting jacks. It is therefore very likely for this load to again play an important role during the construction phase.

Estimates for the 2nd Beneluxtunnel

This subject has been extensively investigated for the 2nd Beneluxtunnel and therefore useful estimates of these forces are available. The situation when the tunnel elements were lowered onto tiles in the trench was modelled. Given that the navigation channel has not increased, it can be assumed that these values would still be valid today. The river flow is already included. [27]

The results of this study are given in table 5.3.

	Passing distance 0m		Passing distance 210m		
	Max	**Min**	**Max**	**Min**	
F_X	29910	-30020	3826	-3406	kN
F_Z	16090	-11480	11800	-2213	kN
M_Y	174000	-173600	21770	-19980	kNm
M_X	0	0	70090	-373800	kNm
M_Z	0	0	117200	-111300	kNm

Table 5.3: The extreme values of ship flow forces on the elements of the 2nd Beneluxtunnel.

Estimates for the 1st Beneluxtunnel

To estimate the loads for the 1st Beneluxtunnel, the results will be factored. The point loads will be factored according to the area they are based on and the moments will be factored according to another characteristic value $1/12*L_1\,(L_2^2+L_3^2)$ in which L_1 is the length of the side perpendicular to the direction of the axis and L_2 and L_3 are the lengths of the other two sides.

The results are given in table 5.4.

		Passing distance 0m		Passing distance 210m		
	factor	**Max**	**Min**	**Max**	**Min**	
F_X	0.61	18358	-18426	2348	-2091	kN
F_Z	0.35	5645	-4028	4140	-776	kN
M_Y	0.20	34502	-34422	4317	-3962	kNm
M_X	0.23	0	0	16392	-87420	kNm
M_Z	0.39	0	0	46124	-43802	kNm

Table 5.4: The extreme values of ship flow forces on the elements of the 1st Beneluxtunnel, based on the 2nd Beneluxtunnel.

According to the factors, the differences between the 1st and the 2nd Beneluxtunnel are very large. Whether these factors are representative will not be answered within this report. But given the fact that during the design of the 2nd Beneluxtunnel this aspect was only addressed at a very late stage, it could be expected that the forces on the long and very wide elements of the 2nd Beneluxtunnel were much larger than what could logically be expected based on the experiences of earlier tunnels.

Present state

Currently however, the tunnels are fully covered in soil, meaning they are only subjected to relatively slow groundwater flows. Also, the surrounding soil provides an immediate reaction force. Apart from minor deformations, no problems are to be expected. If soil is removed and the tunnel becomes exposed, this load will become important.

5.2.6 Temperature

In this section, temperature loads will be discussed. Temperature differences can exist in time, causing thermal expansion and contraction, but these differences can also exist within the concrete itself causing internal stresses. The latter is very important for the durability of the concrete, but it will not be discussed in this section.

Thermal expansion

Thermal expansion is the expansion of the tunnel caused by difference in water temperature relative to the temperature at the time of construction. In the transverse direction, this load results in an increase in soil pressure as indicated earlier in this section. In the longitudinal direction, the effects of this expansion depends on the way the joints of the tunnels are constructed.

For the 1st Beneluxtunnel no significant use of compressible materials in any of the joints is applied. A temperature increase of 20^o C would therefore result in an increase in length of about 179 mm which has to be absorbed by the land parts. For the 2nd Beneluxtunnel, 10 mm thick layers of tempex are applied in the immersion joints which accompany the compressive capabilities of the GINA-profiles. If these layers would be able to be fully compressed, similar conditions would result in an increase in length of 142 mm.

In practice, much of the expansion will be counteracted by shear stresses in the interface between the concrete and the soil and will therefore not reach the land parts but will be converted to soil stresses and concrete stresses.

If a section of concrete is fully restrained, a 20^o C increase in temperature would result in a compressive stress of 9.3 Mpa, which is well within the capacity of the concrete, but nevertheless quite a lot. Possibly a 20^o C temperature differences is unrealistically high for the sheltered conditions of immersed tunnels. Also, the temperature at the time of construction determines the initial situation. It therefore differs if the tunnel is constructed in the summer or in the winter. It is therefore assumed that the temperature can ultimately be 5^o C higher or lower than the initial temperature, resulting in a maximum compressive stress of 2.3 Mpa.

Thermal contraction

The segment joints of both tunnels allow contraction of the concrete, preventing tensile stresses.

Another problem that often occurs related to the temperature load is when the segments are cold and the joints are open, soil gets into the joints. If the segments now expand again and the joint closes, the soil will be trapped and will slowly start to push the segments from each other. For this reason, often protective plates outside of the joints and elastic filling materials inside of the joints are applied.

5.2.7 Settlement

As the tunnel elements are directly founded on soil, settlement can occur. As with all civil structure, settlement only becomes a structural issue when it occurs unevenly. The amount of settlement is determined by the loading of the soil relative to the strength of the soil. As these can both locally vary, it is clear why differential settlement can occur.

Immersed tunnels are designed to allow differential settlement as the segment joints (section 2.4.3) have limited freedom of rotation. This way, the segments of a tunnel settle in a so called chain line. However, the tunnel is also attached to the land parts, which has piled foundations and is therefore significantly less prone to settlement.

Beneluxtunnel

Measurements regarding settlement of the Beneluxtunnels are found in appendix E. The settlement lines of the latest measures of the 1st Beneluxtunnel are displayed in figure 5.6. This figure clearly shows the unsettled segments near the land part and the differences in settlement in between. The maximum settlement relative to the 1st measurement is about 24 mm.

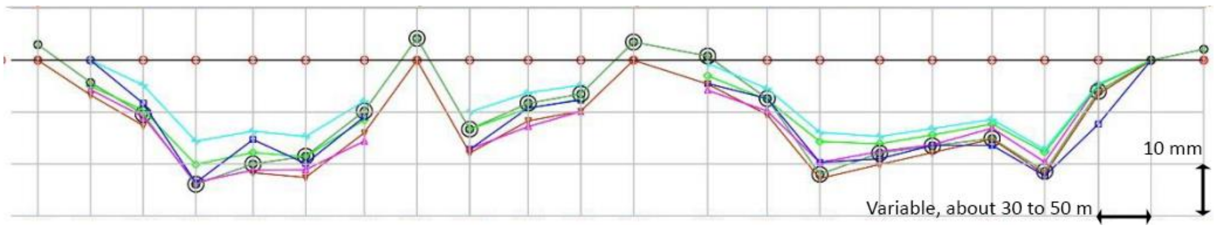


Figure 5.6: The settlement lines of the 22th measurement (2000) until the 27th measurement (2011) of the 1st Beneluxtunnel, showing the displacements of the main axis in the z-direction relative to the 1st measurement in 1982. More information on the locations and the axes can be found in appendix E.

5.2.8 Other

Other loads that can act on immersed tunnels are loads associated with the use and misuse of the functions of the tunnel.

Traffic

Normal traffic use can cause loads of up to 15 kPa for large trucks. Traffic accidents can cause explosions and collisions. Explosions can be schematised as a static load of 100 kPa. Collisions are expected to have less impact, partly due to their direction relative to the tunnel walls. Traffic accidents can also cause fires that decrease the strength of the tunnel which could definitely be a problem. This subject is extensively investigated and in the past decades and many tunnels have been coated on the inside with fire delaying protection materials.

Traffic loads act opposite of the hydrostatic pressure and are therefore less governing. Given the fact that the tunnel will not be used during construction, it is chosen not to incorporate these traffic loads into the structural analysis.

Navigation

Navigation can also cause loads other than their induced flows. Anchors can for instance be dropped onto the tunnels or be dragged along the bottom. It is also possible that a ship would sink right above the tunnel. These loads act in the same direction as the hydrostatic pressure and can therefore important.

However, it is assumed these accidental navigation loads are not present during the construction phase and can afterwards be withstand by the tunnel and its bottom protection.

5.3 Cross-section mechanics of the immersed part

In this section, the mechanics of the cross-section of the tunnel will be analysed. Only the 1st Benelux-tunnel will be treated as only for this tunnel, information regarding its reinforcement is available.

5.3.1 Loads

In the cross-section of the tunnel, the hydrostatic load is by far the largest the largest and therefore mainly determines the mechanics of the cross-section. Interesting is that the vertically directed hydrostatic loads and the horizontally directed hydrostatic loads counteract each other's effect.

However, while significantly smaller, other loads do have an important effect on the cross-section. The other loads combined, result in a downward instead of an upward directed vertical load resultant. Also the difference between the roof and the floor are reduced as resulting from these downward directed loads acting on these slabs, which is increases the load on the roof and decreases it on the floor. The presence of pre-tensioning decreases the load on both of these slabs. The soil pressure increases the load on all slabs.

Figure 5.7 shows the cross-sectional load resultants on the roof slab, side slabs and floor slab, in which the following loads are incorporated:

- A hydrostatic pressure at a maximum depth of NAP -24.5 m (roof at NAP -18.2 m) and a maximum water level of NAP +3.75 m. An extra 2 m of water is added to take waves and other irregularities into account. (Section 5.2.1)
- The self-weight of the slabs and the ballast. (Section 5.2.2)
- Soil pressure, including the reaction force of the subsoil. (Section 5.2.3)
- Transverse pre-tensioning. (Section 5.2.4)

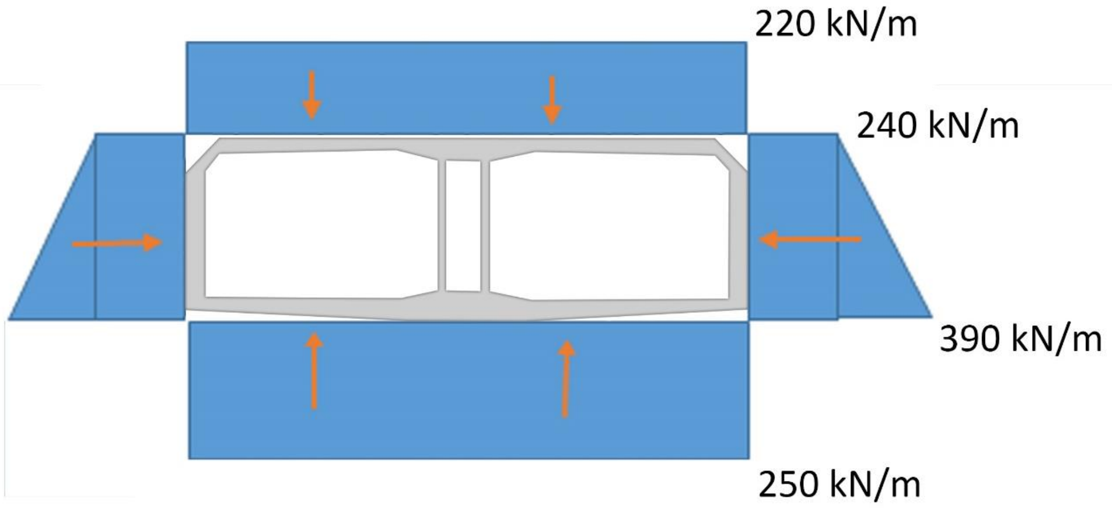


Figure 5.7: This figure shows the load scheme of the 1st Beneluxtunnel

5.3.2 Mechanics

In this section, the internal loads in the cross-section will be estimated.

Structural features

The cross-section of an immersed tunnel is more or less the same throughout the length of the tunnel. Small differences exist at the locations where doors and ventilations are located, but it is assumed that at these locations are designed to be at least as strong regular cross-section. Near the joints different rules apply and are therefore treated separately.

What can be noticed regarding the concrete design is that, especially for the first Beneluxtunnel, all slabs have different thicknesses. Near the inner walls, slanting sections in both tunnels are used to resist the shear forces.

The reinforcement design is only known for the 1st Beneluxtunnel. What can be noticed is that the reinforcement is – unlike the concrete – not uniformly applied throughout the length of the tunnel. The locations are similar, but different diameters are applied.

Furthermore, transverse pre-tensioning is present which has already been discussed as an external load.

Model

To determine the effects of the different loads on the structure, a model is made using Matrixframe. Effort has been made to incorporate as many of the structural features as possible in the model, as it is expected to influence the distribution of the internal forces. To validate the model, also a simplified version of it is made. Information regarding the model, its features and its validation can be found in appendix F. A graphical representation of the model is displayed in figure 5.8.

The moments and shear forces are shown in figures 5.9 and 5.10.

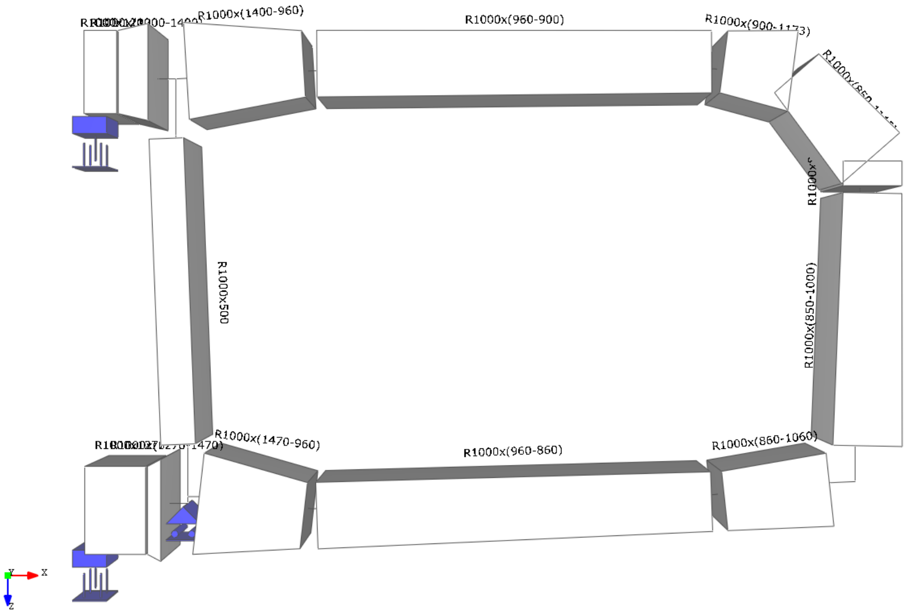


Figure 5.8: A graphical representation of the Matrixframe model

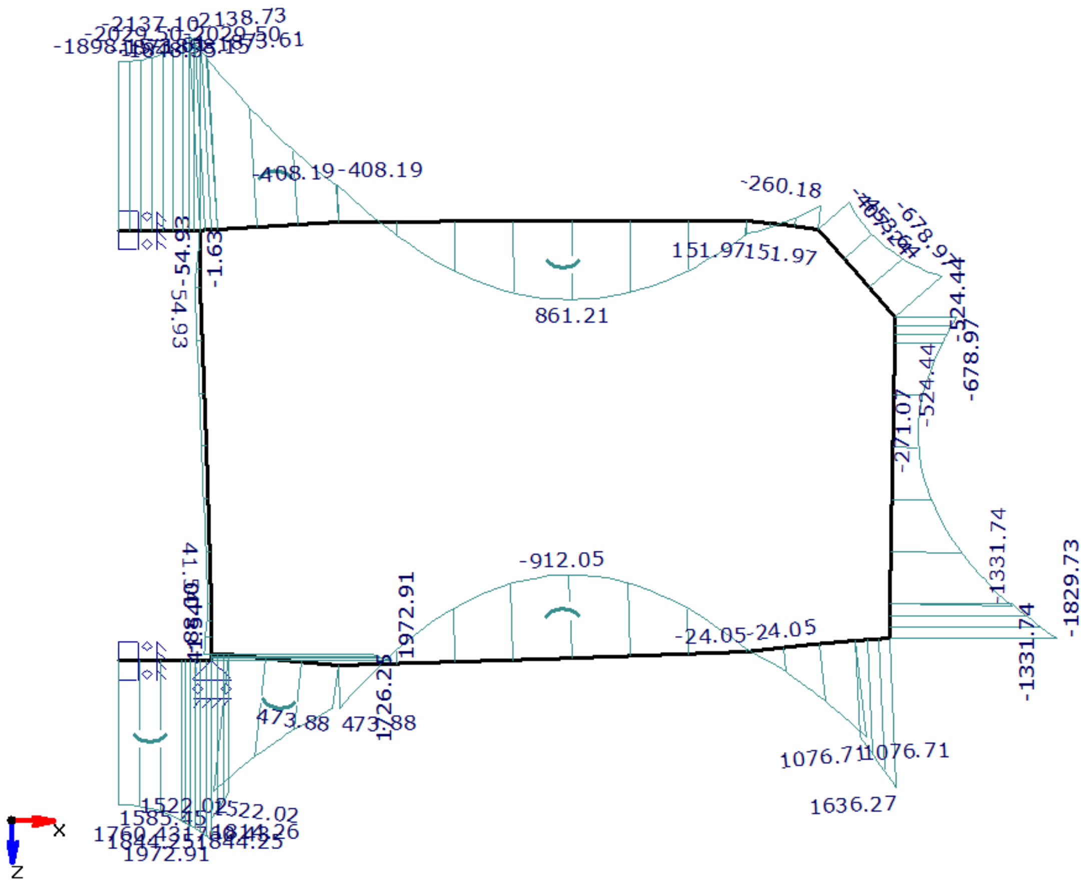


Figure 5.9: The moments acting on the cross-section of the 1st Beneluxtunnel in [kNm]

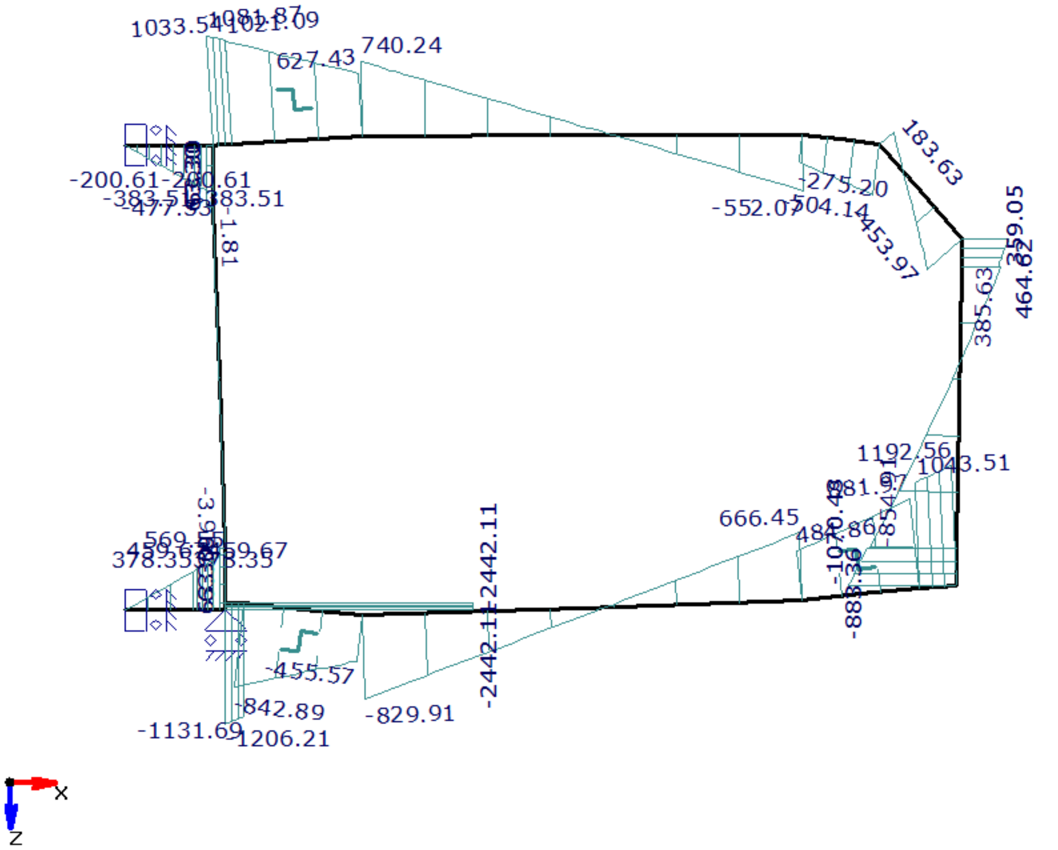


Figure 5.10: The shear forces acting on the cross-section of the 1st Beneluxtunnel in [kN]

5.3.3 Structural capacity

To determine the structural safety of the tunnel, a number of cross-sections are analysed regarding moment and shear capacity. As the reinforcement differs throughout the length of the tunnel, one of the lowest tunnel segments that is not next to an immersion joint (segment 4c) is chosen to determine this capacity.

The choice of locations is indicated in figure 5.11.

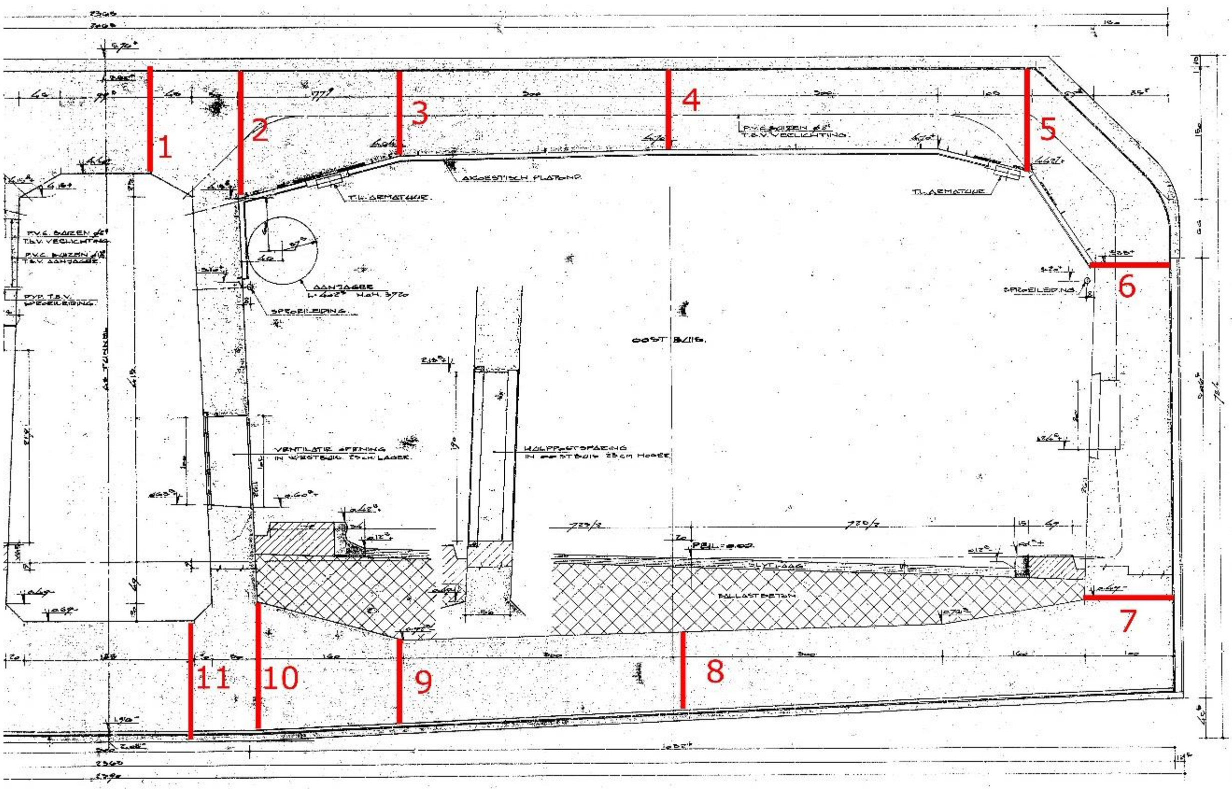


Figure 5.11: The locations where a safety check has been performed

Calculation methods

To determine the moment capacity, the height of the compression zone is determined using the maximum elastic yielding capacity of the reinforcing steel, which turned out to be governing rather than the concrete or the pre-tensioning steel.

To determine the shear capacity, the method proposed in the Eurocode has been used. However, its suitability is questionable. For the Maastunnel, different methods to determine shear capacity have been compared. The method that was regarded most reliable (IBBC) gave a shear capacity of about 1.5 times the capacity determined with the eurocode method, in case there would be no shear reinforcement. It also showed that for the most complex model (finite element model using Atena) the structure would fail on moment capacity rather than shear capacity and that the differences with or without shear reinforcement were minimal. The report claims that the Eurocode method is more suitable for beams with limited width and not for continuous slabs like in immersed tunnels. Also, the unpredictability of shear capacity resulted in large safety factors. [47, 61]

Given the conceptual nature of this study, it is decided to use the Eurocode method to determine the shear capacity. However, the results will also be given multiplied with a factor 1.5. Furthermore, the same cross-section is chosen for the moment capacity as for the shear capacity, which is also too conservative as the loads near support are directly transferred to the support.

Results

The results are shown in table 5.5. The calculations can be found in appendix G. The unity check is defined as:

$$Unity\ check = \frac{Resistance}{Load}$$

	Loads				Resistance			Unity checks		
point	width	N	M	Q	M_{ULT}	Q_{ULT}	Q_{ULT}*1.5	$U_{C.M}$	$U_{C.Q}$	$U_{C.Q*1.5}$
	[m]	[kN]	[kNm]	[kN]	[kNm]	[kN]	[kN]			
1	1200	2098	1898	200	6854	1187	1780	3.61	5.93	8.90
2	1440	2099	1873	1021	8345	1278	1917	4.46	1.25	1.88
3	940	2038	408	740	2681	869	1303	6.57	1.17	**1.76**
4	900	2038	861	0	2277	803	1204	**2.64**		
5	1170	2088	260	504	2331	834	1251	8.97	1.66	2.48
6	850	1371	524	386	1586	668	1003	3.03	1.73	2.60
7	1000	1480	1332	855	2566	790	1185	**1.93**	0.92	**1.39**
8	900	2478	912	0	2593	877	1316	2.84		
9	960	2573	474	829	2079	857	1286	4.39	1.03	**1.55**
10	1470	2617	1522	843	5037	1124	1687	3.31	1.33	2.00
11	1270	2442	1760	378	4224	1038	1557	**2.40**	2.75	4.12

Table 5.5: Unity checks on the cross-sections at the locations indicated figure 5.11, the three lowest are highlighted.

Some comments:

- A unity factor of more than one means the structure is safe according to the safety conditions as described in sections 3.4.1 and 5.2. Consequently, the unity factors as displayed in table 5.5 can be interpreted as additional load factors.
- As the applied loads are not factored, a unity check of 1 does not mean that the structure would satisfy structural safety laws.
- The reliability of the shear capacity estimates is very questionable. While already multiplied with 1.5, the shear capacity estimates still seem too conservative as it would be expected for the shear capacity to be much closer to the moment capacity as otherwise, different design decisions would have been made. It would be advised to apply different methods to estimate the shear capacity.

Critical points

Point 7 seems to be the weakest location for both the moment and the shear capacity. Before applying a multiplication factor of 1.5, the unity check for the shear capacity would even be below one, meaning it would be unsafe.

However, given that the same cross-section has been used to calculate the moment capacity and the shear capacity, the shear capacity results for the points located close to a support are expected to be too conservative as part of the load will be transferred directly to the support. According to the Eurocode, the shear force within a 45^o angle is transferred directly to the support, consequently, the acting shear force will be lower. Apart from point 7, this will also influence points 1, 2, 10 and 11.

If we would now re-examine point 7 using a the shear loading at 45^o from the support as is indacated in figure 5.12, the unity check for the shear capaciy becomes $U_{\mathbf{c.Q}} = 1.51$ ($U_{\mathbf{c.Q*1.5}} = 2.26$). Consequently, point 9 becomes critital point regarding shear capacity.

Another influencing aspect is the structural contribution of the ballast concrete, which will influence both the moment capacity and the shear capacity. However, it is chosen not to take this into account as the strength of the ballast concrete is not expected to be reliable. Consequently, point 7 remains the critical point regarding moment capacity.

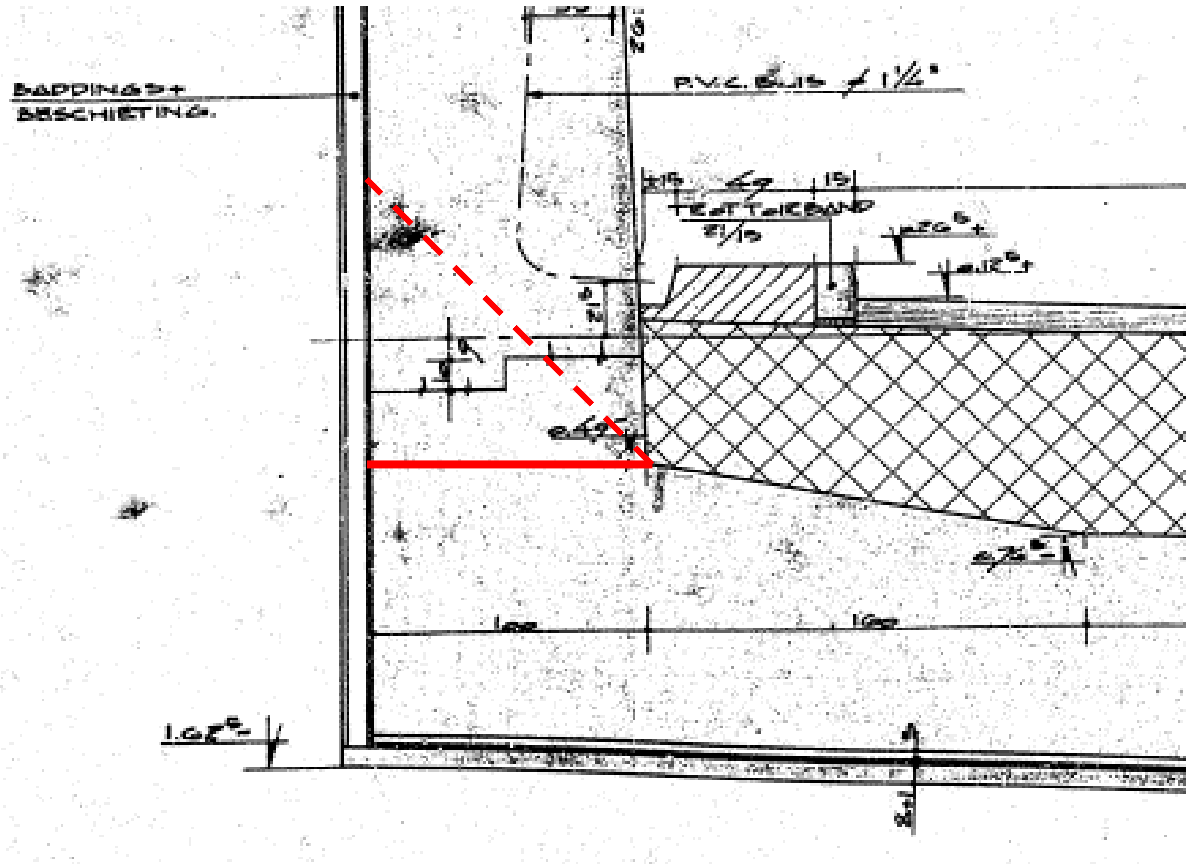


Figure 5.12: This image shows point 7 in more detail. The solid line displays the initial cross-section for which the shear capacity was determined, the dashed line displays the alternative cross-section for determining the shear capacity at a 45^o angle from the support. All shear forces below this line are transferred directly to the support.

5.4 Longitudinal mechanics of the immersed part

In this section, the longitudinal mechanics of the Beneluxtunnels will be discussed. First the longitudinal and cross-sectional load resultants will be determined. Next, the mechanics of these loads combined and their effects on the tunnel segments will be discussed. Next, the joints and their shear capacity are treated and finally, the influence of temperature loads will be discussed.

5.4.1 Loads

Longitudinal loads

When the elements are inundated and transported, the longitudinal pressure is mainly determined by longitudinal pre-tensioning in combination with hydrostatic pressure. When the element is immersed, the hydraulic pressure increases and becomes more dominant. The associated shortening of the concrete reduces the effect of the pre-tensioning. After immersion, the strands are cut and the hydrostatic pressure becomes the solely responsible for the longitudinal pressure.

Because the tunnel differs in height over its length and the longitudinal pressure depends on the magnitude of the hydrostatic pressure which depends on the height, the longitudinal pressure differs over the length of the tunnel.

When the final element is immersed, the mechanisms of the closure joints prevents the elements from expanding, trapping the longitudinal load caused by the hydrostatic pressure. Due to time dependant processes creep, shrinkage and relaxation, the longitudinal load decreases over time. However, these effects do tend to stabilise over time, meaning the elements remain connected. The amount of remaining pressure is rather uncertain, but is estimated at about 60% of the initial pressure. [62]

The longitudinal loads caused by the hydrostatic pressure are given in table 5.6.

		1st Beneluxtunnel		2nd Beneluxtunnel		
		Highest point	**Lowest point**	**Highest point**	**Lowest point**	
Hydrostatic load	**t=0**	10.3	42.4	19.1	100.5	mN
	t=∞	6.2	25.4	11.4	60.3	mN
Compression	**t=0**	0.17	0.69	0.13	0.70	MPa
	t=∞	0.10	0.42	0.08	0.42	Mpa

Table 5.6: longitudinal loads caused by hydrostatic pressure

Cross-sectional loads

The cross-sectional loads have been discussed in section 5.2.2. Only their resultant is important for the longitudinal mechanics. In undisturbed conditions, the resultant of the transverse directed loads would be zero. The vertical load depends on the amount of bottom protection and on the salinity of the water. The boundary values are given table 5.7. The situation underneath the dike is not included. Furthermore, the presence of traffic and navigation can temporarily and locally increase these loads.

	Minimum	Maximum	
1st Beneluxtunnel	280	661	kN/m
2nd Beneluxtunnel	594	1329	kN/m

Table 5.7: lower and upper boundary values of the vertical load resultants (downward is positive)

The reaction force of the foundation soil counteracts thee vertical loads. However, as the soil is not homogeneous and the loads are not constant in time as well in place, deformations can occur. Consequently, these deformations can influence the course of the longitudinal loads.

5.4.2 Mechanics of the tunnel segments

In this section, the mechanics of the segmented tunnel will be discussed by analysing the effects of longitudinal and vertical forces on the tunnel and its foundation. The process is divided into three stages.

Even though this section treats vertical loads and the effects of the subsoil, the same mechanisms would occur if the tunnel would be subjected to loads in the transverse direction.

Stage 1

If in an area underneath the tunnel the soil settles, the reaction force of the soil on the tunnel will decrease and the tunnel will have to span this area, causing a moment. As the tunnel is longitudinally compressed, the settlement will not automatically lead to tension in the bottom of the tunnel. The tunnel will therefore span the area. As the compression is larger at the bottom of the tunnel than at the top of the tunnel, which could be interpreted as a longitudinal moment, the capacity is even larger.

At this stage, it does not matter if the settled area lies underneath a joint or not. This situation is indicated in figure 5.13

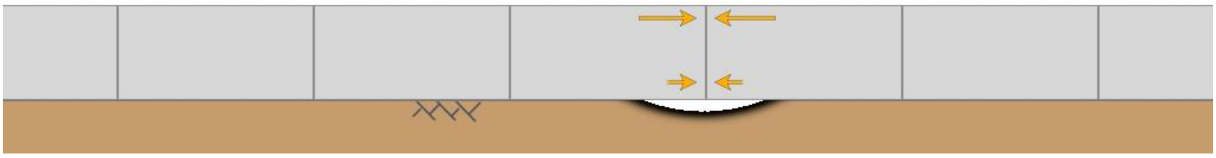


Figure 5.13: Settlement of the soil without deformation of the tunnel segments

If the span is widened, the spanning moment increases and at a certain point, tensile stresses in the concrete will start to occur. However, these tensile stresses can only develop over the distance where no joints are present as tensile stresses in the joint could not exists. Instead, they would result in the opening of the joint, stopping the expansion of the concrete and therefore limiting the tensile stresses.

This situation could be described as an on two sides inclined beam, spanning a distance L. The spanning moment would in this situation be largest near the supports and could be determined using $1/12qL^2$. Figure 5.14 shows the span at which the first tensile stresses are created in this situation, for the load cases as indicated in table 5.7. This figure also shows that tensile stresses start to occur at larger spans if the depth is larger, which is a result from the increased longitudinal loads.

As the tensile stresses only stop increasing when a joint is reached, it could be possible for the concrete to reach its tensile strength before this. This risk increases if the spanning distance is small relative to the distance between the joints, which is the case if loads are large and the depth is small.

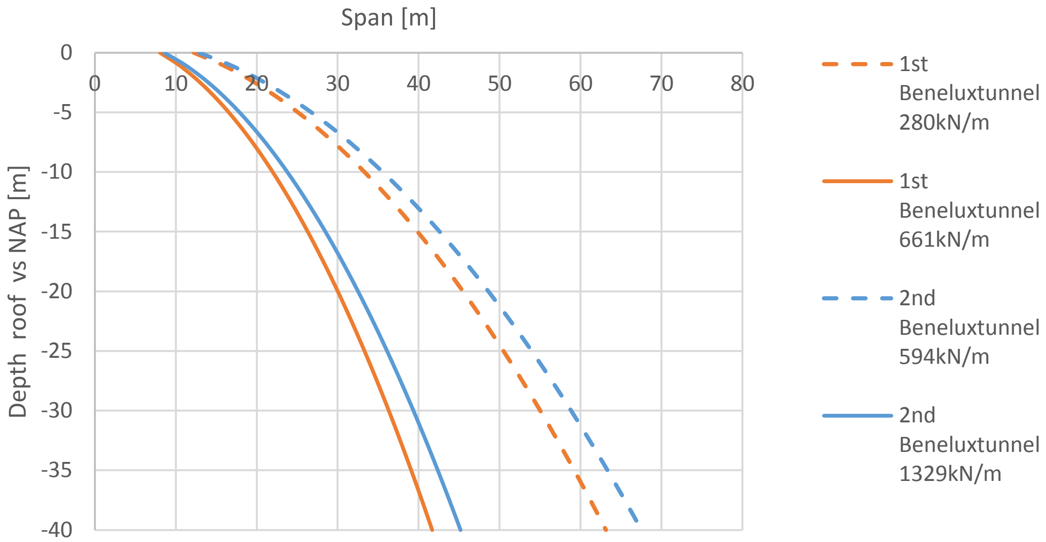


Figure 5.14: This figure shows the span at which the first tensile stresses in the concrete start to occur in case the situation is modelled as an on two sides inclined beam, for different depths and for the four load scenarios indicated in table 5.7.

Stage 2

Once the joint starts to open, the section properties of the joint and the areas around it start to change. Consequently, the tensile stresses in the joint increase, further opening the gap and therefore amplifying the process. The compressive stresses also increase which at some point will reach the plastic phase of the concrete. The pressure gradient will decrease until at some point, the interface is no longer able to transfer moments. This will allow the section properties to remain stable and the process will stop. A plastic hinge has been created.

Resulting from this process, the moment will be redistributed over the span. First the other support will increase up to $1/8qL^2$ and the nearest joint will also become a plastic hinge. Consequently, the spanning moment will increase up to $1/8qL^2$ and resulting in another plastic hinge.

At this point, all deformations in the joint and in the concrete were a result of deformations caused by deformations in the concrete. Figure 5.15 shows estimates of the deflection of the centre and the rotation at the supports of the beam model for the case regarding the 1st Beneluxtunnel loaded with 280 kN/m.

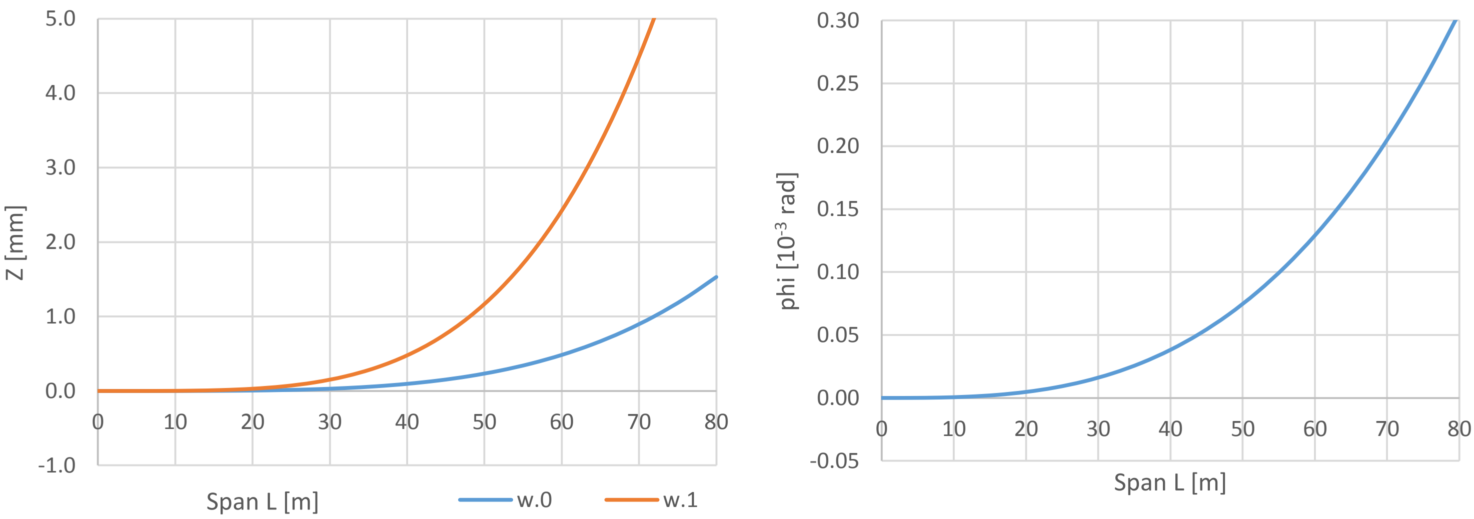


Figure 5.15: These graphs show the deflection (left) and rotation (right) for the beam model of the 1st Beneluxtunnel at a load of 280kN/m. W.0 is the deflection before redistribution of the moment and W.1 is the deflection afterwards.

Stage 3

Figure 5.16 shows a possible situation after stage 2. The joints have been opened resulting in pressure lines between the plastic hinges. Deformations and rotations are still relatively small, but given the fact that it is now a hinged structure, more rotations and therefore deformations are possible.

However, rotations do not only cause vertical deformations, but due to the shape of the tunnel segments, also horizontal deformations are caused. The magnitude of these horizontal deformations relative to the vertical deformations, depend on the angle of the pressure line and therefore on the length and height of the segments, and on the amount of segments between two hinges.

Horizontal displacement will either excite reaction forces in the concrete, or it could also move the concrete segments towards a segment joint, if present, but this will be counteracted by soil shear. Either way, reaction forces would counteract the rotations.

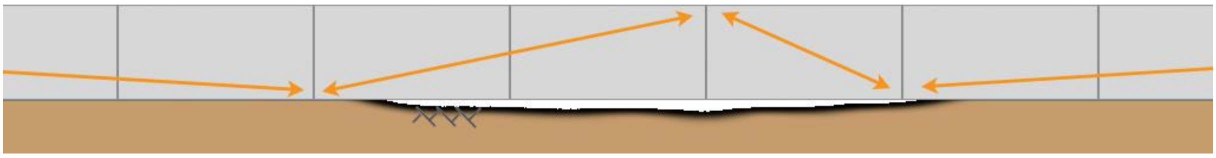


Figure 5.16: This figure shows a model of a tunnel after plastic hinges are created. At this point, deformations are still very small.

If the situation of figure 5.16 would be modelled as a hinged frame following the pressure lines, the horizontal forces on the supports (the lower outer hinges) for the case regarding the 1st Beneluxtunnel loaded with 280 kN/m would be about 12.9 mN. The magnitude of these horizontal forces would slightly increase if the location of the upper hinge lowers (about 1.6 kN/mm).

How this situation will develop depends on the reaction of the concrete on this initial load and on the additional load if the vertical and therefore horizontal displacements continue. Figure 5.17 shows the relationship between vertical and horizontal displacements and the associated reaction force of the concrete. Both graphs give results for 1, 2 or 3 segments. For the displacement this means the number of segments that rotate, for the reaction force this means the number of segments that participate in the reaction. These figures show that reaction forces are extremely large and would prevent the tunnel segments from rotating enough for vertical deformations to follow the settlement of the soil.

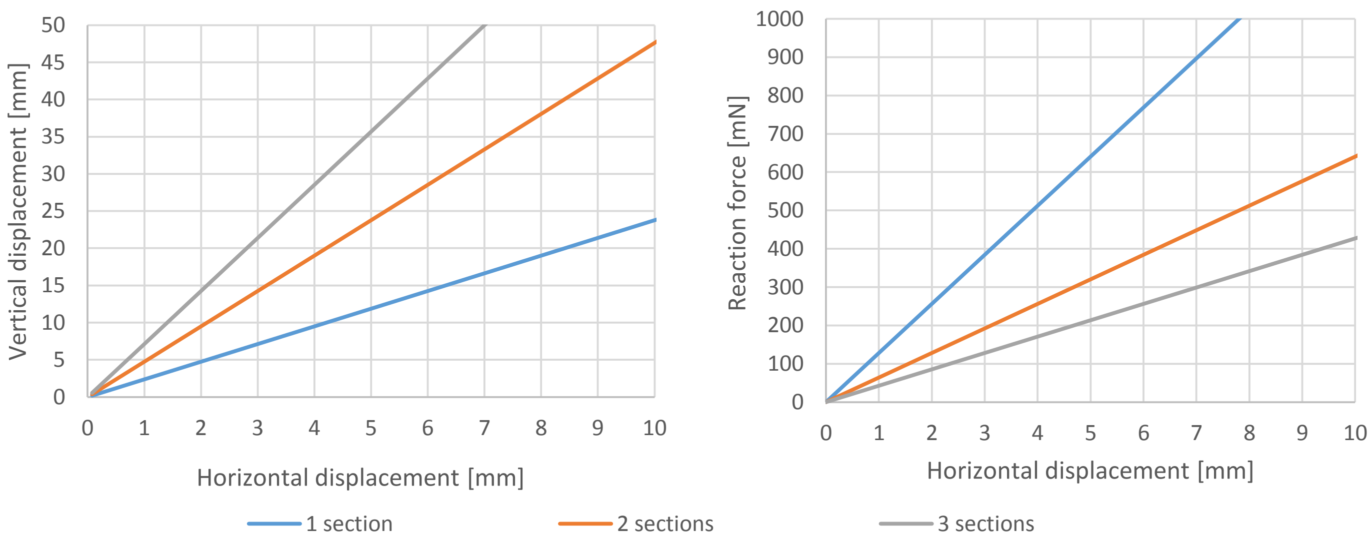


Figure 5.17: These graphs show the relationship between vertical and horizontal displacement (left) and reaction forces on these horizontal displacements (right).

Reality

The results of stage 3 do not correspond with the settlement data discussed in section 5.2.7 and in appendix E. These measurements – like most immersed tunnels – clearly show settlement differences of up to 25mm over length of just a few tunnel segments. It seems either the spanning moments are larger or the reaction force is smaller.

If the spanning moment is larger, the span would have to be larger or the load would have to be larger. In the three stages, the settlement of the soil is represented as a span where vertical soil reactions are no longer possible. In reality, only the carrying capacity of the soil would decrease, resulting in soil pressure decrease, but it would not continue subsiding if the tunnel elements would no longer be resting on it. In reality the vertical loads would therefore be smaller.

Can the span be larger? In stage 1 it was explained that the increase in span was the very reason for the joints to start deforming. However, in stage 3, a new longitudinal load was introduced. The reaction force caused by the horizontal displacements after rotation can have its effect elsewhere in the tunnel. Due to this load, it could be possible for spans to become much larger than described in stage 1, without exciting tensile stresses in the concrete.

It could also be that the reaction force is smaller. This would be the case if there would be easier ways to deform. The compressible materials in the land parts or in the immersion joints in case of the 2nd Beneluxtunnel could for instance allow horizontal deformations. It could also be possible that the elements themselves start to deform. Local deformations near the plastic hinges could for instance be possible or the roof and the floor of the tunnel can move on opposite directions of each other.

Figure 5.18 shows the situation in case the soil has settled in a more irregular shape. Due to these irregularities and secondary effects, is very hard to estimate the actual moments and shear forces in the tunnel in the longitudinal direction.

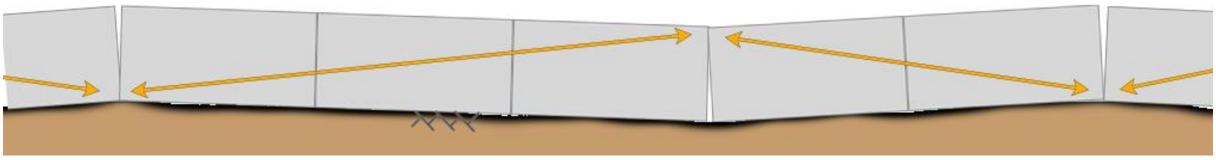


Figure 5.18: Settlement of the soil while the tunnel segments deform accordingly.

5.4.3 Segment joints

The segmented tunnel design allows differential settlements and therefore minimises internal stresses in the concrete. However, a consequence of this method is that the joints between these segments become the weak points of the construction as they have to be able to be subjected to rotation. In this section, the failure mechanisms of these joints will be discussed and the shear capacity of will be estimated.

During this section, one must keep in mind that joints can be closed as in their original state, or they can be opened and subjected to rotation as explained in the previous section. This subject will be discussed in more detail in section 6.4.2.

Failure mechanisms

The most important failure mechanisms regarding these joints are indicated in figure 5.19. They can be caused by longitudinal loads (blue) or by transverse loads (orange).

The longitudinal load is transferred to the next segments through the interface between in these joints. If the joint is opened, the interface has been decreased significantly. The longitudinal load is concentrated in within this area. The interface itself has been deformed into a plastic hinge as is discussed in stage 2. If the deformations continue, parts of the concrete could be pushed out (1).

Also, the longitudinal loads need to be transferred to the other slabs. As the joints can be rotated, the load possibly has to be transferred from the roof to the floor or vice versa. This would impose large shear forces on the interface between the roof or floor and the walls, or vice versa (2).

The joints can also be loaded by transverse loads. In section 2.4.3, the functioning of the collars as a shear keys has been explained. The most obvious failure mechanism is for the collar itself to fail. There is a difference between the outer collar (3) and the inner collar (4) as the inner collar is at some locations connected to walls, providing additional capacity. Given the differences in stiffness, the collars near the walls are expected to attract more load than the spanning sections, which emphasizes the difference between the two.

It could also be possible that the collar would remain intact but the interface with the wall could be torn of (5). This is only possible on the side of the outer collar.

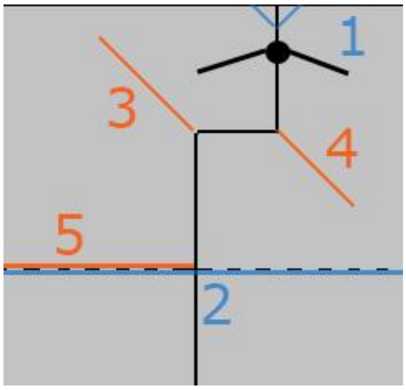


Figure 5.19: This figure shows the most important failure mechanisms of the segment joint.

Collar capacity

Determining the capacity of the joint is complex and unpredictable and will therefore not be extensively performed. The Longitudinal capacity will not be determined.

A first indication of the shear capacity of only the collar (failure mode 3 or 4) of the 1st Beneluxtunnel is determined using a truss model. This method is indicated in figure 5.20. The capacity is determined by the vertical and longitudinal reinforcement, situated in the crack plane. It is assumed that the anchorage length of both reinforcement bars is sufficient for the yielding strength of the reinforcement to be governing.

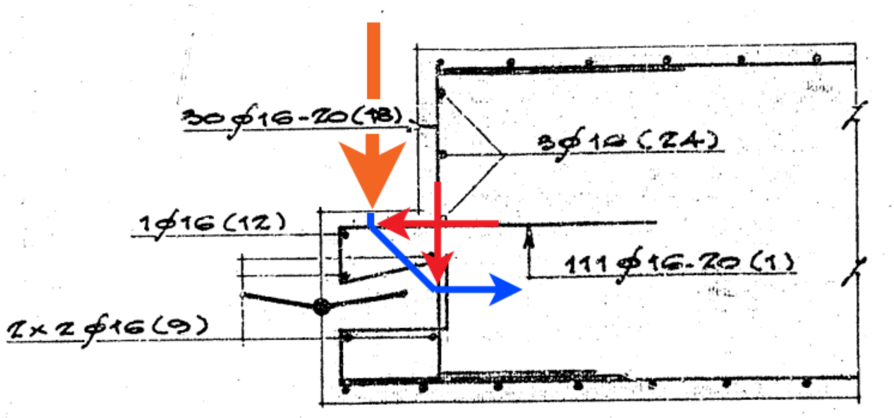


Figure 5.20: A truss model to determine the capacity of the collar

According to this method, the shear capacity of the 1st Beneluxtunnel for the floor collar indicated in figure 5.20 would be 206 kN/m. For entire cross-section of the 1st Beneluxtunnel this would be about 4.9 mN. If the same strength would be assigned to the collars of the 2nd Beneluxtunnel, the capacity would be about 9.3 mN.

This estimate gives the shear capacity of only the collar. The capacity of the interface with the wall (failure mode 5) is not included.

Frictional shear capacity of the segment joints

Apart from the shear capacity provided by the collar, also the shear capacity caused by friction in the interface of the joint contributes to the shear capacity of the segment joints. This interface friction is highly dependent on the magnitude of the longitudinal force and therefore varies over depth.

Figure 5.21 shows the shear capacity of the segment joints (at t = ∞) if only the friction in the interface is taken into account. The interface of the 1st Beneluxtunnel is treated with a layer of bitumen which possibly has a negative effect on the amount of friction. To incorporate the uncertainty regarding the interface friction, the results are given with friction coefficients ranging from μ = 0.5 to μ = 1.0.

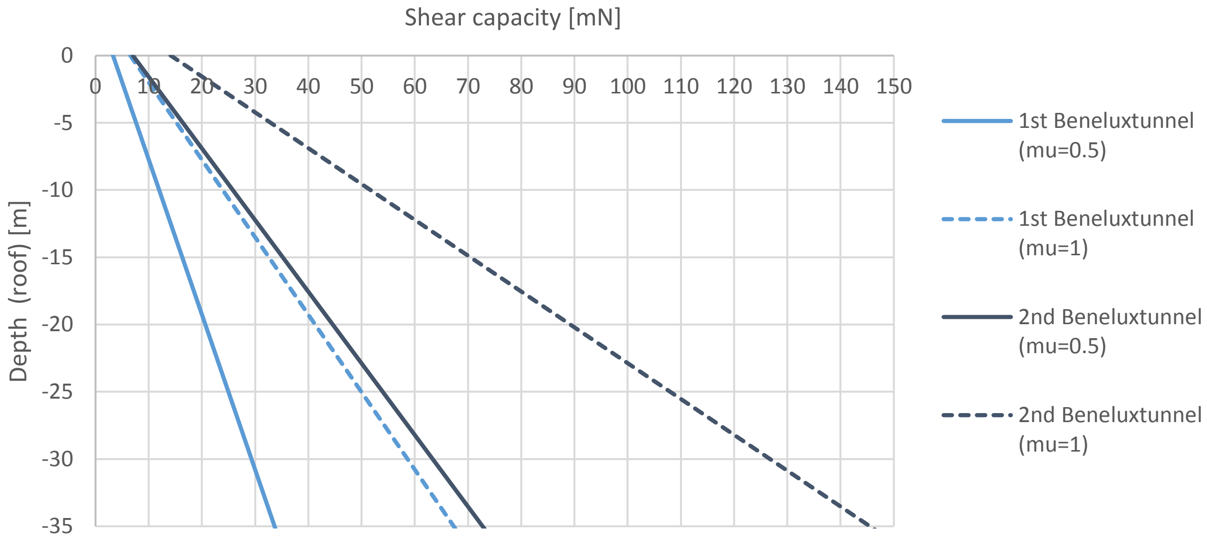


Figure 5.21: The shear capacity of the segment joints due to only interface friction.

Combined shear capacity

The shear capacity of the segment joint is a combination of the shear capacity of the collar and the frictional shear capacity. However, they cannot simply be added, nor are they completely independent of each other. For the reinforcement of the joint to be activated, the collar must be subjected to displacement which would mean the adjacent segment must be displaced relative to the segment of the collar. But if this is the case, the static friction coefficient is no longer valid but a lower, kinematic friction coefficient should be used to describe the friction in the interface.

For the Kiltunnel, the shear capacity of a certain segment joint was determined using more complex methods that combined the friction capacity and the capacity of the collar. This study claimed that if the difference in stiffness in different sections of the roof and floor slab would be ignored, the shear capacity for this joint would be 28.8 mN in one direction and 19.2 mN in the other direction. [34]

The cross-section of the Kiltunnel is very similar to the 1st Beneluxtunnel so let us compare these findings to the capacity estimates given in this section by determining the shear capacity of the 1st Beneluxtunnel at the same depth as was used in the shear capacity estimates of the Kiltunnel.

The roof of the tunnel above the examined joint of the Kiltunnel lies a depth of about NAP - 7.5 m. If we would assume high water conditions, the depth would be about 10 m. According to figure 5.21 this would give a frictional shear capacity of about 12 mN to 24 mN. Let us assume the collar in figure 5.20 is representative for all collars and ignore the reduction of the friction coefficient, the combined shear capacity would be 17mN to 29mN.

These values are relatively close to the values found for the Kiltunnel. However, whether high water conditions were used for the Kiltunnel estimates is unknown and the effect of kinematic friction has yet been ignored. Also the differences in shear force capacity of the Kiltunnel concerning the direction of the force is quite large. It is therefore expected that local configurations of the concrete plays a much larger role than could be estimated with the simple estimates used in this section. This presumption is strengthened by the fact that for the Kiltunnel, a larger friction coefficient turned out to give a lower shear capacity of the joint due to different failure mechanisms.

Despite the expected shortcomings of the method used for the Beneluxtunnel, the estimates in this section are relatively accurate. Even more so if we would include the fact that the capacity of the collar is determined using elasticity limits while the capacity of the Kiltunnel is determined using the actual failure capacity of the segment.

5.4.4 Immersion joints

As discussed in section 2.4.3, the interface conditions of the immersion joints are very different from the segment joints. The interface consists of a compressed GINA-profile and some concrete parts that are casted after the lowering process meaning they were originally uncompressed. Relaxation of the GINA-profiles will have caused some compression later on though.

The shear capacity of these joints are not expected to be very dependent on friction. It is expected that the shear capacity of the immersion joints is mainly determined by the anchor rods that are shown in figure 2.24.

The actual capacity of these joints will not be determined.

5.4.5 Influence of temperature

Thermal expansion and contraction can despite of the fact that actual temperature differences are relatively small, play an important role in the longitudinal mechanics of the tunnels. This subject will be discussed first without taking settlement into account and subsequently with settlement taken into account.

Temperature effects in the initial situation

As indicated in table 5.6, the hydrostatic pressure would cause compression of the concrete of maximally 0.7 mPa. In section 5.2.6, it was mentioned that fully restrained concrete increased in temperature with 5^{o} C, would cause a concrete compression stress of 2.3 mPa, which is much larger than the compressive stresses caused by hydrostatic pressure.

In the initial situation, this would therefore imply that the temperature fluctuations could cause segmental pressures to fluctuate with a magnitude of multiple times the hydrostatic pressure. Consequently the segmental joints can become opened entirely, lowering the shear capacity to only the capacity of the collars.

Temperature effects with settlement taken into account

If settlement would be taken into account, the mechanism appear to work differently. In section 5.4.2, it was mentioned that the settlement line of a tunnel was a result of the vertical loads trying to let the segments follow their settlements in an undisturbed chain line, and the longitudinal loads trying to push the segments together in a straight line. Initially the longitudinal load is determined by the hydrostatic pressure, but if settlements and the associated joint rotations increase, the reaction force of the concrete on horizontal displacements resulting from these rotations becomes more important.

If temperature fluctuations would now be introduced, the associated fluctuations of the longitudinal load could cause fluctuations in the settlement line. If the load is decreased, more rotations would be possible and certain segments would settle. If the load is increased, earlier settled segments could be forced back into their initial alignment meaning parts of the tunnel will be lifted.

This also implies that if enough irregular settlement has occurred to activate horizontal reaction forces and no horizontal space is available to allow full settlement, temperature fluctuations would no longer result in pressure fluctuations in the interface and the shear capacity would remain constant.

5.4.6 3d effects

In section 5.4.2 it was mentioned that the executed analysis is valid for displacements in both the vertical and the transverse direction. But if displacements occur in both of these directions at the same time, 3d effects will occur.

Two types of 3d effects can be distinguished. The presence of longitudinal loads causes concentrated loads and the presence of shear loads causes torsion.

Concentrated loads

Each segment has to transfer a certain compression load to the next segment through the interface of the segment joint. If the segments are rotated, the area of the interface changes which means the load concentrates over a smaller area and the pressure is therefore increased. This process has been described in section 5.4.2 and results in plastic deformations of the concrete

However, these concentrated loads can be rather large which could threaten the longitudinal capacity of the concrete at these locations, especially if these rotations occur in both the vertical and the transverse direction. Table 5.8 gives an indication of the relative pressure in the segment joints for different interface conditions.

interface conditions	**relative pressure**	
	1st Beneluxtunnel	2nd Beneluxtunnel
full (no rotation)	1.0	1.0
roof (only vertical rotation)	2.6	3.2
sidewall (only transverse rotation)	7.8	17.0
corner (vertical and transverse rotation)	61.3	144.6

Table 5.8: The relative pressure in the segment joints for different interface conditions.

According to these values, only a small load is required to cause local damage. The effect for the 2nd Beneluxtunnel is much larger as the initial interface area is much larger.

Torsion

If a difference in settlement would exist between two sides of the tunnel, torsion would develop. Within the segments, this torsion would be resisted by the shear capacity of the outer fibres of the concrete. At the segment joints however, another situation occurs. In the joint interface, torsion is primarily resisted by frictional shear capacity, but in contrast to transverse shear capacity, rotational shear capacity declines as the interface area decreases. If the joint is subjected to vertical and transverse rotation, the rotational shear capacity can be neglected and the interface becomes a rotation point. Consequently, rotation will develop at shear loading.

The secondary resistance is provided by the collars. Whether the capacity of the collar is sufficient for these 3d effects will not be determined.

5.5 Structural analysis of the land part

In this section, the land part of the Beneluxtunnel will be structurally analysed. The analysis will be performed similar to the analysis of the immersed part in the previous sections, but less extensive. In practice this means that a qualitative approach will be used meaning magnitudes will not be determined.

First the loads acting on the land part will be discussed and subsequently, the mechanics of the land part.

5.5.1 Loads

The loads acting on the land part are quite similar to the loads acting on the immersed part. The most important loads are:

- Hydrostatic pressure
- Self-weight
- Soil pressure
- Temperature load
- Traffic loads
- Loads from immersed part

Only hydrostatic pressure, soil pressure and the loads from the immersed part will be discussed. The other loads are not expected to play an important role in the adjustment of the land part.

Hydrostatic pressure

The hydrostatic pressure acting on the land part is in most aspects very similar to the hydrostatic pressure acting on the immersed part, discussed in section 5.2.1. However, there are some important differences that should be mentioned.

The most important difference between the immersed part and the land part, is that the land part is part of the flood protection system. Three areas can be distinguished:

- The area on the riverside of the abutments. This area lies outside the flood protection system and is therefore subjected to the same water level regime as was discussed for the immersed part of the tunnel, having a maximum water level of NAP +3.75 m.
- The area on the landside of the abutments, but outside of the land part. This area lies inside the flood protection system and is therefore subjected to a water level regime which is mostly based on the groundwater level which is kept at constant level and only has minor fluctuations in case of extreme levels. The maximum will not be defined but is expected to be around NAP.
- The area inside the land part. No water should be present within this area, but this area does lie outside of the flood protection system meaning that if the tunnel is flooded, this area will also be flooded. As the tunnel will have most chance of failure at high water levels, the theoretic maximum of this part will also be NAP +3.75 m, meaning an outward directed load will occur. For this study however, this scenario will not be taken into account.

The hydrostatic pressure acts on all sides of the land part.

Soil pressure

Soil can induce vertical and horizontal pressure. For the land part however, only the horizontal soil pressure is important. Downward soil pressure is not present as the upper side of the land part is open and upward soil pressure is also not present as the vertical load resultant is directed upwards and does not rely on the subsoil as foundation, apart from the upper sections of the ramp, but they will not be taken into account.

The horizontal soil pressure acts on the side walls for both tunnels. The abutments are also subjected to horizontal soil pressure, but the abutments of the 1st Beneluxtunnel are built partly in the water, meaning the level of the soil is substantially lower. On the ramp side, the slope is too low for horizontal soil pressure to act.

The surface around the land part is mostly NAP + 4 m. Also surcharge has to be taken into account.

Loads from immersed part

The transition point is the interface between the land part and the immersed part. This interface is very similar to the immersion joints within the immersed part of the tunnel, meaning similar loads will act at the transition point as described for the immersion joints, which includes all longitudinal and transverse loads described in section 5.4.1.

5.5.2 Mechanics

As discussed in section 2.4.2, each of the land parts of both the 1st and the 2nd Beneluxtunnel can be interpreted as a giant box with two side walls, an abutment functioning as riverside wall and an access ramp functioning both as a landside wall and as a floor.

When discussing the mechanics of the land part, this box could be considered as a whole, determining the overall stability of the system, or the different components of the box could be considered separately, determining the local stability.

For the vertical, the longitudinal and the transverse direction, the overall and local stability will be discussed.

Vertical stability

The vertical stability of the land part is threatened by the upwards directed resultant of the hydrostatic pressure. In contrast to the immersed part, the magnitude of this load depends on the depth.

This force could for the higher parts of the ramp be counteracted solely by using the self-weight of the floor, but for both Beneluxtunnels, we see that tension piles have been applied well above NAP and therefore well above the expected high groundwater water levels. Consequently, the overall vertical stability is mainly provided by the friction between these tension piles and the surrounding soil.

The local stability is determined by the distance between the piles and the strength of the underwater concrete floor above.

Transverse stability

The transverse stability is threatened by horizontal water and soil pressure. However, as the water and soil conditions are more or less similar on both sides of the land part, overall stability is not threatened.

The local stability in the transverse direction on the other hand is one of the most important aspects of the land part. The side walls are significant soil and water retaining structures which require much effort in order to obtain stability.

Figure 5.22 shows the water and soil pressures on one of the side walls of the 1st Beneluxtunnel during construction. Also the locations where the anchors and underwater concrete floor are to be positioned are indicated. These components are very important in providing stability of the wall.

For both tunnels, the retaining walls are very similar to this and they both use anchors an underwater floors to provide horizontal stability.

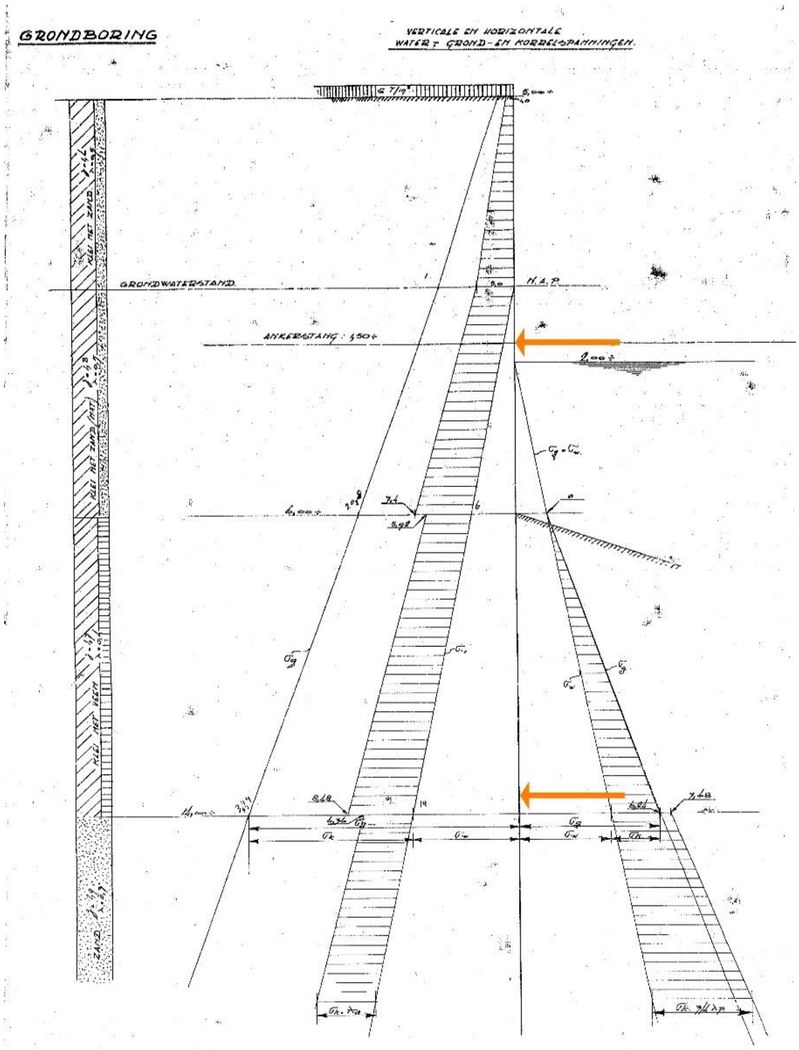


Figure 5.22: This figure shows the water and soil pressure on one of the side walls of the 1st Beneluxtunnel during construction. The orange arrows indicate the locations of the anchors and the underwater concrete floor. [10]

Longitudinal stability

Also in the longitudinal direction, an equilibrium must exist. As the water level on the waterside can be much larger than on the landside of the tunnel, a significant force pushes the access ramp landwards. This can be resisted mostly by soil friction of the floor or by the pile foundations. Given that the pile foundations are straight, the load may not become too much. But given the large amount of piles this is not to be expected.

Local stability has to be provided by the abutment. The waterside of the abutment has to be strong enough to resist these loads, and they are transferred to the floor and through the inner walls in the lower parts of the ramp.

5.6 Summary

In this chapter, the existing Beneluxtunnel has been structurally analysed in order to determine its potential weaknesses.

Loads

First the loads acting on the immersed part of the tunnel are be determined. The most important load is the hydrostatic pressure, which depends on the depth and applies a large pressure on all outer slabs of the tunnel. Other loads that have been analysed are:

- Self-weight
- Soil pressure
- Pre-tensioning
- Flows and currents
- Temperature loads

Cross-section mechanics

In the least favourable scenario, the outer slabs of the tunnels are subjected to loads ranging from 220 kN/m to 390 kN/m. The mechanics of the cross-section have been determined using a frame analysis model. Subsequently, a safety check regarding the capacity of numerous locations has been performed. The most critical is the shear capacity of point 9, just outside of the slanting section in the floor underneath the road, near the inner walls.

Longitudinal mechanics

The magnitude of the longitudinal load depends on the hydrostatic pressure, but also the cross-sectional load resultants are important for the longitudinal mechanics. The tunnel segments allows deformations of the tunnel, but longitudinal pressure counteracts these deformations, which could result in spans consisting of multiple segments if longitudinal deformations are prevented.

The segment joints of immersed tunnel have proven to be a weak point. Multiple failure mechanisms of these joints are mentioned and the shear capacity as a combination of frictional shear capacity and collar shear capacity has been estimated. The capacity of the immersion joints has not been analysed.

Because of the influence of temperature, the deformations of the segment joints are constantly changing. The 3d effects concentrated loads and torsion could influence the longitudinal capacity and shear capacity, but their influence is not determined.

Land part

For the land part, also the loads and mechanics are determined, but only qualitatively. Hydrostatic pressure and soil pressure are the most important loads regarding the land part. The mechanics of the land part could be divided in a vertical, a transverse and a longitudinal component, which are briefly discussed.

Chapter 6

The effects of lowering on the structure

Figure 6.1: This image shows the construction of the 1st Beneluxtunnel. The last element is being town towards its desired position. [63]

6.1 Introduction

In this chapter, the structural analysis of chapter 5 will be used to determine the effects of lowering on the mechanics of the Beneluxtunnel and to estimate the associated limitations.

The following subjects will be discussed:

Effects of lowering First, the effects of lowering the tunnel and the influence of these effects on the functionality of the tunnel will be determined. (Section 6.2)

Maximum depth increase Next, the maximum depth increase of the tunnel will be determined by analysing the effects of the processes associated with depth increase on the immersed part and the land part of the tunnel. (Section 6.3)

Maximum joint rotations Also, the maximum rotation of the segment joints and immersion joints will be determined. (Section 6.4)

Vertical profiles Finally, the maximum depth increase and maximum rotations will be used to determine the possible vertical profiles of the tunnel according to technical limitations. (Section 6.5)

Finally, a summary will be given of the most important findings within this chapter.

6.2 Effects of lowering on the functionality of the tunnel

In this section, the effects of lowering the alignment of the tunnel will be explained, independent of the construction methods used to acquire this lowering. Subsequently, the functionality of the tunnel will be defined by determining the possible causes of failure.

6.2.1 Effects of lowering

If the profile of the tunnel is lowered in order to increase the draught, two mechanisms will occur:

- The depth of certain parts of the tunnel will increase.
- The vertical profile of the tunnel will change.

Depth increase

Depth increase is an inevitable consequence of lowering as the depth of the navigation channel should be increased.

Because the depth is increased, certain loads will be increased of which the most important is the hydrostatic pressure. This will affect the mechanics of the tunnel. The maximum depth increase will be determined in section 6.3.

Vertical profile changes

Changes in the vertical profile of the tunnel are also an inevitable consequence of lowering as the lowered parts of the tunnel have to reach up to the initial level of the road at some point.

To allow changes to the vertical profile, certain parts of the tunnel must be rotated relative to each other. At the land parts this could be accomplished by reconstructing the road at a slightly different angle which is not expected to give any technical difficulties and will therefore not be elaborated.

To allow rotations in the immersed part of the tunnel on the other hand, an alternative solution must be found as the tunnel is made up of stiff and unadaptable segments. However, as both the segment joints and the immersion joints have limited freedom of rotation, this could be utilised to allow changes to the vertical profile as is discussed in section 3.2.3. The maximum joint rotations will be determined in section 6.4.

6.2.2 Functionality of the tunnel

Failure of the tunnel occurs when it is no longer accessible for traffic. This could imply that the tunnel would become physically inaccessible for traffic, but in most realistic scenarios, inaccessibility of the tunnel would be caused by the disability to meet safety requirements.

Many different causes could be responsible for such failure, but only a few of these possible causes are expected to be affected by the lowering process. Consequently, if the following three conditions are met, the tunnel will remain functional:

- Water tightness
- Structural safety
- Traffic requirements

These conditions will be checked with regards to depth increase and joint rotations.

6.3 Determining the maximum depth increase

In this section, the maximum depth increase of the tunnel will be estimated.

First, some important conditions applying to this subject are mentioned. Next, the structural safety of the immersed part and the land part of the tunnel is analysed with respect to depth increase. Finally, the conclusions regarding depth increase will be given.

6.3.1 Conditions

In this section, some important conditions regarding depth increase are treated.

Situations requiring depth increase

Not all sections of the tunnel necessarily have to be lowered to allow deepening of the navigation channel. The situations in which depth increase is required are:

- Regarding the immersed part, the depth will only be increased if the vertical position of tunnel elements is adjusted. Consequently, this applies to the scenarios “keep elements immersed” and “re-float elements”.
- Regarding the land part, the depth will only be increased if the external boundaries of the land part are adjusted. Consequently, depth increase only occurs in case of “general lowering”.

Subjects determining depth increase

Depth increase could result in an increase of hydrostatic pressure and soil pressure, which could affect the structural safety of the land part and the immersed part. Consequently, to determine the maximum depth increase, the following subjects will be discussed:

- Structural safety regarding the cross-section of the immersed part.
- Structural safety regarding the longitudinal section of the immersed part.
- Structural safety regarding the vertical stability of the land part.
- Structural safety regarding the horizontal stability of the land part.

The effects of depth increase on the water stops is closely related to joint rotations and will therefore be discussed in section 6.4.

6.3.2 Structural safety of the cross-section of the immersed part

In this section, the analysis of the cross-section mechanics as treated in section 5.3, will be used to estimate the effects of depth increase on the structural safety of the immersed part.

Effects of lowering on the cross-section mechanics

If the elements of the tunnel become situated at an increased depth, the most important aspect that is changed regarding the cross-section is the increase in hydrostatic pressure.

The magnitude of the soil pressure depends on the amount of soil and bottom protection on top of the elements. As the elements are lowered to increase the depth, the amount of soil on top of the elements will be kept minimal. However, near the land parts and in the centre of the navigation channel, the amount of soil could be increased. However, it seems to be more a decision regarding design and maintenance than a consequence of lowering and it will therefore not be included.

All other loads are expected to remain constant and also the strength is expected to remain constant.

To determine the influence of depth changes on the mechanics of cross-section, the model used in section 5.3 has been adjusted for different depths ranging from 10 m above the original depth to 15 m below the original depth.

It turns out that the moments, shear forces and normal forces behave linear to the depth, which could be expected as the relation between external forces and internal forces is also linear. However, the rate of this behaviour is different per location, which is a consequence of the differences per location in the amount the hydrostatic load contributes to the total load.

The initial load multiplied with this rate results in the change of load per meter depth increase or decrease. For three safety levels, the maximum possible depth increase is indicated at which the acting loads remain below the structural capacity

Results

The results for the most critical situations are presented in figure 6.2.

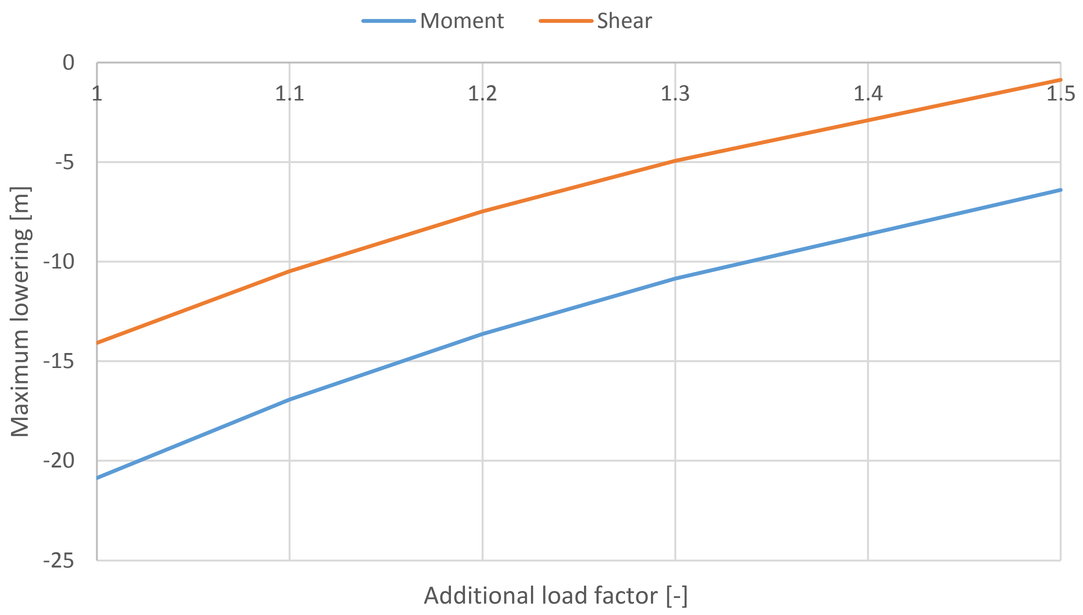


Figure 6.2: The lowering capacity of the deepest point of the 1st Beneluxtunnel according to the strength of the cross-section.

Some comments:

- Load factor 1 gives the situation as discussed before, with unfavourable conditions as discussed in section 5.2. This safety level is expected to be representative as it is based on the physical maximum water level, increased with an extra safety margin of 2 m.
- Load factor 1.3 is approximately the load factor one would obtain if the unfavourable loads are multiplied by 1.2 and the favourable loads are multiplied by 0.9. This safety level is on the conservative side, but has more chance of being approved according to safety regulations.
- A material factor of 1.5 was used for the concrete, 1.15 for the reinforcement and 1.10 for the pre-tensioning steel. (Section 3.5.3)
- Point 9 is governing regarding shear capacity. (Section 5.3.3)
- Point 7 is governing regarding moment capacity. (Section 5.3.3)

Conclusions

Figure 6.2 shows that regarding the strength of the cross-section, the maximum depth increase of the deepest sections of the immersed part of the 1st Beneluxtunnel is:

- 20.9 m according to the moment capacity given a load factor of 1
- 10.8 m according to the moment capacity given a load factor of 1.3
- 14.1 m according to the shear capacity given a load factor of 1
- 4.9 m according to the shear capacity given a load factor of 1.3

As mentioned in section 5.2.3, the structural features of the cross-section differ throughout the length of the tunnel. Consequently, the possible depth increase of the deepest sections is not representative for the higher sections of the tunnel. To estimate the maximum depth increase throughout the entire length of the tunnel, it is assumed that the relative maximum depth increase its equal for the entire immersed part of the tunnel.

Consequently, the maximum depth increase of the immersed part is estimated at 58% of the maximum depth increase at the deepest sections.

6.3.3 Structural safety of the longitudinal section of the immersed part

In this section, the analysis of the longitudinal mechanics as treated in section 5.4, will be used to estimate the effects of depth increase on the structural safety of the immersed part.

Effects of lowering on the longitudinal mechanics

As discussed in section 5.4.1, the magnitude of the longitudinal load depends on the depth. Consequently, if the depth of a section of the tunnel is increased, the longitudinal load would also increase, which would have the following effects:

- The compression of the concrete will become slightly larger, but is expected to remain well within the capacity
- The spanning moment capacity would increase.
- The frictional shear capacity of the segment joints would increase.

The loads nor the strength are expected to be significantly negatively affected by this increase in longitudinal load. Similar to the existing situation, the influences of temperature changes and 3d effects as discussed in sections 5.4.5 and 5.4.6 are expected to have a much larger influence on the structural safety of the tunnel, but these effects are hardly affected by depth increase.

Conclusions

The structural safety of the longitudinal section of the immersed part is not expected to impose limitations to depth increase.

6.3.4 Structural safety regarding the vertical stability of the land part

In this section, the analysis of the land part as treated in section 5.5, will be used to estimate the effects of depth increase on the structural safety of the vertical stability of the land part.

Effects of lowering on the vertical mechanics

To allow a lowered trajectory, the height of the underwater concrete floor has to be decreased. Regarding the vertical stability, this will have the following effects:

- The upward directed hydrostatic pressure will increase.
- Regarding the pulling capacity of the tension piles, two situations could occur:
  - The piles cannot be driven further into the ground which would decrease the effective length of the piles and therefore the pulling capacity.
  - The piles can be driven further into the ground which would prevent the pulling capacity from decreasing.

To counteract the increase in upward pressure and the possible decrease in pile capacity, it is possible to increase the thickness of the underwater concrete floor.

To determine the effectivity of this measure, the effects of depth increase on the hydrostatic pressure, the capacity of the piles and the thickness of the underwater concrete floor have been estimated for the two situations as described above.

The initial situation was used to determine the effectivity of the piles. Consequently, the results show the requirements to obtain the same safety level as in the initial situation. Excess capacity has not been taken into account.

Conditions

Furthermore, the following conditions were applied:

- These results apply to the deepest sections of the land part of the 2nd Beneluxtunnel as for the 1st Beneluxtunnel, technical information regarding the piles and the underwater concrete floor was unclear. The conclusions are expected to be applicable to both tunnels.
- The following heights were used
  - A maximum water level of NAP + 0 m.
  - The bottom of the underwater concrete floor at NAP - 13.3 m.
  - The bottom of the underwater concrete floor at NAP - 14.5 m.
  - The pile head at NAP - 30 m.
- A pile to pile distance was used of:
  - 2.1 m in the longitudinal direction.
  - 2.7 m in the transverse direction.
- The capacity of the piles is expected to be given solely by shaft friction and is assumed to be constant over depth. Consequently, a pile capacity of 56 kN/m is found

Results

The results of the analysis are displayed in figures 6.3, 6.4 and 6.5, for the two situations described on the previous page, namely:

- Situation 1, in which the piles cannot be driven further into the ground.
- Situation 2, in which the piles can be driven further into the ground.

Figure 6.3 shows the changes in the vertical force equilibrium of the depth is increased and figure 6.4 shows the associated thickness of the underwater concrete floor. These two graphs clearly show large differences between the two situations.

In situation 1, the increase in upward pressure and the decrease in pile capacity requires much additional concrete, meaning the underwater concrete floor gains thickness fast. As this additional concrete has to be situated on the below ground to allow depth increase, the upwards pressure increases even faster and the effectivity of the piles decrease even faster. At a depth increase of about 4.2 m, the underwater concrete reaches all the way to the tip of the piles, meaning the piles are fully ineffective. Consequently, this situation is not feasible for depth ranges larger than about 1 m.

In situation 2, the capacity of the piles is assumed constant, resulting in a much more gradual increase of the thickness of the underwater concrete floor. However, whether it is possible to drive the piles further into the ground is questionable. In this study it is assumed not to be possible.

Figure 6.5 shows situation 1 if the thickness of the underwater concrete floor is kept constant and an alternative method is used to provide the vertical force equilibrium. The required downward force at a depth increase of 5 m is about 90 kN/m^2, which is about 500 kN per pile. Inserting extra piles might affect the functioning of the existing piles and is therefore not recommended. Another method would be to insert anchors which would reach deeper than the existing piles. One anchor between each group of four piles would be sufficient to allow 5 m depth increase, or even more. [64]

Conclusions

According to this analysis, increasing the thickness of the underwater concrete floor is not enough to allow general lowering. If the piles cannot be driven deeper, which is expected to be the case, additional measures are required. Applying anchors between the piles would allow 5 m depth increase.

More depth increase would even be possible according to this method, but new problems are likely to arise and also the constructability becomes questionable. Consequently, a maximum depth increase of 5 m for the deepest parts of the land part is used as a maximum within this study.

Figure 6.3: This figure shows the vertical forces for situations 1 and 2.

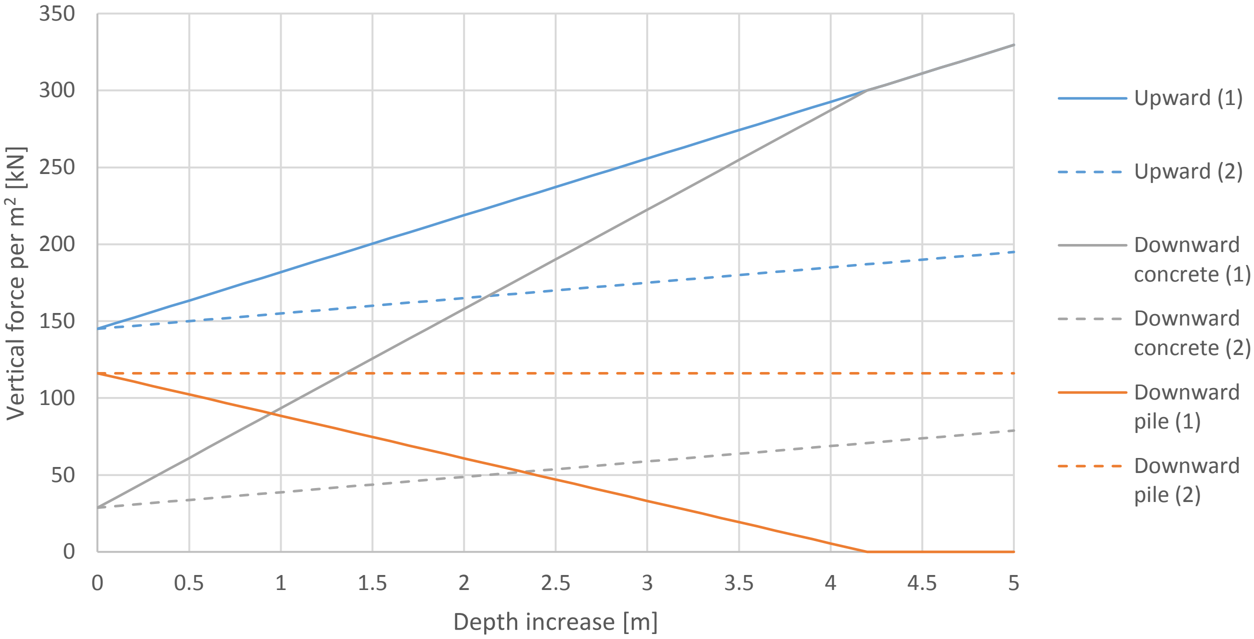


Figure 6.4: This figure shows the required thickness of the underwater concrete floor for situations 1 and 2.

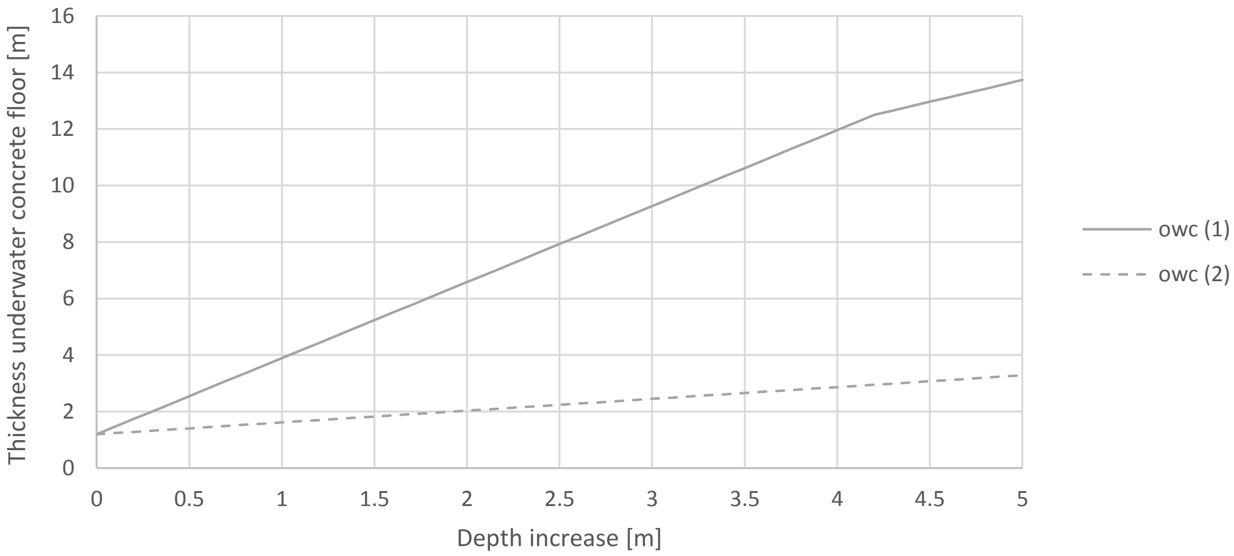
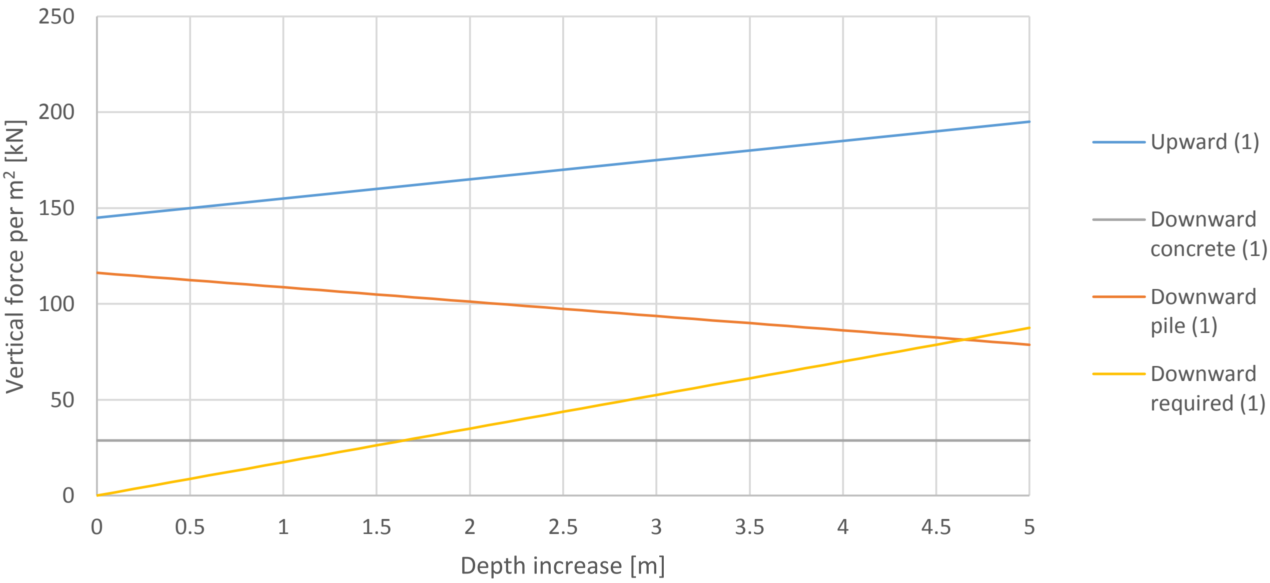


Figure 6.5: This figure shows the vertical forces if the thickness of the underwater concrete floor is maintained.



6.3.5 Structural safety regarding the horizontal stability of the land part

In this section, the analysis of the land part as treated in section 5.5, will be used to estimate the effects of depth increase on the structural safety of the transverse and horizontal stability of the land part.

Effects of lowering on the transverse mechanics

The effects of lowering on the transverse mechanics of the land part depends on the method of lowering, which will be discussed in section 7.5.

- If the land part is lowered in its entirely, all surrounding soil will also be lowered. Consequently, only the hydrostatic pressure acting on the walls will increase, not the soil pressure.
- If the only the floor of the land part is lowered, the vertical positioning of the lower strut will be decreased. Consequently, both the hydrostatic pressure and the soil pressure acting on the walls will increase.

In both situation, the horizontal pressure will have been increased and measures will be required. Similar to the vertical situation, anchors could be applied to regain stability. If the land part is lowered in its entirely, also temporary struts are required for the lowering process. If only the floor is lowered, the anchors should be inserted before removing the floor and as low as possible to take over the strutting function of the floor when it is removed.

Given the fact that the driving depth of the walls is about 10 - 15 m below the bottom of the underwater concrete floor, a maximum lowering of 5 m is also for this method a good estimate.

Effects of lowering on the longitudinal mechanics

Regarding the longitudinal mechanics. The loads depend very much on the question whether it is chosen to reconstruct the dike on the waterway side of the abutment, as is currently the case with the 2nd Beneluxtunnel, or to make the abutment part of the dike as is currently the case with the 1st Beneluxtunnel.

If the latter is assumed, only the hydrostatic pressure will be increased. The increase of hydrostatic pressure occurs both on the landside and on the waterway side of the land part, but as the water level on the waterway side can become much higher than the groundwater level on the land side, the absolute pressure increase will be larger on the waterside, resulting in a landwards directed resultant load.

The design of the new abutment must be able to provide local stability. Overall stability will be provided by the piles and the by the surrounding soil. This capacity will not be determined, but given the large amount of piles, this effect is not expected to be governing relative to the vertical and the transverse stability issues.

6.4 Determining the maximum joint rotations

As indicated in section 6.2.1, joint rotations are required to allow changes in the vertical profile of the immersed part. In this section, the maximum allowed joint rotations will be estimated according to the three conditions of section 6.2.2, namely: Water tightness, structural safety and traffic requirements.

First, some important conditions applying to this subject are mentioned. Next, the joints of the tunnel will be analysed to determine their maximum rotation. Subsequently, the long term behaviour of the joints will be analysed and finally, the conclusions regarding the maximum joint rotations will be given.

6.4.1 Conditions

In this section, some important conditions regarding joint rotations are treated.

Situations requiring joint rotations

Rotations in the alignment are not limited to the immersed part. Depending on the design decisions regarding the land part (section 3.2.2), alternatives are possible that cause rotations in only the immersed part, only the land parts or in both parts, as was indicated in section 4.4. Consequently, rotations in the alignment of the immersed part are not strictly necessary to allow lowering, but for most lowering options it is required.

Whether rotations in the alignment require joint rotations depends on the design decision regarding the immersed part:

- If the immersed part is lowered while immersed, joint rotations are required.
- If the immersed part is lowered by re-floating the elements, it depends on the amount of adjustments on the elements whether joint rotations are required. If the same elements are used, joint rotations are required, while if new elements are fabricated, joint rotations are not required.

Joint types

As mentioned in section 2.4.3, each tunnel has three types of joints which can possibly be used as rotation points. Of these, only the immersion joint and the segment joint will be discussed. The third joint, the closure joint, is not designed to allow rotation nor will it be easily be adjusted to allow rotation. Hence, the closure joint will not be discussed.

Consequently, to determine the maximum joint rotations, the following subjects will be discussed:

- Water tightness of the rubber profiles.
- Structural safety of the segment joints.
- Structural safety of the immersion joints.
- The effect of joint rotations on traffic requirements.

6.4.2 Water tightness of the rubber profiles

The water tightness of the joints is ensured by the rubber profiles. To help understand the functioning and capacity of these profiles, industry specialist Marcel de Vos has been consulted.[7]

If the joints are subjected to rotation in order to lower the trajectory of the tunnel, two mechanisms will influence the capacity of the rubber profiles:

- As discussed in section 6.2, the lowering of the tunnel increases the hydrostatic pressure. The maximum hydrostatic pressure with a lowering of 5 m will be about 0.35 MPa, so a safe upper bound of 0.4 MPa is chosen.
- The profiles will be elongated as a result of the rotation.

The effects of these mechanisms will be discussed for the GINA-, OMEGA- and W9Ui-profiles. Background information regarding these profiles can be found in section 2.4.3.

GINA-profiles

The GINA-profiles secures water tightness because it is compressed against the adjacent element. The profiles usually have a height of 180 mm and are compressed to about 140 mm which should be enough to withstand the water pressure while remaining enough tolerance for temporary variations and relaxation. If the profile is stretched too much, the water pressure will cause leakages. If the profile is compressed and deforms too much, it might become damaged which can also result in leakages.

Although there have never been cases of leaking GINA-profiles, the long term behaviour of these joints is relatively unpredictable and therefore not specified by the manufacturer. However, a simple estimation is made.

Using the initial loads in the immersion joints (table 5.6 in section 5.4.1) and their initial compression length of 40 mm, the length at which a pressure of 0.4 mPa can be maintained is calculated under the assumption that it increases linear. For both tunnels, the result of the estimates for the deepest points of the tunnel is about 3.5 mm, allowing an expansion of 36.5 mm. A safe estimate of 35 mm be used.

It is assumed for the compression length of the GINA-profile to be similar to the expansion length, which would be sensible regarding the design. Consequently, a maximum compression length of 35 mm is expected.

OMEGA-profiles

As large deformations were expected, a special OMEGA-profile is used for the 2nd Beneluxtunnel consisting of the flange of the OS 360/70 profile and the radius of the OS 400/100 profile. Figure 6.6 shows the allowed gap width of the OMEGA-profile type used for the radius in the 2nd Beneluxtunnel including a safety factor of 2.5. [38]

If there are two or three plies is at this moment unknown so two plies are assumed. Consequently, the profiles can be elongated with 60 mm and compressed with 100 mm.

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[7] Marcel de Vos is an engineering manager at Trelleborg, a company that specialises in engineered polymer solutions and has provided many of the Dutch immersed tunnels with GINA, OMEGA and W9Ui profiles. .

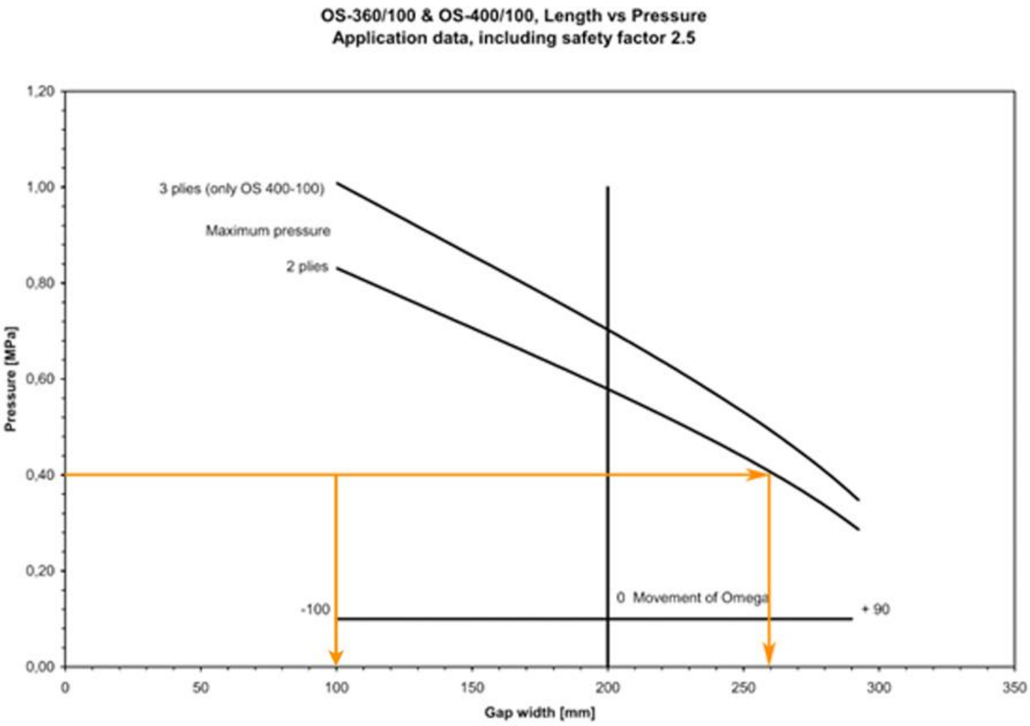

Figure 6.6: safety levels OMEGA-profiles [65]

The immersion joints of 1st Beneluxtunnel are not designed to allow rotation. As discussed in section 2.4.3, these joints are designed with a metal strip instead of a rubber profile, which is less flexible. Also, these joints are monolithic connected by reinforced concrete and thus no rotation is possible without demolition.

If it would be required to use the immersion joints of the 1st Beneluxtunnel in order to obtain the required lowering of the elements, major adjustments would be required to joints, including the removal of the monolithically constructed concrete sections. The metal strips can consequently be reached and a new OMEGA-profile could be installed. A similar maximum gap width would be acquired as with the 2nd Beneluxtunnel.

W9Ui profiles

The water tightness of the segment joints of the 2nd Beneluxtunnel is provided by a W9Ui profile. The 1st Beneluxtunnel however, uses an older profile, without the possibility of injection. It is assumed that this older profile has the same properties as the standard W9Ui-profiles that are used in the 2nd Beneluxtunnel.

Figure 6.7 shows the possible elongation of the W9Ui profile in SLS conditions. According to this graph, the W9Ui-profile can be elongated up to 22 mm.

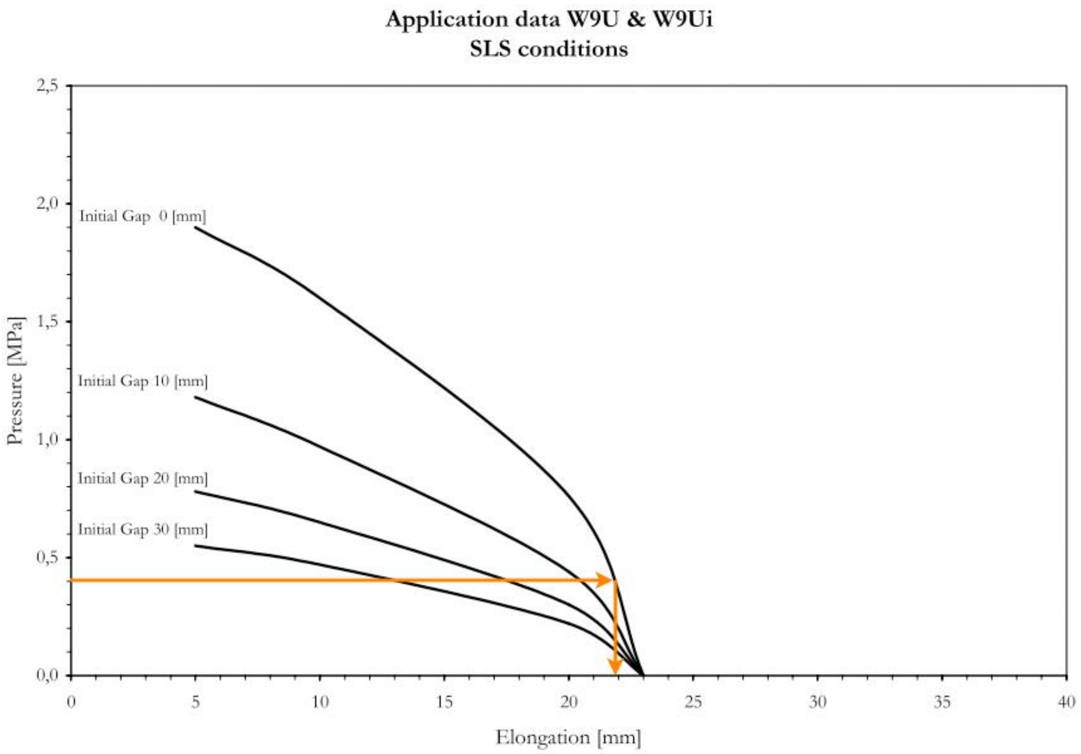

Figure 6.7: safety levels for the elongation of the segment profiles [66]

6.4.3 Structural safety of the segment joints

The mechanics of the segment joint were discussed in section 5.4.3. To determine the effects of rotation on the structural capacity of the segment joints, the effects of rotation on the five failure mechanisms as shown in figure 5.19 (section 5.4.3) will be discussed.

Important in case of rotation is the presence of a push and a pull side, both having different mechanics. The push and the pull side are indicated in figure 6.8.

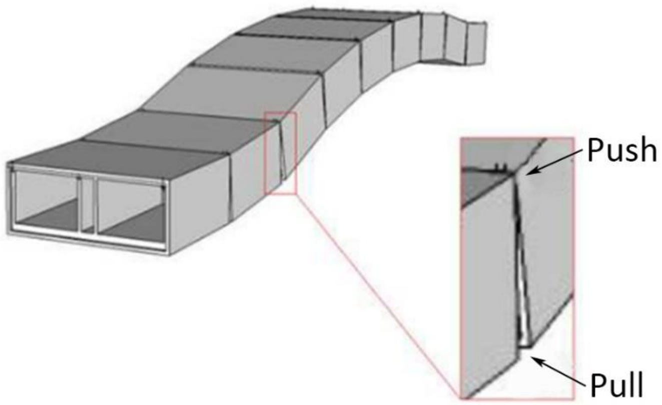

Figure 6.8: Rotation in tunnel joints [24]

Push side

On the push side, the longitudinal load is active and spread over an interface created by plastic deformation. If the joint rotation is increased, the local plastic deformations (1) will have to increase to maintain this interface. The limitations regarding this mechanism are unknown but given the relatively low rotations, it is expected not to become problematic.

The risk of shear failure force on the interface with the walls (2) caused by the longitudinal load will decrease as the impact angle pushes the roof (or floor) onto the walls.

The effect of the shear mechanisms on the push side is expected to remain unchanged as the displacements due to rotation are negligibly small.

Pull side

On the pull side, the longitudinal load is not present. The differences in the pull side are only relevant for the shear mechanisms. A graphical representation of the elongation of the pull side of the joint is given in figure 6.9. This figure shows a point force (orange arrow) that represents the shear force of the upper segment onto the lower segment. If the horizontal distance between the segments increases, the attachment point of this force moves towards the edge of the collar.

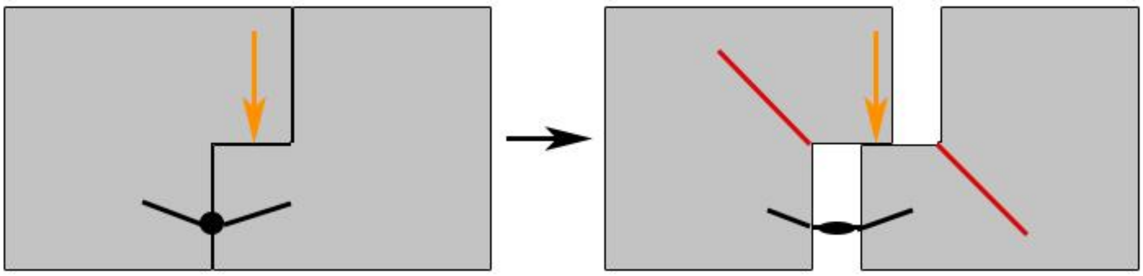

Figure 6.9: This figure shows the collar on the pull side when elongated. The displacement in this figure is about 150 mm.

If the collar were to be represented as an inclined beam, the inclination moment would increase. However, given the large height of the collar compared to its width, this would be a false representation of the situation.

If the situation would be interpreted as the truss model displayed in figure 5.20 (section 5.4.3), the situation would hardly be different as still the same reinforcement will be activated. Only the compression in the rod would end up at a slightly lower height, but for displacements of up to 22 mm, this will not be expected to be problematic. This statement hold for both the outer (3) and the inner (4) collar. Also the situation regarding the attachment to the walls (5) is not expected to have changed significantly.

Conclusions

It is expected that the structural capacity of a segment joints which is rotated to maximally cause a gap of 22 mm will not be significantly lower than when only subjected to the rotations discussed in section 5.4.3.

6.4.4 Structural safety of the immersion joints

The immersion joints of the 2nd Beneluxtunnel act similar to the segment joints, except for a few aspects. The presence of the GINA-profile prevents plastic deformations of the concrete. However, if the longitudinal load is concentrated on specific parts of the GINA-profile, it could be compressed beyond its capacity, but this has already been discussed regarding the water tightness.

Furthermore, the shear key of the immersion joint functions similar to the segment joints. Longitudinal movement is however possible as a 10 mm thick layer of tempex has been applied. Another important difference is the fact that these shear keys are located on the inside of the watertight profiles and are therefore possible to be redesigned according to adjusted requirements.

The immersion joints of the 1st Beneluxtunnel are monolithically constructed and can therefore not be subjected to any rotation. But similar to the 2nd Beneluxtunnel, this can be adjusted if desired.

6.4.5 Effect of joint rotations on traffic requirements

Joint rotations and the associated changes in vertical profile influence the traffic requirements in three ways:

- Joint rotations cause kinks in the road.
- Joint rotations cause vertical curves with a certain radius.
- Joint rotations cause an increase or decrease in slope.

These aspects are treated with help of figures 6.10 through 6.12. These figures all show a certain aspect of the traffic function in relation to the rotation of a joint. The values in these graphs apply to the 1st Beneluxtunnel.

Rotation versus elongation and contraction

Figure 6.10 shows the relationship between joint rotations and the maximum elongation and contraction of the watertight profiles discussed in section 6.4.2. For the segment joints, the W9Ui-profile has a maximum elongation of 22 mm and contraction is not possible. The rotation point of these joints is assumed to be at the horizontal section of the collar, at 500 mm from the outer fibre of the joint.

For the immersion joints, a maximum elongation and contraction of 50 mm are both applied. Consequently, the rotation point lies in the centre of the joint which causes the possible rotation to be about twice as much as for the segment joint.

Kinks in the road

Figure 6.11 gives an indication of the amount of ballast concrete that have to be removed to smoothen out kinks in the rotate resulting from joint rotations. The vertical displacement of the road is the distance with which the road has to be lowered at the location of the joint to remain a continuous road profile in case of a top curve. In case of a bottom curve, the concrete has to be removed in the centre of the segments. This distance indicates the amount of concrete that has to be removed, which may not be too much as weight problems could occur if too much ballast is removed.

The entire layer of ballast is about 1 m thick. According to the figure, the loss of ballast for the segment joint is negligibly small. For large rotations in the immersion joints however, problems could occur as up to 18% of the ballast would be removed which is about 72 kN/m. Measures should be taken to increase the safety level in these circumstances, but these problems will not be unmanageable.

Radius

Figure 6.12 shows the acquired radius of the top curve of the road for different joint rotations of either the segment joints or the immersion joints. It also shows the minimum radius for different speeds as indicated in section 4.2.1. This figure also indicates of the effectiveness of rotation in the segment joints relative to rotation in the immersion joints, which is a result of the smaller lengths. It also indicates that in most cases, technical limitations are governing for determining the vertical profile rather than the functional requirements determined in section 4.4.

The figure shows that the traffic requirements for the 50 and 80 km/scenarios are not affected within the feasible ranges of joint rotation. The 100 km/s scenario and the scenario of the currently applied radius do propose boundaries on the joint rotation regarding the radius of the road. However, the associated rotations and profile elongations are for both profiles in the higher and less realistic ranges.

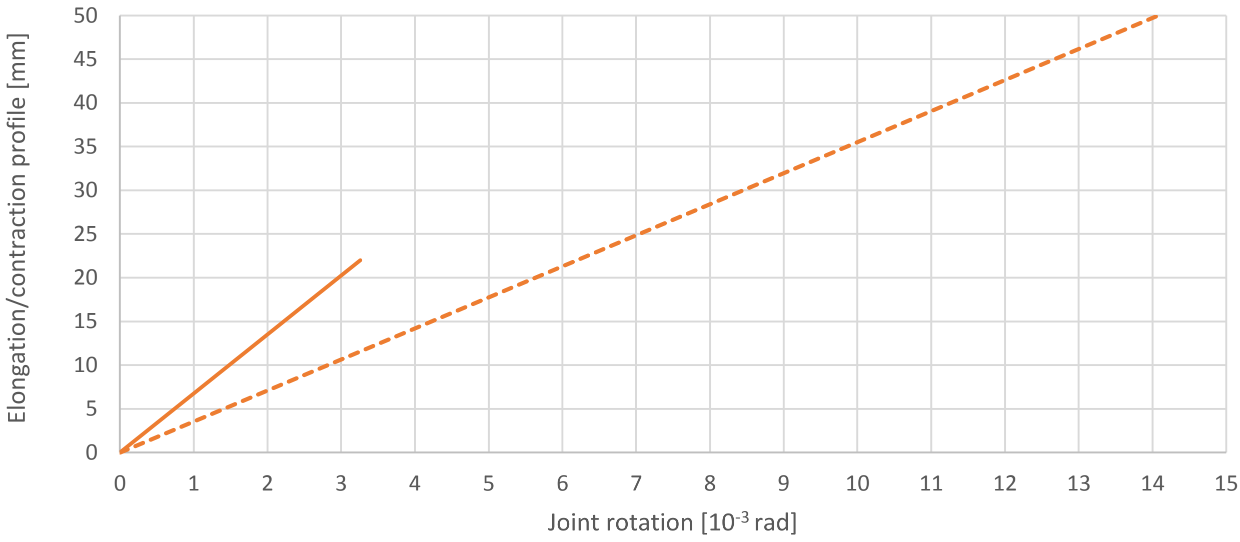

Figure 6.10: Joint rotation versus elongation or contraction of profiles

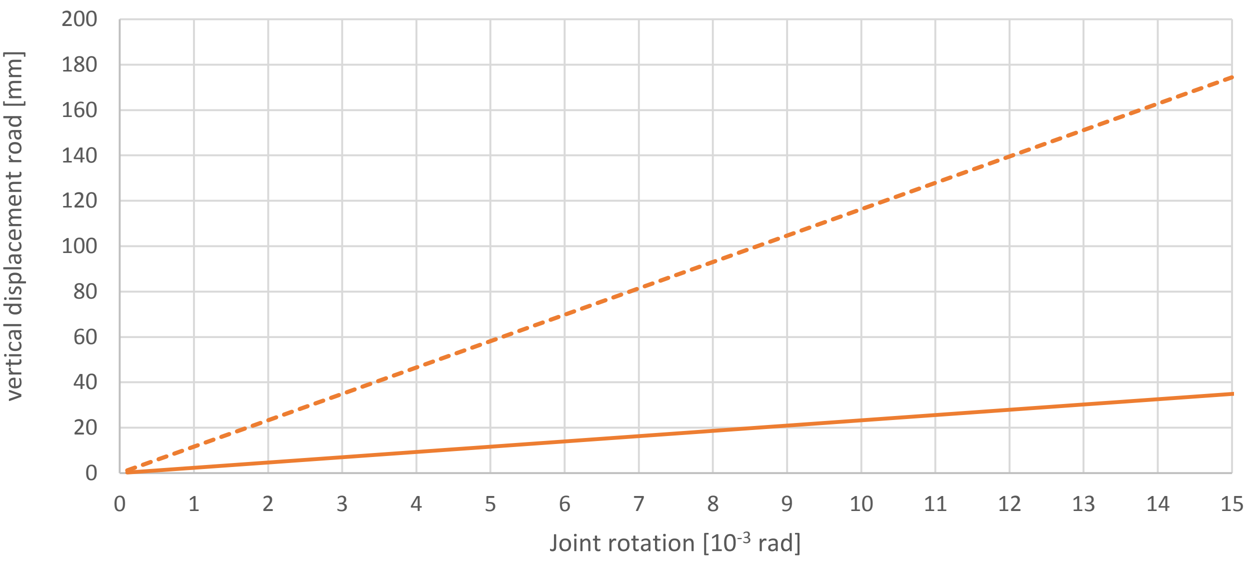

Figure 6.11: This figure shows the required vertical displacements in joints to prevent kinks in the road for certain joint rotations.

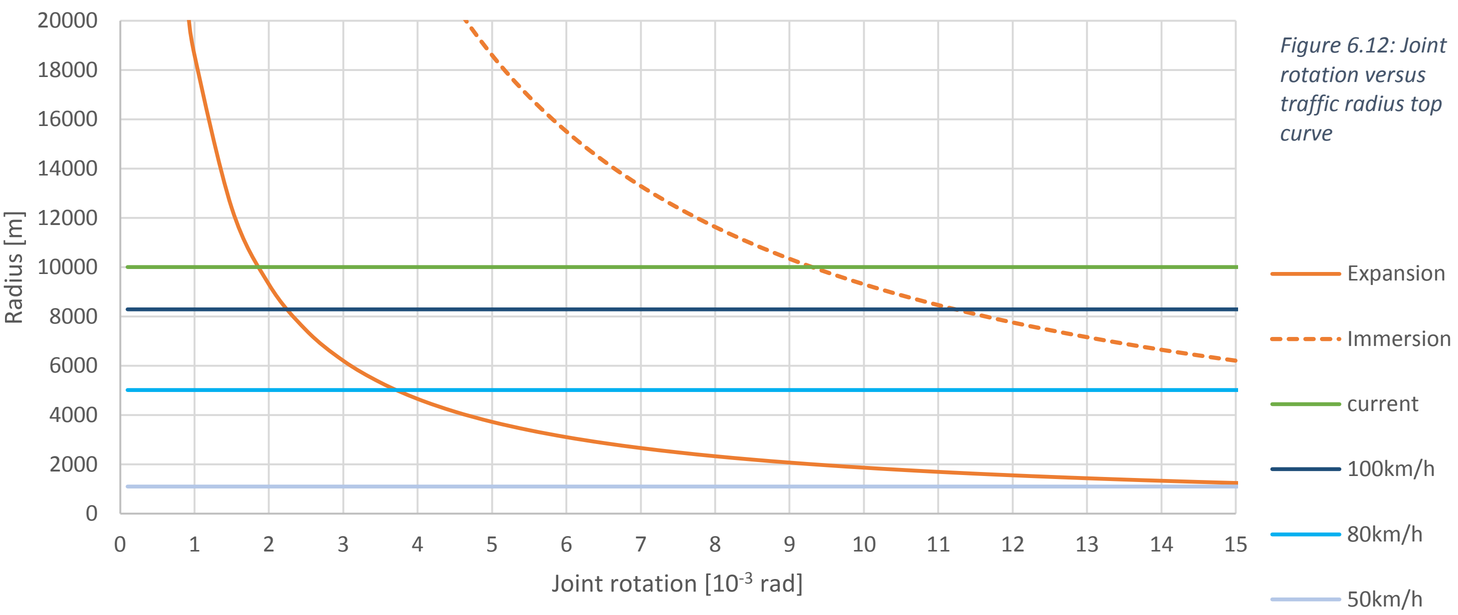

Figure 6.12: Joint rotation versus traffic radius top curve

6.4.6 Time dependent behaviour of the joints

The long term behaviour of the joints depend on the deformations of the tunnel segments during its lifetime. This has to be taken into account when determining the maximum rotation.

Settlement data

To determine the maximum joint deformations, existing settlement data can be reviewed. However, as the foundation of the tunnels will be renewed, the foundation data cannot be directly implemented. It does however, give an idea of the maximum possible settlements of the local situation and can therefore be used.

Figure 6.13 shows settlement data of four interesting points of the 1st Beneluxtunnel with large settlements. The first measurements regarding this tunnel date from about 15 years after construction of the tunnel, meaning the important first part of the settlement is missing. This first part is important because the settlement of soil is expected to develop logarithmic in time.

To estimate the maximum settlement differences for the 1st Beneluxtunnel, a single logarithmic development line is plotted onto four of the most extreme points of the 1st Beneluxtunnel, which is quite a rough and inaccurate technique, but serves the purposes of this study.

This analysis shows that the maximum settlement over 100 years would be about 40 mm relative to the reference height. If a longer lifetime would be taken into account, about 50 mm of settlement would be expected for these points.

Settlement estimates

As shown in figure 6.13 and appendix E, the least settled points are around the reference height. The current maximum settlement difference is therefore about 25 mm. If these points would remain stable, the maximum expected settlement differences would be about 50 mm.

However, given that these values are relative to a certain reference line which is defined bout 15 years after construction, settlement of the points relative to each other can be much larger. It is possible for the points that have had little settlement according to the measurements to also have had little settlement in the first 15 years and the points that have much settlement to have had much settlement in the first 15 years. The latter is indicated in the graph which shows that for these points, the reference height must have settled about 55 mm. The logarithmic function is however not very suitable for backcasting. But if we would assume this is true and if we would also assume that the stable points would have had only say 5 mm of settlement, total settlement differences of up to 100 mm could exist.

Design codes advice to use 50 mm additional height near the land parts, which is indicates a settlement range of 0 to 100 mm is not a bad estimate.[4]

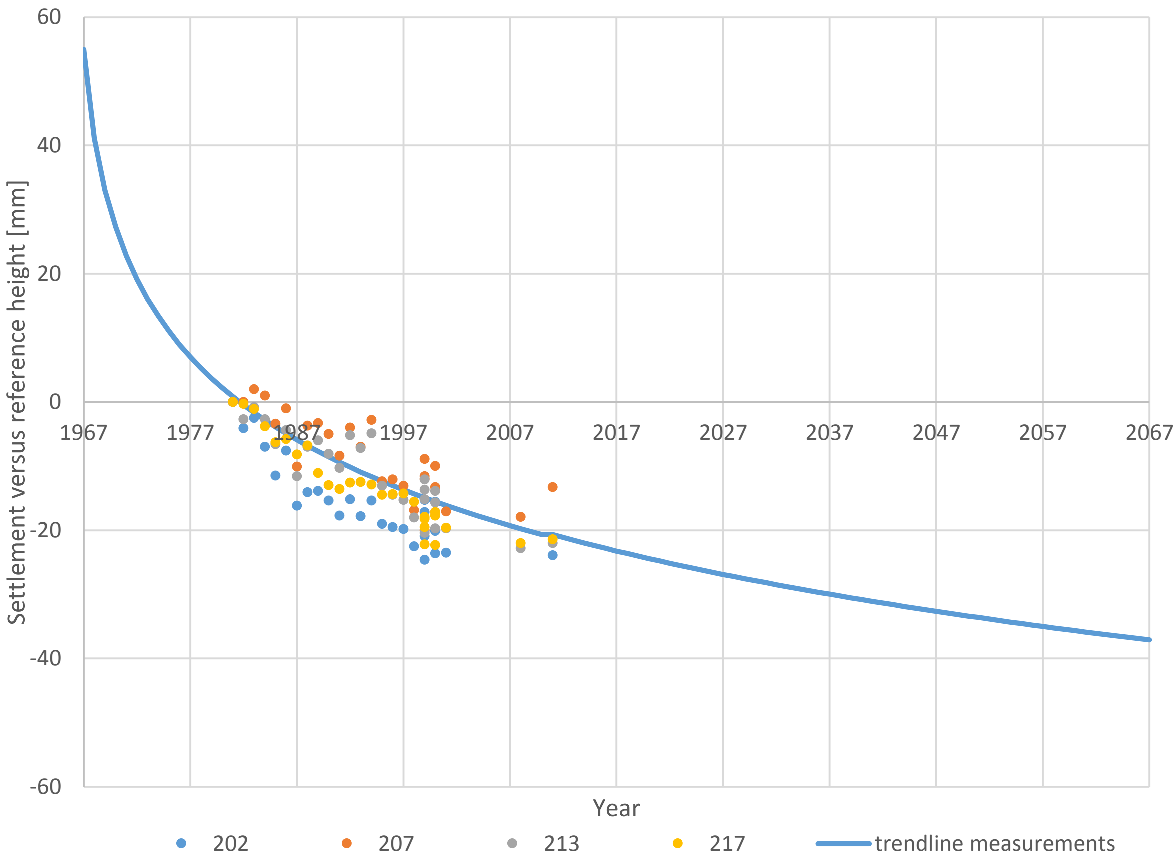

Figure 6.13: The settlement of the 1st Beneluxtunnel of the points: 202, 207, 213 and.217 (R^2 is not

Settlement differences

The settlement on its own is not important for the rotation of the joints and the associated horizontal displacements as also the horizontal distance between these settlements is important. Figures 6.14 and 6.15 show the elongation of W9Ui-profiles and the joint rotation in the most extreme scenario for different horizontal distances ranging from one segment to one element. The 25 mm, the 50 mm and the 100 mm scenarios are displayed. In these scenarios, the considered joint would be either in a positive or negative extreme point. The rotation is therefore the effect of the situation on both sides of these joints. The joints that lie between these points are assumed not rotated, which is especially for larger horizontal distances not expected to be true.

These figures clearly indicate the large possible rotations and elongations of the W9Ui-profiles that could exist for the 1st Beneluxtunnel in different scenarios ranging from likely to unlikely. If we would compare these estimates to other tunnels, it turns out the values for the maximum rotation of joints range from $0.2*10^{-3}$ rad to $2.5*10^{-3}$ rad. [56]

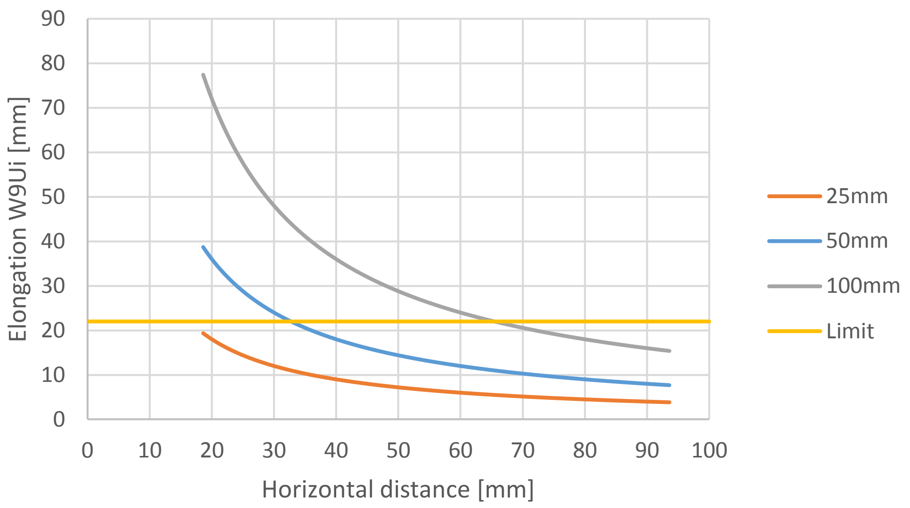

Figure 6.14: The Elongation of the W9Ui profile relative to the horizontal distance over which extreme settlement differences occur. The horizontal distance of one segment is 18.6m.

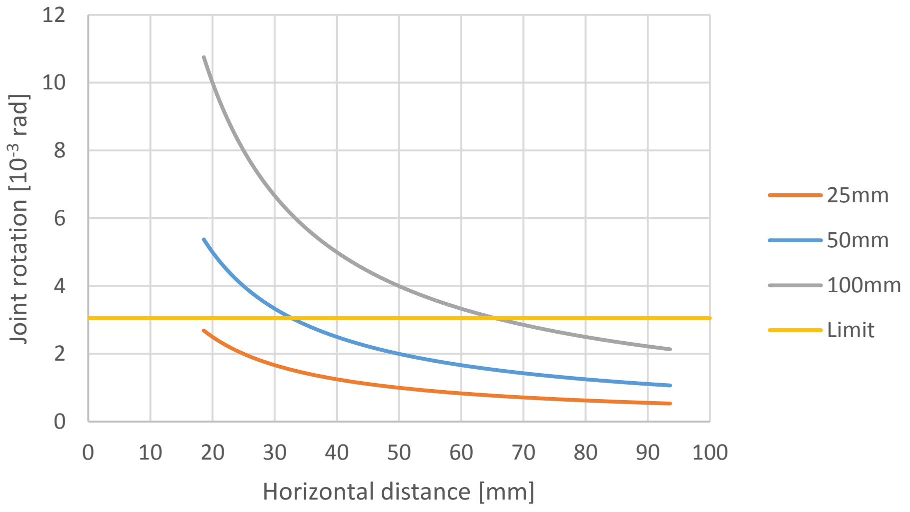

Figure 6.15: The joint rotations relative to the horizontal distance over which extreme settlement differences occur. The horizontal distance of one segment is 18.6m.

Given the fact that the adjusted tunnel would develop a completely new settlement line which could have many possible scenarios, and given the large range of possible outcomes in existing tunnels, no reliable estimates regarding the maximum rotations and maximum elongations of the W9Ui-profile can be given.

Consequently, the problem becomes a probability issue. To find a limit value, the probability of extreme settlement differences could be estimated and combined with the required probability of safety. This will however not be performed.

In practice

The findings in this section indicate that large elongations of the W9Ui-profiles are very well possible as a result rotations caused by settlement, which could jeopardize this design alternative. It also indicates that for existing tunnels, the W9Ui-profiles are likely to be already elongated beyond their capacity. This could suggest problems with these profiles in existing cases.

These problems certainly occur as is extensively documented by Ing. L. Leeuw. These leakages are however generally the result of imperfections regarding the connection between the concrete and the metal strip and not because of damages within the profiles themselves caused by elongation. All of these leakages were managed or repaired, most of them by injecting a sealing fluid within the leak. [39]

Consequently, it seems that capacity of the rubber profiles themselves might be larger than the design graphs in figures 6.6 and 6.7 would suggest. This presumption is one of the subjects that is currently being investigated as is indicated in figure 6.16.

Figure 6.16 - Testing the rubber profile of an segment joint [67]

Conclusions

In short, deformations, rotations and elongations of the W9Ui-profiles can occur in large magnitudes, but it seems the capacity of the rubber profiles can also be much larger than expected. The influence of initial displacements in the order of 10 - 20 mm would therefore be regarded less than figures 6.6 and 6.7 would suggest. However, if rotations become much larger, conclusions regarding the structural capacity of these joints might be different from the structural capacity analysis within this section.

For the remainder of this study, it will be assumed that 50% of the maximum rotations are possible:

- The maximum rotation in the segment joints will be assumed $1.6*10^{-3}$ rad, based on a maximum elongation of the W9Ui-proilfe of 11 mm.
- The maximum rotation in the immersion joints will be assumed $7.0*10^{-3}$ rad, based on a maximum elongation of the W9Ui-proilfe of 25 mm.

These rotations allow large settlement differences to exist without overstretching the rubber profiles. However, if extreme settlement differences occur, this will no longer be sufficient. Whether this would result in leakages depends on the excess strength of the rubber profiles. As these leakages can be repaired, failure of the profiles would not result in an unmanageable situation.

To determine the maximum rotations, a probabilistic analysis should be performed, which would have to include the probability of extreme deformations, the excess strength of the rubber profiles and the costs of repairing the profiles.

6.5 Vertical profiles according to technical limitations

In this section, the maximum depth increase determined in section 6.3 and the maximum joint rotations determined in section 6.4 will be used to determine the possible vertical profiles according to technical limitations. Together with the functional limitations determined in section 4.4, this will allow us to determine the overall depth limitations and therefore the effectivity of the design options.

To determine the effect of joint rotations on the vertical profile of the tunnel, a model is created that determines the displacements of the corner points of the segments within the immersed part of the tunnel, if the joints are subjected to a certain rotation.

The following aspects will be analysed:

- The influence of the rotations per joint type.
- The influence of the magnitude of rotations.
- The influence of the design options.

First the conditions applied to this analysis will be defined and finally the conclusions will be given.

6.5.1 Conditions

The following conditions apply to this analysis:

- All information presented in this section applies to the 1st Beneluxtunnel only.
- The influence of the horizontal curve is ignored.

The following subjects have been used as input parameters for this analysis:

- Information regarding the geometry of the 1st Beneluxtunnel as determined in sections 2.4 and 4.2.1.
- Information regarding the relationship of the tunnel profile and the navigation channel as determined in section 4.3.
- The maximum depth increase as determined in section 6.3
  - The maximum lowering of the immersed part is determined is 14.1 m at the deepest point of the tunnel and 8.2 m near the land parts.
  - The maximum lowering of the land part is 5 m.
- The maximum joint rotation as determined in section 6.4.
  - The maximum rotation of the segment joints is $1.6*10^{-3}$ rad.
  - The maximum rotation of the immersion joints is $7.0*10^{-3}$ rad.

Figure 6.17 shows the original situation and figures 6.18 through 6.32 show adapted situations. For these graphs holds:

- The graphs shows the vertical profile of the entire immersed part of the 1st Beneluxtunnel.
- The tunnel is displayed in the tunnel domain while the navigation channel is displayed in the navigation domain. The difference is explained in section 4.3.1. The navigation channel would be about 20% wider in the tunnel domain. Consequently, the margins between the tunnel and the navigation channel are smaller than they would appear on the graphs.

Original situation

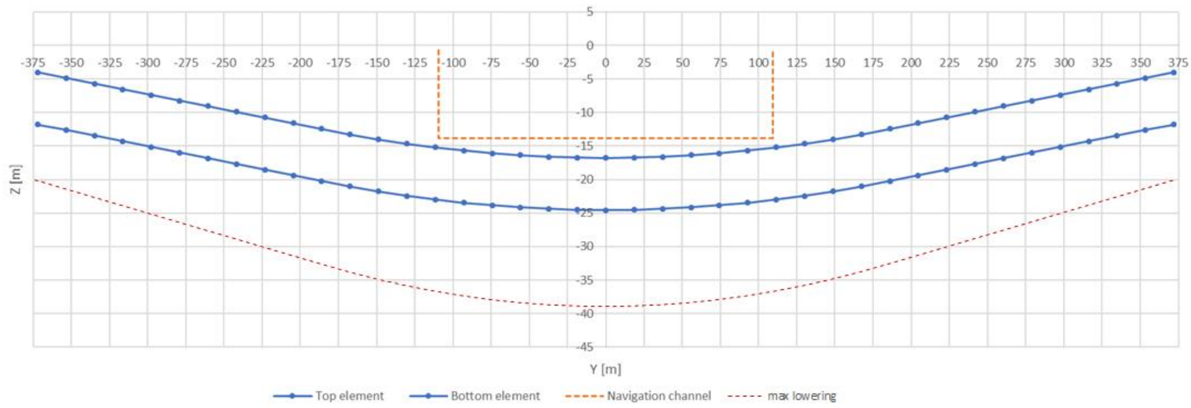

Figure 6.17: This figure shows the original longitudinal profile of the 1st Beneluxtunnel and the associated navigation channel.

6.5.2 The influence of rotations per joint type

In this section, the influence of joint rotations on the vertical profile of the tunnel will be analysed for three situations:

- Rotation in only the segment joints.
- Rotation in only the immersion joints.
- Rotation in both the segment joints and the immersion joints.

The results are shown in figures 6.18 through 6.20 and in table 6.1.

Only segment joints

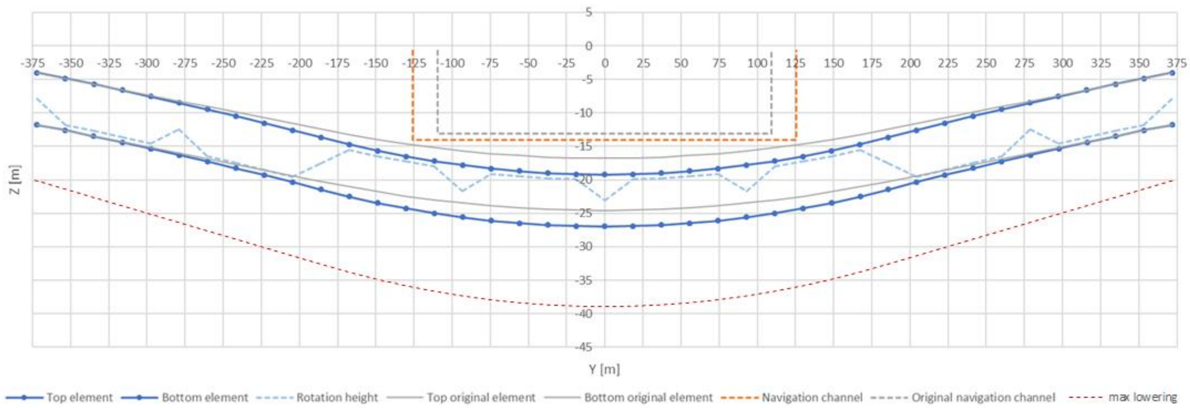

*Figure 6.18: This figure shows the longitudinal profile of the 1st Beneluxtunnel and the associated navigation channel in case only the segment joints are rotated with 1.6*10^{-3} rad.*

Only immersion joints

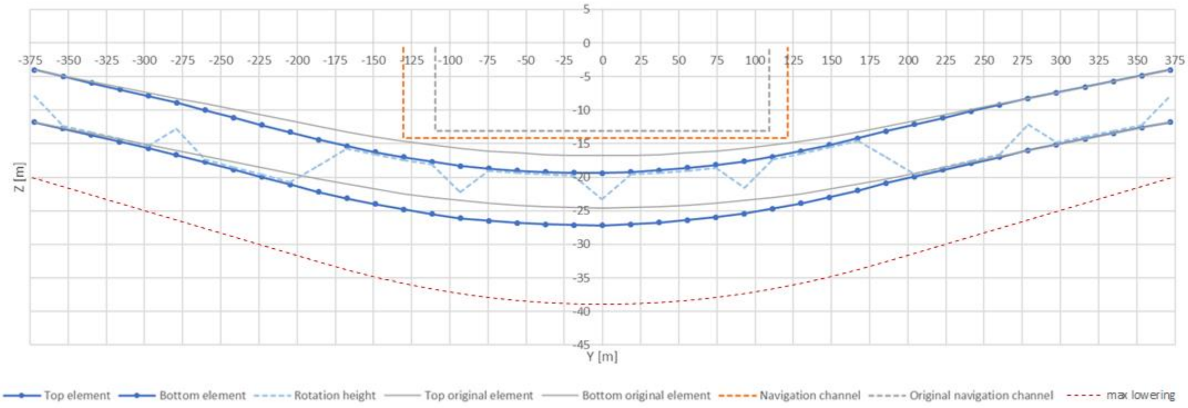

*Figure 6.19: This figure shows the longitudinal profile of the 1st Beneluxtunnel and the associated navigation channel in case only the immersion joints are rotated with 7.0*10⁻³ rad.*

Both segment and immersion joints

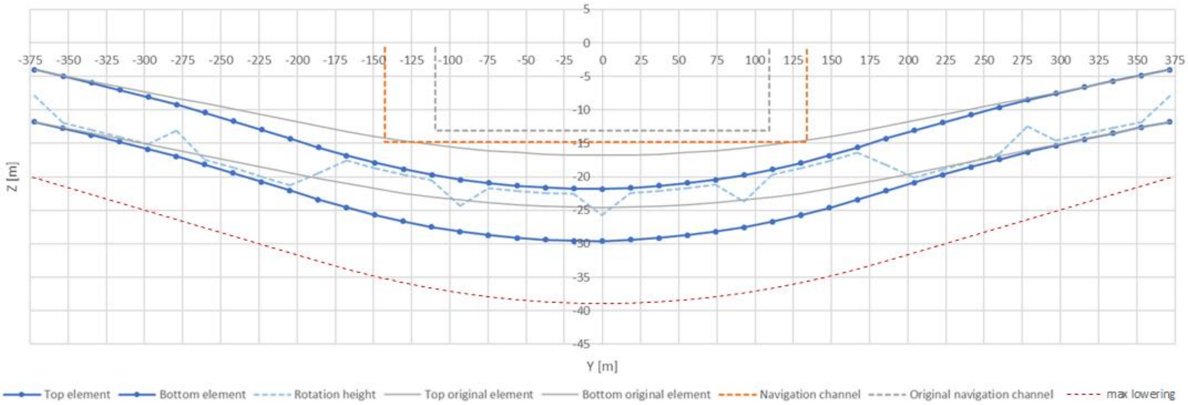

*Figure 6.20: This figure shows the longitudinal profile of the 1st Beneluxtunnel and the associated navigation channel in case the segment joints are rotated with 1.6*10⁻³ rad and the immersion joints are rotated with 7.0*10⁻³ rad.*

Results

The most important values of this analysis are given in table 6.1:

	Original	Only segment	Only immersion	Both joints	
Lowest point (vs NAP)	-24.56	-26.98	-27.18	-29.60	m
Lowering	-	2.42	2.62	5.04	m
Navigation depth (at NAP)	13.86	14.80	14.82	15.54	m
Deepening	-	0.94	0.97	1.68	m
Navigation width	218.79	251.26	252.13	276.67	m
Widening	-	32.48	33.34	57.88	m
Longitudinal displacement	-	208	223	474	mm
Max slope	4.55	5.85	5.96	7.27	%

Table 6.1: The effects of the type of joint rotations on the vertical profile.

6.5.3 The influence of the magnitude of rotation

In this section, the influence of joint rotations on the vertical profile of the tunnel will be analysed for different magnitudes, ranging from 0.5 times the rotation as defined in section 6.5.1, up to 2 times this rotation. Also, the effects of only rotating one of the joint types is included in the analysis. The results of the 2 times situation is shown in figure 6.21, the figures of the other examples are not given.

Note: 2 times the rotation as defined in section 6.5.1 is equal to the maximum rotation as defined in section 6.4, without applying a safety margin for settlement.

Times the rotation

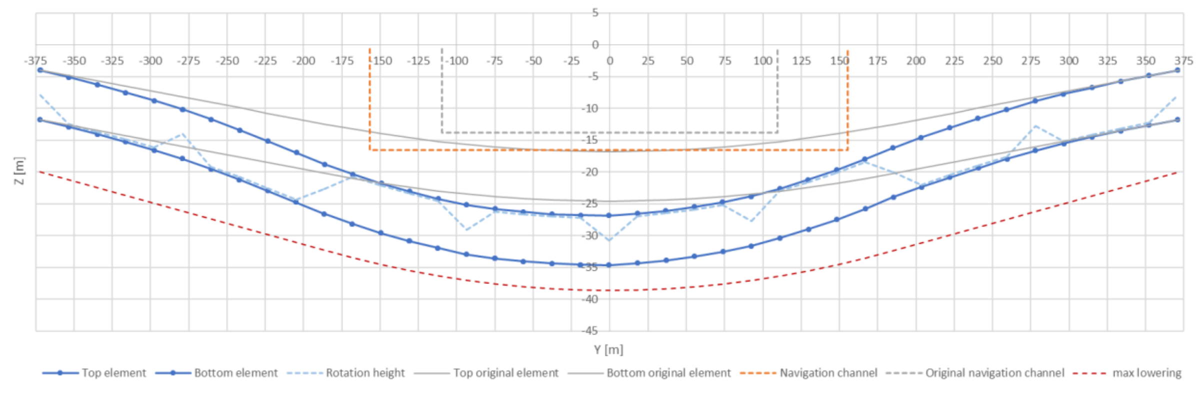

Figure 6.21: This figure shows the longitudinal profile of the 1st Beneluxtunnel and the associated navigation channel in case the segment joints are rotated with $3.2*10^{-3}$ rad and the immersion joints are rotated with $14.0*10^{-3}$ rad.

Results

The most important values of this analysis are given in table 6.2:

	Original	0.5 x	1.5 x	2.0 x	2.0 x segment	2.0 x immers.	
Lowest point (vs NAP)	-24.56	-27.08	-32.12	-34.64	-29.41	-29.80	m
Lowering	-	2.52	7.56	10.08	4.85	5.24	m
Navigation depth (at NAP)	13.86	14.81	16.12	16.56	15.51	15.58	m
Deepening	-	0.96	2.26	2.71	1.66	1.73	m
Navigation width	218.79	251.66	296.54	311.91	275.84	278.14	m
Widening	-	32.87	77.76	93.12	57.05	59.35	m
Longitudinal displacement	-	214	779	1131	460	501	mm
Max slope	4.55	5.92	8.68	10.08	7.19	7.40	%

Table 6.2: The effects of the magnitude of joint rotations on the vertical profile.

6.5.4 The influence of the design options

In this section, the influence of joint rotations on the vertical profile of the tunnel will be analysed for the different design options defined in section 3.2

The same combinations will be used as in section 4.4.1:

Primary land part (south)	**Secondary land part (north)**
Do nothing	Do nothing
Local adjustments	Local adjustments
General lowering	General lowering
Do nothing	Local adjustments
Do nothing	General lowering

However, some exceptions are maintained:

- Do nothing combined with local adjustments will not be modelled.
- Local adjustments combined with general lowering will also be analysed.
- The 50 km/h situation will not be analysed for all alternatives.

The results are shown in figures 6.22 through 6.32 and in tables 6.3 through 6.6.

Do nothing on both sides

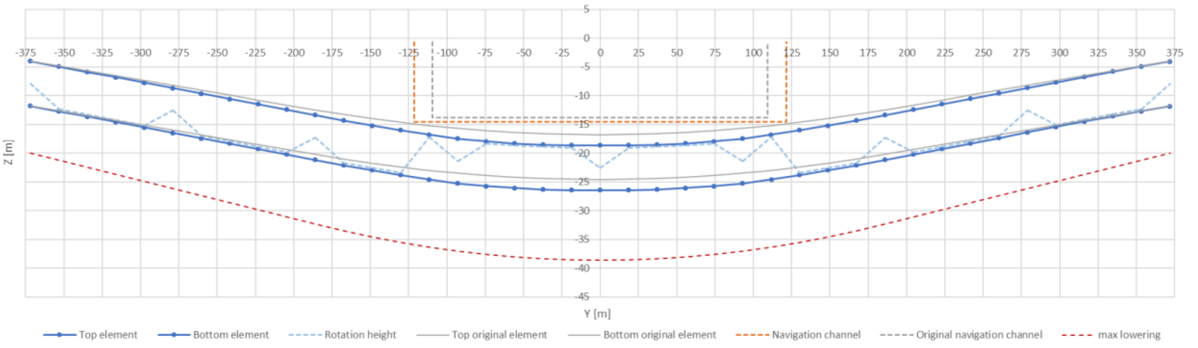

Figure 6.22: This figure shows the vertical profile of the 100 km/h alternative, having a maximum slope of 5%

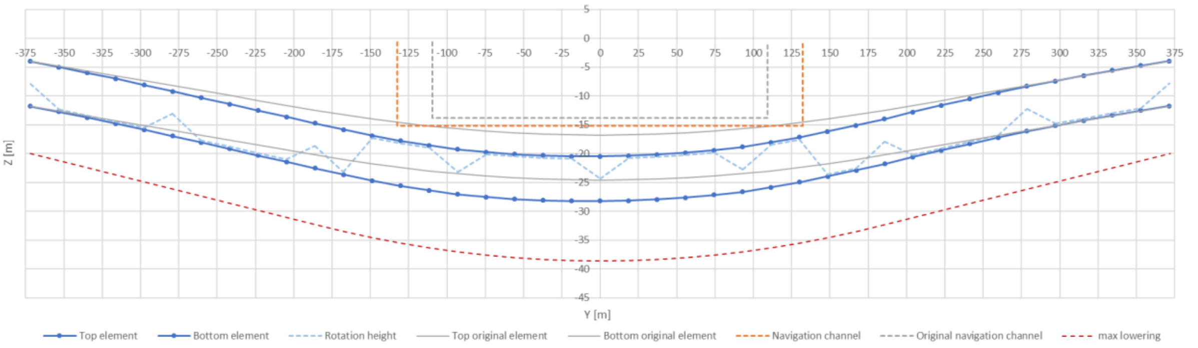

Figure 6.23: This figure shows the vertical profile of the 80 km/h alternative, having a maximum slope of 6%

	Original	5%	6%	
Lowest point (vs NAP)	-24.56	-26.09	-30.69	m
Lowering	-	1.53	6.13	m
Navigation depth (at NAP)	13.86	14.47	14.73	m
Deepening	-	0.62	0.88	m
Navigation width	218.79	240.04	249.03	m
Widening	-	21.25	30.24	m
Longitudinal displacement	-	112	650	mm
Max slope	4.55	5.04	6.19	%

Table 6.3: The effects of joint rotations on the vertical profile for the alternative "do nothing on both sides".

Local adjustments on both sides

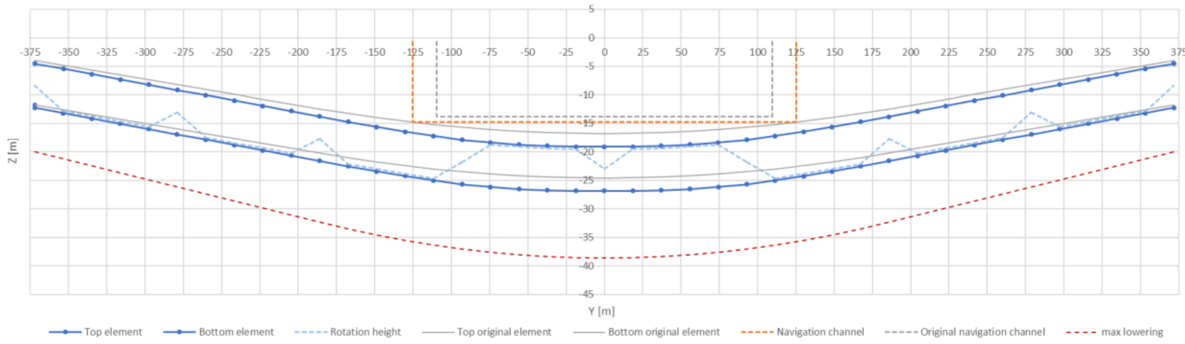

Figure 6.24: This figure shows the vertical profile of the 100 km/h alternative, having a maximum slope of 5%

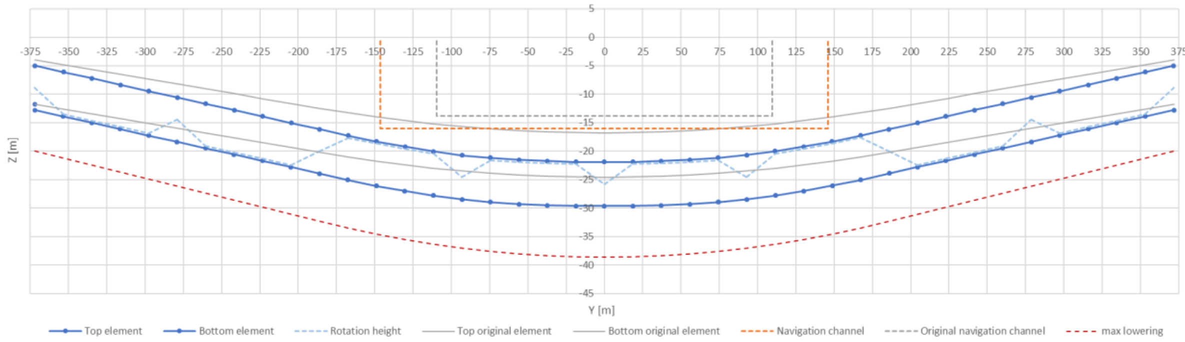

Figure 6.25: This figure shows the vertical profile of the 80 km/h alternative, having a maximum slope of 6%

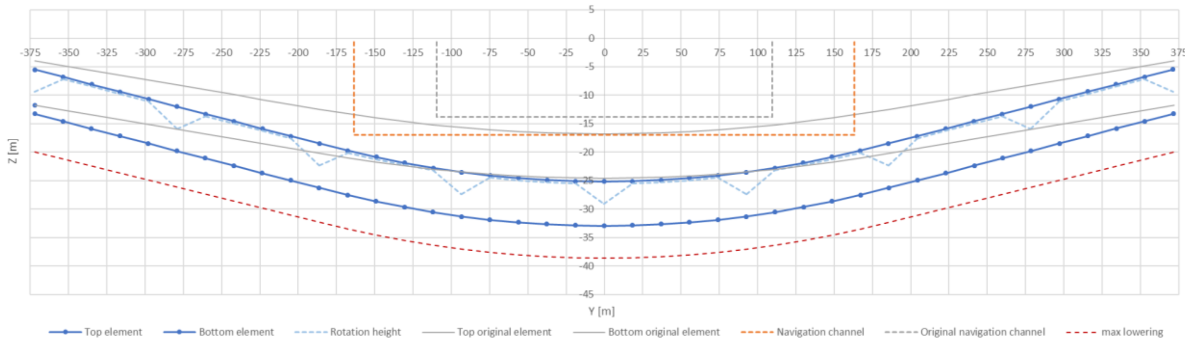

Figure 6.26: This figure shows the vertical profile of the 50 km/h alternative, having a maximum slope of 7%

	Original	5%	6%	7%	
Lowest point (vs NAP)	-24.56	-26.87	-29.66	-32.96	m
Lowering	-	2.31	5.11	8.40	m
Navigation depth (at NAP)	13.86	14.78	16.00	16.99	m
Deepening	-	0.93	2.14	3.14	m
Navigation width	218.79	250.63	292.42	326.78	m
Widening	-	31.84	73.63	108.00	m
Longitudinal displacement	-	180	274	519	mm
Max slope	4.55	5.05	6.02	7.03	%

Table 6.4: The effects of joint rotations on the vertical profile for the alternative "local adjustments on both sides".

General lowering on both sides

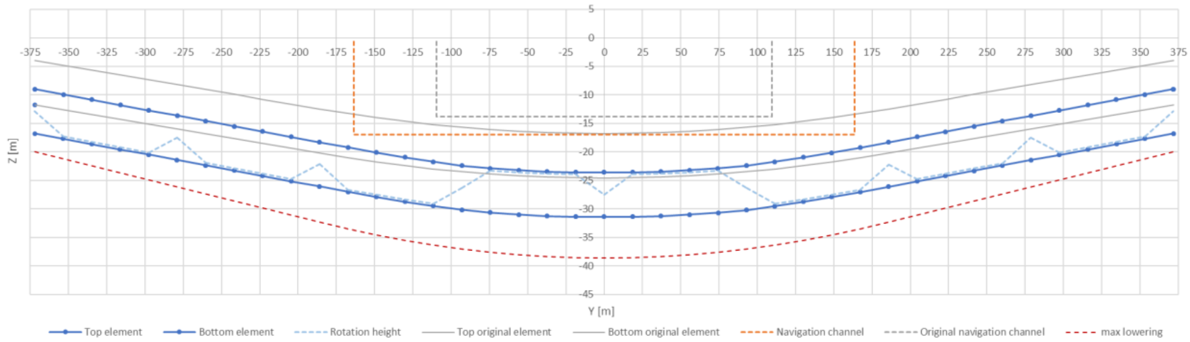

Figure 6.27: This figure shows the vertical profile of the 100 km/h alternative, having a maximum slope of 5%

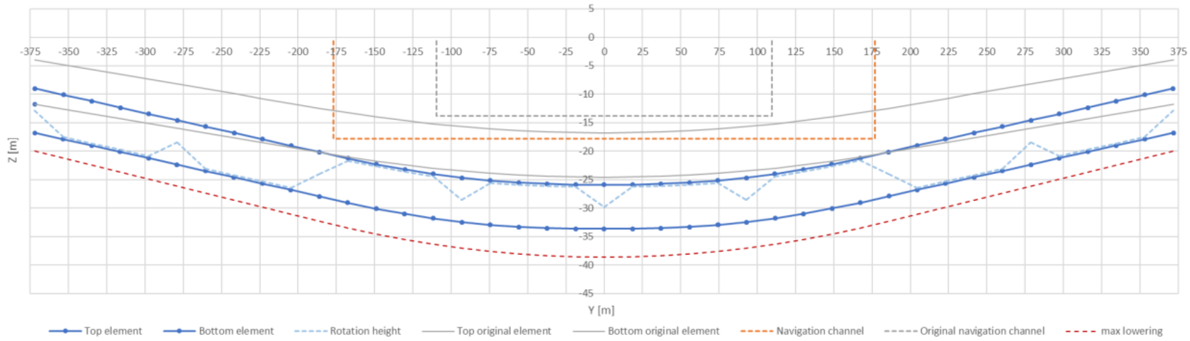

Figure 6.28: This figure shows the vertical profile of the 80 km/h alternative, having a maximum slope of 6%

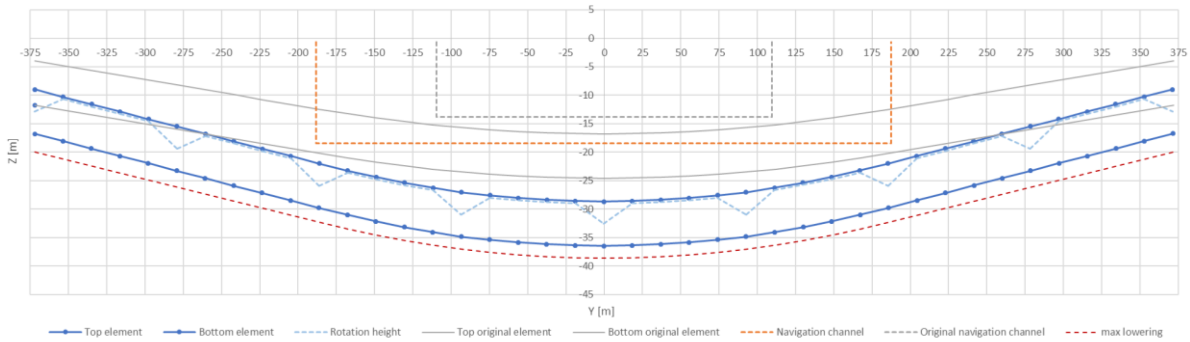

Figure 6.29: This figure shows the vertical profile of the 50 km/h alternative, having a maximum slope of 7%

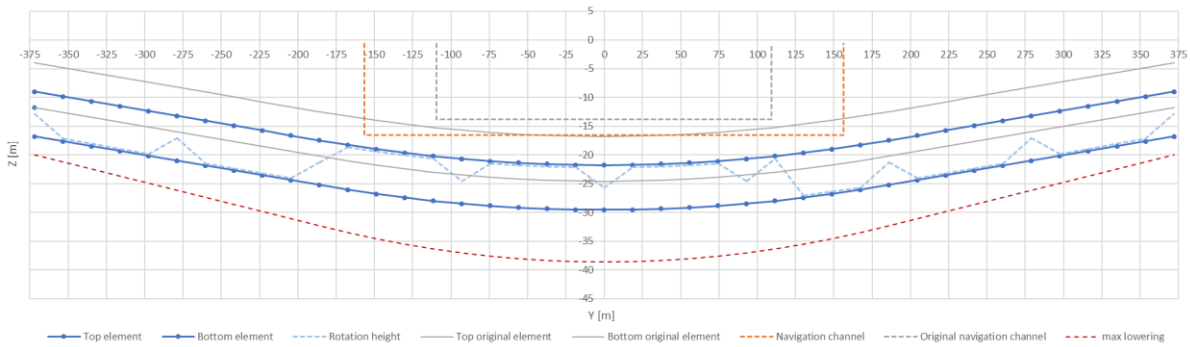

Figure 6.30: This figure shows the vertical profile of alternative in which the current slopes are applied, having a maximum slope of 4.5%

	Original	5%	6%	7%	4.5%	
Lowest point (vs NAP)	-24.56	-31.37	-33.66	-36.46	-29.56	m
Lowering	-	6.81	9.11	11.90	5.00	m
Navigation depth (at NAP)	13.86	17.01	17.78	18.42	16.59	m
Deepening	-	3.15	3.92	4.57	2.74	m
Navigation width	218.79	327.18	353.75	375.95	312.95	m
Widening	-	108.40	134.96	157.17	94.17	m
Longitudinal displacement	-	180	161	519	0	mm
Max slope	4.55	5.05	6.05	7.03	4.55	%

Table 6.5: The effects of joint rotations on the vertical profile for the alternative "general lowering on both sides".

General lowering on one side

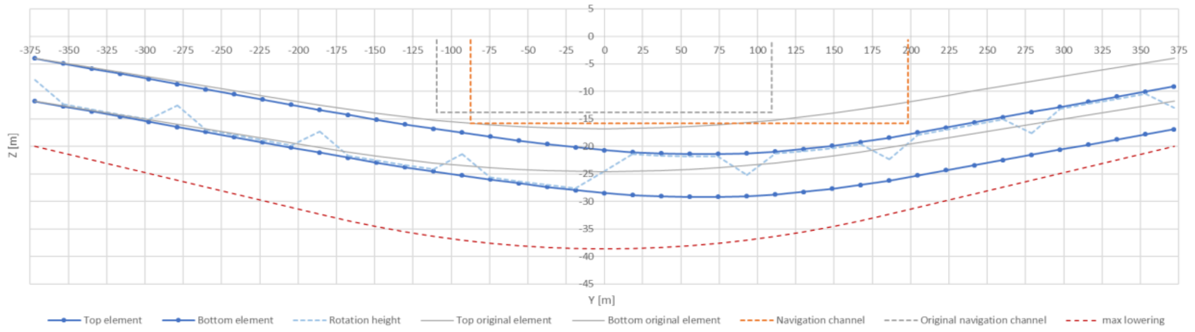

Figure 6.31: This figure shows the vertical profile of the 100 km/h alternative, having a maximum slope of 5%

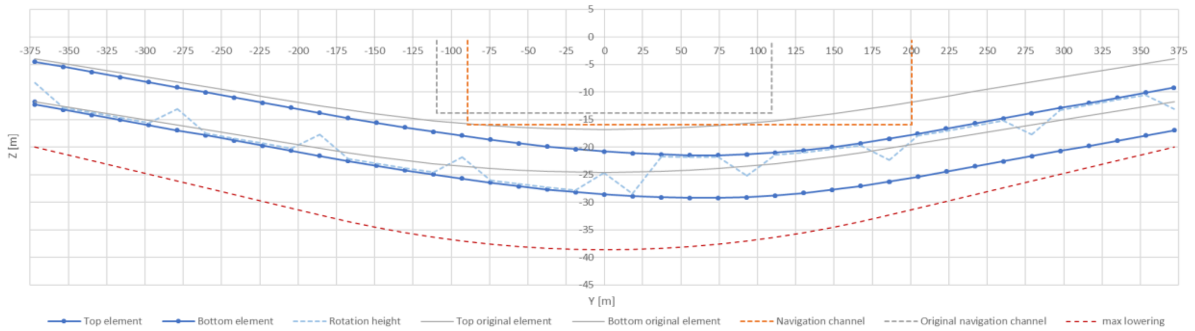

Figure 6.32: This figure shows the vertical profile of the 100 km/h alternative for the alternative general lowering combined with local adjustments, having a maximum slope of 5%

	Original	0-gen		loc-gen		
		5%	6%	5%	6%	
Lowest point (vs NAP)	-24.56	-29.18	-30.80	-29.24	-32.22	m
Lowering	-	4.62	6.24	4.68	7.67	m
Navigation depth (at NAP)	13.86	15.80	16.23	15.93	16.87	m
Deepening	-	1.95	2.38	2.08	3.02	m
Navigation width	218.79	285.80	300.59	290.32	322.60	m
Widening	-	67.02	81.80	71.53	103.81	m
Longitudinal displacement	-	136	177	-35	143	mm
Max slope	4.55	5.03	6.04	5.01	6.04	%

Table 6.6: The effects of joint rotations on the vertical profile for the alternative "general lowering on one side".

6.5.5 Conclusions

General conclusions

Regarding all three analyses, the following conclusions are given:

- Joint rotations are able to cause significant lowering of the alignment of the tunnel.
- Joint rotations also cause significant horizontal displacements, ranging up to about 65 cm.
- In none of the treated scenarios, the depth limitations as determined in section 6.3 were breached.

Conclusions regarding the influence of the rotations per joint type

In section 6.5.2, the influence of rotating only the segment joints, rotating only the immersion joints or rotating both type of joints was analysed. The following conclusions are given:

- If only the segment joints or only the immersion joints are rotated, the navigation channel will be deepened with about 0.95 m. Consequently, the influence of these both joint types are similar.
- If both the segment joints and the immersion joints are rotated, the navigation channel will be deepened with about 1.68 m, which is about 75% more than if either one of the joints is rotated.

Conclusions regarding the influence of the magnitude of rotations

In section 6.5.3, the influence on the vertical profile of rotating only the segment joints, only the immersion joints or rotating both type of joints was analysed. The following conclusions are given:

- The effectivity of joint rotations is almost proportional. As is indicated in figure 6.33, the effect of increased rotations on the depth of the navigation channel slightly decreases for larger rotations.
- If the rotations are twice as large, the effect of rotating both joint types is about 60% larger than for a single joint type. This difference is smaller than for the initial rotation discussed in section 6.5.2. This could very well be a consequence of the decrease in effectivity for larger rotations.

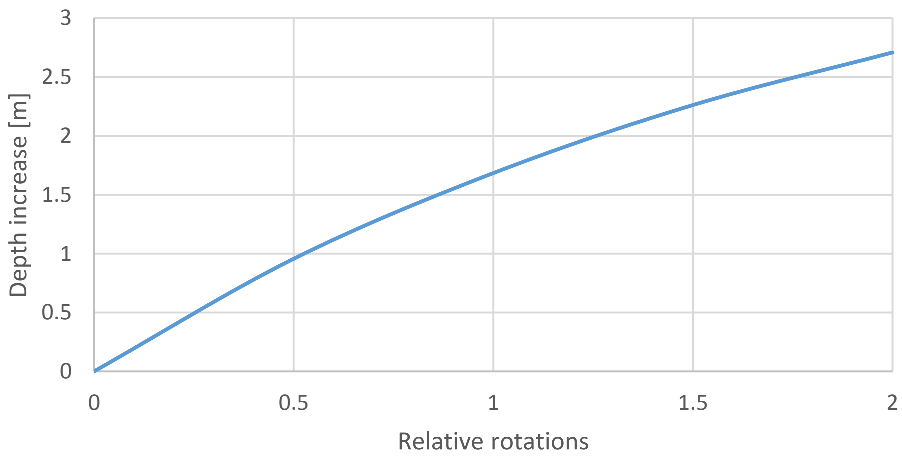

Figure 6.33: The effectivity of joint rotations

Conclusions regarding the influence of the design options

In section 6.5.4, the influence of the design options on the vertical profile of the 1st Beneluxtunnel was analysed. The results of the analysis are combined into figure 6.34. Most values are obtained directly from tables 6.3 through 6.6, others are interpolated using the relationships determined in this section. Also the functional limitations determined in section 4.4 are shown.

Consequently, this figure shows the maximum possible lowering for both technical and functional limitations, for different maximum speeds and for the following scenario's concerning the land parts:

0-0	Do nothing on both sides.
loc-loc	Local adjustments on both sides.
gen-gen	General lowering on both sides.
0-loc	Local adjustments on one side.
0-gen	General lowering on one side.

The * is used to indicate the use of the existing traffic requirements. Furthermore, the differences between functional and technical limitations are:

- Regarding the technical limitations, a distinction has been made between the joint types that are rotated, as is discussed in section 6.5.2. These limitations apply to the situation in which tunnel sections are lowered while they remain immersed.
- If the elements are re-floated, they also need to be adapted to the required rotations, but it is expected that the technical limitations as determined in section 6.4 do not apply. Consequently, the functional limitations are valid for this scenario.

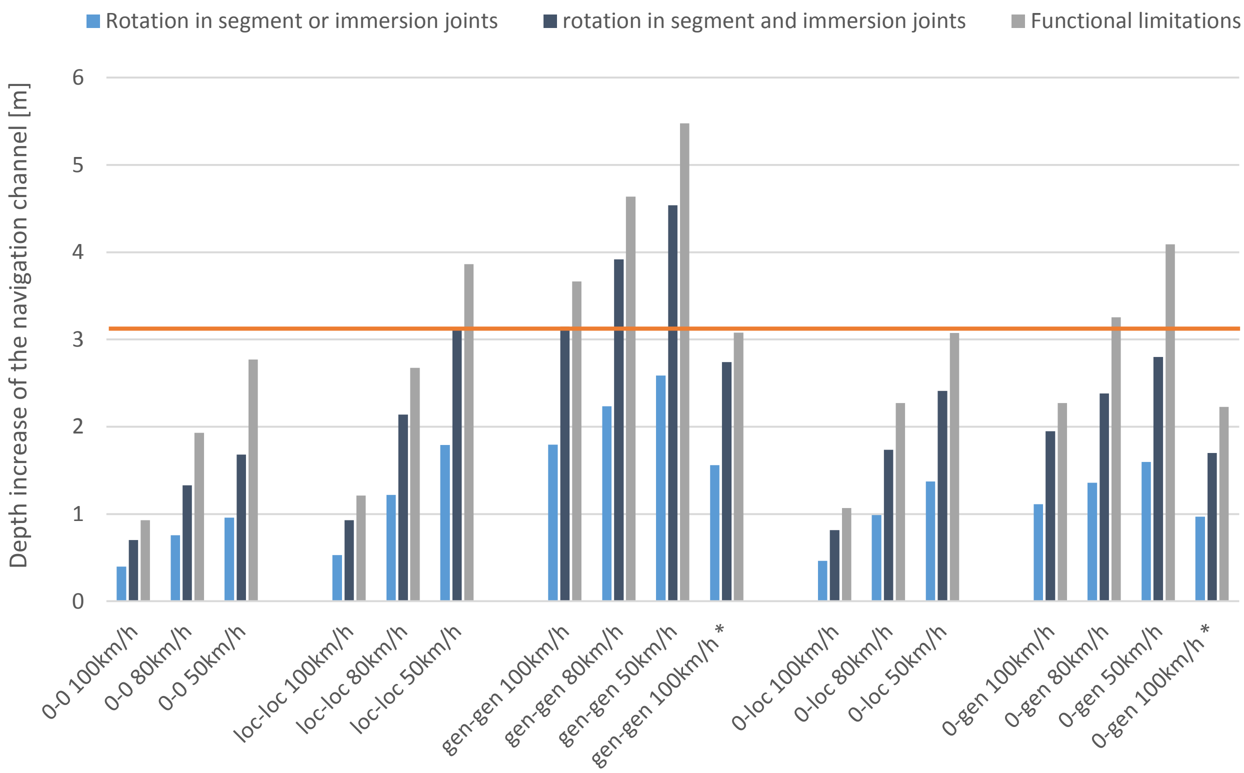

Figure 6.34: This figure shows the depth increase limitations of the design options. The orange line indicates the depth of the Maeslantkering.

The differences between the technical limitations and the functional limitations are rather large, which is a consequence of the presence of the bottom curve. The required radius according to functional requirements is much smaller than the applied radius, which stretches over 4 elements.

For the design options, three areas can be distinguished along the length of the tunnel:

- Near the land parts, the sections rotate downward (compression in floor), until the desired slope is reached. Given the initial slope of 4.5%, this does not require much length. If the slope is already changed at the land part – which is the case for local adjustments and general lowering – this area is not required.
- The second area lies near the edges of the navigation channel. The bottom curve of the tunnel starts well before the navigation channel is reached. To lower the alignment, this effect has to be counteracted, requiring downward rotations (compression in the floor).
- Near the deepest point of the tunnel, the direction of the alignment needs to change from downwards to upwards, requiring large changes in slope. Consequently, upward rotations (compression in roof) are required, decreasing the radius of the bottom curve.

It depends heavily on the situation which area is governing. Also, these areas are found on both sides of the tunnel, except for the asymmetric options.

6.6 Summary

In this chapter, the effects of lowering on the mechanics of the Beneluxtunnel have been analysed in order to determine the most important lowering limitations. To do so, the structural analysis of chapter 5 is used to determine the effects of certain changes.

Effects of lowering

If the profile of the tunnel is lowered in order to increase the draught, two mechanisms will occur:

- The depth of certain parts of the tunnel will increase, which results in an increase of hydrostatic pressure and in some cases soil pressure.
- The vertical profile of the tunnel will change, which results in rotations in the segment and/or immersion joints.

Maximum depth increase

To determine the maximum depth increase, the effects of lowering on the land part and the immersed part are estimated. The cross-sectional capacity of the immersion joints allows a lowering of 20.9 m regarding the moment capacity and 14.1 m regarding the shear capacity. Using an additional safety factor of 1.3, the values would be respectively 10.8 m and 4.9 m. The longitudinal mechanics are not expected to be significantly negatively influenced by depth increase.

If the land part is subjected to "general lowering", both for the vertical stability and the transverse stability measures are required. Consequently, an earlier estimate of 5 m maximum lowering seems to be quite accurate.

Maximum joint rotations

The maximum joint rotations are determined by the deformation capacity of the rubber water stops. Structural aspects only become important if the rotations are much larger and the effects on traffic requirements will be manageable.

Settlement can cause rotations that are much larger than the maximum rotations allowed by the rubber profiles, however the probability of such extreme rotations is expected to be very low. To determine the actual influence of settlement on the maximum rotations requires a probabilistic analysis on the expected settlement differences. For now, maximum rotations are assumed 50% of the maximum rotations according to the rubber profiles.

Vertical profiles

To determine the effects of joint rotations on the vertical profile, a model is made. Using this model, it is shown that the joint rotations can be utilized to allow significant depth increase to the navigation channel. If one of the joint types is used the depth increase is about 55% of the case in which both types are used, which is about 60 - 80% of the functional lowering.

Chapter 7

Construction of alternatives

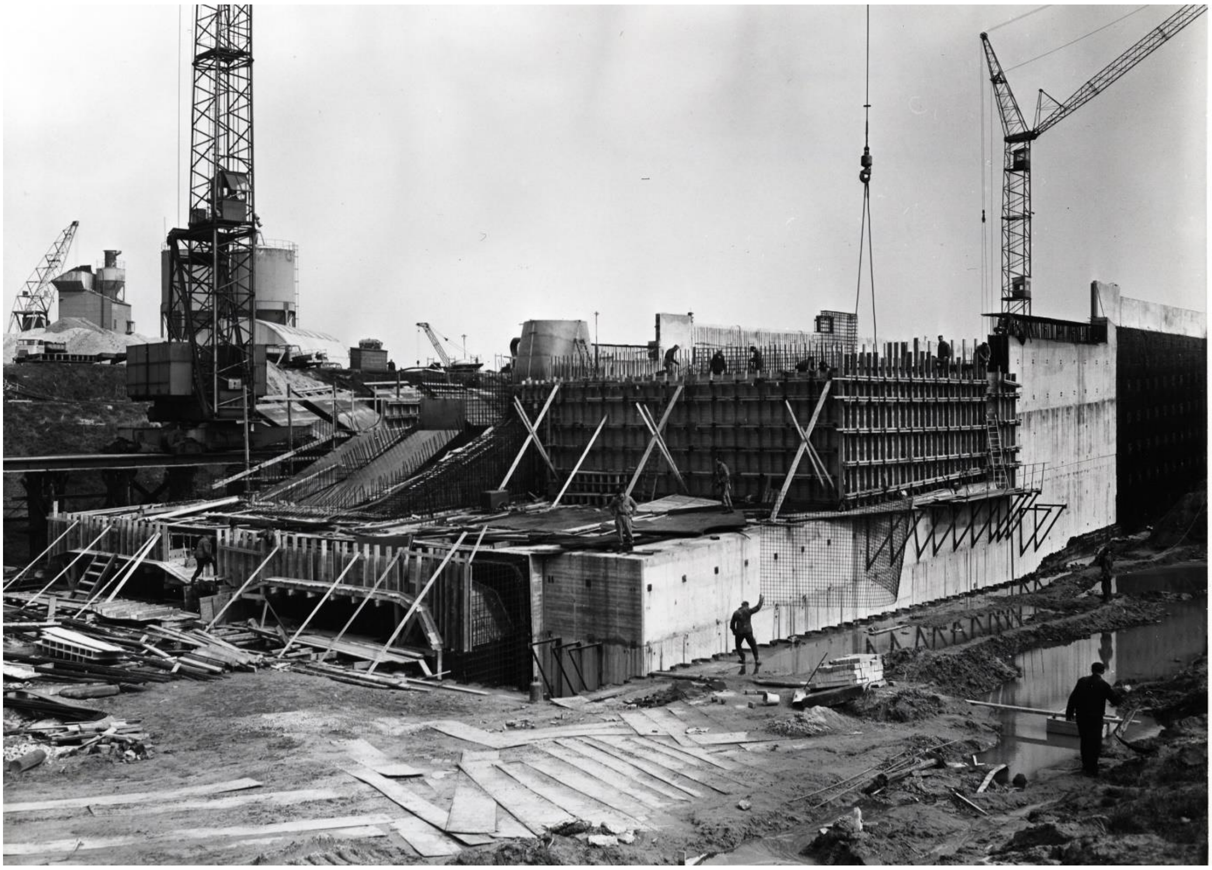

Figure 7.1: The construction of one of the abutments of the 1st Beneluxtunnel [40]

7.1 Introduction

In this chapter, the construction methods required to execute the design alternatives for the lowering of the Beneluxtunnel are elaborated.

The main purpose of this chapter is to establish a certain level of technical feasibility as is discussed in section 3.3.2. However, as a secondary purpose, the construction cost estimates treated in chapter 8 will be based on the construction sequences defined in this chapter.

All design options discussed within this chapter were proposed in section 3.2.3.

Immersed part

The following subjects will be discussed regarding the immersed part:

Keep elements immersed The first design option to be discussed is the possibility of lowering the tunnel while keeping the elements immersed. (Section 7.2)

Re-floating elements The second design option regarding the immersed part is the possibility of lowering the tunnel by re-floating the elements. (Section 7.3)

Note: the option "adapt river bottom" will not be discussed as the effectivity of this method is regarded very low. (Section 4.3.1)

Land part

The following subjects will be discussed regarding the land part:

Local adjustments The first design option to be discussed regarding the land part, is the possibility of locally adjusting the transition point. (Section 7.4)

General lowering The second design option regarding the land part is the possibility of generally adapting the land part. (Section 7.5)

Note: the design option "do nothing" will not be discussed as it does not require any constructional effort. (Section 3.3.2)

7.2 Immersed part 1: Keeping elements immersed

The first design option to be discussed is perhaps the most conceptual. The objective of this method is to lower the segments of the immersed tunnel according to an adjusted vertical profile without re-floating the elements. Consequently, the elements will remain immersed and

In section 5.4 the possibility of lowering the tunnel by exploiting the freedom of rotation in the segment joints and the immersion joints was examined.

Doing so, some difficulties are to be expected:

- How can the soil, tiles and gravel underneath the elements be removed?
- How can the stability of the elements be maintained if all surrounding soil is removed?
- How can the elements be lowered to their desired position in a controlled and manageable fashion?
- What will be the influence of the construction sequences on the structural safety of the elements?

These subjects will be discussed in this section. Subsequently, the main construction sequences will be given.

7.2.1 Soil removal

On all sides of the elements lies soil. In order to lower the elements, at least the volume of the soil below the elements should decreased. Four options to accomplish this will be discussed in this section:

- Dredging from vessel
- Access from land
- Access from tunnel
- Compaction

Consequently, also the influence of siltation and the removal of the foundation tiles will be discussed. Finally, an advice regarding these options will be given.

Dredging from vessel

The most common method to remove soil from river bottoms is by dredging. To remove the soil on top and on the sides of the tunnel, normal dredging equipment can be used. Bottom protection that is too large to be removed by dredging can be removed by cranes on barges.

However, to reach underneath the elements is a much greater challenge. To find out if this it all possible, industry specialist Pieter Koman[8] has been consulted. According to his knowledge, it would be possible to use a dustpan type of dredger to reach underneath the elements, as long as the angle would be small enough for it to reach until halfway, meaning the slope and depth of the trench should be sufficient as is indicated in figure 7.2. Consequently, removing only a small layer of soil underneath the elements would not be possible. To reduce the impact on the trench requirements, the equipment can possibly be modified.

[8] Pieter Koman is a senior project manager at Van Oord and has had much experience with immersed tunnels.

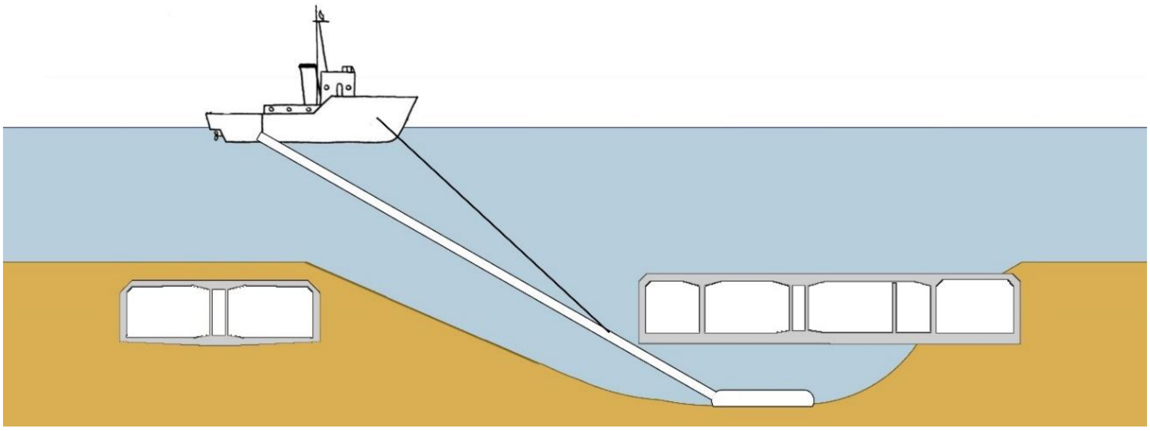

Figure 7.2: This figure shows a graphical representation of the dustpan dredger below the 2nd Beneluxtunnel.

Furthermore, removing gravel would not be a problem for normal dredging equipment and dredging at large depths is also common practice.

This advice is regarded sufficient to expect removing the soil by dredging is possible and would provide an executable method to remove the soil.

Important with dredging is the influence of the trench of one tunnel on the presence of the other tunnel. This also indicated in figure 7.2, although the situation in this figure would not occur as it would be advised to start construction on the 1st Beneluxtunnel. Nevertheless, this aspects should be taken into account.

A clear disadvantage of this method is that for removing the soil underneath the elements, also the soil surrounding the elements will have to be removed. This will be quite costly as will be discussed in section 8.2.3.

The removal of this soil would also make the elements subjected to flow forces (section 5.2.5) and siltation.

Access from land parts

A method that would limit the removed soil to only the soil underneath the elements is to remove soil by gaining access from the land parts. From there, the entire tunnel could be undermined using the caisson method.

However, in order to work in such a space, it should also be water tight, requiring some type of screens. In the transverse direction, such a screen could for instance be provided by inserting sheet piles along both sides of the tunnel while closing the gap between the tunnel and the sheet pile wall with grout or chemical injection. Given a required space of 5m, a sheet pile would have to be inserted about 20m into the bottom of the waterway. As the depth the waterway is at some points NAP -16.5m, the feasibility of this method is questionable.

Furthermore, water tightness will only be provided if also in the longitudinal direction a screen exists. Luckily, sheet pile walls that would provide this are used during the construction of the land part and are presumably still present, as removing these sheets could damage the abutments. However, these walls only reach up to NAP -22 m for the 1st Beneneluxtunnel and NAP -24 m for the 2nd Beneluxtunnel, which is not enough to act as a screen for the entire tunnel. Hence, intermediate screens should be provided, which will have to be installed in a confined space. Figure 7.3 shows this option.

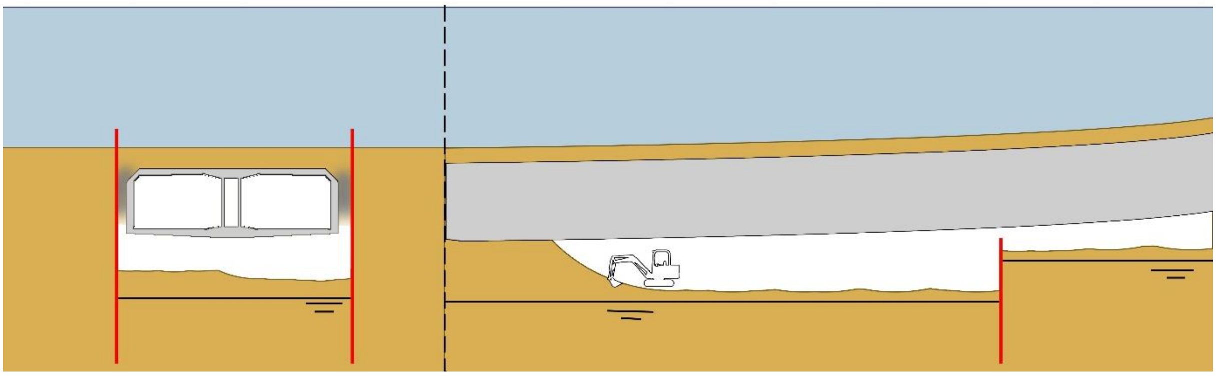

Figure 7.3: This figure shows a graphical representation of digging underneath the tunnel.

More technical difficulties are to be encountered like the fact that the area should be pressurised which immensely increases labour costs, and the space that is created will create a very large buoyancy load on the tunnel which will have to be counteracted by ballast and will also cause a risk of blowouts.

Removing soil from tunnel

Another method that would limit the removed soil to only the soil underneath the elements is to remove soil by gaining access from inside the tunnel. The general idea behind this method is that enclosed spaces inside the tunnel are created in which the pressure could be increased up to the level outside of the tunnel. Here, holes could be made in the floor allowing access to soil underneath.

One could make such a hole large enough for equipment and workers to be lowered through, however, the same problems would arise that are to be encountered in case access is provided from the land parts as is described above.

Instead, another strategy could be maintained. Holes could be bored through the floor which would be connected to a system of pipelines in which the pressure can be controlled. If the pressure inside such a pipeline is lowered, water would start to flow into this pipeline. If flow speeds increase, water starts transporting the soil from underneath the tunnel into the pipelines.

If the holes are created at the right locations, local pressures could be adjusted and the elements could be lowered in controlled fashion.

This method can in some extent be interpreted as the inversed version of internal soil flowing method, used to apply the foundation underneath the element. This method is shown in figure 7.4

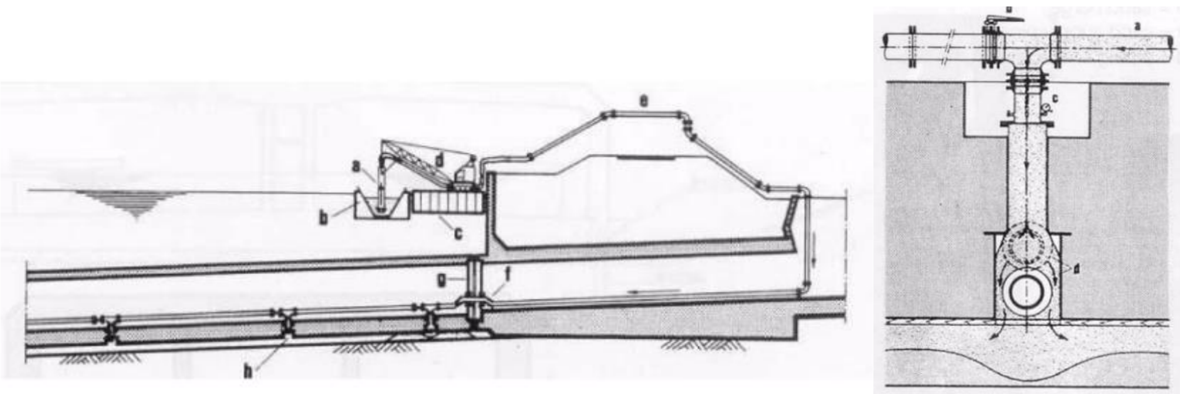

Figure 7.4: This figure shows the internal soil flowing method (left) and a close up of the hole in the tunnel floor (right). [4]

Furthermore, in the 1st Beneluxtunnel the reinforcement spacing is about 20cm, meaning about 15cm holes could be bored without damaging reinforcement. Also, some kind of protruding pipe could be inserted to subject deeper layers which could for instance be used in case of gravel foundations.

If the lowering process is finished, the holes can be refilled with concrete. They could even be closed in a way they could be re-opened in the future when again, settlement or straightening of the elements is required.

A major challenge for this method is the strength of the tunnel elements. The elements have much excess strength as is determined in section 5.3.3, but if this is enough to resist the unpredictable loads of this method is yet unknown. Also, the effects of the soil near the corners of the tunnel might become problematic.

This method seems very promising, but its success depends in large extent on how the soil reacts, making it unpredictable and experimental.

Compaction

Another method to decrease the volume of soil is through compaction. Due to the digging of the trench, the subsoil is relatively relieved and also, the underflown soil is not compacted. The effects of this will however be very limited as the layer is only up to a meter thick.

It is expected this method will not be able to provide sufficient lowering. [56]

Siltation

In section 2.4.3, it was mentioned that the initial attempt to lower the elements of the 2nd Beneluxtunnel onto the gravel foundation was complicated by the presence of silt. For this tunnel, the expected rate of siltation in the trench was estimated using the modelling programme SURTRENCH. The results were:

- 0.2 - 0.3 m/week for the mid sections of the waterway
- 0.04 - 0.1 m/week for the banks of the waterway
- 0.1 -0.2 m/week near the land parts
- 4000 - 5000 t/week for the entire tunnel

Also, the shape of the trench tends to flatten which without bottom protection would cause the other tunnel to be revealed a few weeks after the trench is made.

This siltation only affects the dredging option. However, given the fact that to be able to dredge underneath the elements, the trenches have to be relatively deep. Siltation does not become problematic quickly.

Tile removal

Apart from soil, also hard obstacles are found underneath, namely temporary tile foundations on which the tunnels were lowered before sand was pumped underneath[9]. They can be up to 5 x 5 m. An example of a tile foundation for the 1st Beneluxtunnel is shown in figure 2.30.

The distance between these tiles and the tunnels was initially about 0.2 m which is not enough to allow sufficient lowering, meaning the tiles are to be removed from their current position before lowering is possible.

To remove these tiles, they will have to be reached and physically removed as it is expected that any other effort to remove them would threaten the tunnel. To do so, the surrounding soil will have to be removed by dredging.

Once the tiles are reached, divers can attach cables to them. Consequently, they can be pulled out horizontally. For the tiles of the 1st Beneluxtunnel, the horizontal pulling force would need to be about 60kN to move these tiles, only taking into account the friction force of the subsoil. Most small boats are capable of providing this. However, effort must be made to assure the pulling is indeed carried out horizontally enough not to damage the tunnel, which possibly has consequences for the slope of the trench, or perhaps some kind of anchor could be used instead.

Conclusion

In this study, the dredging method will recommended. Although this method has major disadvantages compared to the other methods because the elements have to be revealed, meaning there are subjected to flow forces, siltation, very much additional soil needs to be removed and the dredging vessels might hinder navigation, this method is by far the most feasible of all.

- Constructing a watertight space underneath the tunnel is hard to achieve and the risk of flooding is high.
- Gaining access from tunnel is very experimental and therefore risky.
- Compaction does not provide sufficient lowering.

An additional advantage of this method is that given the fact that the tiles have to be removed by dredging, already a significant amount of soil is removed.

However, the other methods might propose interesting and more cost effective alternatives in later stages of design. Especially the method of removing soil from within the tunnel is very interesting as it – in addition to the advantages mentioned above – does not require any construction work outside of the elements, including temporary supports. It could also be an interesting idea for future tunnels.

To estimate the benefits of this experimental alternative, it will be included in the evaluation in section 8.2.1.

[9] Although the 2nd Beneluxtunnel was partly founded on gravel beds, tiles were still placed underneath as lowering on the gravel beds was initially not planned.

7.2.2 Stability

In the previous section, methods were proposed to remove the soil underneath the elements. Doing so, one creates an obvious problem: Stability. In this section, the problems regarding stability will be analysed and methods to provide vertical stability will be proposed.

The assumed soil removal method used in this section will be dredging. The conceptual method in which soil is removed from within the tunnel provides its own vertical stability.

First, a general outline regarding the stability of the tunnel will be given. Afterwards, the following subjects will be treated:

- Load resultants
- Tile removal
- Supports
- Lowering process

General outlines

To understand the problems and solutions regarding the stability of the tunnel, the lowering process will be briefly explained.

Initially, the tunnel will is fully embedded in soil. When the soil around the tunnel is removed as is explained in the previous section, the tunnel will be subjected to different loads, including navigation loads. These loads act in all directions. At some point, too much soil will be removed for it to support the tunnel and it should be manually supported. Once all soil is removed, the elements can be lowered to their new position. During this lowering sequence it is assumed that navigation forces will not be present as no vessels would be allowed. Once the tunnel, or a section of the tunnel, is lowered, the underflowing and backfilling process will start in which the tunnel can yet again be subjected to navigation.

During this entire process, stability has to be provided partly by the strength of the tunnel in the longitudinal direction and partly by supports. The locations of these supports therefore depends on the longitudinal strength.

This mechanism is closely related to longitudinal mechanics discussed in section 5.4 in which the displacements and rotations of segments were explained as if the tunnel was spanning a certain distance. With regards to vertical stability, the tunnel actually is spanning a distance. Three stages were distinguished:

Stage 1: The spanning distance is present but not large enough to cause tension in the tunnel.

Stage 2: The spanning distance is large enough to cause tension in the tunnel. Increase of tension will be prevented by the presence of the joints which will be opened. Further rotation will however be prevented by the reaction forces in the adjacent sections and settlement will be minimal.

Stage 3: The horizontal forces of the hinged span become larger than the reaction forces of the concrete and the segments will start to rotate until a new equilibrium is reached.

These stages will be used to discuss the vertical stability problems and solutions during the different construction sequences.

Load resultants

Once the soil and the bottom protection is removed, the tunnel would initially be subjected to the minimum vertical load resultant as indicated in table 5.7. The downward vertical load without navigation would be 280 kN/m for the 1st Beneluxtunnel and 594 kN/m for the 2nd Beneluxtunnel.

The influence of navigation is discussed in section 5.2.5. However, the results were expressed as moments and point loads acting on an entire elements if a ship passes either above the element or at 210m from the centre of the element. At this point, the moments will be ignored and only the vertical and transverse loads will be taken into account. However, over the entire length of the tunnel, the maximum load will be assumed present in case of navigation. The navigation loads per meter are given in table 7.1. The transverse load can act in both directions.

	downwards	upwards	transverse	
1st Beneluxtunnel	61	-43	198	kN/m
2nd Beneluxtunnel	115	-82	214	kN/m

Table 7.1: Navigation loads per meter tunnel.

The resultant loads with or without navigation are given in table 7.2.

	without navigation		with navigation		
	Vertical	Transverse	Vertical	Transverse	
1st Beneluxtunnel	280	0	340	198	kN/m
2nd Beneluxtunnel	594	0	709	214	kN/m

Table 7.2: Resulting loads with or without navigation.

The influence of the upward navigation load is not enough to counteract the weight of the elements and is therefore ignored. If there is no navigation present, the transverse loads will in reality not be 0 as there will always be some flow present. For this analysis this is ignored.

With these loads determined, a renewed version of figure 5.14 in which the maximum span at which the tensile stresses do not occur in the tunnel is given in relation to the depth. The spanning moments are estimated using $^1/_8\,qL^2$ to incorporate the possible effect of hinged joints.

The results are shown in figure 7.5. What may be noticed is that the transverse spans are much larger than the vertical spans, which is partly a result of the smaller loads in the transverse direction, but more importantly is the difference in section modulus. This also explains the large difference between the 1st and the 2nd Beneluxtunnel.

In figure 7.6, the same relationship between depth and span is shown, but regarding the maximum possible shear force. The shear force is estimated as $^1/_2qL$ and the shear resistance is expected to be lowest in the segment joints and is estimated using only interface friction with a friction coefficient of $\mu = 0.5$. It seems the shear force capacity is much larger than the moment capacity. However, as discussed in section 5.4.3, the shear force capacity estimates in the segment joints are very uncertain as is the assumption that it would keep increasing if the depth increases. For this study however, these estimates are regarded sufficient.

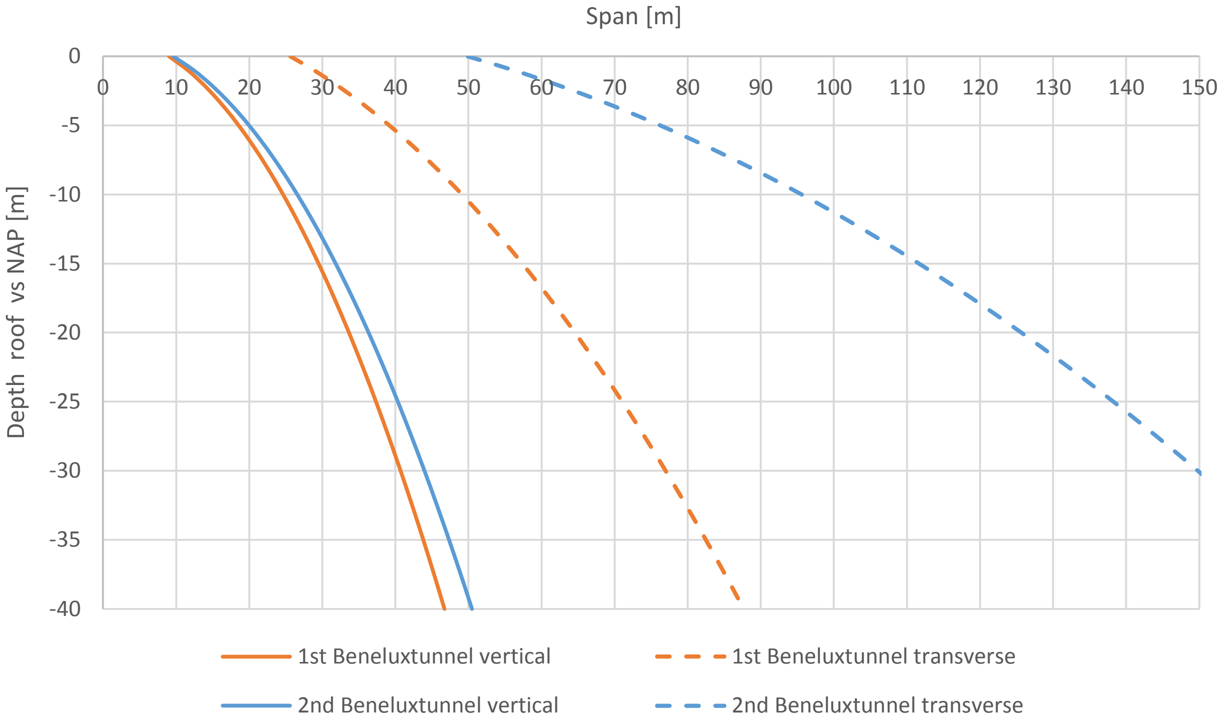

Figure 7.5: This figure shows the maximum span at which no tensile stresses occur in the tunnel for different depths in case navigation loads are included.

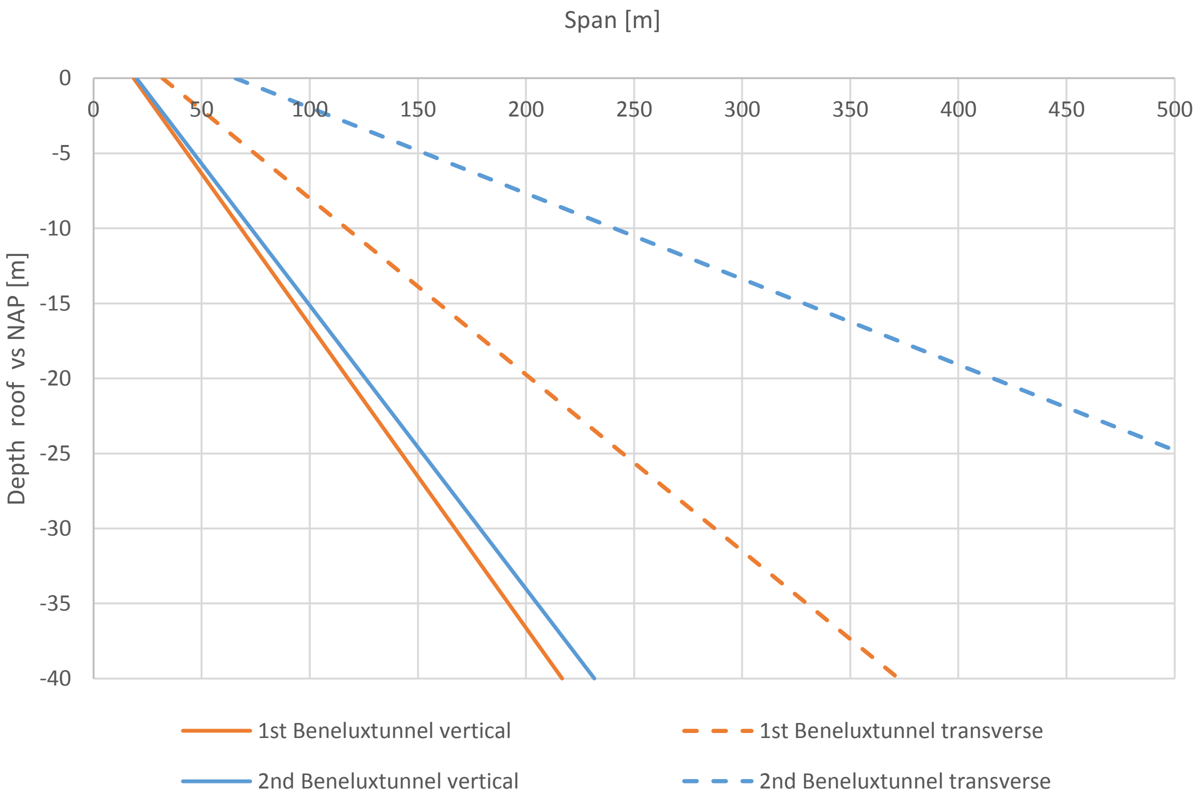

Figure 7.6: The relationship between maximum span and the depth of the roof for maximum allowed shear force.

Tile removal stability

As discussed in section 7.2.1, the foundation tiles need to be removed before the tunnel can be lowered. Doing so, the local foundation of the tunnel is removed. If a slope of 1/3 is maintained, an area of about 5 m around the 5x5 m tile would be unsupported. Given the larger stiffness of the tunnel in longitudinal direction resulting from the presence of walls in this direction, it is expected that the load would be transferred longitudinally. This would result in a span of up to 15 - 20 m.

The section of the tunnel that is subjected to horizontal load can also be expressed as a span. The supports however does not lie at the same location as the vertical support because the surrounding soil has a slope which influences both the area of the tunnel that is subjected to the load, as well as the point where the tunnel can be regarded inclined in the soil. The difference between the horizontal and vertical span in this situation is indicated in figure 7.7.

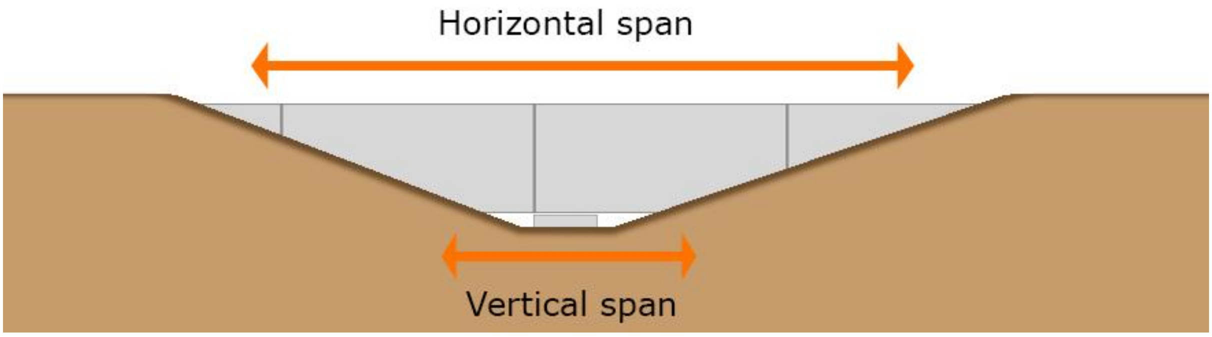

Figure 7.7: The vertical and horizontal span in case the soil around the tunnel is removed to give access to the tile.

It is assumed that the support would be at ¾ of the height of the tunnel as seen from the bottom. Consequently, the span horizontal span would be 25 - 30 m.

Looking at figure 7.5, it seems problems could occur if the depth of the tunnel is less than 5 m which would mean problems could occur near the abutments. However, tiles are only placed near the immersion joints within the waterway. The roof of the tunnel at the first immersion joint lies at a depth of about NAP - 8.5 m for both tunnels. Looking at the shear capacity of figure 7.6, no problems are to be expected.

According to the moment capacity and shear capacity of the span, it can be concluded that for the depths of the tunnel near the tiles, the possible span would be sufficient to remove the tiles. However, the influence of rotations has not been taken into account. Minor displacements are no problem as this situation is only temporary and the sections will be straightened and lowered on a new bed.

If this situation is considered too risky, one could decide to apply some local pre-tensioning on the inside of the tunnel.

Place supports or increase the span

If more soil is removed and spans would increase, supports will be required. However, it could also be chosen to apply external pre-stressing to increase the possible span. This principle has of course already been used during the immersion of the tunnels.

In theory, the possible span could be made extremely long to minimise the number of spans. However, this possibility will not be studied as there are very good reasons to maintain a certain span, namely: the length of the elements. Consequently, supports would be placed at the immersion joints.

The following reasons are proposed:

- As the tiles are to be removed, the soil around the immersion joints is removed which would provide a good starting point for supporting the tunnel.
- The immersion joints are weak and unpredictable points.
- Applying the support at the immersion joints gives the possibility of disconnecting the elements during the lowering process.
- If the length of the spans become too large, the curves in the tunnel will become more important.
- Given the fact that the tunnel elements are designed at this length, it can be assumed that it is able to withstand most structural risks.

Design of supports

The design of the supports has to fulfil a number of functions. It has to:

- Withstand the vertical load of a tunnel element including fluctuations caused by (navigation) flow.
- Allow and control lowering of the element ranging from less than a meter near the land parts to up to possibly more than 10 m near the centre.
- Resist transverse load fluctuations caused by (navigation) flow.
- Allow and control longitudinal displacements required for the lowering process.
- Resist rotation of the elements in all directions.
- Be lowered onto the river bottom, stabilise itself and be recovered when the lowering process is finished.
- If desired, allow the elements to be lowered separately.
- Not interfere navigation.

Given the requirements for strength and flexibility, a mechanical solution would be most obvious.

Figures 7.8 and 7.9 show a mechanical solution used by Strukton for the positioning of tunnel elements on the bottom of a 50 m deep seabed for the Busan-Geoje Fixed Link project in South Korea. It allowed the tunnel elements to be lowered onto the seabed relatively coarse and connect and straighten the element afterwards.

A solution for the supports could be roughly based on this. However, this solution does have slightly different functions. It is expected that the solution for the Beneluxtunnel would require more load resistance in all directions and must therefore have a more robust design. It must also be able to lower the elements so the legs must be much longer. Possibly a type of truss could function as the frame of the structure and the tunnel could hang from its corners, resembling a gantry crane.

It is expected that a solution would be possible. The actual design of this solution will not be determined within this study.

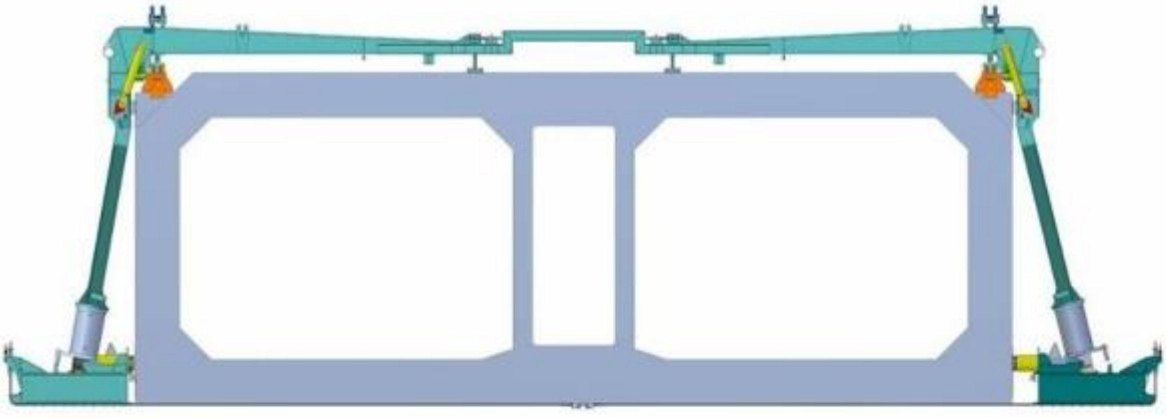

Figure 7.8: External positioning system (EPS) used by Strukton for the positioning of the elements for the Busan-Geoje fixed Link Project in Korea. It connects to the tunnel with jacks and the legs of the system are able to rotate relative to the foundation blocks, which are connected to each other by a cable underneath the element. [68]

Figure 7.9: A picture of the External positioning system (EPS) [68]

Spanning the supports by pre-tensioning

Once the supports are placed, the elements can be further excavated. According to figure 7.5, a span of 93 m for the 1st Beneluxtunnel and 140 m for the 2nd Beneluxtunnel is not possible without causing tensile forces in the concrete. It is therefore advised to apply pre-tensioning.

However, pre-tensioning is not necessarily required. To understand why, let us analyse the situation without longitudinal reinforcement. With such large spans, the situation would develop from stage 1 to stage 2 as defined in the beginning of this section. This means that some of the segment joints will start to open, but it does not necessarily result in failure of the span. As long as the reaction force of the tunnel in the longitudinal direction is sufficient, rotations in the segment joints will not become large enough to damage the W9Ui-profiles and vertical displacements will be minimal, as was shown in figure 5.15 (section 5.4.2) for the 1st Beneluxtunnel.

Whether this situation is desired is another question. This mechanism is less predictable than a situation in which joint rotations are prevented and failure would have dramatic consequences. Therefore it would be advised to apply longitudinal pre-tensioning and to remain in stage 1, as long as people are inside the tunnel.

Figure 7.10 shows the number of pre-tensioning strands that would have to be applied to assure no tensile stresses would occur, if it is assumed that the strands are applied evenly and therefore only cause compression. The type of stand is a 7-wire strand with a wire diameter of 18 mm and an area of 200 mm^2. It is assumed to be loaded at 90% of its initial stress level which is 1395 mPa. This results in a compression load per strand of about 250 kN. The two situations that are shown are the situation in which the effect of the hydrostatic pressure is reduced to 60% to represent the compression loss in time – as treated in section 5.4.1 – and the situation in which the full hydrostatic pressure is acting.

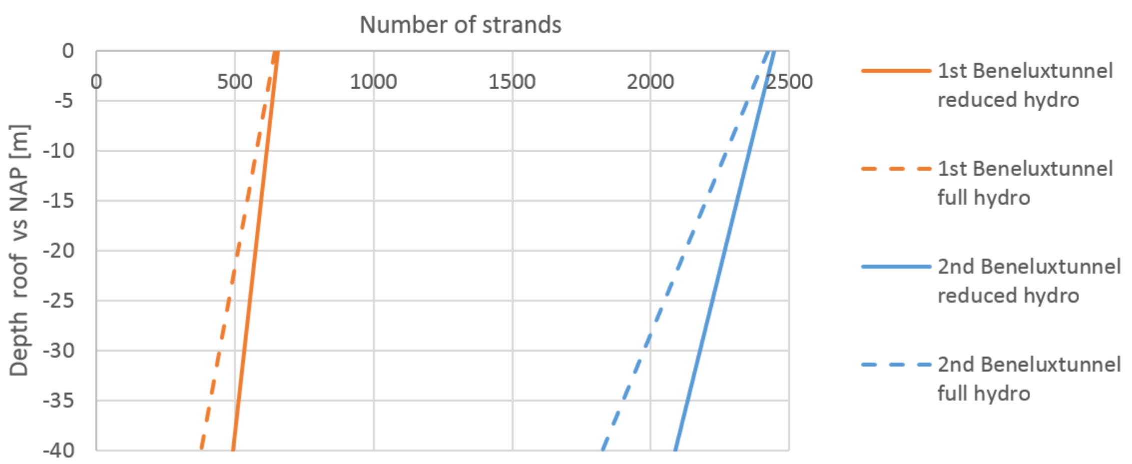

Figure 7.10: Number of strands required to assure no tensile stresses occur, for different depths. The type of stand is a 7-wire strand with a wire diameter of 18mm and an area of 200mm^2. It is assumed to be loaded at 90% of its initial stress level which is 1395 mPa.

The number of strands is relatively high for both tunnels, but much higher for the 2nd Beneluxtunnel. This difference is mainly caused by the fact that the span of the 2nd Beneluxtunnel (140 m) is about 1.5 times the span of the 1st Beneluxtunnel (93 m).

Furthermore, the loads are relatively high as no ballast material has been removed in this analysis. Removing a part of the ballast would also significantly lower the pre-tensioning requirements. Figure

7.11 shows the pre-tensioning requirements if the load is decreased to 1.5 times the difference between upward and downward navigation load, which would be the maximum possible load including safety factor that could occur if it is chosen to remain assure downward directed loads. The weight of the tunnel could be further decreased, but then measures should be taken at the supports to prevent the tunnel from being lifted by upward navigation loads.

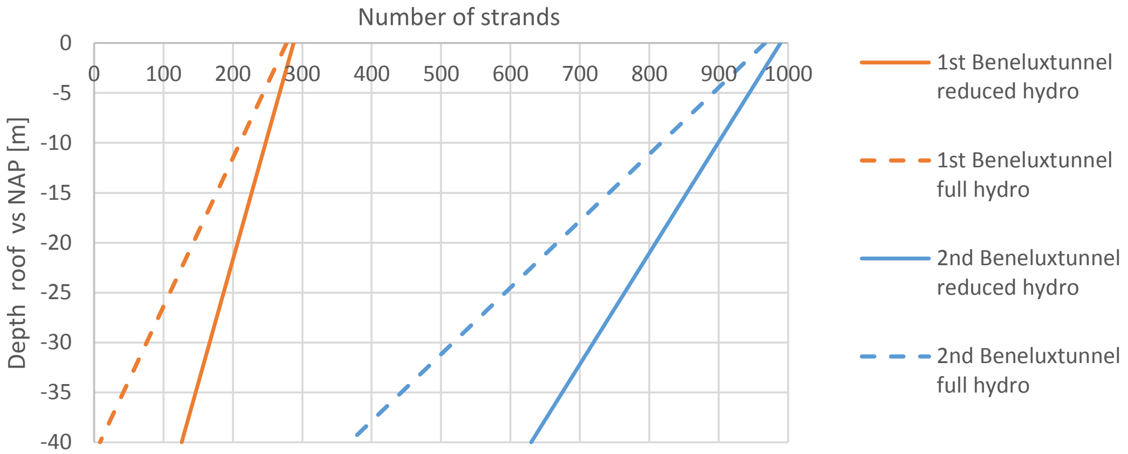

Figure 7.11: Same as 6.44, but with loads deceased to 1.5 times the difference between the maximum and the minimum navigation load. The resulting loads are 156 kN/m for the 1st Beneluxtunnel and 294 kN/m for the 2nd Beneluxtunnel.

To further reduce the number of strands, they can be applied unevenly over the height of the cross-section which would cause a moment, similar to the moment caused by the hydrostatic load. They could even be installed curved which would also cause moments.

Important with pre-tensioning is the connection with the tunnel. During the immersion process, ducts inside the slabs are used to house the pre-tensioning which allows the load to be perfectly transferred into the concrete. It is unlikely for these ducts to be reused which means the pre-tensioning will have to be applied externally. The connection points require special attention, but this will not be treated within this study. However, it will limit the amount of strands per bundle.

The amount of pre-tensioning seems to be a consideration between the costs of removing ballast and possibly adjusting supports and costs of applying external pre-tensioning. The maximum possible amount of pre-tensioning is determined by the effect of this pre-tensioning on the connection points of the concrete and by the available space within the tunnel.

One should also be aware that these pre-stressing loads are in most cases, much larger than the longitudinal loads caused by settlement (section 5.4.2). The joints that were opened as a result of settlement will therefore be pushed back together. Displacements in all directions will occur.

7.2.3 Lowering process

If the elements rest on supports near the immersion joints, external longitudinal pre-tensioning is applied to allow the entire element to be spanned between the supports without tensile stresses occurring and the soil around and underneath the element is removed until at least the desired depth, the element can be lowered.

In this section, the method of lowering will be explained.

Longitudinal displacement

Important is the occurrence of longitudinal displacements if the tunnel is to be lowered as discussed in section 6.4.5. If these displacements would be accepted, the length of the pressure-line would increase, decreasing the longitudinal pressure. As the safety of the tunnel depends in large extent on the presence of this pressure, this is not preferred. It will be required to disconnect the elements at a certain point – it does not matter where – to maintain the longitudinal pressure.

Disconnecting could be performed either by re-flooding one of the immersion joints, or by re-opening the closure joint. If the latter is not required in later stages of design, the first alternative is preferred.

Doing so, the longitudinal pressure would be provided directly by the hydrostatic pressure. This is similar to the construction phase and would increase the magnitude of the longitudinal load back to its original level and independent from time-dependent losses.

Also, if enough space is available, longitudinal displacement is no longer restricted, meaning joint rotations are now possible without causing large reaction forces in the concrete. If all elements are pre-tensioned meaning all spans are in stage 1, this will have no consequences. However, if a part would be in stage 2, there will no longer be a reaction force meaning the segments will start rotating until a new equilibrium is reached. If the trench underneath the elements is deep enough, this would result in failure of the W9Ui-profiles. This situation is displayed in figure 7.12.

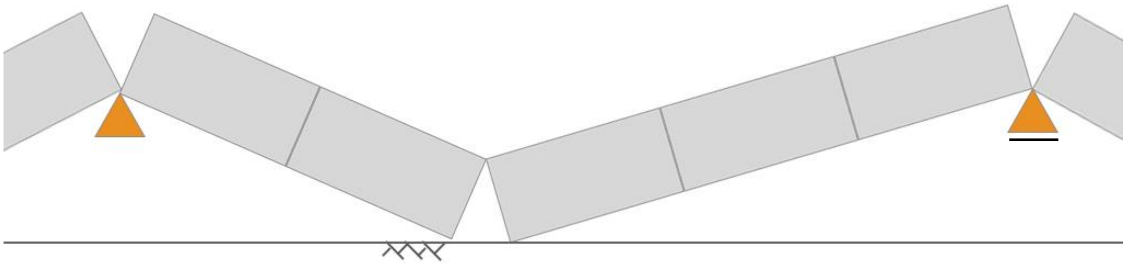

Figure 7.12: This figure indicates what will happen if spanning tunnel segments are suddenly allowed to displace longitudinally.

This mechanism stresses the importance of pre-tensioning before the supporting soil is removed and before one of the elements is disconnected. But more importantly, it also shows that longitudinal pre-tensioning prevents the segments to lower into their desired position as they are unable to rotate relative to each other.

Lowering strategy

Before continuing on this subject, a distinction must be made regarding the lowering methods. As mentioned in section 6.4.5, it is possible to cause lowering by allowing rotation in the segment joints, in the immersion joints or in both type of joints.

Rotation in the immersion joints is not prevented by longitudinal pre-tensioning. This method could therefore simply be executed by lowering the vertical position of the supports, either simultaneously or by first disconnecting the elements and reconnecting them when they are both at their desired height. Only the rotation of the segment joints is prevented by longitudinal pre-tensioning.

Disconnecting the elements has the advantage that the elements can be lowered one by one, which will require much less equipment as only a single element is lowered at a time. However, doing so, the element that is lowered can no longer be reached through the other tunnel. An alternative method of entering should be used.

In this study it will be assumed that both types of rotation are required and that the tunnel is lowered in its entirety. These decisions will be discussed in during the evaluation is section 8.2.1.

Inducing rotations in segment joints

To allow rotation in the segments without causing failure of the tunnel, a solution would have to be found. This solution would have to satisfy the following conditions:

- The offset of rotation must be controlled.
- The direction of a rotation must be controlled.
- The amount of rotation needs to be bounded.
- The system may not induce tensile stresses in the concrete.

The proposed solution is to incorporate special components in the external pre-tensioning. An example of such a component is shown in figure 7.13 before opening of the joint and in 7.14 after opening of the joint.

This solution will now be explained:

First it must be determined in which direction each joint has to rotate to cause the desired shape. Consequently, each joint will have push and a pull side. The locations of the push and pull sides for the treated scenarios of the 1st Beneluxtunnel were determined in section 6.4.5. At this stage, pre-tensioning will only be assumed present on the roof or floor of the tunnel.

- On the push side, normal pre-tensioning should be applied. The strands must be fixed and tensioned in the longitudinal direction near the immersion joints. In between, they must at some points be fixed in the vertical and transverse direction to counteract the bending of the tunnel alignment in these directions. An important additional requirement is that the strands are not fully pre-tensioned as rotation of the joint will also at the push side cause some lengthening of the wire. With a lengthening of the W9Ui-profile of 11 mm, as was used in section 6.4.5, the strands on the push side will lengthen about 2.5 mm per joint which for both tunnels result in an additional tensile stress of about 20 mPa.
- On the pull side, components as displayed in figure 7.13 and 7.14 must be applied between the strands at the location of the joints. The components must be fixed in the vertical and transverse direction to counteract the bending of the tunnel, similar to the strands on the push side. After tensioning, which is also executed at the immersion joint, the components must also be fixed in the longitudinal direction. At this point, the tunnel is fully compressed and the tunnel is at both sides of the joint connected to the pre-tensioning wire.

Next, the opening process. This part of the procedure could optionally be remotely controlled to allow complete closure the subjected element, either for safety or to allow the element to be disconnected on both sides during this process.

- To initiate the opening process, the wires that connect the two parts of the components must be cut one by one. The stress increase on the remaining wires will cause them to elongate. If this process is continued gradually, the opening of the joint will be a smooth process. This mechanism has two additional requirements. There must be enough wires that are small enough for this process to develop gradually and prevent a large percentage of the wires to be removed if a single wire cut, which would cause shocks to the system. Also the wires must be long enough to prevent rupture. During this process, the fixation to the tunnel will have caused it to follow the displacements of the components.
- At a certain point, the wires will have yielded up to the point where the bolts take over the longitudinal load. The bolts should be significantly stronger and stiffer than the wires to prevent further opening of the joints. The size of the bolts depends on the size of the longitudinal load and on the desired yielding of the bolt which consequently depends on the predictability of the load. If the load is very predictable, the yielding of the bolt can be incorporated in the required length of the bolt to produce the desired opening of the Bolt. At this point, all joints on the pull side of the tunnel will be opened and the desired vertical profile of the element will be reached.

Finally, the gap can be filled with a material with a high compressive strength which will secure the vertical profile of the element during underflowing and backfilling. Once this is finished, first the longitudinal fixation of the components should be removed and afterwards, the pre-tensioning can be lowered and removed.

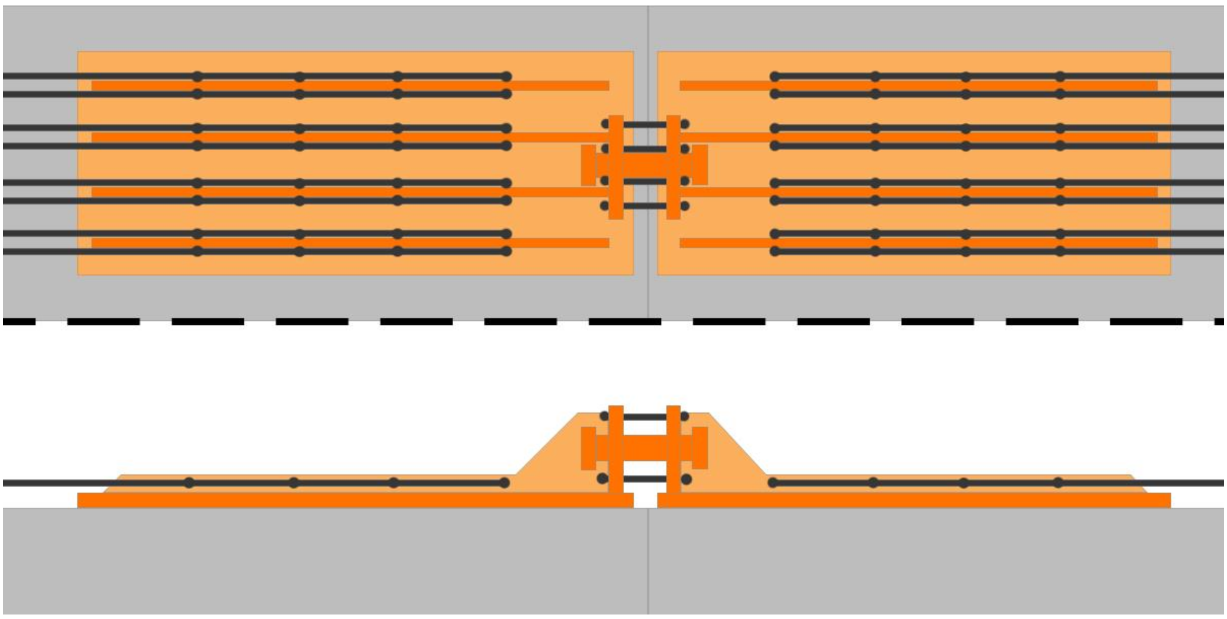

Figure 7.13: The key component of a solution which allows joint rotation to be controlled. The orange parts are made of steel and are significantly stronger and stiffer than the black pre-tensioning strands. The strands are assumed to be entirely secured to the components. (Top view above, side view below)

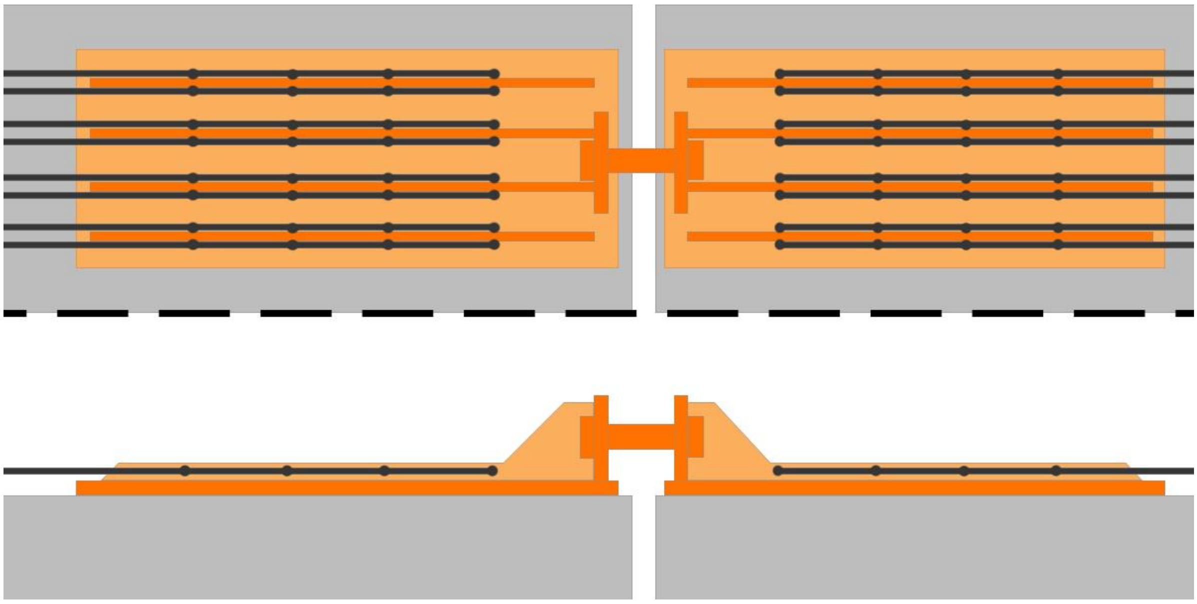

Figure 7.14: The same component as indicated in figure 7.13, but after opening of the joint. The steel bolts have taken over the longitudinal load. (Top view above, side view below)

Conclusion

Rotation is possible for both the segment joints and the immersion joints. Rotation in the immersion joints could be directly applied and controlled by the mechanic supports discussed in section 7.2.2. Rotation in the segment joint requires special components which are described in this section.

7.2.4 Construction sequences

In this section the construction sequences required to execute this alternative will be discussed.

1. **Remove bottom protection**
    - To reach the tunnel, the bottom protection will have to be removed by cranes on barges.
    - If the bottom protection is removed, the tunnel could become subjected to anchor forces. Perhaps measures will have to be taken to prevent this from happening during construction.
    - This subject is treated in section 7.2.1.
2. **Dig around tiles**
    - The soil around the tiles will have to be removed to be able to reach them.
    - This subject is treated in section 7.2.1.
3. **Remove tiles**
    - To allow lowering of the elements, the tiles must be removed.
    - This subject is treated in section 7.2.1.
4. **Longitudinal pre-tensioning**
    - To allow soil to be removed underneath the elements, the elements must be able to span this length. This will be provided by applying external longitudinal pre-tensioning.
    - To decrease the amount of pre-tensioning, ballast concrete could be removed.
    - This includes the installation of the lowering components.
    - This subject is treated in sections 7.2.2 and 7.2.3.
5. **Place supports**
    - The support structures must be installed at the immersion joints.
    - The support structures must be able to lower the tunnel to its desired height, meaning the soil around it and underneath the tunnel at its location would have to be removed if this is not yet executed during the removal of the tiles.
    - This subject is treated in section 7.2.2.
6. **Remove remaining soil**
    - To allow lowering, the soil underneath the elements must be removed.
    - This subject is treated in section 7.2.1.
7. **Disconnect one joint**
    - To allow longitudinal displacements, at least one joint has to be disconnected.
    - This subject is treated in section 7.2.3.
8. **Lower elements**
    - The most important step is the lowering of the elements. It is assumed that this is will be performed simultaneously for the entire tunnel.
    - It is also possible to do this one element at a time.
    - It is assumed that both types of joints will be rotated.
    - The rotation of the immersion joints of the 1st Beneluxtunnel is in reality not possible as they are monolithically constructed. To be able to allow rotation at these joints, the connecting concrete should be removed.
    - This subject is treated in section 7.2.3.
9. **Underflowing**
    - When the elements and their segments are secured at their desired position, their soil foundation should be returned.

10. **Remove pre-tensioning and supports**
    - When the elements are founded again, the external pre-tensioning on the inside of the tunnel can be removed.
11. **Remove supports**
    - Also supports on the outside of the tunnel can be removed.
12. **Backfilling**
    - The soil and the bottom protection around the elements should be returned.
    - During backfilling, the stability of the elements should be maintained. Consequently, soil on either sides of the element should not differ more than 1 m in height.
13. **Closure joint**
    - The longitudinal deformations caused by the lowering process will have created a gap. This gap should be closed similar to a closure joint.
14. **Finishing**
    - Finally, the interior of the tunnel should be finished

Table 7.3 shows the planning of the construction sequences required to realise this alternative. A distinction has been made between the activities that take place inside and outside the tunnel as these activities can occur simultaneously.

nr.	inside tunnel (a)	outside tunnel (b)
1		Remove bottom protection
2		dig around tiles
3		remove tiles
4-5	longitudinal pre-tensioning	place supports
6-7	Disconnect one joint	Remove remaining soil
8	lower elements	
9		underflowing
10-11	Removing pre-tensioning	Remove supports
12		Backfilling
13	Closure joint	
14	finishing	

Table 7.3: An overview of the construction sequences for immersed part alternative 1: lowering while immersed.

7.3 Immersed part 2: Re-floating elements

If the elements are to be lowered, it might be useful to utilise their floating capabilities and temporarily remove them from the site by re-floating them. Doing so, the trench can be deepened and all required modifications to the land parts can be executed relatively undisturbed.

The process of re-floatation can be regarded as the inversed immersion process.

However, disconnecting and re-floating tunnel elements has never been executed and is therefore experimental. The following aspects will therefore be discussed before proposing a construction method:

- How can the first element be removed?
- Should the elements be lifted or floated to the surface?
- How will stability within the elements be provided during this process?

These subjects will be discussed in this section. Subsequently, the main construction sequences will be given.

7.3.1 Disconnecting elements

To allow an element to be re-floated, it has to be disconnected from its neighbouring elements and it has to have certain margins in order to be safely removed from its position. In case of the design option discussed in section 7.2, this space was provided by the longitudinal displacement of the elements during the lowering process, which for this design option is not the case. Consequently, this space must be created manually.

A method of doing this is by re-opening the closure joint. This option will be discussed in this section, as well as an alternative option that does not require for the closure joint to be re-opened. Also the opening of the regular immersion joints will be discussed.

Sealing the closure joint

To re-open the closure joint, it has to be sealed just like it was during the immersion process. The initial seal is still present but to use it would be unwise as its functioning is likely to be affected resulting from time related mechanisms. Furthermore, it would complicate the demolition of the adjacent concrete slabs. It is therefore advised to provide an external seal over the initial seal.

To install an external seal, the closure joint will have to be reached. Soil on top, on the sides and below the closure joint should therefore be removed. As indicated in section 7.2.1, this can be executed by dredging.

Consequently, a gap would be created underneath the closure joint. To safely install an external seal, a depth of about 2 m underneath the existing seal would be required. Given the width of the current seal of about 2 m (for both tunnels) and applied slopes of 1:3, a gap of 14 m would be required. In figure 7.5 is indicated that for these circumstances such a gap would not result in tensile stresses in the concrete and could therefore be applied without having to take additional measures.

Concrete removal techniques

Once the seal is placed, the existing lining of the closure joint can be removed. To accomplish this, let us first briefly discuss the available concrete removal techniques. A quick glance through the available solutions learns that a general distinction can be made between demolition and cutting:

Demolition Demolition methods rely on penetrating into the concrete and applying high energy impact loads on the concrete intended to provoke tensile splitting in the adjacent sections. Examples are drilling or blasting.

These methods are very useful for removing large quantities of concrete. They are however not very precise and should therefore not be used too close to parts of the tunnel that should remain unaffected. Also, reinforcement has to be removed separately.

Cutting Cutting methods rely essentially on the same principle, but then on molecular level. Examples are sawing, water jetting or wire cutting. All of these methods can be used to cut concrete slabs of over 1m thick.

These methods are useful to cut through sections of concrete without affecting adjacent sections. Also reinforcement can be cut. However, to remove embedded sections of concrete this method cannot be used.

Combined A method that uses combines both techniques is hydro-demolition. It achieves the same result as the demolition techniques, but uses the method of the water jet. Consequently, adjacent sections of concrete will not be damaged.

Looking at these concrete removal techniques, a wide range of possibilities to remove the tunnel lining is possible. The choice of method depends on the risk of damaging other sections and the costs of doing so. [69-72]

Opening the joint

While opening the joint, one should keep in mind the mechanics of the joint both before and after it is inundated

- Before the joint is inundated it should remain able to transfer the longitudinal compression force and, if present, shear forces.
- After the joint is inundated the acting forces may not damage the adjacent tunnel sections.

As indicated in section 5.4.1, the longitudinal load is expected to have decreased in time resulting from time dependent processes in the concrete. This would imply that if the closure joint is inundated and the hydraulic pressure would yet again reach the same value as outside, the lining within the joint would be in tension.

An interesting solution would be to cut all reinforcement within the lining and as much as the concrete as required for it to be able to withstand compression and shear forces but to fail at tension. However, at least about 98% of the surface area should be cut to accomplish this effect, which is hard to achieve with certainty.

Another method would be to make two cuts – for instance by wire cutting – in a slight angle meaning they would be slightly further apart at the top. This section could easily be lifted out when the joint is inundated and opened. However, the effects of the longitudinal and shear forces on the cutting process and on the cut itself is possibly problematic.

Perhaps it is easier and safer to manually remove the concrete from inside the tunnel and replace them with struts. Also shear struts will have to be applied. If the joint is immersed, the struts will become loose and can be removed.

Alternative solution

If all elements should be removed, sooner or later the closure joint will have to be opened. It therefore makes sense to start with it. However, if for instance only the land part opposite to the closure joint would be adapted, the closure joint doesn't necessarily have to be removed. In this case it might be beneficial to use another method to break the chain.

An interesting alternative solution would be to remove the GINA-profiles. First the immersion joints on both sides of the element should be prepared for inundation, meaning the concrete that has later been applied around the joints will have to be removed, bulkheads should be applied and the OMEGA-profile should be removed. When the joint is inundated, divers can then manually cut and remove the GINA-profiles on both sides of the element. Subsequently, there would be a gap of about 10 cm on both sides. This could be sufficient to carefully remove the element, but perhaps a type of guidance rails could be used to prevent the elements from damaging each other.

However, this option is only possible if the shear keys are removed from within the tunnel.

Opening immersion joints

To allow for the tunnel elements to be disconnected, the concrete lining in the immersion joints would have to be removed. This is especially challenging for the 1st Beneluxtunnel in which the immersion joints were constructed monolithically, as was mentioned in section 2.4.3. For both tunnels, the lining functions as shear keys.

The concrete lining can be removed using similar deconstruction techniques as discussed for the closure joint. Using an automated process from the inside of the tunnel, it is possible to execute this while the bulkheads are in place which would allow for minor or major leakages to occur without personal safety risks. If the automated process fails, it should be finished by divers from outside of the tunnel.

Also temporary shear struts should be applied to prevent sudden shear displacements.

Conclusion

In this study, the method of opening the closure joint by internally removing concrete sections and applying struts will be used.

7.3.2 Lifting or floating

If an element is disconnected from its adjacent elements, it is ready to be transported to the water level. To do so, generally three options could be distinguished.

- The elements could be floated which would require the removal of ballast.
- The elements could be lifted instead of floated without removing the ballast.
- The elements could be lifted with removing the ballast.

Re-floating

The most obvious choice would be to apply the inverse method of immersion. The weight of the element would have to be decreased for it to regain its floating capabilities which would mean the ballast concrete would have to be removed. To remain immersed during the process, water tanks can be installed.

However, this method has some disadvantages which might make it less suitable for this purpose:

- The ballast concrete would need to be removed without damaging the tunnel. This could possibly be a time consuming task as rough methods cannot be used.
- Once the element is ready to float, it should be kept stable. A system of cables usually provides this during the immersion process, but as the floating process is inversed, these cables would have to be attached to the bottom of the waterway. This would require anchors on the bottom of the river for cables to be pulled through.

Lifting without removing ballast

Without removing the ballast, the immersed weight of an element of the 1st Beneluxtunnel would be about 31 mN and an element of the 2nd Beneluxtunnel would be about 99 mN. If the elements were to be lifted without removing ballast, the lifting process would be very demanding on all aspects involved.

To determine the effects on the strength of the elements, the forces have been estimated. For the 1st Beneluxtunnel being lifted at four points placed at 1/4L and 3/4L, this would imply:

- A longitudinal moment would be present of about 60 mNm which would cause tensile stresses in the roof at the supports of 0.38 mPa.
- A shear force in the outer joints of about 6 mN would act which is larger than the shear resistance of the joints if no frictional shear capacity would be present.

To counteract resist these forces, the elements should be pre-tensioned. The required amount of pre-tensioning is comparable to the estimates given in figure 7.10, with a depth of 0 m for the roof.

Another disadvantage is the demands of the large weight on the pontoons and hoisting equipment required to lift the element to the surface.

Lifting with removing ballast

An intermediate solution is to lift the elements, but first remove enough of the ballast for the weight of the elements to be minimal. Consequently, the advantage of control during the lifting process can be utilised, while the demands on pre-tensioning and hoisting equipment remains low.

To accomplish this, the ballast needs to be removed. Demolition techniques should be used that do not damage the structural concrete of the element. If conventional demolition techniques turn out to cause too much vibrations, possibly the method of hydro-demolition treated in section 7.3.1 could be used.

The purpose of this method is to minimise the weight, but if the weight is decreased by too much, the elements will start to float which should be prevented. The loads caused by passing navigation can also result in an upward directed load resultant. To prevent this, the weight should be kept sufficiently large during the construction process. One could maintain a safe margin of ballast concrete but this will increase the demands on pre-tensioning and hoisting equipment. A better solution is to use similar water tanks as were used during the immersion process to provide the required ballast during the construction stages in which the element could be subjected to navigation forces.

Conclusion

Lifting with removing ballast is expected to be the most advantageous solution.

7.3.3 Construction sequences

In this section the construction sequences required to execute this alternative will be discussed.

1. **Remove bottom protection**
    - To reach the tunnel, the bottom protection will have to be removed by cranes on barges.
    - If the bottom protection is removed, the tunnel could become subjected to anchor forces. Perhaps measures will have to be taken to prevent this from happening during construction.
    - This subject is treated in section 7.2.1.
2. **Remove the soil around closure joints**
    - The soil around the closure joints will have to be removed to be able to reach them.
    - Also underneath the closure joint, the soil must be removed.
    - This subject is treated in section 7.2.1.
3. **Open the closure joint**
    - The closure joint must be opened to allow the first elements to be removed.
    - This subject is treated in section 7.3.1.
4. **Remove the soil around the element**
    - To be able to re-float the elements, the soil on top and on the sides should be removed.
    - Apart from the soil around the elements, also some strips of soil underneath the elements must be must be made to prevent the elements from being sucked to the bottom.
    - This subject is treated in sections 7.2.1
5. **Prepare element for re-floating**
    - To prepare the elements for re-floating, certain aspects would have to be prepared.
    - Ballast concrete has to be removed.
    - Water tanks have to be installed.
    - Pre-tensioning has to be applied.
    - Bulkheads have to be installed.
    - The later installed shear keys have to be removed as is discussed in section 7.3.1.
    - The OMEGA-profiles have to be removed.

6. **Re-float elements**
    - When an element is prepared and the surrounding soil is removed, it can be re-floated. This will be performed by lifting rather than floating as this allows better handling of the elements.
    - This operations should be performed at still water conditions.
    - This subject is treated in sections 7.3.2
7. **Transport elements to temporary location**
    - The re-floated elements have to be transported to a temporary location.
    - Possibly the Madroelhaven could be used as a temporary location. It is situated next to the southern land part and was also used during the construction of the 1st Beneluxtunnel. Otherwise, building dock Barendrecht could be used which was used for the 2nd Beneluxtunnel.
8. **Renovate elements**
    - Whether the elements can be renovated depends on the decision whether to remove them from the water or not. This is quite costly and therefore not ideal. However, if renovating the elements would significantly increase their lifetime, it is probably wise to make this investment.
    - If the elements are required to have a changed profile, this should also be executed during this phase. This could be achieved by lowering the elements on a bed with the desired shape and gradually lowering the pre-tensioning force. To allow for the element to obtain the same shape.
    - It is also possible to construct new elements, but this is expected to be much more expensive.
9. **Prepare bed**
    - While the elements are renovated, the bed must be prepared at the desired depth and also the construction works on the land part must be finished.
    - Also the tiles must be removed.
10. **Transport back to site**
    - When the bed and the elements are ready, the elements must be transported back to site.
11. **Re-immerse**
    - When they arrive, the elements can be immersed. This is executed like the normal immersion process described in section 2.2.2.
    - Given the experiences with the 2nd Beneluxtunnel, it would be advised to lower the elements on gravel.
12. **Backfilling**
    - When the elements are immersed, the surrounding soil and the bottom protection can be returned.
    - During backfilling, the stability of the elements should be maintained. Consequently, soil on either sides of the element should not differ more than 1 m in height.
13. **Closure joint**
    - To finish the immersion process, the closure joint must be closed.
14. **Finishing**
    - Finally, the interior of the tunnel should be finished. This includes returning the ballast concrete and the road.

7.4 Land part 1: Local adjustments

The objective of this design option is to change the boundary conditions of the immersed part by locally adjusting the transition point. The main advantage of this method is that with minimal constructional effort, much impact on the course of the vertical profile can be achieved. However, this method is only useful if the traffic requirements are adjusted as is explained in section 4.2.1. Otherwise, an increased slope would not be allowed.

The following difficulties are expected regarding this method:

- What space is available which would allow the adjustment of the transition point?
- How can the transition point be reached and reconstructed if it also has to function as an immersion joint?

These subjects will be discussed in this section. Subsequently, the main construction sequences will be given.

7.4.1 Geometric constraints

For this method, the existing abutments as displayed in figure 2.17, will be adapted. As there will be no adjustments required in the transverse direction, only the vertical and longitudinal direction will be treated. Taking the traffic requirements and the local geometry of the construction into account, the limitations of this method can be determined. Doing so, an upper and a lower boundary can be distinguished.

Figure 7.15 shows the situation for the abutment of the 2nd Beneluxtunnel. The orange and blue lines give the different slopes ranging from 4.5% to 7%, similar to section 4.4. Interesting points and areas are indicated in red.

It will be assumed that the 1st Beneluxtunnel gives similar conclusions.

Abutment

The part of the trajectory that lies inside the abutment is the transitional part.

On the top side of the transitional part, the available space is bounded by a roof, which is also the bottom of the service building. Theoretically, if the slope is increased, the available height over the distance of the transitional part will decrease and thus either a lowered alignment or adjustments the roof will be required. However, given the small differences in slope, the extra vertical distance required is limited to about 0.5 m.

On the bottom side of the transitional part, the space that is being used by the dewatering cellar can be utilised. The real limitation is the underwater concrete floor, as adapting this would result in an inflow of water. An extra margin of 1m above the underwater concrete floor is taken to allow for a road to be constructed. Thus, allowing a lowering of about 2.5 m as is shown in figure 7.15

Access ramp

The effects of such a lowering would not only act at the abutment, but also further along the access ramp. If the road profile would be constructed at the minimum height, a large stretch of road will need to be reconstructed. This effect will be larger if the slope is smaller.

However, if the desired slope is larger than the original slope, it will soon enough reach above the original level and will require a stretch of road to be constructed above the original road deck.

Either way, parts of the road will have to be reconstructed. A larger slope gets up to the original level sooner and can therefore be constructed slightly lower. However, a lowering as large as 2.5 m is expected to cause problems with the thickness of the road.

Conclusion

The following values are based on some exploratory calculations and will be used in this study:

- 0.5 m lowering for a slope of 5%.
- 1.0 m lowering for a slope of 6%.
- 1.5 m lowering for a slope of 7%.

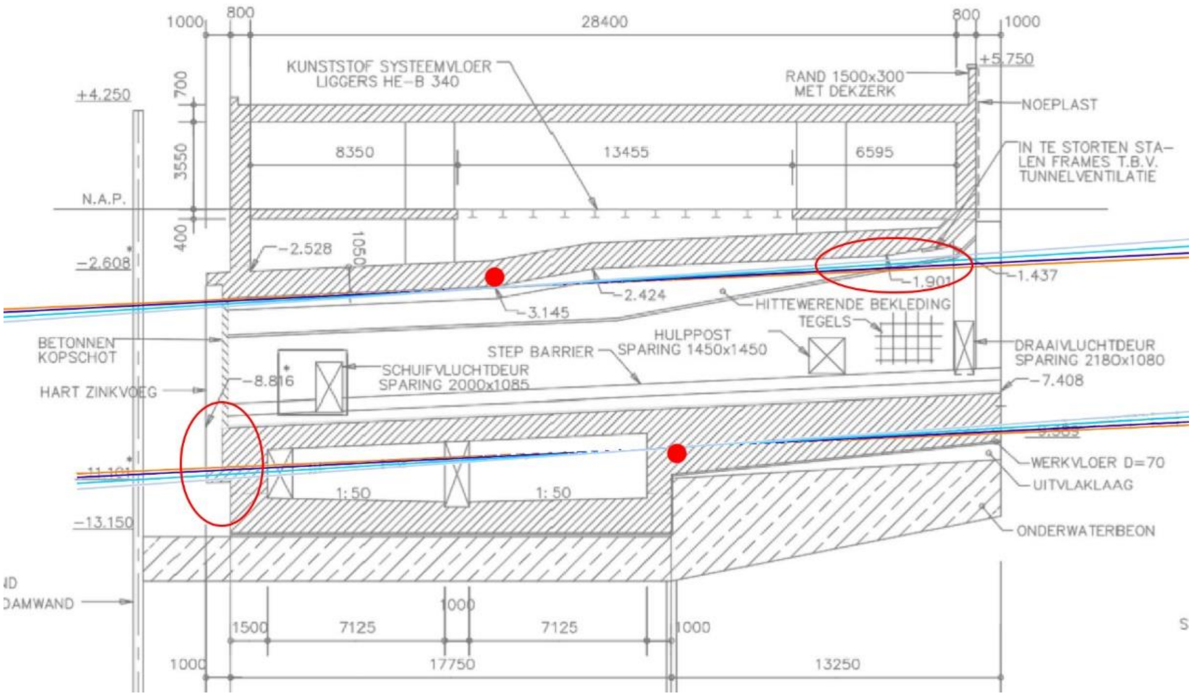

Figure 7.15: Local influence of increased slopes on abutment

7.4.2 Reaching the transition point

To be able to rebuild the transition point, one should be able to reach it. However, as it also functions as an immersion joint. It allows the axial force of the elements to be transferred to the abutment and ensures water tightness of the connection. Any adjustment to this point would compromise these functions.

To overcome this problem, basically two types of measures are possible. One could either remove the element or one could try to build a new transition point within the abutment without removing the element. Both options will be discussed.

Removing the element

To remove the elements, first bulkheads should be placed on both sides of the transition point, after which the element can be disconnected and removed. Where it goes and how this is performed depends on the decisions regarding the immersed part.

Subsequently, a watertight space should be provided around the initial transition point which should be large enough to allow the reconstruction of the transition point at the desired height and angle. This space could be provided by inserting sheet piles at the minimum distance on the riverside of the original sheet piling and injecting the soil in between. Consequently, the original underwater concrete floor could be reused. The sheet piling could on the upper side be supported against the top side of the initial transition point, which does not necessarily has to be removed.

Another method would be to lower a prefabricated screen which would connect to the abutment using rubber profiles.

Without removing the element

If the element cannot be removed, which could be the case in certain design options regarding the immersed part, the new transition point could still be constructed. Important to allow this is the extra space provided by the inevitable longitudinal displacement treated in section 6.4.5. This 20 to 50 cm displacement allows the elements to be disconnected and repositioned, without being removed.

Two strategies remain:

- The new transition point could be constructed within the abutment. The extra space provided by the dewatering cellar should be sufficient to allow this. Water tightness will be provided by the presence of the tunnel and the old transition point. Measures should be taken to allow transfer of the axial force during this reconstruction. Finally, the element can be disconnected, the last parts of concrete will have to be removed by divers and the element could be connected to the new transition point as an immersion joint.
- The new transition point could be constructed similar to the closure joint. The element should first be disconnected and placed into its new position. Wedges should be placed to transfer the axial force to the abutment. Screens should now be placed around the element to create a watertight space, large enough to reconstruct the transition point.

Conclusion

The choice of method depends on the boundary conditions provided by the immersed part. For this study, the option where the element is removed and water tightness is provided by sheet piling will be used.

7.4.3 Construction sequences

1. **Remove soil**
    - To gain access to the transition point, the surrounding soil has to be removed.
    - This subject is treated in section 7.2.1.
2. **Separate immersed part from land part**
    - To allow access to the transition point, the element has to be disconnected and removed.
    - This subject is treated in section 7.4.2.
3. **Create watertight screen**
    - To be able to rebuild the transition point, a watertight area has to be created.
    - This subject is treated in section 7.4.2.
4. **Demolish concrete sections**
    - All that slabs obstructing the construction of a new transition point should be removed. This includes mostly sections of the roof of the dewatering cellar.
    - It should be checked whether the stability of the service building is not compromised.
    - This also includes demolishing the parts of the road that will have to be replaced.
    - Concrete removal is treated in section 7.3.1.
5. **Rebuild transition point**
    - When all obstructing concrete is removed, the new transition point can be constructed.
    - Effort should be made for this transition point to be able to transfer the axial force of the elements towards the land part. This probably is the most challenging at the top side where the force has to be transferred into the roof.
    - This also includes reconstructing the road.
6. **Create new dewatering buffer**
    - As the dewatering cellars are used for adapting the transition points, the buffer function of these cellars should be taken over by a new mechanism.
    - Possibly there is some excess space in the new situation, otherwise, a pumping system with sufficient capacity is possible.
7. **Remove watertight screen**
    - When the new transition point is finished, it should be fitted with bulkheads.
    - Subsequently, the construction dock can be filled with water and the sheet piling can be cut by divers and removed.
8. **Reconnect immersed part with land part**
    - The element can be reconnected as a regular immersion joint.
    - This includes underflowing of soil underneath the elements.
9. **Return soil**
    - Finally, the soil around the elements should be returned.

7.5 Land part 2: General lowering

If the adjusting the transition point does not provide sufficient lowering, one could choose to use a more severe approach. The objective of this design option is to allow significant lowering of the transition point. Consequently, general lowering is referred to all methods that would affect the presence or position of the underwater concrete floor.

The following difficulties are expected regarding this method:

- What method can be used to allow general lowering of the land part?

These difficulties will be discussed in this section. Subsequently, the main construction sequences will be given.

7.5.1 Choosing a lowering mechanism

As general lowering an extensive execution, various methods are possible. This sections discusses a number of design options that follow from a first exploration of the subject. These options are evaluated qualitatively, aiming to provide a founded advice concerning the execution methods.

In figure 7.16 the design options regarding the execution methods for general lowering of the land part are indicated. The initial situation of the land part is indicated in figure 7.17.

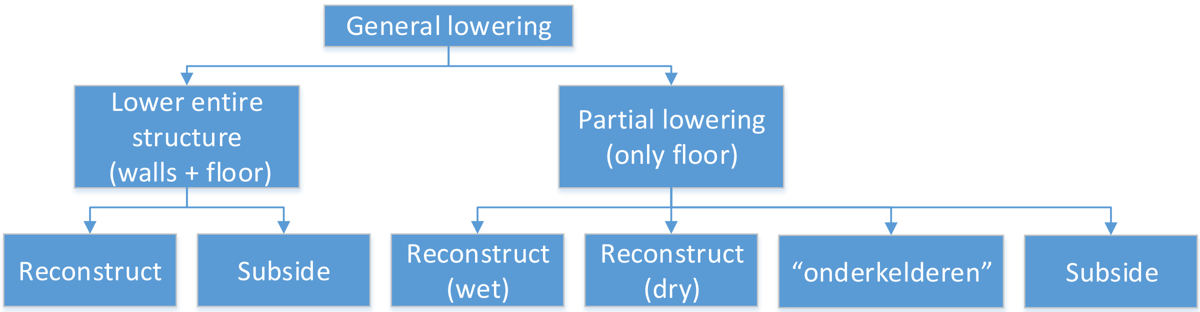

Figure 7.16: Design options regarding the execution methods for general lowering of the land part

To help distinguish execution methods, it is chosen to divide the methods based upon their approach regarding the structure:

- Two methods aim to lower the structure entirely which in practice means affecting both the walls and the floor of the structure, which is indicated in figure 7.18.
- Four other methods aim to lower only a part of the structure which in practice means affecting only the floor of the structure, which is indicated in figure 7.19.

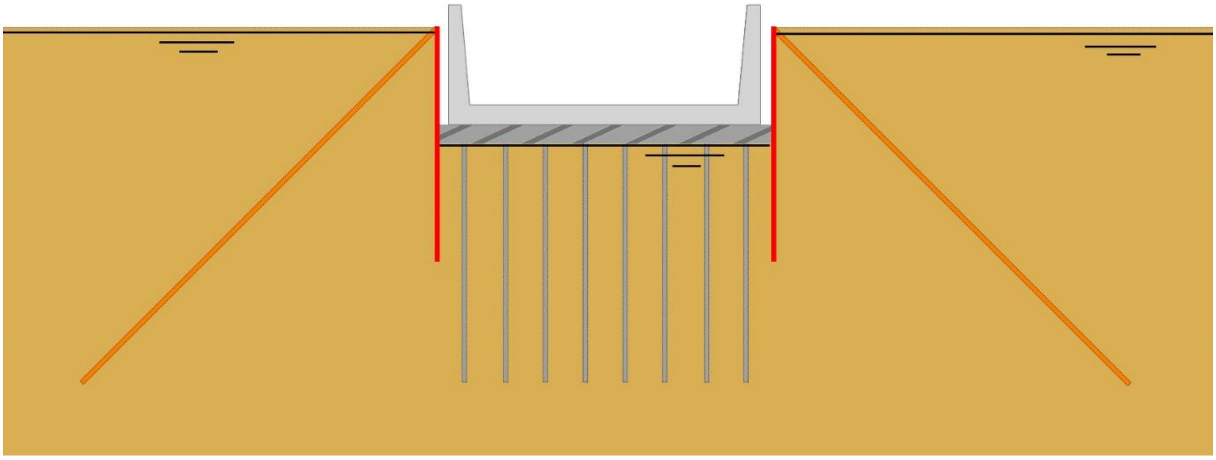

Figure 7.17: The initial situation of the land part.

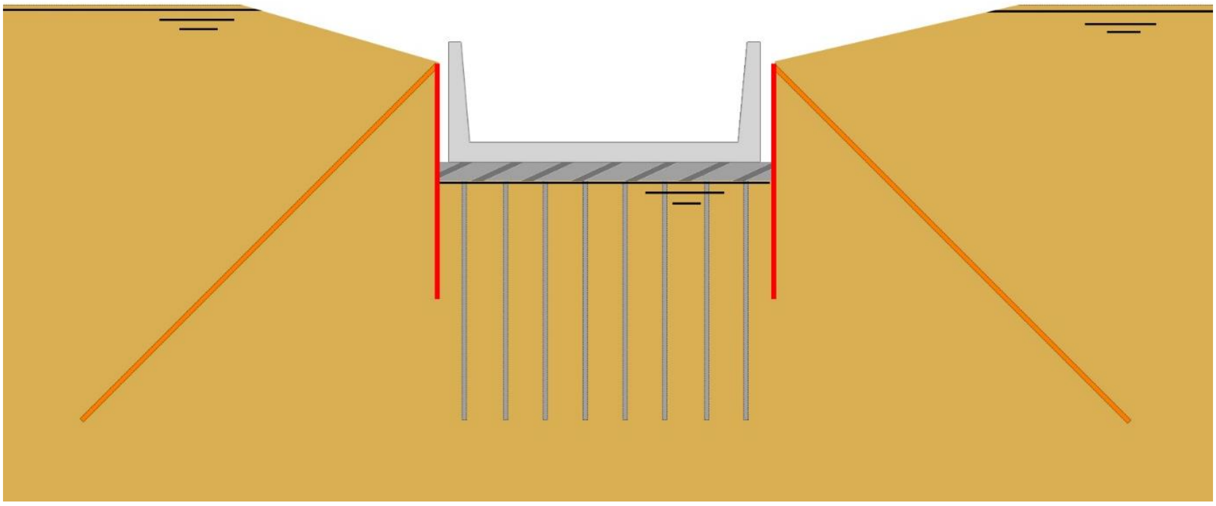

Figure 7.18: The lowering of the entire land part.

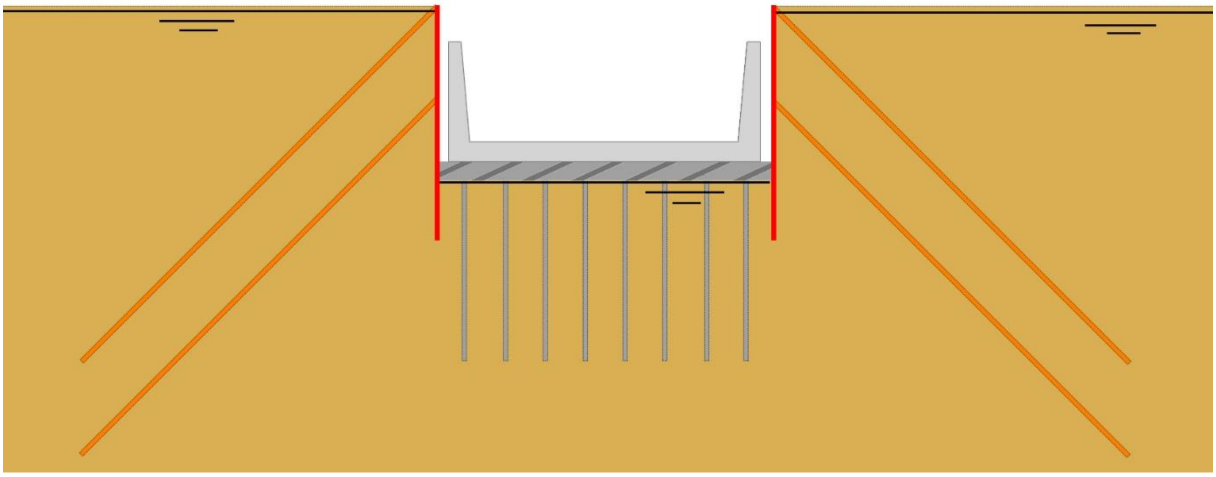

Figure 7.19: Lowering of only the floor of the land part.

Lower entire structure

It would be ideal if the entire structure could be lowered to the desired height. A possible option to do so would be to remove and rebuild the entire structure. However, it seems illogical and very cost-ineffective not to use any of the existing features of the current land parts. This method is therefore regarded not economically feasible unless reusing features of the current land parts seems to be impossible.

The other method is to subside the entire structure. The term subsiding is used to indicate that this method is adjusting only the vertical position of the structure without adjusting the layout of the structure itself. Such lowering is triggered if the vertical stability is out of equilibrium in favour of the downward directed forces, which can be achieved by increasing the downwards directed forces, lowering the bearing capacity of the soil or a combination of both.

If executable, this method would be very beneficial as no major construction has to take place – except possibly some measures to provide a new vertical equilibrium in a lowered position – saving time and money. But the technical feasibility of this method is questionable due to the following reasons:

- The tension piles, the anchorage of the walls as well as the friction between the wall and the soil, make the structure very intertwined with the surrounding soil, meaning subsidence would have to be triggered by altering the subsoil at a level deep enough to influence the entire area.
- If subsidence is triggered by altering deeper layers of subsoil, this will most likely not only influence the tunnel, but also the surrounding environment, resulting unacceptable settlements.
- Executing subsidence deliberately is very non-conventional as it is very hard to perform it evenly. Uneven settlements result in very large stresses moments within the structure.
- New water retaining measures will have to be taken to prevent water from flowing over the walls.

Subsiding large embedded structures in an urban environment does not seem technically feasible.

Lower only the floor

If the floor is separated from the walls and only the floor is lowered, the functionality of the walls and the anchors remains and the surrounding soil will not be affected.

However, lowering the floors will affect the horizontal stability of the walls. It is expected that the walls are not designed to have much excess capacity meaning measures are to be taken. The proposed solution for all four execution methods is to provide a second row of anchors as is indicated in figure 7.19.

A similar situation exists concerning the vertical stability, as both the upwards directed force will increase because of the larger depth and the capacity if the piles may decrease. Vertical stability could again be obtained by applying extra weight.

The four execution options will now be briefly discussed.

Wet reconstruction Wet reconstruction is perhaps the most standard method. It is based on the same principle as the construction, meaning the ramp will be flooded to prevent problems with water tightening and hydraulic loads. Consequently, the existing underwater concrete floor should be demolished from floating equipment and a new underwater concrete floor can be constructed on a lower level.

The major downside of this method is that it requires much time in which the access ramp cannot be used and that the underwater demolition might damage the tension piles.

Dry reconstruction With dry reconstruction, another method of preventing up bursting of water should be applied. A possible method is to inject grout or chemicals below the desired depth. Consequently, the underwater concrete floor can be demolished with much more precision that during wet demolition, with much less chance of damaging the piles. The soil underneath can easily be removed and a new floor does not require underwater concrete.

However, whether it is feasible to create a watertight layer by injecting in the soil, through the floor, between the walls and around the piles is questionable. Also this layer should be thick and heavy enough to reassure vertical stability.

Caisson method Another interesting method is to undermine the ramps using the caisson method. A similar method would be used as treated in section 7.2.1. A shaft is built outside of the ramp which gives access to the area below the existing floor. This area should be kept under high pressure to decrease the water level. Soil can subsequently be manually removed and a large part of the new floor can be construction, while the road above remains functional.

However, if something is possible does not mean it will be regarded safe by authorities. It is expected that it will be very hard to get permission for such an experimental solution.

Subside An even more experimental solution would be to try to lower the floor of an entire segment of the ramp in its entirety. To do so, first a screen should be provided similar to dry reconstruction. Next, the floor should be cut around the piles. A similar method can then be applied as discussed in section 7.2.1 for the immersed part. Water should be pumped in to liquefy the soil below, after which it is pumped out. The piles could be used to stabilise the floor.

The advantage of this method is that it is the only method that does not require for the entire access ramp, including the inside walls and abutment. However, as it is very experimental it is not regarded feasible.

Conclusions

The following conclusions are made:

- Partial lowering seems to be both much more feasible and cost effective and is therefore preferred.
- Wet reconstruction is chosen as the favourable method as the risks of the other methods are simply too high to regard them feasible. In later stages of design however, the options dry reconstruction and the caisson method could be reconsidered if the method of wet reconstruction turns out to be very risky or expensive. The method subside is regarded infeasible.

7.5.2 Construction sequences

In this section the construction sequences required to execute this alternative will be discussed.

1. **Insert new anchors**
    - First, a new row of anchors should be applied to assure the stability of the walls in the new situation.
    - This subject is treated in section 7.5.1.
2. **Disconnect element**
    - To allow construction on the land part, the adjacent element should be removed.
    - This subject is treated in sections 7.3.1 and 7.3.2.
3. **Insert sheet pile wall**
    - On the riverside of the abutment, a sheet pile wall has to be constructed to protect the area from waves and to later function as a water retaining wall.
    - These walls have to be strutted against the side walls.
    - The area between the wall and the abutment could be dewatered which would allow for the entire abutment to be deconstructed in dry conditions but this would require a temporary watertight layer. It is chosen not to apply this strategy.
4. **Demolish all possible components**
    - As deconstruction is easier in dry conditions, the top layers of the access ramp can be removed, as well as all other components that are not required for the stability of the ramp, the walls and the abutment.
5. **Inundate**
    - To allow the last components to be demolished, the land part has to be inundated.
6. **Demolish remaining components**
    - The underwater concrete floor and the abutment can now be removed.
    - The piles and walls of the abutment may not be damaged.
7. **Dig until the desired depth**
    - To allow the new ramp to be constructed at the desired depth, the top layers of soil have to be removed.
    - The soil has to be removed around the piles.

8. **Underwater concrete**
    - When the desired depth is reached, a layer of underwater concrete can be poured to create a watertight floor. The layer must be thick enough to counteract the loss of tensile capacity of the piles.
    - Possibly, the pile heads must be adapted to allow good attachment to the underwater concrete.
9. **Dewatering**
    - When the floor is ready, the water can be pumped out of the building pit.
10. **Rebuilding**
    - When the building pit is empty, the new ramp and abutment can be constructed.
11. **Cut sheet piles on waterside**
    - When the abutment is finished and fitted with bulkheads, the space between the abutment and the waterside retaining wall can be filled with water.
    - Consequently, divers must cut the sheep piles at a level significantly below the desired height of the element.
12. **Reconnect element**
    - When the abutment is finished and connected to the waterway, the adjacent element can be returned.
13. **Finishing**

7.6 Summary

In this chapter, the constructability of the design options is determined. For two design options regarding the immersed part and for two design options regarding the land part, important considerations have been elaborated and a series of construction sequences has been proposed.

Immersed part 1: Keeping elements immersed

To lower the immersed part while it remains immersed, the soil must be removed below the elements by special dredging equipment. Also the tiles must be removed. The large longitudinal pressure is expected to allow this without supports. However, for the lowering process, applying pre-tensioning and temporary supports is required. Special components connected to the pre-tensioning should allow to control the lowering process. Finally, the soil can be returned and the tunnel can be finished.

Immersed part 2: Re-floating elements

To re-float the elements the closure joint must be re-opened. Next, the weight of the elements must be reduced by removing the ballast. Instead of re-floating it is chosen to lift the elements which allows better control of the process. The elements can now be transported to a location where they can be stored and adapted. When the bed is lowered and the land parts are finished, the elements can be re-immersed.

Land part 1: Local adjustments

To locally adjust the land part, access is required which requires for the connecting element to be removed and for a watertight screen to be placed. Next, the old transition point can be demolished and the new transition point can be constructed within the existing abutment, utilizing the space provided by the dewatering cellar.

Land part 2: General lowering

General lowering can be achieved by wet reconstruction. First additional anchors have to be inserted in the walls. Next the land part can be flooded and the entire abutment and underwater concrete floor must be demolished. A new floor should be constructed at a few meters below, but first additional anchors must be inserted into the soil to provide vertical stability. Finally, water could be pumped out and everything could be reconstructed.

Chapter 8

Evaluation

Figure 8.1: This figure shows the northern land part of both Beneluxtunnel, just after the 2nd Beneluxtunnel was finished.

8.1 Introduction

In this chapter, the design options discussed throughout this report will be evaluated according to the strategy treated in section 3.3.3.

Cost factors	The costs of the design options will be estimated, which includes the construction costs of the land and the immersed part, but also the economic consequences of traffic disturbance. (Section 8.2)
Cost analysis	Next, the costs will be analysed. The costs will be compared and the influence of the cost factors will be determined. Also the influence of depth limitations will be determined. Consequently, the most cost effective solutions for different depth ranges will be given. (Section 8.3)

The applied cost factors and some calculations can be found in appendix H.

8.2 Cost factors

In this section, the following subjects will be discussed.

- Costs of a new tunnel.
- Economic consequences of traffic disturbance.
- Construction costs.

8.2.1 Costs of a new tunnel

As discussed in section 3.2.1, it is possible to build a new immersed or bored tunnel and remove the old one. The cost of this alternative will be compared to the costs of adapting the tunnel.

Method

To estimate the costs of a new tunnel, the costs of existing tunnels have been analysed. Unfortunately, the costs of existing infrastructure is not easily found and is expressed in many different ways, taking certain aspects into account or not, depending whether the source wants the costs to seem high or low.

However, for 4 immersed tunnels and 3 bored tunnels, the costs have been found. As in some cases, only the costs of tunnel part is given and in other cases the costs of the entire project, they have to be made equal. Luckily, for the 2nd Beneluxtunnel, both the costs of the tunnel part and the costs of the immersed part were available. The costs of the tunnel part were about 63% of the total costs of the project, which will also be used for the other cases.

Using this information, the costs per meter per traffic lane for a new tunnel is estimated. A new tunnel is assumed to have 11.5 lanes, which is equal to the current capacity of the 1st and the 2nd Beneluxtunnel combined.

Results

The results are:

- Regarding the immersed tunnel, the costs would be € 41,000 /m /lane, giving a total of € 630 million.
- Regarding the bored tunnel, the costs would be € 60,000 /m /lane, giving a total of € 915 million.

Apart from the costs of building a new tunnel, also the costs of demolishing the old tunnels should be taken into account, which is roughly estimated at about € 100 million.

8.2.2 Societal costs of traffic disturbance

In this section, the societal costs of traffic disturbance will be determined. Calculations regarding these estimates can be found in appendix J. Similar to the construction costs, the costs estimates presented in this section are assumed to have a margin of error of about plus or minus 50%.

Temporary traffic disturbance

Temporary traffic disturbance is caused by construction on the tunnels. Given the vitality of the connection, it is important to incorporate this in the evaluation process. Consequently, design options that can be executed quickly have an advantage regarding traffic disturbance costs.

To estimate these costs, three alternative routes are proposed that can be used to reach the other side of the waterway. The extra time it takes for traffic to use these alternative routes is used to approximate societal costs of traffic disturbance. The available routes are:

- The Benelux trajectory (5.4 km)
- The Blankenburg trajectory (20.6 km)
- The Maastunnel trajectory (17.5 km)
- The Van Brienenoordbridge trajectory (34.4 km)

Certain factors are applied to incorporate additional effects. The other traffic functions are estimated at another 10% of the total costs for road traffic. The situation when only one of the tunnels is opened is estimated at 40% of the total costs.

Consequently, the societal costs of closure of the tunnel is estimated at 2.3 million per week.

As these are two tunnels, the costs of closing a single tunnel would be less. The expected costs of the four situations regarding the tunnel given in table 8.1.

	Situation	road traffic	other traffic	costs / week
1	no tunnels closed	0%	0%	€ 0
2	1st Beneluxtunnel closed	40%	0%	€ 0.9 million
3	2nd Beneluxtunnel closed	40%	10%	€ 1.1 million
4	Both tunnels closed	100%	10%	€ 2.5 million

Table 8.1: The costs per week of temporary traffic disturbance.

At this stage of design, a mean value of € 1.0 million per week will be used if one of the tunnels is closed.

Long term traffic disturbance

Long term traffic disturbance is the costs of speed reduction throughout the lifetime of the tunnel. It can be approximated applying the method used to approximate the societal costs of traffic jams. As future traffic intensity as well as the lifetime of the tunnel are very uncertain parameters, the outcomes of these approximations are also very uncertain and should not be implemented blindly. This method does however provide a reasonable method of evaluating the effects of speed reduction.

The following conditions were applied:

- Functional lifetime of 50 years.
- The mean speed is assumed to be the maximum speed.
- An average intensity of 200,000 vehicles per day is applied.
- The societal costs are estimated to be:
    - € 11.72 /h for cars based on 1.2 persons per car an derived from the societal value of time [73]
    - € 85.00 /h for trucks
- A tunnel length of 1900 m is used which is roughly the length of the tunnel system from pivot dike to pivot dike.
- Correction factors are used to incorporate the positive effects of speed reduction on environment, traffic intensity and traffic jam sensitivity.

The results are:

80 km/h € 335 million (€ 394 million without correction)

50 km/h € 1103 million (€ 1575 million without correction)

8.2.3 Construction costs

In this section, the costs of the construction alternatives will be estimated based on the construction sequences presented in chapter 7. The four alternatives are:

- Immersed part 1: Keeping elements immersed.
- Immersed part 2: Re-floating elements.
- Land part 1: local adjustments.
- Land part 2: General lowering.

As the cost estimates are based on a conceptual design, there are still many uncertainties in the construction methods and in the costs estimates. Hence, the costs estimates presented in this section are assumed to have a margin of error of about plus or minus 50%.

The applied cost factors can be found in appendix J.

Simplified representative tunnel

To simplify calculations, the costs estimates of the tunnel are based on a simplified representative tunnel of which the dimensions are approximately the mean of the 1st and the 2nd Beneluxtunnel.

Important dimensions are:

- Tunnel width: 35 m.
- Tunnel height: 8 m.
- The tunnel consists of 6 segments per element of each 20 m.
- The tunnel consists of 7 elements of each 120 m.

Using this representative tunnel, the construction costs will be estimated for a single land or immersed part of a single tunnel. Consequently, to obtain the total project costs, the estimates have to be multiplied by the either two or four. Scaling advantages will at this point not be taken into account.

Immersed part 1: Keeping elements immersed

The fixed costs of this alternative are:

nr.	Phase		cost estimate
1	Remove bottom protection		€ 619,000
2	Dig around tiles		€ 2,460,000
3	Remove tiles		€ 19,000
4	Longitudinal pre-tensioning		€ 3,831,000
5	Place supports		€ 1,671,000
6	Remove remaining soil		€ 14,125,000
7	Disconnect one joint		€ 791,000
8	lower elements		€ 454,000
9	underflowing		€ 17,204,000
10	remove pre-tensioning		€ -
11	backfilling		€ -
12	closure joint		€ 94,000
13	finishing		€ 3,400,000
			€ 44,667,000
	Risk	30%	€ 13,400,000
	indirect	25%	€ 11,167,000
	total construction costs		€ 69,234,000
	profit etc.	10%	€ 6,923,000
	total costs		€ 76,158,000

The variable costs of this alternative are:

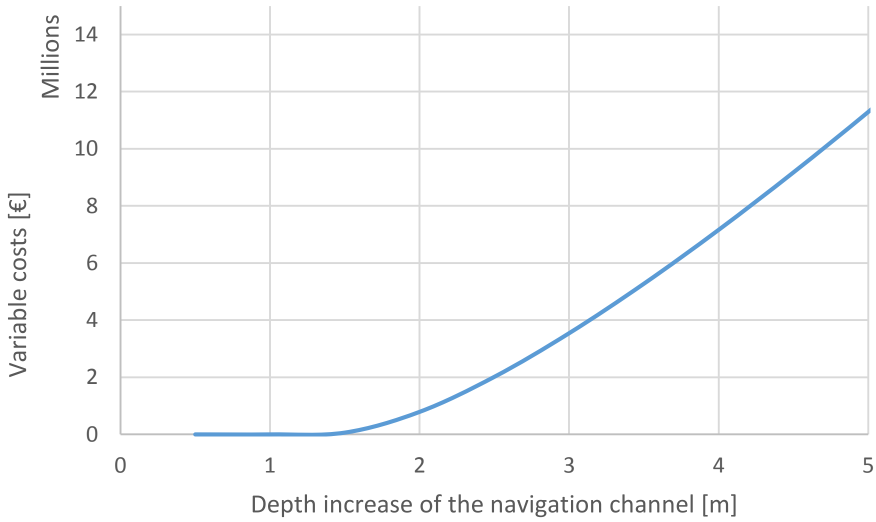

Figure 8.2: The variable costs of the immersed part alternative: Keeping elements immersed

Some comments:

- The total costs of this alternative are about € 76 million.
    - The lower boundary of the cost range is € 38 million.
    - The upper boundary of the cost range is € 114 million.
    - Variable costs (up to € 11 million are not included)
- This estimate applies to a single tunnel, both tunnels would cost twice as much.
- A time estimate of 62 weeks applies to this design option.
- About 80% of the costs of this alternative is related to earthworks. For a large part this is the soil that must be removed underneath the elements which has to be at least a layer of about 5 m to allow for the soil to be reached. Consequently, variable costs are 0 for the first few meters of lowering.
- Other important contributors are the costs of longitudinal pre-tensioning and the closely related removal of the ballast concrete (about 17%). To lower these costs, it might be wise to consider placing more supports.
- In section 7.2.1, the option was discussed lower the elements through holes in the floor. As this option would not require earthworks nor pre-tensioning or ballast removal, this could be an interesting alternative which could significantly lower the costs.
- In section 6.5, the distinction was made between lowering while only using one joint type (segment joints or immersion joints) and lowering while using both joint types. The cost estimate of this section applies to using one joint type. It is assumed for the option of using both joint types to cost an additional € 5 million per tunnel, consequently, these options will be referred to as:
    - Keeping immersed 1 is the option in which a single joint type is utilized.
    - Keeping immersed 2 is the option in which both joint types are utilized.
- The costs of removing pre-tensioning (10) are already included in applying it (4).
- The costs of backfilling (11) are already included in underflowing (9).

Immersed part 2: Re-floating elements

The fixed costs of this alternative are:

nr.	Phase			cost estimate
1	Remove bottom protection		€	619,000
2	Dig around closure joint		€	25,000
3	Open closure joint		€	140,000
4	Remove soil around elements		€	4,990,000
5	prepare elements for floating		€	14,193,000
6	re-float elements		€	958,000
7	transport to temp location		€	76,000
8	Renovate elements		€	68,740,000
9	Prepare river bottom		€	368,000
10	transport back to site		€	76,000
11	re-immerse		€	958,000
12	backfilling		€	4,919,000
13	closure joint		€	94,000
14	finishing		€	-
			€	96,154,000
	Risk	30%	€	28,846,000
	indirect	25%	€	24,038,000
	total construction costs		€	149,039,000
	profit etc.	10%	€	14,904,000
	total costs		€	163,943,000

The variable costs of this alternative are:

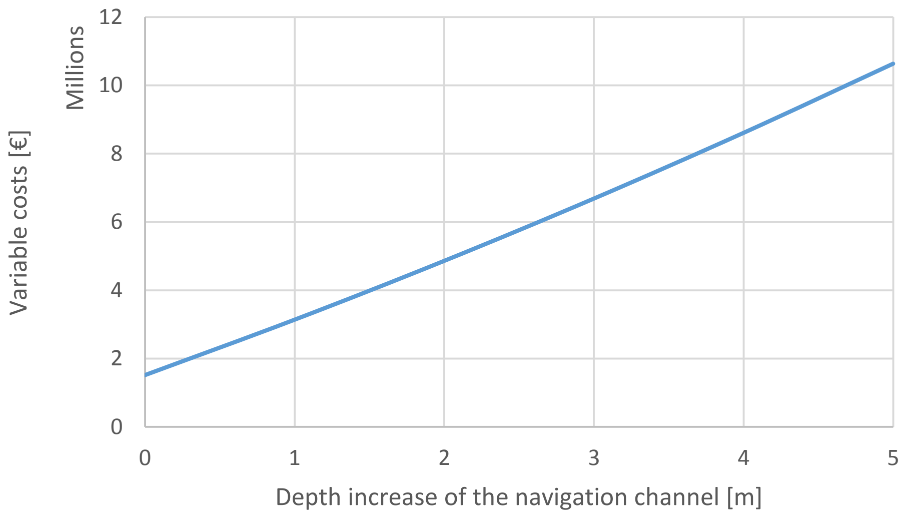

Figure 8.3: The variable costs of the immersed part alternative: Re-floating elements

Some comments:

- The total costs of this alternative are about € 164 million.
    - The lower boundary of the cost range is € 82 million.
    - The upper boundary of the cost range is € 246 million.
    - Variable costs (up to € 11 million are not included)
- This estimate applies to a single tunnel, both tunnels would cost twice as much.
- A time estimate of 73 weeks applies to this design option.
- About 71% of the costs is spend on renting the construction dock and renovating the elements. These costs could be significantly decreased if it is chosen not to renovate the elements but to keep them afloat and reuse them in the same condition.

Land part 1: Local adjustments

The fixed costs of this alternative are:

nr.	Phase		Cost estimate	
1	Remove soil		€	-
2	Separate		€	-
3	Watertight screen		€	117,000
4	Demolish concrete		€	217,000
5	Rebuild		€	1,027,000
6	Dewatering buffer		€	500,000
7	remove Watertight screen		€	24,000
8	Reconnect		€	-
9	return soil		€	-
			€	1,884,000
	Risk	30%	€	565,000
	indirect	25%	€	471,000
	total construction costs		€	2,920,000
	profit etc.	10%	€	292,000
	total costs		€	3,212,000

No variable costs are determined for this alternative.

Some comments:

- The total costs of this alternative are about € 3.2 million.
  - The lower boundary of the cost range is € 1.8 million.
  - The upper boundary of the cost range is € 4.6 million.
- This estimate applies to a single land part of a single tunnel, both land parts of both tunnels would cost four times as much.
- A time estimate of 20 weeks applies to this design option.
- All costs related to the immersed part (1,2, 8 and 9) are not included.

Land part 2: General lowering

The fixed costs of this alternative are:

nr.	Phase			
1	Insert new anchors		€	495,000
2	Disconnect element		€	-
3	Insert sheet piles		€	467,000
4	Demolish		€	1,260,000
5	Inundate		€	10,000
6	Demolish 2		€	2,856,000
7	Digging		€	3,100,000
8	Underwater concrete		€	8,232,000
9	dewater		€	20,000
10	rebuild		€	8,316,000
11	cut sheet piles		€	24,000
12	Reconnect elements		€	-
13	finishing		€	-
			€	24,769,000
	Risk	30%	€	7,431,000
	indirect	25%	€	6,192,000
	total construction costs		€	38,392,000
	profit etc.	10%	€	3,839,000
	total costs		€	42,231,000

No variable costs are determined for this alternative.

Some comments

- The total costs of this alternative are about € 42 million.
    - The lower boundary of the cost range is € 21 million.
    - The upper boundary of the cost range is € 63 million.
- This estimate applies to a single land part of a single tunnel, both land parts of both tunnels would cost four times as much.
- A time estimate of 84 weeks applies to this design option.
- All costs related to the immersed part (2, 12 and 13) are not included.

8.3 Cost analysis

In this section, the costs will be analysed. First the costs of the design alternatives will be compared. Next, the influence of depth increase limitations will be determined. And finally, conclusions and advice regarding the value of the solutions will be given.

8.3.1 Costs of alternatives

The costs estimates of the previous sections will be summarized:

- The costs of building a new tunnel and removing the old tunnel are:
  - A new immersed tunnel: € **730** million.
  - A new bored tunnel: € **1115** million.
- The costs of traffic disturbance are:
  - Temporary traffic: € **60 - 84** million per tunnel, or € **120 - 168** million in total.
  - Long term traffic: € **335 - 1103** million.
- The costs of lowering the immersed part of the tunnel are:
  - Keeping immersed 1: € **76 - 87** million per tunnel, or € **152 -174** million in total.
  - Keeping immersed 2: € **81 - 92** million per tunnel, or € **162 -184** million in total.
  - Re-floating: € **164 - 175** million per tunnel, or € **328 - 350** million in total.
- The costs of adapting the land part are:
  - Do nothing: € **0** per land part, or € **0** in total.
  - Local adjustments: € **3** million per land part, or € **12** million in total.
  - General lowering: € **42** million per land part, or € **168** million in total.

Consequently, the cheapest combination costs € 272 million and the most expensive combination costs € 1789.

Figure 8.4 indicates the differences between the construction costs of the technical design options. For all relative design combinations, the cumulative costs are displayed in figure 8.5. These costs include the variable costs of the maximum possible depth increase for the design options.

The figures show the design options for different maximum speeds and for the following scenario's concerning the land parts:

0-0	Do nothing on both sides.
loc-loc	Local adjustments on both sides.
gen-gen	General lowering on both sides.
0-loc	Local adjustments on one side.
0-gen	General lowering on one side.

The * is used to indicate the use of the existing traffic requirements. Also, the three options regarding the immersed part are displayed for each alternative:

- The left bar of each alternative shows keeping immersed 1, in which the tunnel is lowered using only one joint type.
- The middle bar of each alternative shows keeping immersed 2, in which the tunnel is lowered using both joint types.
- The right bar of each alternative shows re-floating.

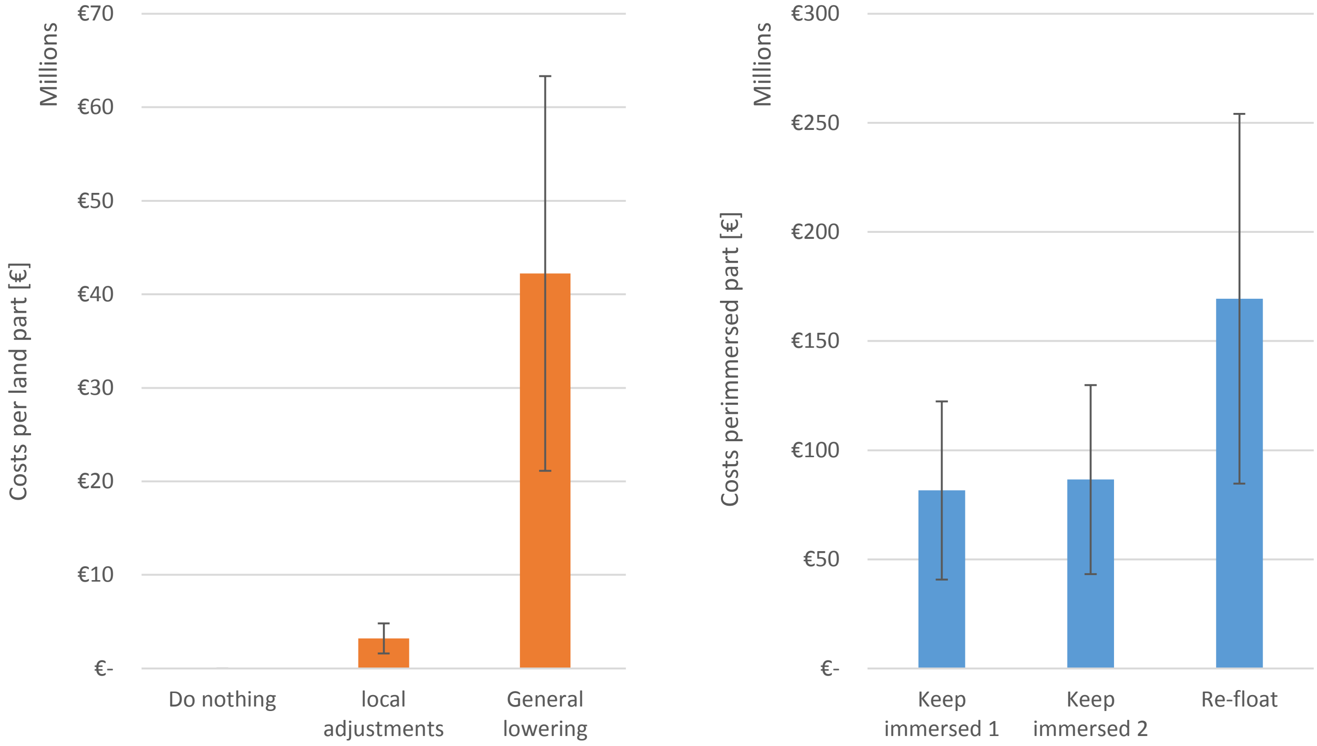

Figure 8.4: These graphs show the costs of the design options for one land part (left) and one immersed part (right).

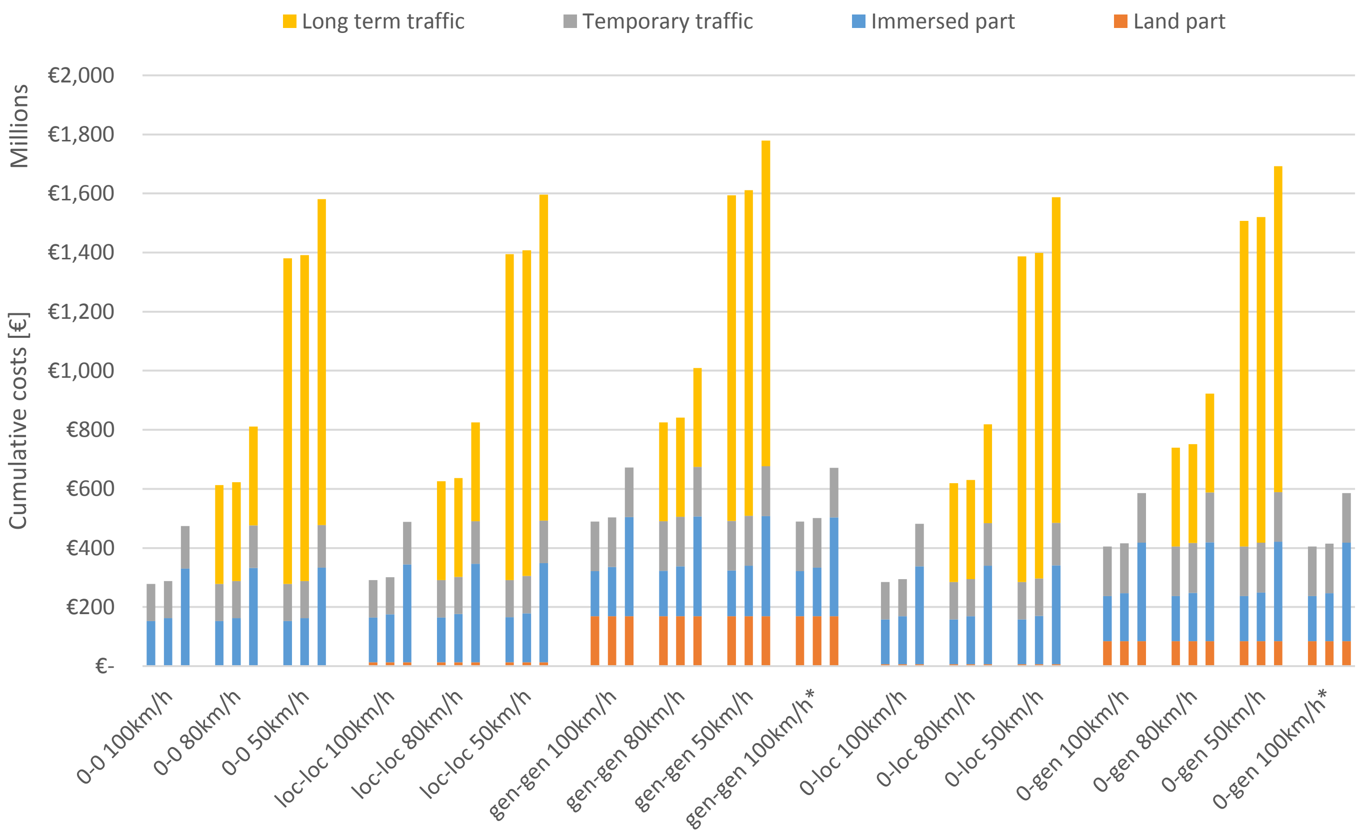

Figure 8.5: This figure shows the costs of the design options for the maximum possible depth increase

8.3.2 Costs versus depth increase

In this section, the depth increase will be taken into account.

The most cost effective solutions

Figure 8.6 shows the development of the most cost effective solution relative to the depth. Also the associated cost components are indicated. If the depth increases, lowering limitations are breached and less cost effective solutions become the alternative.

Figure 8.7 shows the same values, but divided by the achieved depth, showing the costs per meter.

At certain depths, the costs suddenly increase to a larger level. The associated cost levels will be discussed.

0 - 0.9 m	At these depths, all 100 km/h alternatives that do not generally lower the land part are preferred.
0.9 - 1.9 m	In this range, generally adapting one of the land parts is required.
1.9 - 3.2 m	In this range, adapting both land parts is required.
3.2 - 3.6 m	At 3.2 m, the limitations regarding joint rotations prevent the elements to be lowered while they remain immersed. Consequently, it is required to re-float the elements.
3.6 - 3.9 m	From 3.6 m, the traffic requirements require the speed to be dropped to 80 km/h. Doing so, it is yet again possible to lower the elements while they remain immersed.
3.9 - 4.6 m	At this point, it is again required to re-float the elements.
4.6 - 5.5 m	At 4.6 m, traffic requirements require further decrease of the speed down to 50 km/h. At 5.5 m, also the limitations regarding this speed regime can no longer be met. Consequently, this is the maximum possible depth.

Some comments:

- The required depth increase to reach the depth of the Maeslantkering is about 3.1 m
- In this section, the costs most cost effective options are described. However, given the risks and uncertainties in the estimates, it could be possible for another method to be preferred. This will be described in the next section.
- These graphs show the entire costs of the project, for the 1st and the 2nd Beneluxtunnel combined.

Regarding the costs / meter, it can be concluded that for depth ranges of 1.8 to 3.6 m, the costs / meter are about € 200 million.

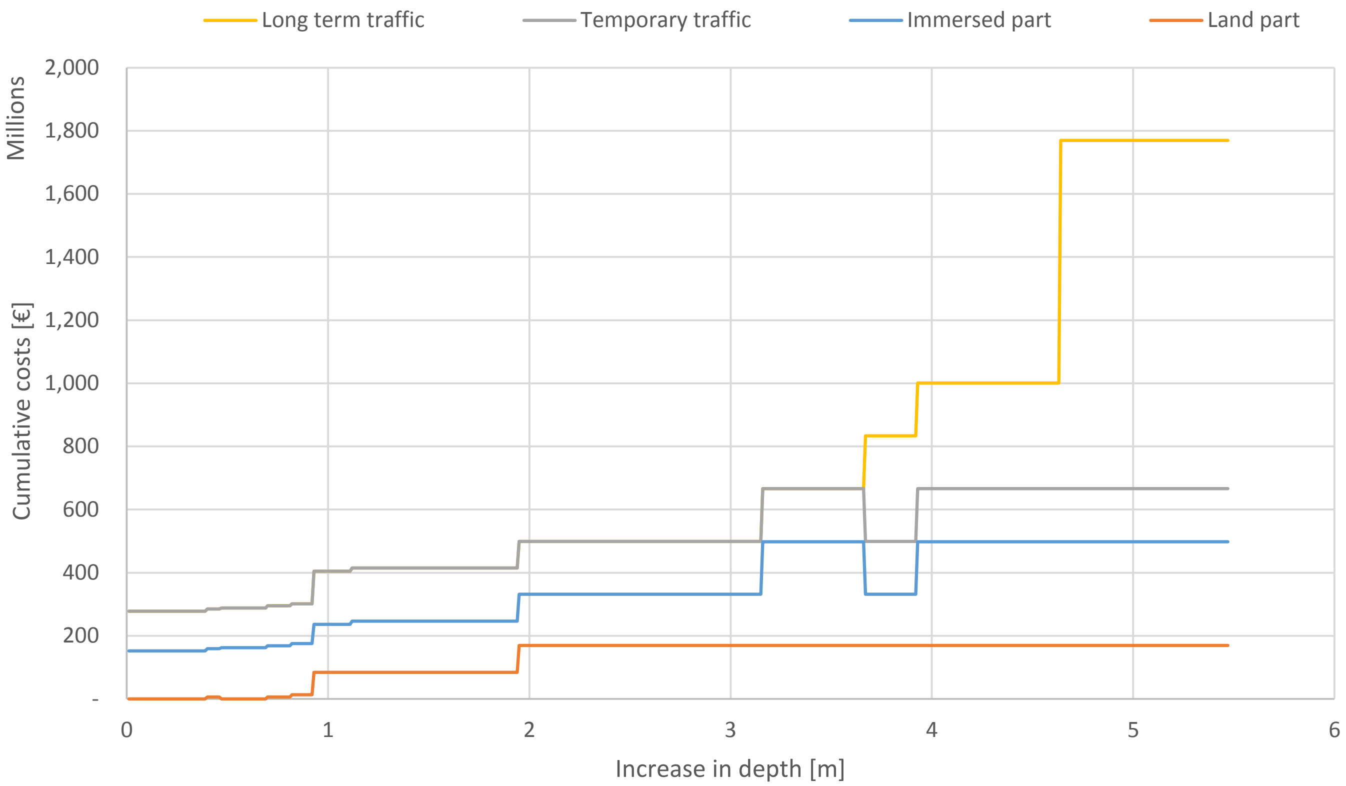

Figure 8.6: This graph shows the cumulative cost estimates for the most cost-effective solutions relative to the increase in depth.

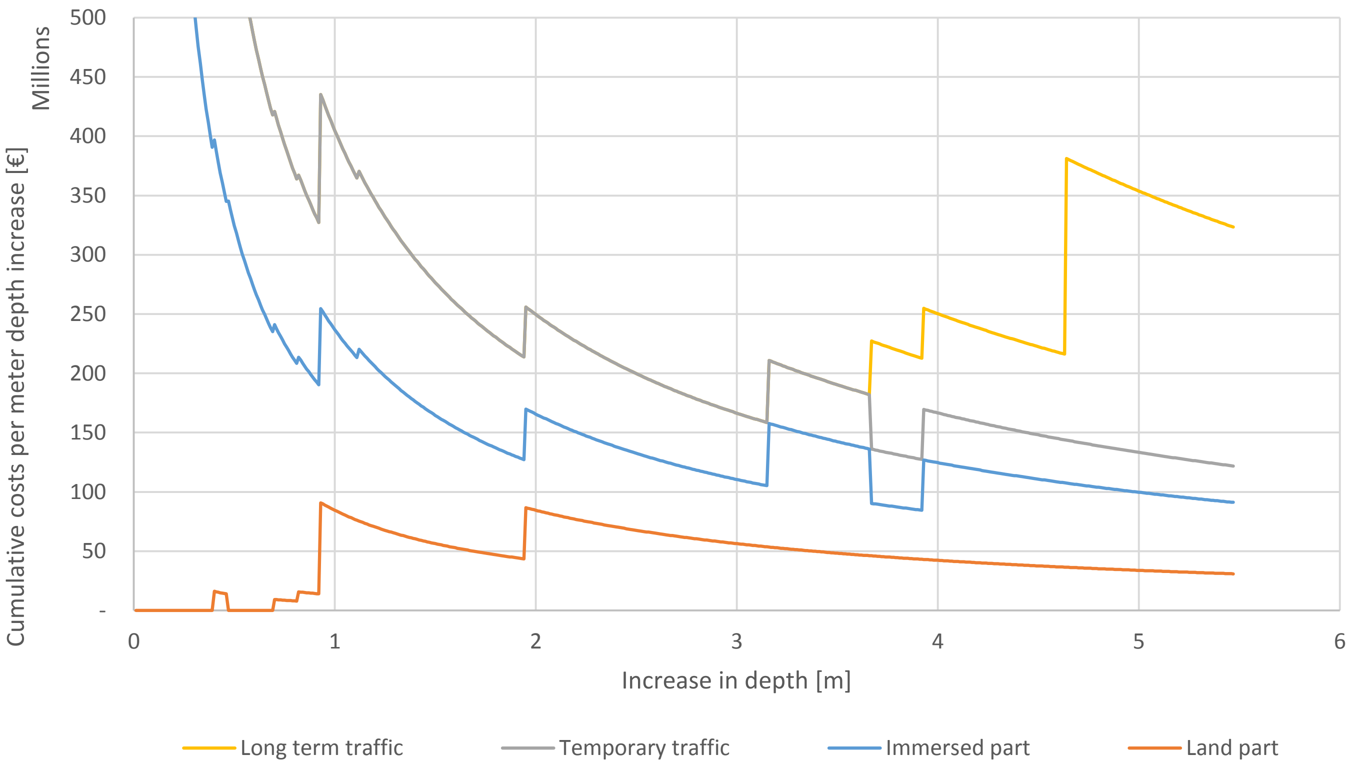

Figure 8.7: This graphs shows the cumulative costs per meter lowering for the most cost-effective solutions relative to the increase in depth.

Situation without long term traffic

Figure 8.8 shows the total costs for the scenario displayed in figure 8.6, but also for the scenarios in which the long term traffic costs are excluded. Figures 8.9 and 8.10 show the associated cumulative costs.

From these graph can be concluded that these different approaches would result in different design decisions. The most important differences that may be noticed are:

- The first range described for the original scenario (0 - 0.9 m) would reach to 2.2 m if 80 km/h would be possible and to 3.2 m if the 50 km/h scenario would be possible. The associated cost differences with the original scenario are about € 200.
- The second range described for the original scenario (0.9 - 1.9 m) in which only one land part is generally adapted, would for the 80 km/h scenario be preferred in the range of 2.2 -2.4 m. The associated costs differences are about € 85 million.
- The scenario in which none of the land parts were generally adapted, combined with the immersed part alternative of re-floating is not beneficial for the original situation. For the 80 km/h scenario it is beneficial but at little cost differences. For the 50 km/h scenario however, the costs differences are large. This ranges from 3.1 - 3.9 m. The associated costs differences are about € 85 million.
- From 3.6 m, lowering without decreasing the speed is no longer possible.

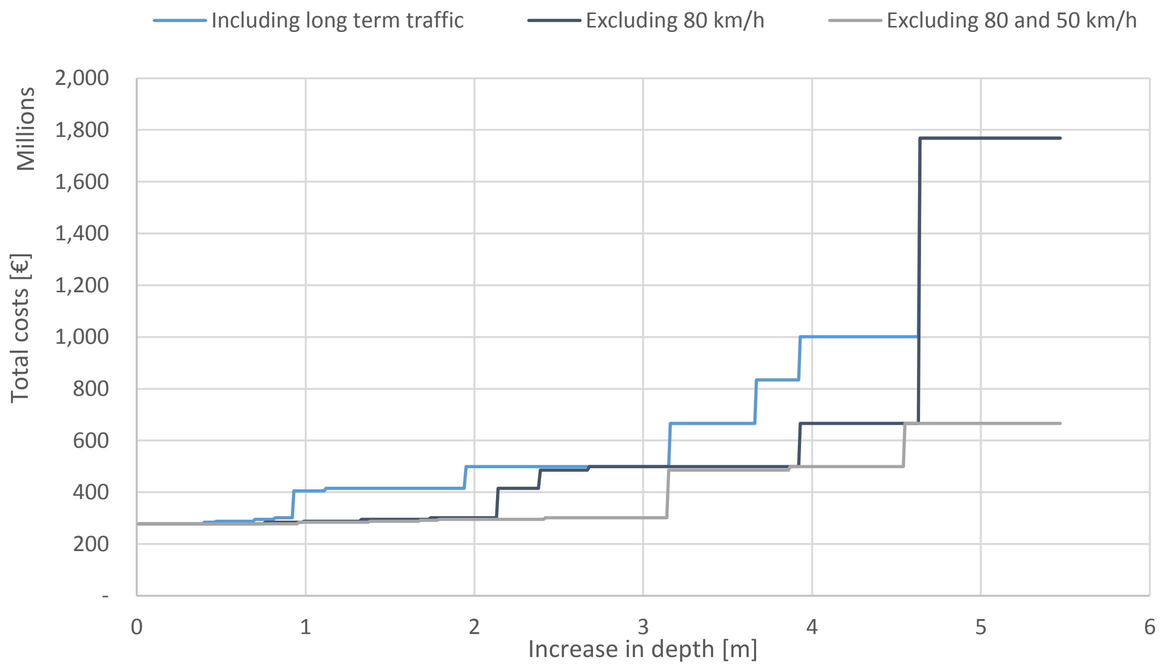

Figure 8.8: The graphs shows the most cost-effective solutions for different scenario's regarding long term traffic relative to the increase in depth.

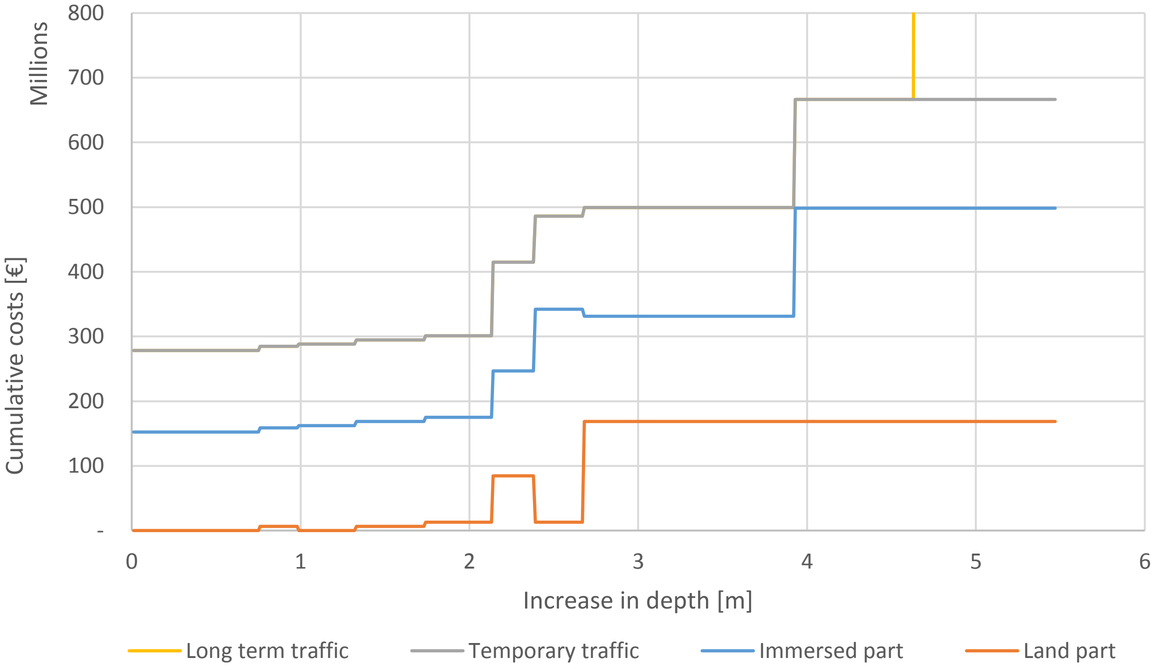

Figure 8.9: The graphs shows the cumulative costs of the most cost-effective solutions in case the long term traffic costs are excluded for the 80 km/h scenario, relative to the increase in depth.

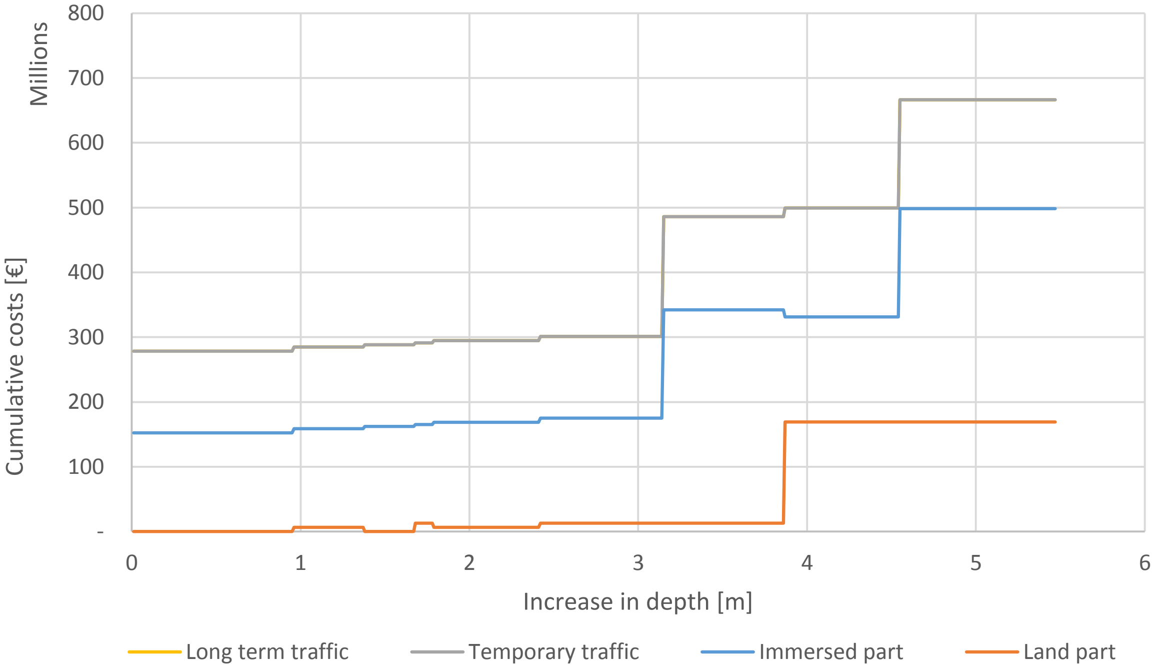

Figure 8.10: The graphs shows the cumulative costs of the most cost-effective solutions in case the long term traffic costs are excluded for the 80 km/h and the 50 km/h scenarios, relative to the increase in depth.

The top range of solutions

As there are many design options and the costs estimates of these design options are subjected to a large uncertainty (+/- 50%), the differences of the top range of solutions are analysed. Table 8.2 shows three of the top solutions for depth ranges 0.5 to 3 m. However, the proposed solutions are not the top 3 solutions. Given a total of 51 possible solutions, many have only minor differences. It has been tried to propose the best solutions with significant differences in costs or methods. The associated ranks range from number 1 to number 12. The numbers 1, 2 and 3 indicate the immersed part alternative as explained in section 8.3.1.

Figure 8.11 gives a graphical representation of the results.

	0.5 m	1 m	1.5 m	2 m	2.5 m	3 m	3.5 m
rank	1	1	1	1	1	1	1
name	0-0 100km/h 2	0-gen 100km/h 1	0-gen 100km/h* 2	gen-gen 100km/h* 2	gen-gen 100km/h* 2	gen-gen 100km/h 2	gen-gen 100km/h 3
costs	€ 288	€ 405	€ 415	€ 500	€ 501	€ 503	€ 672
rank	5	4	3	3	3	5	2
name	0-gen 100km/h * 1	0-loc 100km/h 3	gen-gen 100km/h* 1	0-gen 100km/h* 3	gen-gen 100km/h* 3	0-gen 80km/h 3	gen-gen 80km/h 2
costs	€ 405	€ 481	€ 489	€ 585	€ 671	€ 922	€ 839
rank	9	12	9	5	5	7	4
name	0-0 100km/h 3	0-0 80km/h 2	0-loc 80km/h 2	loc-loc 80km/h 2	loc-loc 80km/h 3	loc-loc 50km/h 2	loc-loc 50km/h 3
costs	€ 474	€ 623	€ 630	€ 637	€ 825	€ 1,407	€ 1,595

Table 8.2: This table shows the top 5 design options for depth increase ranging from 0.5 m to 5 m. Costs are in millions.

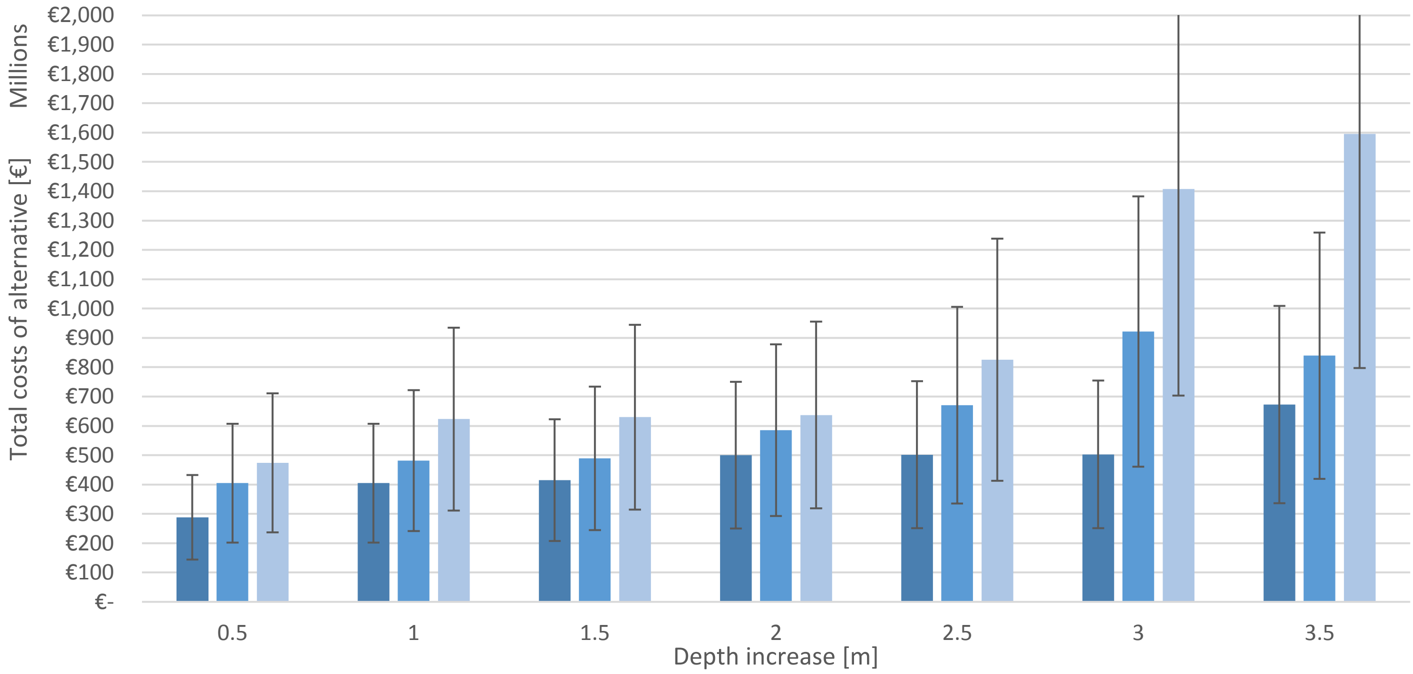

Figure 8.11: This figure shows the costs of the alternatives displayed table 8.2, intended to give graphical insight in the differences between the costs and the influence of uncertainty. The three colours are display the 1st, 2nd and 3rd rows of the table.

The main purpose of this overview is to indicate the effects of uncertainties on the conclusions. If the cost deviate much from their estimates, other solutions could be preferred. Consequently, one must not jump to conclusions too quickly.

8.3.3 Conclusions

As indicated in section 3.3.3, the value of the solutions is very similar, except for the aspects:

- Construction costs
- Consequences for traffic
- Consequences for navigation

Now that the construction costs and the consequences for traffic has been estimated for different depth ranges, conclusions regarding the cost-effectiveness of the solutions can be given.

However, the only exception to this methodology is the possibility of building a new tunnel, which could also differ in value for other aspects. Hence, this will be discussed first.

New versus old

The costs of building a new tunnel are much larger than the costs of adapting the existing tunnel for depth requirements up to 3.6 m. However, to determine which decision is preferred, also the pros and cons of these options should be taken into account.

Important pros and cons regarding building a new tunnel are:

+ A new tunnel could be made to perfectly fit the requirements of the connection for both the traffic and the navigation function.
+ A bored tunnel could even tunnel parts of the land, which would be very beneficial for the surrounding area.
+ A new tunnel would be made according to present day standards and would therefore be very durable.
+ If the new tunnel would be built on a different location, it could be constructed without causing temporary traffic disturbances.

Important pros and cons regarding adapting the existing tunnel are:

+ Adapting the tunnel is economically more attractive until a depth requirement of 3.6 m.
+ Adapting an existing tunnel is much more sustainable than building a new one.
– The old tunnel could have a restricted technical lifetime.
– The possible depth increase of the old tunnel is restricted.

The large cost differences are expected to outweigh these differences for most depth ranges. Consequently, for a certain depth requirements it is expected for a new tunnel to be more beneficial. If all costs would be taken into account, this tipping point would be at 3.6 m depth increase.

Long term traffic disturbance

Long term traffic disturbance has been included in the costs estimates to determine the influence of changes in traffic requirements on the value of the solutions. According to this cost analysis, it is in no scenario preferred to lower the speed. The expected economic impact outweighs the additional construction costs for depth increase up to 3.6 m and for larger depth requirements it is preferred to build a new tunnel.

Given the fact that the long term traffic disturbance is based on a functional lifetime of 50 years, the actual economic impact could even be much larger.

On the other hand, the estimates are very uncertain and should be interpreted cautiously.

Also, there could very well be other reasons to lower the speed. There are numerous examples of highways in urban areas where the maximum speed is lowered to decrease the environmental impact of the highway on its surroundings. These types of rules and legislations would primarily apply to the open stretches of highway on either sides of the tunnel, but traffic safety might require for the tunnel itself to maintain the same speed.

If there are reasons to lower the speed, the associated decrease of construction costs could reach up to € 200 million or even € 500 million in extreme scenarios.

Consequently, lowering the speed as a design option is clearly not advised according to the long term cost analysis as executed within this study. But if there are reasons to lower the speed regardless of this advice, the associated decrease of construction costs can be very high.

Temporary traffic disturbance

Temporary traffic disturbance has been included in the costs estimates to determine the influence of closing the tunnel on the total costs. The magnitude of this aspect is based on the construction time and is therefore determined by the longest construction time of either the land part or the immersed part.

For depth ranges up to 3.2 m, temporary traffic disturbance is expected to be about 35% to 45% of the total construction costs, ranging between € 60 million and € 85 million.

It is interesting to see that if the land parts are not generally lowered, the immersed part is governing. If general lowering has to be applied, the land part is governing.

Immersed part

The costs of re-floating are estimated at about twice the costs of keeping the elements immersed. Such large cost differences would generally be sufficient to discard the expensive option, this is however not advised. The following reasons are proposed.

- Both options could be regarded highly experimental and therefore risky. However, the lowering while immersed could be regarded the most unconventional solution of both and would therefore be riskier.
- The costs of re-floating could be significantly decreased if the costs of the construction dock could be decreased. Also, new elements could be constructed which would significantly lower the temporary traffic disturbance of the project.
- Based on the limited joint rotations determined in this report, the lowering range of re-floating is much larger.

- Re-floating allows the elements to be adapted to increase their technical lifetime.
- To adjust the land parts, space is required. Consequently, it would be preferred if at least one of the elements is removed.

Consequently, the large cost differences can better be used to conclude the inverse statement:

- Lowering while immersed seems to be a costs saving alternative to the more conventional option of re-floating and should therefore not be discarded. Especially if the costs of soil removal could be decreased, this option could be a very interesting way to decrease the vertical position of immersed tunnels.

Land part

Regarding the land part, two statements can be concluded:

- The cost differences between doing nothing and making local adjustments are relatively small.
- The cost differences between local adjustments and general lowering are relatively large.

Consequently, it would be cost-effective for lower ranges of lowering to apply local adjustments as the marginal costs of doing this are relatively small.

General lowering should only by applied if it is absolutely required to gain sufficient depth. The costs are higher, but also the risks are much higher than the risks associated with local adjustments.

Land part and immersed part combined

An aspect that has not been included in the cost analysis is the effect of decisions regarding the land part for the constructability of the immersed part and vice versa.

- To lower the immersed part while keeping the elements immersed, entrance to the elements must be provided from at least one of the land parts.
- To locally or generally adjust one of the land parts, space is required. This would require for the adjacent elements to be removed and could therefore better be combined with re-floating.

Costs versus depth

The most cost effective solutions when taking the depth into account are:

Lowering range	Land part	Immersed part	Costs (millions)
0.0 - 0.9 m	Local adjustments on both sides	Remain immersed	€ 295
0.9 - 1.9 m	General lowering on one side	Remain immersed	€ 415
1.9 - 3.2 m	General lowering on both sides	Remain immersed	€ 500
3.2 - 3.6 m	General lowering on both sides	Re-float	€ 665
> 3.6 m	New immersed tunnel		€ 730

Table 8.3: The most cost effective solutions to lower the Beneluxtunnel. The costs include the effect of traffic disturbance.

Chapter 9

Conclusions and recommendations

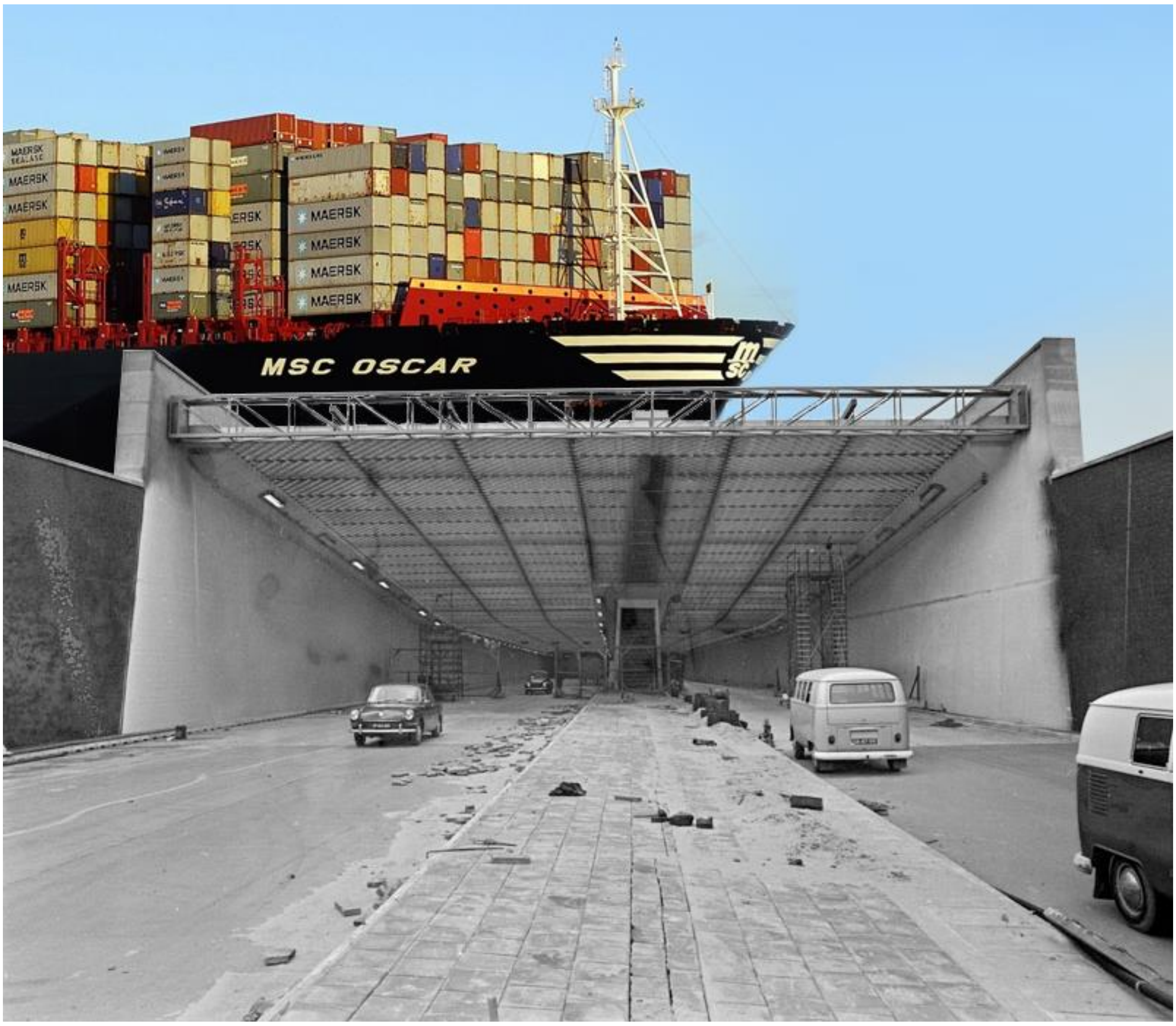

Figure 9.1: In this figure, the past and the possible future are combined. Will the largest container vessel in the world ever be able to cross the Beneluxtunnel?

9.1 Conclusions

The main objective of this study was to:

Propose a conceptual design for adapting the Beneluxtunnel in order to effectively increase the available draught for navigation.

To satisfy this objective, the following design options were proposed, elaborated and evaluated:

- Investigated options regarding the land part:
  - Do nothing
  - Local adjustments
  - General lowering
- Investigated options regarding the immersed part:
  - Adapt river bottom
  - Keep elements immersed
  - Re-float elements
- Investigated options regarding traffic requirements:
  - 100 km/h scenario
  - 80 km/h scenario
  - 50 km/h scenario
  - 100 km/h scenario with existing traffic requirements.

These design options have been analysed separately and combined to determine their technical feasibility and value.

Technical feasibility

The strength of the elements of the Beneluxtunnel is sufficient to allow lowering.

- The cross-sectional moment and shear capacity is sufficient to allow lowering within the required depth ranges.
- The longitudinal mechanics of the tunnel are not expected to be negatively influenced by the increase in hydrostatic pressure.
- The effect of lowering on the rubber profiles is very limited.

Discussion:

- The cross-sectional capacity of the 2^{nd} Beneluxtunnel has not been determined as no information regarding the applied reinforcement was available. Given the outcomes of the 1^{st} Tunnel, it is expected for the 2^{nd} Beneluxtunnel to have similar excess strength. However, it is not unlikely for the safety philosophy to have changed over the 35 year period between the construction of both tunnels, resulting in a less conservative design with less excess safety.
- The shear capacity estimates are based on a method that was expected to be very conservative and are therefore multiplied by 1.5. This method is however not expected to be very accurate.
- The strength of the concrete is based on assumed material parameters.

Joint rotations can be utilised to allow significant lowering of the tunnel.

- For the segment joints it is expected that the W9Ui profiles are governing with regard to the possible rotation. This profile allows elongation of up to 22 mm. The associated rotations are not expected to significantly affect the structural capacity of the segment joints as the mechanical behaviour in case of 1 mm elongation is very similar to the case of 22 mm.
- For the immersion joints it is expected for the GINA-profiles to be governing regarding the possible rotation. This profile is expected to allow elongation and compression of 35 mm.
- If 50% of the maximum elongation is maintained, the associated rotations are large enough to allow changes in tunnel profile that provide significant depth increase. Rotation of only the segment joints or only the immersion joints give similar results; rotation of both joints combined gives about 75% more depth increase.

  Discussion:

- Joint rotations are affected by settlement deformations during the lifetime of the tunnel. These rotations can become intolerably large in extreme scenarios, but they are very hard to predict. This is however already the case in existing tunnels and successful control measures are available.
- The immersion joints are analysed less extensive than the expansion joints, which mainly follows from the fact that for the 1st Beneluxtunnel the immersion joints were constructed monolithically, resulting in a focus on lowering methods without opening the joints.

Preferred design options

For the land part, the preferred option depends on the local situation and on the depth requirements.

- For the norther land part, the option “do nothing” is preferred because near this land part, the closure joints are located.
- For the southern land part, the option “local adjustments” is preferred because of the relatively low costs and large effect.
- Although the option “general lowering” is much more expensive and risky than the other two options, for depth requirements of larger than 1.0 m, it is preferred to generally lower the southern land part.

For the immersed part, one option can be discarded. The other options can be combined.

- Adapting the river bottom is not effective as it is already fully utilized at the edges of the navigation channel.
- Keeping elements immersed is the most cost effective option for the immersed part, it also has various drawbacks meaning it is not directly the preferred option. Combining this option with re-floating of elements would allow for the advantages of both options to be used.

The high expected costs of long term traffic disturbance results in major constructional interventions being preferred over lowering the speed.

Preferred design combinations

As the benefits associated with depth increase are unknown, it is not possible to propose a single best solution to lower the Beneluxtunnel. Instead, two alternatives are proposed:

Proposed alternative 1: Little effect, low costs.

Southern land part	Northern land part	Segment joints	Immersion joints	Re-float	Traffic regime	Lowering	Cost estimate
Local adjust	-	Maximum rotations	-	-	100 km/h, 5% slopes	1.0 m	€ 300 million

- The southern land part is locally adjusted to allow a slope at the transition point of 5% and as much translation as the dewatering cellar allows, which is about 2 m.
- As the closure joint lies near the northern land part for both the 1st and the 2nd Benelux-tunnel, this land part is kept intact.
- The segment joints are rotated as much as possible.
- The immersion joints are not rotated as they are monolithically constructed for the 1st Beneluxtunnel. For the 2nd Beneluxtunnel, the same approach is maintained.
- The entire length of the tunnel is dug out, supported and lowered in a single take.
- A speed limit of 100 km/h is maintained, allowing a maximum slope of 5%.
- The depth increase of this alternative is about 1.0 m for the two-way navigation channel. A one-way channel of at least 1.5 m is also possible. The main axis of the channel would shift about 40 m southwards.
- The costs of this alternative are about € 300 million, including traffic costs.
- The vertical profile of this alternative is shown in figure 9.2

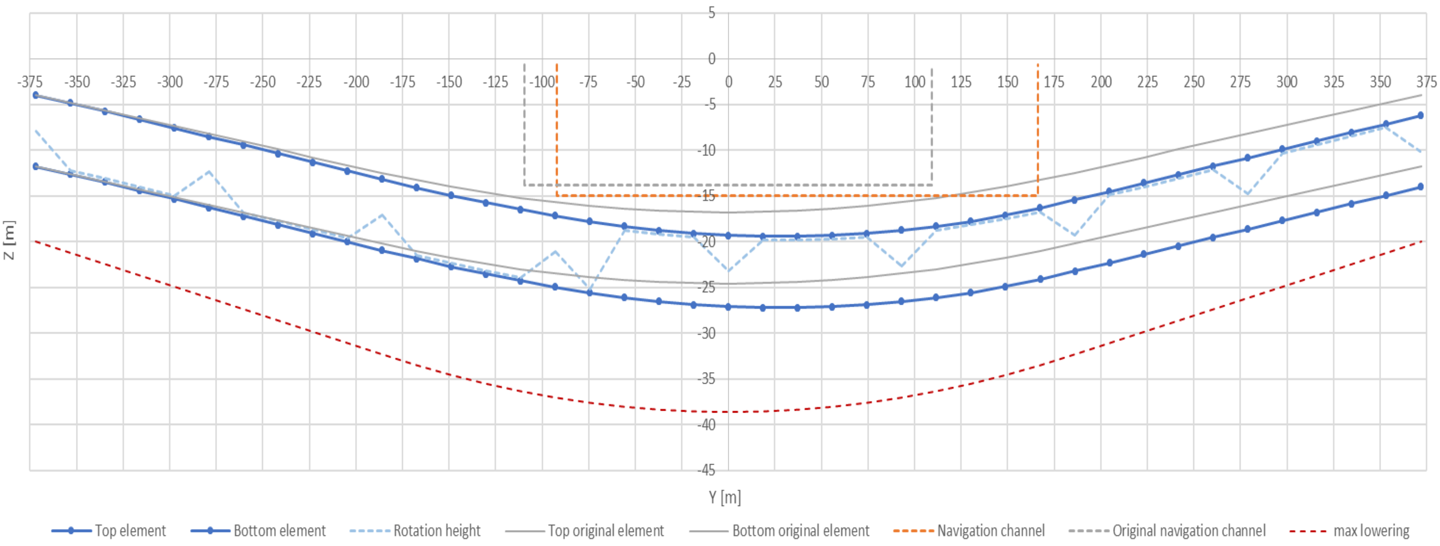

Figure 9.2: The vertical profile of alternative 1: Little effect, low costs.

Proposed alternative 2: Large effect, high costs.

Southern land part	Northern land part	Segment joints	Immersion joints	Re-float	Traffic regime	Lowering	Cost estimate
General lowering	-	Maximum rotations	Maximum rotations	1 element	100 km/h, 5% slopes	2.7 m	€ 450 million

- The southern land part is generally lowered up to 8 m. To allow this, new tension piles will be required.
- As the closure joint lies near the northern land part for both the 1st and the 2nd Benelux-tunnel, this land part is kept intact.
- The segment joints are rotated as much as possible.
- The immersion joints are rotated as much as possible. To allow this, the GINA-profiles will be locally replaced or adapted. To open the joints, hydraulic cutting systems will be used. To prevent flooding, the bulkheads will be inserted during this process.
- The elements of the tunnels are dug out, supported and lowered one by one. First the element next to the southern land part is re-floated to allow this land part to be generally lowered. Next, the other elements are lowered one by one.
- A speed limit of 100 km/h is maintained, allowing a maximum slope of 5%.
- The depth increase of this alternative is about 2.7 m for the two-way navigation channel. A one-way channel of at least 3.2 m is also possible. The main axis of the channel would shift about 100 m southwards.
- The costs of this alternative are about € 450 million, including traffic costs.
- The vertical profile of this alternative is shown in figure 9.3

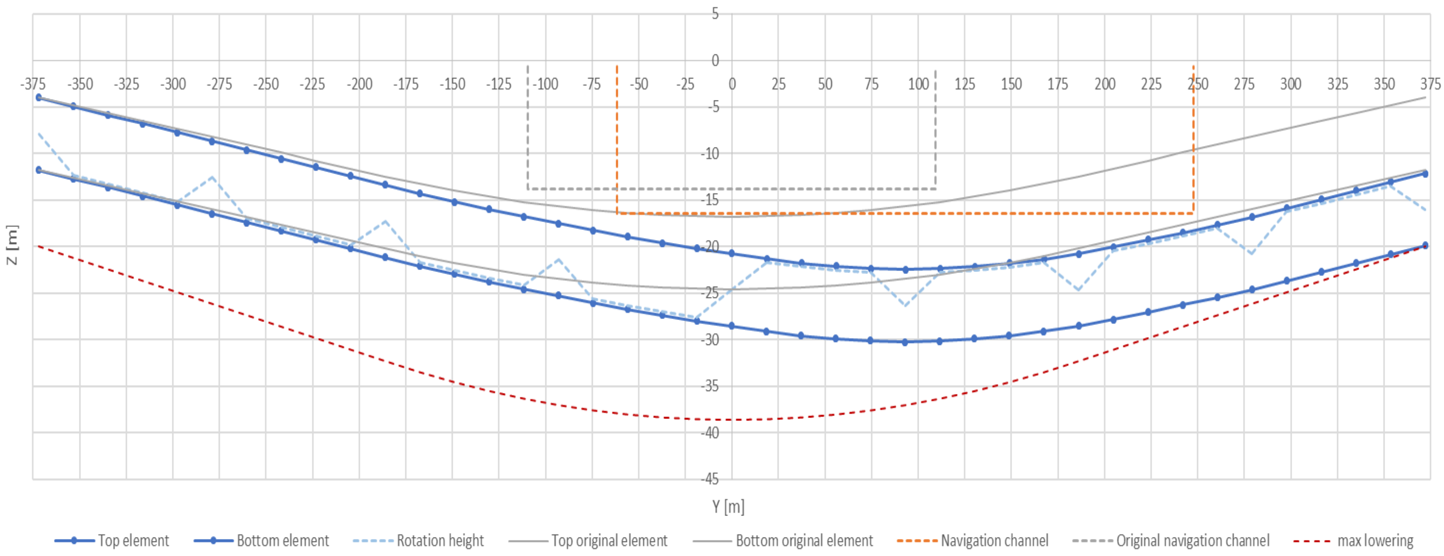

Figure 9.3: The vertical profile of alternative 2: Large effect, high costs.

9.2 Recommendations

Recommendations regarding the Beneluxtunnel

Regarding the possibility of lowering the Beneluxtunnel:

- It is recommended to estimate the economic benefits associated with depth increase and to determine whether it is economically feasible to increase the depth of the Beneluxtunnel and by how much.

Regarding the design option, it is advised to investigate the following aspects:

- If little depth is required, lowering alternative 1.
- If much depth is required, lowering alternative 2.
- The probability of the maximum speed being reduced to 80 km/h in the future.
- Both options regarding the immersed part, and especially the possibility of combining the options.

Regarding technical details, it is advised to investigate the following aspects:

- Obtain the proper structural data of both the 1st and the 2nd Beneluxtunnel, including experimental data regarding the current concrete strength.
- A cross-sectional moment capacity analysis on the 2nd Beneluxtunnel.
- The shear capacity of the cross-section and of the segment and immersion joints, for both tunnels, including the effects of torsion.
- A probabilistic analysis of the possible rotations of the segment joints and immersion joints regarding safety levels given the influence of settlement.
- A feasibility study regarding the possibility of deconstructing the monolithically constructed immersion joints of the 1st Beneluxtunnel and deconstructing the closure joints.
- Estimate the technical lifetime of both of the Beneluxtunnels and determine whether adaptions could be made to increase this lifetime.
- The possibility of shifting the navigation channel southwards.
- It is recommended to execute the further research in close collaboration with people that were involved in the construction of the 1st and the 2nd Beneluxtunnel.

Recommendations regarding immersed tunnels in general

Recommendations regarding other existing immersed tunnels:

- As all immersed tunnels consists of elements and segments, the solutions applying to the immersed part can in large extent be applied to other existing immersed tunnels.
- As all abutments contain dewatering cellars, the option of locally adapting the land part can in large extent be applied to other existing tunnels.

Recommendations regarding the design of new immersed tunnels

- It is recommended to find another method of longitudinal pre-tensioning that does not require to be cut, but could be restored when required.

Chapter 10

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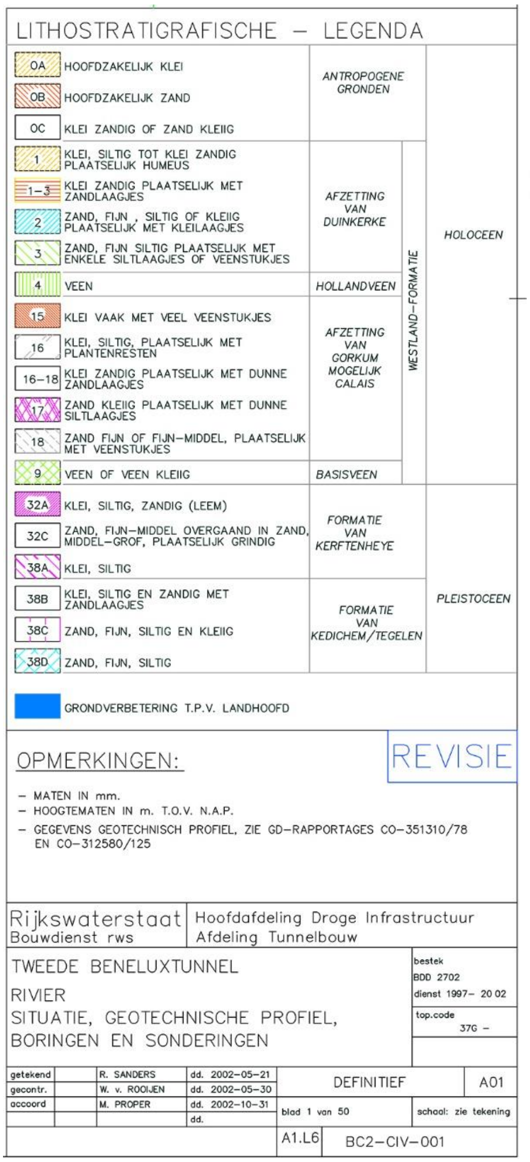
Appendices

Appendix A: Boundary conditions

Soil properties



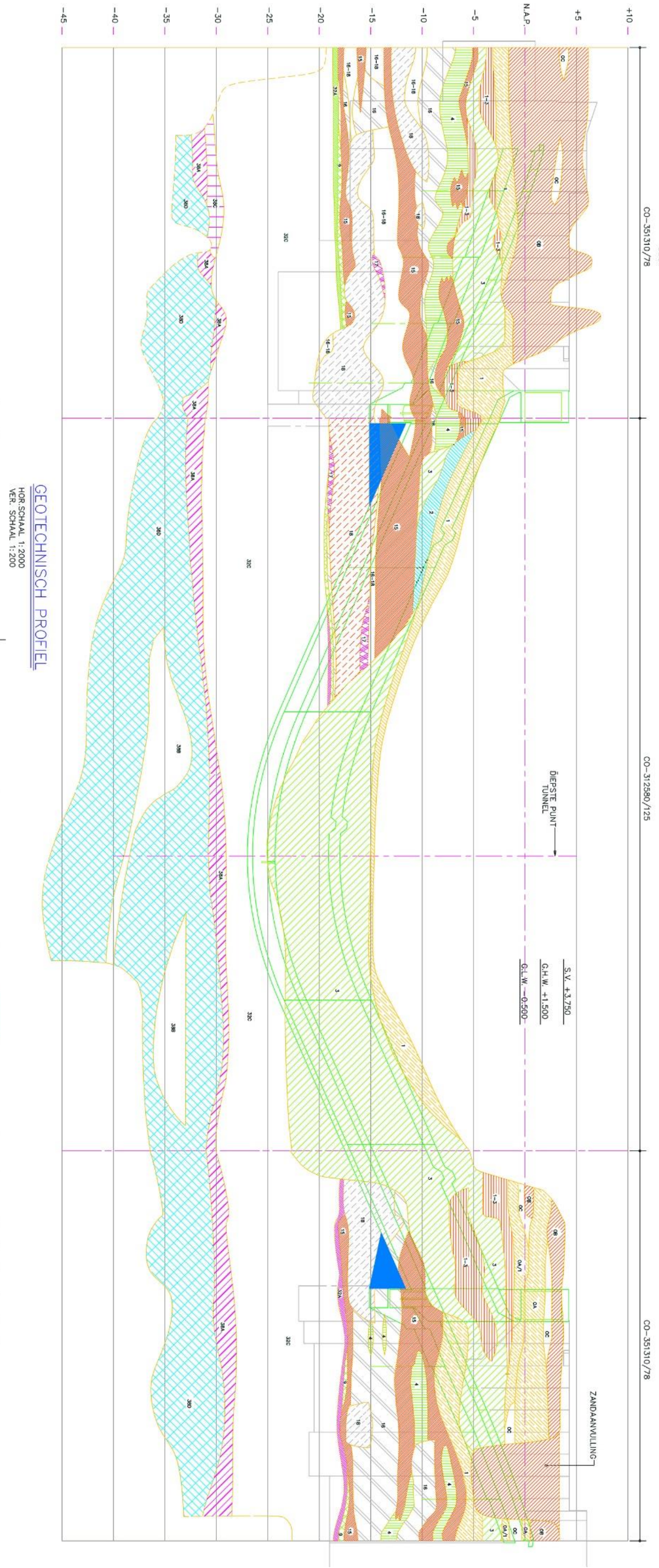
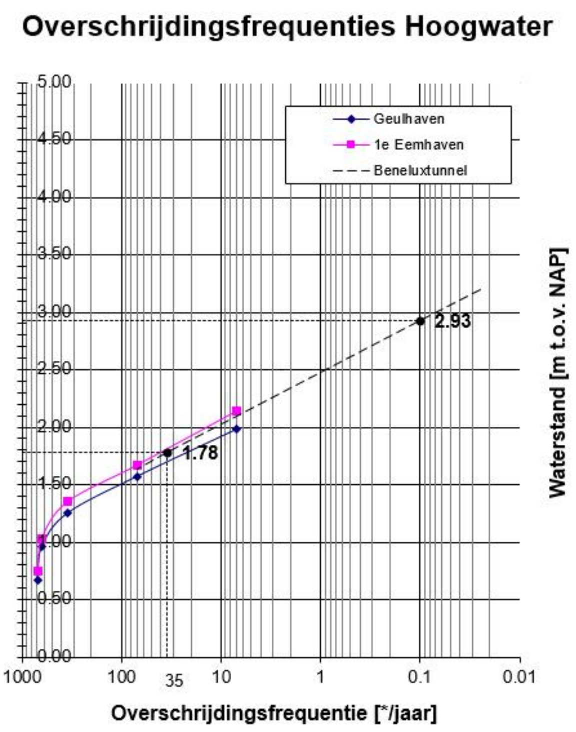
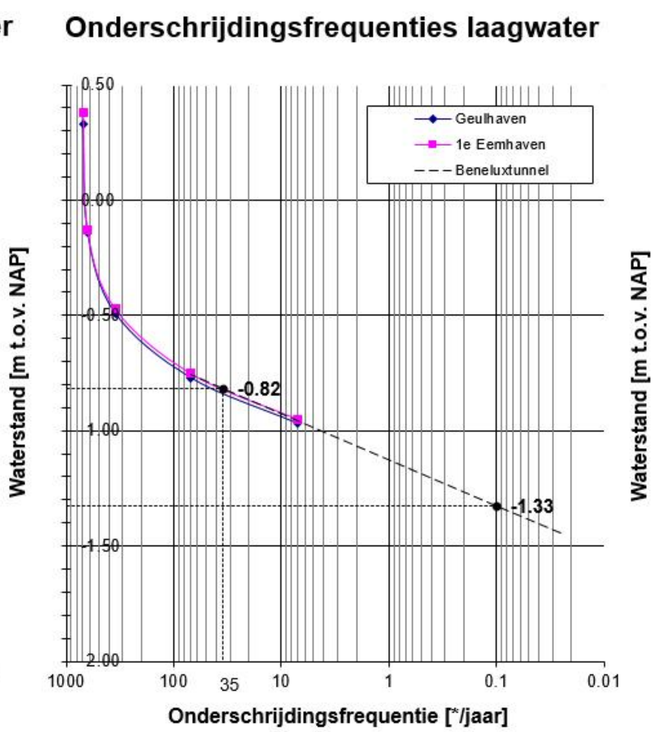


Figure A.1: The geotechnical profile at the location of the Beneluxtunnel [74]

Hydraulic properties





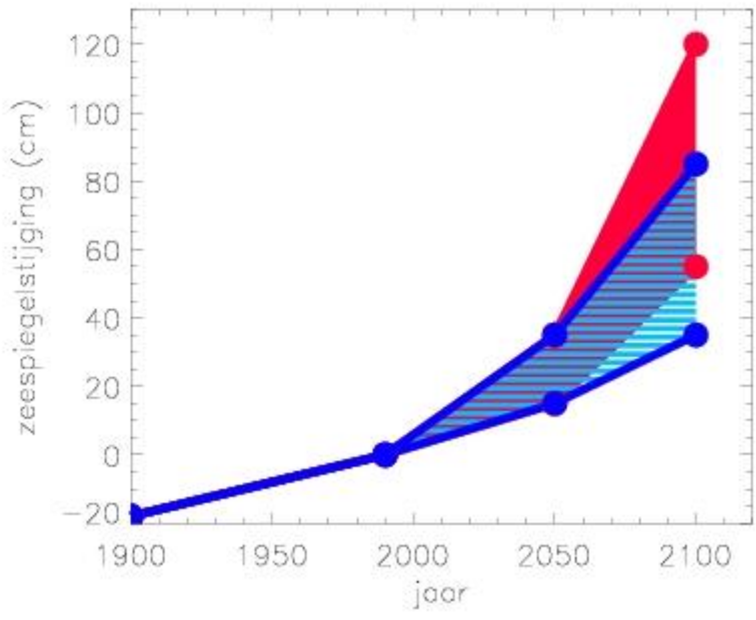


Figure A.2 - Scenario's voor zeespiegelstijging langs de Nederlandse kust voor de 21e eeuw (blauw: KNMI'06 scenario, rood: bovengrensscenario Deltacommissie)

(Source: http://www.knmi.nl/cms/content/115877/zeespiegelveranderingen_in_de_toekomst)

Rotterdam (Nieuwe Maas)

Slotgemiddelden 1998.0

Algemene gegevens	
1813	Aanvang waarnemingen
1 sep 1874	Peilschrijver geplaatst
23 jun 1987	DNM geplaatst

Gemiddelde waterstanden bij gemiddelde afvoer (2200 m^3/s)

type tij	HW-stand in cm + NAP	LW-stand in cm + NAP	tijverschil in cm
gemiddeld tij	**132**	**-39**	**171**
springtij	**148**	**-38**	**186**
doodtij	**112**	**-38**	**150**
gemiddelde waterstand		**24**	

Gemiddelde havengetallen bij gemiddelde afvoer (2200 m^3/s)

type tij cq grootheid	HW-tijd u:min	tijd u:min	LW-tijd u:min
gemiddeld tij	**02:43**		**10:51**
springtij	**02:45**		**10:53**
doodtij	**02:30**		**09:27**
duur rijzing		**04:17**	
duur daling		**08:08**	

Gemiddelde over- en onderschrijdings frequentie per jaar

overschrijding hoogwaterstanden		onderschrijding laagwaterstanden	
frequentie	stand in cm	frequentie	stand in cm + NAP
1x per 10.000 jaar	**358**	1x per 10 jaar	**-120**
1x per 4.000 jaar	**351**	1 x per jaar	**-100**
1x per 1.000 jaar	**342**		
1x per 100 jaar	**326**	OLW 1991.0	**-65**
1x per 10 jaar	**299**		
1x per 2 jaar (grenspeil)	**268**		
1x per jaar	**256**		
Maatgevende waarde	**360**		
(Schieland, 1 x per 10.000 jaar)			
Maatgevende waarde	**350**		
(IJsselmonde, 1 x per 4.000 jaar)			

Bijzonderheden

datum	stand cm + NAP	kenmerkende waarden	periode
3 jan 1976	**286**	hoogst bekende waarde	(periode 1971...1990)
19 jan 1972	**-153**	laagst bekende waarde	(periode 1971...1990)
21 nov 1971	**293**	maximale rijzing	(periode 1971...1990)
4 jan 1976	**273**	maximale daling	(periode 1971...1990)

De stormvloedkering in de Nieuwe Waterweg wordt bij verwachte HW-stand te Rotterdam (Boerengat) boven ca 300 cm + NAP gesloten

Appendix B: Traffic calculations

Specifying design parameters

Design speed	km/h	120	100	80	50	metro	
slope	%	5	5	6	7	3.90625	*in tunnel
max length	m	250	250	175	150		
Top arch radius	m	12375	8284	5011	1095	3100	
Bottom arch radius	m	1200	850	500	200	2000	

Standards

NOA:

heights

car	2.06			95% (SATO)
truck	4	m	law	

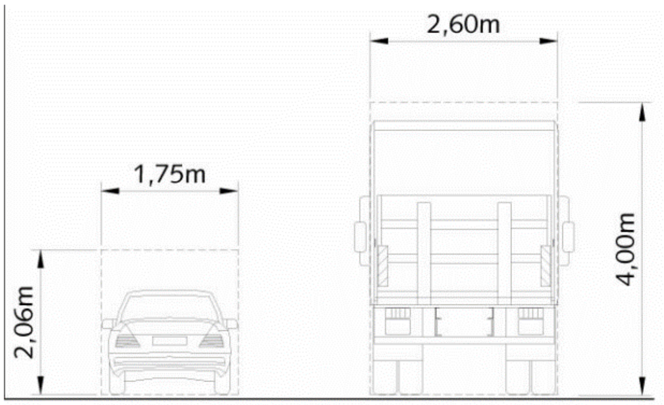
width

car	1.77	m		95%
truck	2.6	m	law	

SATO:

	personenauto	vrachtauto
ooghoogte	1,10 m	2,50 m
rijsnelheid bij $_{v0}$ = 120 km/h	120 km/h	90 km/h
rijsnelheid bij $_{v0}$ = 90 km/h	90 km/h	90 km/h

Tabel 2.3.2 Kenmerken voertuigen



Afbeelding 2.3.4, Afmetingen ontwerpvoertuigen

Slope
NOA:

Tabel 3-18
Eisen aan het maximale hellingspercentage bij grote kunstwerken

ontwerpsnelheid	helling (%)
120 km/h	≤ 5
100 km/h	
80 km/h	≤ 6
50 km/h	≤ 7

SATO:

Ontwerpsnelheid	**50 km/h**	**70 km/h**	**90 km/h**	**120 km/h**
p_{max} hoofdrijbaan	-	-	4%	3%
p_{max} overige rijbanen	7%	6%	5%	-

Tabel 2.3.7 Maximale hellingspercentages wegvakken op aardebaan

Bij bijzondere constructies, zoals tunnels en rivierovergangen worden in verband met de hoge kosten hellingen tot 4,5% geaccepteerd.

$$i_{max} = \frac{200 \cdot \Delta H}{\sqrt{2 \cdot \Delta H \cdot (R_o + R_b)}}$$

waarin:
i_{max} = het maximaal mogelijke hellingspercentage
ΔH = te overwinnen hoogteverschil [m]
$R_o + R_b$ = som van de boogstralen van de top- en voetboog [m]

slope

Design speed	km/h	120	100	80	50	
slope	%	5	5	6	7	
max length	m	250	250	175	150	(only for trucks)

Max length slope (trucks)

NOA

Tabel 3-19

Eisen aan de lengte van de helling (in meters), afhankelijk van het hellingspercentage, waarbij geen extra stroken benodigd zijn, c.q. waarbij de snelheidsterugval van vrachtwagens niet meer dan 20 km/h bedraagt

hellingspercentage	lengte helling
2%	≤ 800
3%	≤ 550
4%	≤ 350
5%	≤ 250
6%	≤ 175
7%	≤ 150

Top arch

NOA

Tabel 3-13

Stralen van een topboog

Figuur 3-28

ontwerpsnelheid	straal (1)	straal (2)
120 km/h	$R_{min} = \dfrac{L_z^2}{2\,(\sqrt{h_o} + \sqrt{h_h})^2}$	Zie figuur 3-28
100 km/h		
80 km/h		
50 km/h		

Verklaring van de tabelkoppen:

- straal (1) = straal van de topboog in de situatie dat de zichtafstand kleiner is dan de booglengte (in meters)
- straal (2) = straal van de topboog in de situatie dat de zichtafstand groter is dan de booglengte (in meters)

Legenda bij de formule:

R_{min} = minimale verticale boog (in meters)

L_z = (maatgevende) zichtafstand (in meters)

h_o = ooghoogte van de bestuurder (1,10 meter)

h_h = hoogte van het waar te nemen object (0 meter bij wegverloop in continue situatie; 0,20 meter bij obstakel op de weg en 0,50 meter bij stilstaande file)

ontwerp-snelheid	toetsing alle wegvakken			alleen plaatselijke toetsing	
	rijzicht: wegverloop continue situatie (m)	stopzicht: zicht op stilstaande file (m)	uitwijkzicht: zicht op obstakel op één strook (m)	discontinuïteit strookbeëindiging (m)	discontinuïteit onverwacht krappe horizontale boog (m)
120 km/h	165	260	235	500	zie figuur 7-2 waarin R_{hor} = de straal van de boog krappe boog
100 km/h	135	170	190	415	
80 km/h	105	105	145	335	
50 km/h	45	40	70	210	

Tabel 7-6

Zichtlengtes per situatie

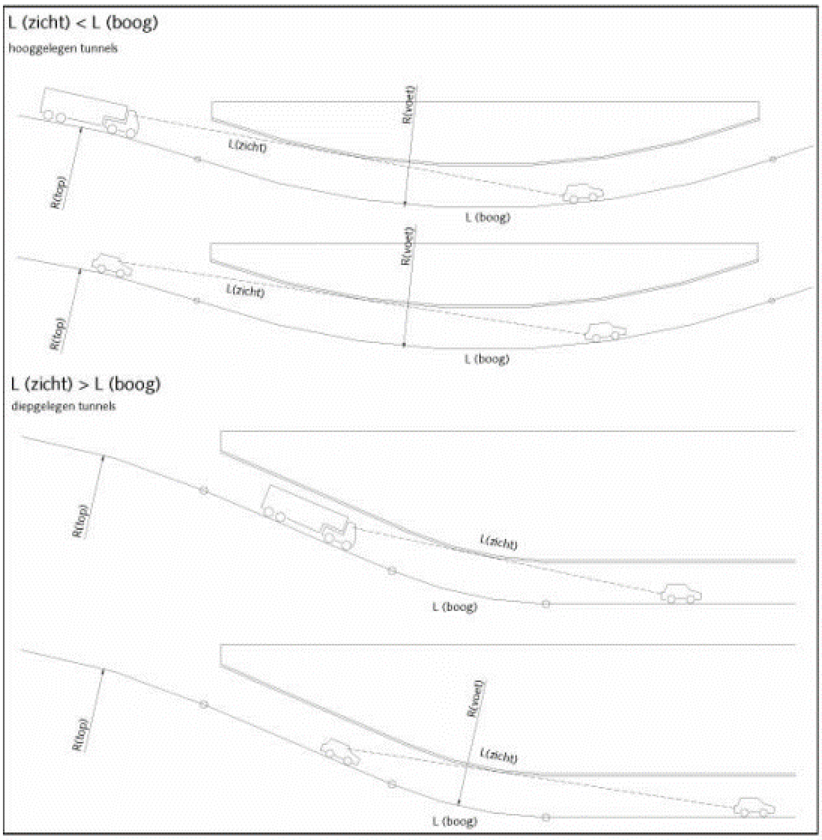
SATO:

Minimale zichtafstanden	wegverloopzicht	stopzicht	Uitwijkzicht
120 km/h	165 m	260 m	235 m
90 km/h	120 m	135 m	165 m
70 km/h	90 m	80 m	100 m
50 km/h	55 m	40 m	70 m
Objecthoogte: h0	1,10 m	1,10 m	1,10 m
Objecthoogte: hh	0 m	0,50 m	0,20 m
Minimale boogstraal: min. Rbol			
120 km/h	12400 m	11000 m	12300 m
90 km/h	6500 m	3000 m	6000 m
70 km/h	3700 m	1000 m	2200 m
50 km/h	1400 m	260 m	1100 m

Tabel 2.3.4 Minimale boogstralen bolle bogen bij 'standaard' prt

Design speed	km/h	120	100	80	50
Lz	m	165	135	105	45
h0	m	1.1	1.1	1.1	1.1
hh	m	0	0	0	0
Rmin		12375	8284.091	5011.364	920.4545
Design speed	km/h	120	100	80	50
Lz	m	260	170	105	40
h0	m	1.1	1.1	1.1	1.1
hh	m	0.5	0.5	0.5	0.5
Rmin		10962.5	4686.629	1787.892	259.4673
Design speed	km/h	120	100	80	50
Lz	m	235	190	145	70
h0	m	1.1	1.1	1.1	1.1
hh	m	0.2	0.2	0.2	0.2
Rmin		12337.57	8064.937	4697.1	1094.687
		12375	8284.091	5011.364	1094.687

Bottom arch



Afbeelding 2.3.2, Berekeningsprincipe voetbogen in tunnels

In Tabel 2.3.6 zijn de minimale stralen als functie van de plafondhoogte en de helling (p) gegeven. Bij een steilere, dalende helling neemt de minimaal gewenste zichtlengte toe, door een langere remweg. De genoemde minimale stralen zijn alleen gebaseerd op de minimale stopzichtlengte. Er is dus geen rekening gehouden met comfort- en wegbeeldeisen.

plafondhoogte	ontwerpsnelheid	min. R_{hol} (stopzicht)		
		p=2%	p=4%	p=6%
4,5 m	120 km/h	2280 m	2530 m	2830 m
	90 km/h	780 m	840 m	910 m
	70 km/h	250 m	270 m	290 m
	50 km/h	80 m	80 m	90 m
5,0 m	120 km/h	2000 m	2220 m	2480 m
	90 km/h	660 m	710 m	770 m
	70 km/h	220 m	230 m	240 m
	50 km/h	70 m	70 m	70 m

Tabel 2.3.6 Minimale boogstralen voetbogen in tunnels

Voetbogen bij kunstwerken die het zicht belemmeren

Uitsluitend bij toepassing van een voetboog in een tunnel is het toegestaan de voetboog alleen op comfort te ontwerpen. Wel is aanvullend hierop een toets noodzakelijk voor de zichtbaarheid van de belijning en eventueel aanwezige informatiedragers direct na het kunstwerk.

Bij voetbogen mag uit het oogpunt van rijcomfort de toename van de verticale versnelling niet meer dan 1,0 m/s^2 zijn. De minimale eisen aan de straal van de voetboog die hieruit voortvloeien zijn:

Tabel 3-16
Minimale eisen aan de straal van de voetboog uit het oogpunt van rijcomfort m.b.t. de toename van de verticale versnelling

ontwerpsnelheid	straal (m)
120 km/h	≥ 1.200
100 km/h	≥ 850
80 km/h	≥ 500
50 km/h	≥ 200

Design speed	km/h	120	100	80	50
bottom arch	m	1200	850	500	200

Height profile

SATO:

Ontwerp-snelheid	Ontwerpvoertuig	Voertuig-hoogte	Marge	Object-afstand	Profiel van vrije ruimte, Hoogte
120 km/h	Personenauto	2,06 m	0,20 m	-	-
90 km/h	Personenauto	2,06 m	0,20 m	-	-
80 km/h	Vrachtauto	4,00 m	0,20 m	0,30 m	4,50 m

Tabel 2.3.9 Profielen van vrije ruimte, breedte (boven) en hoogte (onder)

Appendix C: Navigation channel background

The dimensions of the largest desired vessels to be using the waterway, result in the dimensions for the navigation channel. As this channel should not be blocked, this imposes functional requirements for the tunnel design. The depth and the width of the shipping channel therefore results in minimum depth requirements for the tunnel.

Depth

The required depth of a navigation channel is a sum of 3 types of factors: Water level related factors, ship related factors and bottom related factors. These factors can be very time dependent, making it hard to estimate the required depth over a large period.

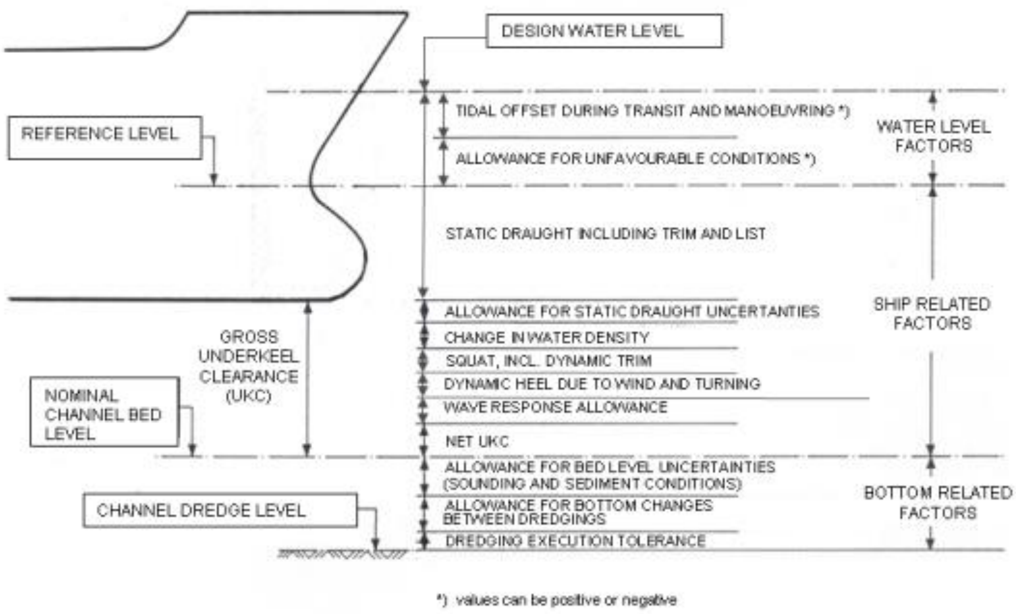


Figure C.1 - Channel depth factors

(PIANC, harbour approach channel design guidelines, 2014)

Water level related factors

The minimum water level, which is far from constant, is dependent on the following factors:

- The local mean water level which is relative to a certain standard date, in the Netherlands this date is NAP and it more or less equals the mean sea level.
- Tidal variations could exist if the waterway is directly connected to sea.
- Discharge variations could exist in rivers.
- Human induced variations could exist in canals.
- Wind induced variations like wind waves or set-up can exist.
- Local effects due to local shapes and flows can exist.

For all these factors it should be reminded that for a local water level change to have an effect on the height of a vessel, it should be spread over a distance that is large enough for the vessel to be affected. Wind waves with wavelengths smaller than the vessel for example would therefore have no effect on the height of the ship. They can however influence the movement of the ship, but this is a ship related factor.

Predicting the minimum water level by finding these factor is to some extent possible but is usually not the preferred method. Water level predictions are more often made empirically, by making use of historic data. By looking at previous extreme water levels and their rate of occurrence at a certain location an estimation can be given for the probability of occurrence for each water level in a certain period.

However, long term effects should be included in this data research method as they would affect the long term trends of the water level which is not included when only using historic data. It should however kept in mind that the impact of these effects could differ for each local situation. Examples of such effects are:

- Sea level rise
- Regional subsidence
- Local settlement

Ship related factors

Ship related factors cover all influences a ship could have on the height of its deepest point relative to the water level. It consists of the static draught and the gross under keel clearance.

The static draught is maximum distance from the deepest point of the vessel to the water level in static conditions. It can be found using the Archimedes principle[10]. As the weight of a ship is very dependent on its load, the static draught is found if the ship is fully loaded and it should include the influence of rotations of the ship, trim and list, following from asymmetrical loading. The value of the static draught is unaffected by local conditions and is therefore given by the ships designers.

The gross under keel clearance consists of all factors that are affected by local conditions. These factors are:

- Water density, which is influenced by atmospheric pressure, salinity and temperature and therefore differs globally as well as locally
- Squat, which is a steady downward displacement consisting of a translation and rotation due to the flow of water past the moving hull (pianc, 2014). It includes sinkage and trim.
- Dynamic heel, which is the effect of rotations following from turning a vessel
- Wave response allowance, which is the effect of waves induced forces on the positioning of a ship.
- Safety margins, which includes the allowance for uncertainties and the net under keel clearance.

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[10] *"Any object, wholly or partially immersed in a fluid, is buoyed up by a force equal to the weight of the fluid displaced by the object."* – Archimedes of Syracuse

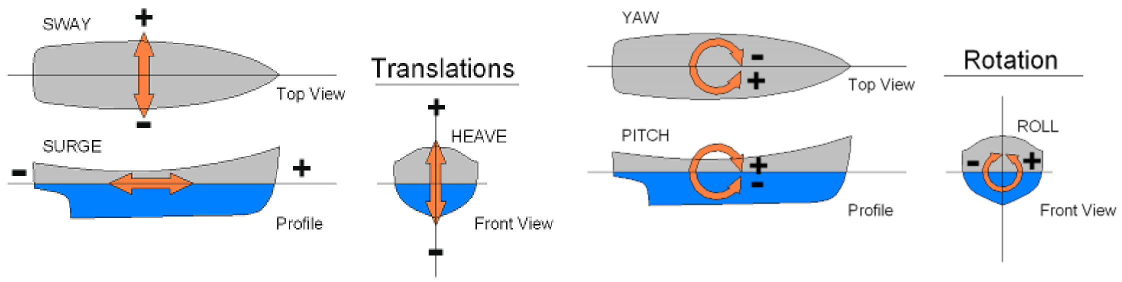


Figure C.2 - Expressions for the translation and rotation of vessels.

Bottom related factors

The third group of factors cover all influences of the bottom uncertainties. The top level of this group is the nominal channel bed level, which gives the guaranteed depth. The actual bottom level should therefore never exceed this level. To accomplish this, a number of factors are taken for channel bottom uncertainties:

- Allowance for bed level uncertainties due to measuring error
- Allowance for bottom changes between dredging due to sedimentation and siltation
- Dredging execution tolerance
- Additional allowance for muddy channel beds

Width

The channel width depends on factors dealing with manoeuvring uncertainties and waterway traffic flow:

Basic manoeuvrability

Between the initiation of a manoeuvre by the ship-handler and the execution of the manoeuvre by the ship, some delay exists. Because of this, the ship-handler is constantly adjusting its own actions and therefore, the ships track is not exactly straight, but rather sinusoidal.

The amount of deviation from the normal depends on all factors influencing the manoeuvring abilities of both the ship-handler and the ship:

- The ability of the ship handler
- The visual information available for the ship handler
- The inherent manoeuvrability of the ship

Additional effects

The negative influence on manoeuvrability can be increased by a number of additional effects. These effects could be environmental or human induced.

- Wind
- Currents
- Waves
- Depth/draught ratio
- Bottom surface
- Aids to navigation

Appendix D: Navigation calculations

Channel alignment

BT1

General input				
Main axis				
Rv	4000	m		
Rh	1300	m		
slope at bend	0.045423	rad		
effect bending				
distance bend/bend				
horizontal				
following axis	363.2613	m		
(angle)	0.279432	rad	section of Rh circle	
straight	362.0806	m		
relative:	**0.99675**			
thus:	ignore			
effect anlge with channel				
min angle	28	degree		
max angle	32	degree		
mean angle	30	degree		
	0.523599	rad		
distance bend/bend				
straight	362.0806	m		
perp. to channel	313.571	m		
relative	**0.866025**			
thus:	important!			
Effect width of tunnel				
Tunnel width	23.9	m		
effect tunnel width	5.975	m		
(Per side)				
perp. to channel	301.621	m		
relative	**0.961891**			
combined effects:				
perp. Dist. Comb.	301.621			
relative	**0.830314**			

BT2

General input			
Main axis			
Rv	3200	m	
Rh	1238.075	m	
x (over axis)	281.328	m	
angle	0.088029	rad	
Metro axis			
Rv	3150	m	
x (over axis)	249.802	m	
	0.079386	rad	
effect bending			
slope at bend	0.043972		
distance bend/bend			
horizontal			
following axis	281.3278	m	
(angle)	0.22723	rad	section of Rh circle
straight	280.7229	m	
relative:	**0.99785**		
thus:	ignore		
effect anlge with channel			
min angle	29	degree	
max angle	35	degree	
mean angle	32	degree	
	0.558505	rad	
distance bend/bend			
following axis	281.3278	m	
perp. to channel	238.5795	m	
relative	**0.848048**		
thus:	important!		
Effect width of tunnel			
Tunnel width	45.25	m	
effect tunnel width	11.98942	m	
(Per side)			
perp. to channel	214.6007	m	
relative	**0.899493**		
combined effects:			
perp. Dist. Comb.	214.6007		
relative	**0.762814**		

Ship proportions

Post panamax

DWT (t)	Δ m (t)	L oa (m)	L pp (m)	B (m)	T (m)	C B (-)	Min. Lateral Windage: Fully Loaded (m 2)	Max. Lateral Windage: In Ballast (m 2)	Approx. Capacity: TEU / CEU	B/T
245,000	340,000	470	446	60	18	0.69	11,000	12,500	22,000	3.3333
200,000	260,000	400	385	59	16.5	0.68	10,700	12,000	18,000	3.5758
195,000	250,000	418	395	56.4	16	0.68	10,100	11,300	14,500	3.525
165,000	215,000	398	376	56.4	15	0.66	9,500	10,500	12,200	3.76
125,000	174,000	370	351	45.8	15	0.7	8,700	9,500	10,000	3.0533
120,000	158,000	352	335	45.6	14.8	0.68	8,000	8,700	9,000	3.0811
110,000	145,000	340	323	43.2	14.5	0.7	7,200	7,800	8,000	2.9793
100,000	140,000	326	310	42.8	14.5	0.71	6,900	7,500	7,500	2.9517
90,000	126,000	313	298	42.8	14.5	0.66	6,500	7,000	7,000	2.9517
80,000	112,000	300	284	40.3	14.5	0.66	6,100	6,500	6,500	2.7793
70,000	100,000	280	266	41.8	13.8	0.64	5,800	6,100	6,000	3.029
65,000	92,000	274	260	41.2	13.5	0.62	5,500	5,800	5,600	3.0519
60,000	84,000	268	255	39.8	13.2	0.61	5,400	5,700	5,200	3.0152
55,000	76,500	261	248	38.3	12.8	0.61	5,200	5,500	4,800	2.9922
										3.1485

Panamax:

60,000	83,000	290	275	32.2	13.2	0.69	5,300	5,500	5,000	2.4394
55,000	75,500	278	264	32.2	12.8	0.68	4,900	5,100	4,500	2.5156
50,000	68,000	267	253	32.2	12.5	0.65	4,500	4,700	4,000	2.576
45,000	61,000	255	242	32.2	12.2	0.63	4,150	4,300	3,500	2.6393
40,000	54,000	237	225	32.2	11.7	0.62	3,750	3,900	3,000	2.7521
35,000	47,500	222	211	32.2	11.1	0.61	3,550	3,700	2,600	2.9009
30,000	40,500	210	200	30	10.7	0.62	3,350	3,500	2,200	2.8037
25,000	33,500	195	185	28.5	10.1	0.61	2,900	3,000	1,800	2.8218
20,000	27,000	174	165	26.2	9.2	0.66	2,400	2,500	1,500	2.8478
15,000	20,000	152	144	23.7	8.5	0.67	2,000	2,100	1,100	2.7882
10,000	13,500	130	124	21.2	7.3	0.69	1,800	1,900	750	2.9041
										2.7263

source: pianc harbour approach channels design guidelines (2014) Table C-1: Typical ship dimensions from ROM 3.1 (Continued)

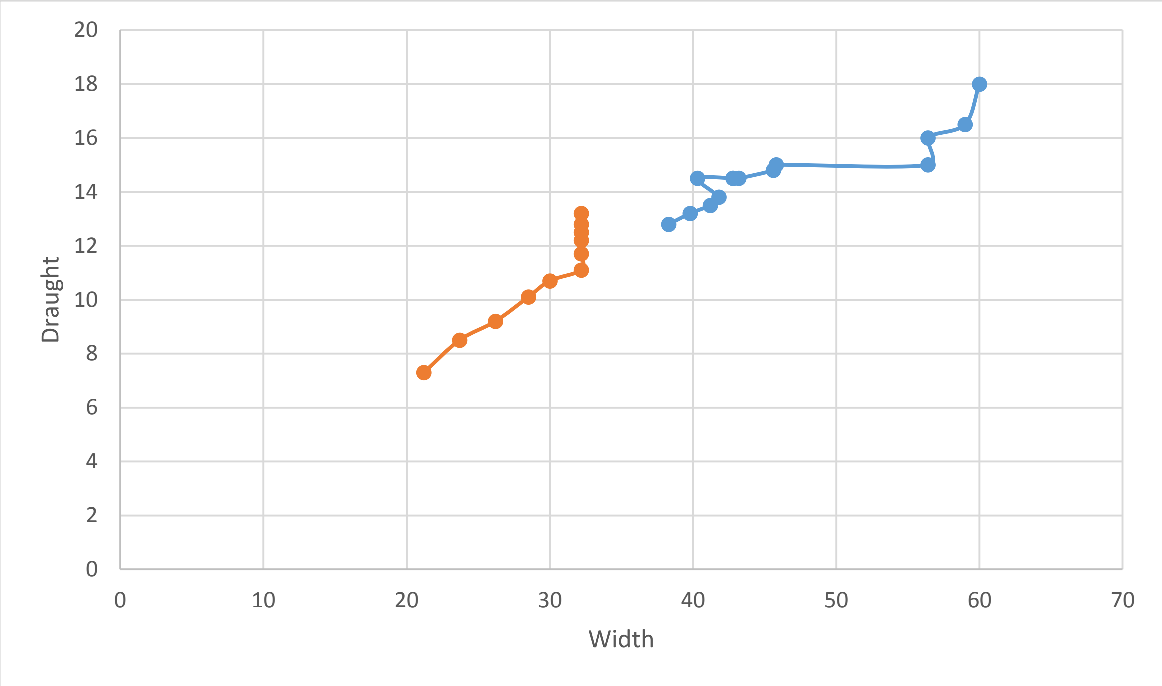


Figure D.1: Proportions large-scale container vessels (orange: Panamax, blue: Post-Panamax)

Channel depth

calculating design vessel according to current standards

Water level related factors

LLWS	-0.7	m	NAP

Sealevelrise

year	Min	Max
2000	0	0
2050	0.16	0.36
2100	0.36	1.2

Thus:

	LLWS		
2000	-0.7	m	NAP
2050	-0.54	m	NAP
2100	-0.34	m	NAP

Ship related factors

1st estimation

D= 1.1 T

Actual depth

	Geometric constraints shipping channel [m]		
	Width	Min depth (vs NAP)	Max depth (vs NAP)
BT1	275	-13	-16
BT2	273.92	- 13.34 / - 13.86	-16.5

D = 12.3 m

T = 11.18182 m

Channel width

1 **calculating design vessel according to current standards**

all figures from pianc

1-way

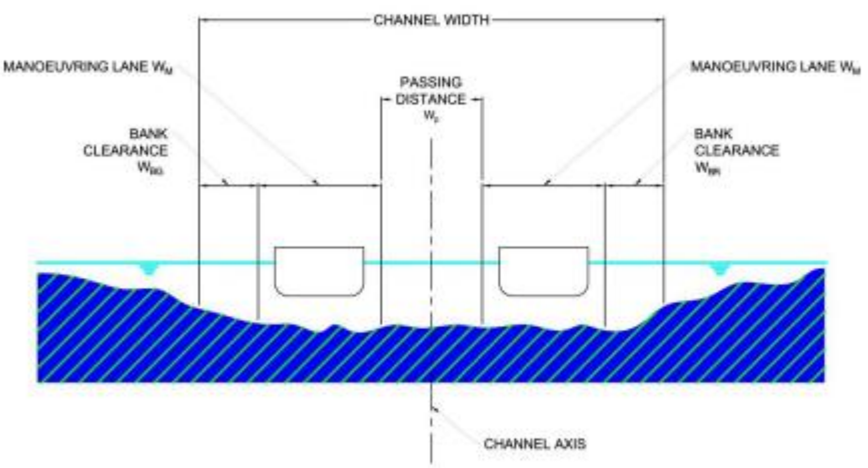
$$W = W_{BM} + \sum W_i + W_{BR} + W_{BG} = W_M + W_{BR} + W_{BG}$$

2-way

$$W = 2W_{BM} + 2\sum W_i + W_{BR} + W_{BG} + \sum W_p = 2W_M + W_{BR} + \sum W_P + W_{BG}$$

Wbm

Ship Manoeuvrability	Good	Moderate	Poor
Basic Manoeuvring Lane, W_{BM}	1.3 B	1.5 B	1.8 B



Wbm= 1.5 B

wi moderate speed

a	0	
b	0.2	
c	0	
d	0	*stroomsnelheid opzoeken
e	0	*waveheight
f	0	
g	0.2	
h	0.4	
i	0	

Wi= 0.8 B

Width for passing distance W_p	Outer Channel (open water)	Inner Channel (protected water)
Vessel speed V_s (knots)		
- fast: $V_s \geq 12$	2.0 B	1.8 B
- moderate: $8 \leq V_s < 12$	1.6 B	1.4 B
- slow: $5 \leq V_s < 8$	1.2 B	1.0 B

Table 3.7: Additional width for passing distance in two-way-traffic W_p

Wbr

Wbr= 0.5 B

Wbg

=Wbr

Wbg= 0.5 B

Wp

Wp= 1.4 B

Current widths:

W= 7 B

W= 225 m --> B = 32.14286 m --> PANAMAX

W= 275 m --> B = 39.28571 m

for 1 way channel:

W= 3.3 B

W= 125 --> B = 37.87879 M

Width for bank clearance (W_{BR} and/or W_{BG})	Vessel Speed	Outer channel (open water)	Inner channel (protected water)
Gentle underwater channel slope (1:10 or less steep)	fast moderate slow	0.2 B 0.1 B 0.0 B	0.2 B 0.1 B 0.0 B
Sloping channel edges and shoals	fast moderate slow	0.7 B 0.5 B 0.3 B	0.7 B 0.5 B 0.3 B
Steep and hard embankments, structures	fast moderate slow	1.3 B 1.0 B 0.5 B	1.3 B 1.0 B 0.5 B
Note: W_{BR} and W_{BG} are widths on 'red' and 'green' sides of channel			

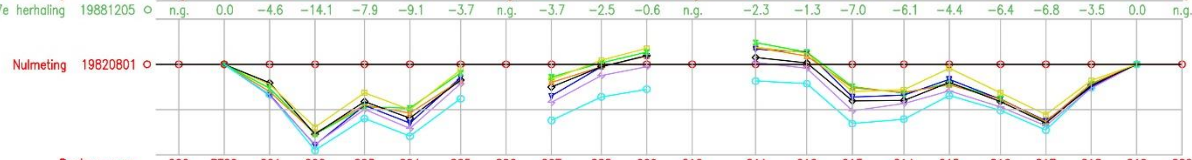
Table 3.6: Additional width for bank clearance W_{BR} and W_{BG}

Appendix E: Settlement

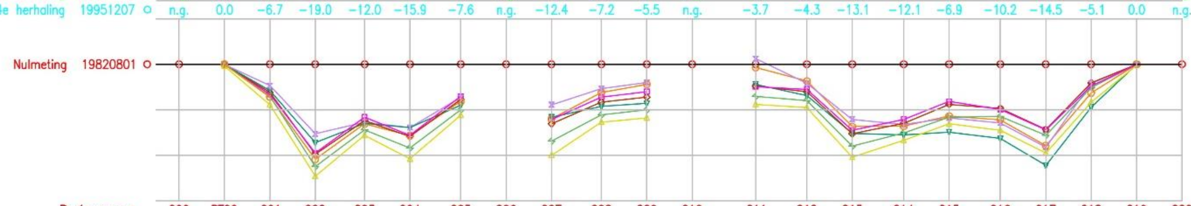
1st Beneluxtunnel[75]

Z-axis [mm]

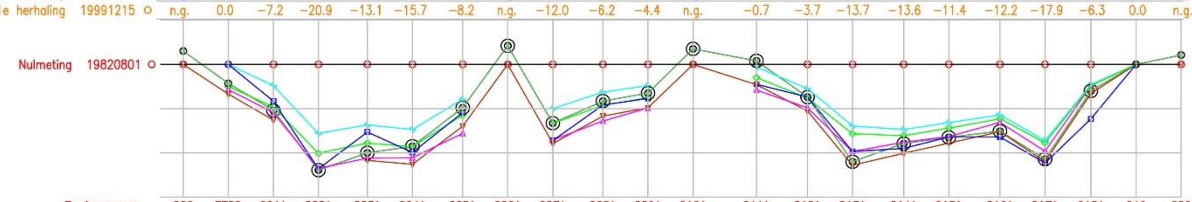
Puntnummers		200	BT20	201	202	203	204	205	206	207	208	209	210	211	212	213	214	215	216	217	218	219	220
1e herhaling	19830215	n.g.	0.2	−1.9	−4.1	1.1	−3.6	2.0	n.g.	NULM	1.1	3.6	n.g.	n.g.	2.8	−2.7	−0.3	2.5	0.5	−0.3	−1.8	0.0	n.g.
2e herhaling	19831115	n.g.	0.0	−0.9	−2.5	−0.4	−2.3	0.4	n.g.	2.0	0.3	2.1	n.g.	NULM	0.3	−0.8	−1.5	−0.9	−0.9	−1.1	−0.6	0.0	n.g.
3e herhaling	19841215	n.g.	0.0	−2.7	−7.0	−3.2	−4.0	−0.8	n.g.	1.0	0.0	2.1	n.g.	−0.9	0.6	−2.7	−3.9	−1.7	−4.0	−3.8	−1.7	0.0	n.g.
4e herhaling	19851211	n.g.	0.0	−3.8	−11.5	−6.3	−7.3	−3.2	n.g.	−3.4	−3.0	−1.3	n.g.	−2.8	−1.4	−6.6	−6.5	−4.0	−6.5	−6.3	−3.8	0.0	n.g.
5e herhaling	19861010	n.g.	0.0	−1.5	−7.6	−6.2	−6.1	−2.3	n.g.	−1.0	−1.1	−0.1	n.g.	−0.8	−1.1	−4.4	−4.9	−4.1	−5.5	−5.8	−1.8	0.0	n.g.
6e herhaling	19871215	n.g.	0.0	−6.0	−16.2	−9.5	−13.3	−6.9	n.g.	−10.1	−6.5	−4.5	n.g.	−6.3	−4.7	−11.6	−9.7	−6.4	−8.5	−8.2	−4.5	0.0	n.g.
7e herhaling	19881205	n.g.	0.0	−4.6	−14.1	−7.9	−9.1	−3.7	n.g.	−3.7	−2.5	−0.6	n.g.	−2.3	−1.3	−7.0	−6.1	−4.4	−6.4	−6.8	−3.5	0.0	n.g.



Puntnummers		200	BT20	201	202	203	204	205	206	207	208	209	210	211	212	213	214	215	216	217	218	219	220
8e herhaling	19900104	n.g.	0.0	−4.4	−13.9	−6.3	−10.0	−1.2	n.g.	−3.3	0.9	3.5	n.g.	3.8	2.5	−6.0	−5.6	−0.9	−6.3	−11.1	−3.6	0.0	n.g.
9e herhaling	19901212	n.g.	0.0	−4.1	−15.4	−8.2	−11.9	−3.5	n.g.	−5.0	−0.5	1.9	n.g.	1.5	0.3	−8.1	−8.0	−4.0	−8.2	−13.0	−4.7	0.0	n.g.
10e herhaling	19911217	n.g.	0.0	−6.9	−17.7	−9.9	−14.2	−4.3	n.g.	−8.4	−2.5	−0.5	n.g.	0.4	−0.9	−10.3	−8.7	−5.9	−9.4	−13.6	−5.3	0.0	n.g.
11e herhaling	19921111	n.g.	0.0	−6.0	−15.2	−9.0	−10.8	−3.5	n.g.	−4.0	−0.5	1.8	n.g.	3.7	1.8	−5.2	−6.2	−4.5	−7.7	−12.6	−4.3	0.0	n.g.
12e herhaling	19931125	n.g.	0.0	−6.7	−17.8	−9.1	−13.0	−2.9	n.g.	−7.0	−0.5	1.8	n.g.	3.4	2.7	−7.2	−6.8	−3.3	−7.7	−12.5	−5.0	0.0	n.g.
13e herhaling	19941130	n.g.	0.0	−5.3	−15.4	−9.4	−9.7	−1.9	n.g.	−2.8	0.3	2.7	n.g.	4.8	2.7	−4.9	−6.4	−4.2	−7.6	−12.9	−4.4	0.0	n.g.
14e herhaling	19951207	n.g.	0.0	−6.7	−19.0	−12.0	−15.9	−7.6	n.g.	−12.4	−7.2	−5.5	n.g.	−3.7	−4.3	−13.1	−12.1	−6.9	−10.2	−14.5	−5.1	0.0	n.g.

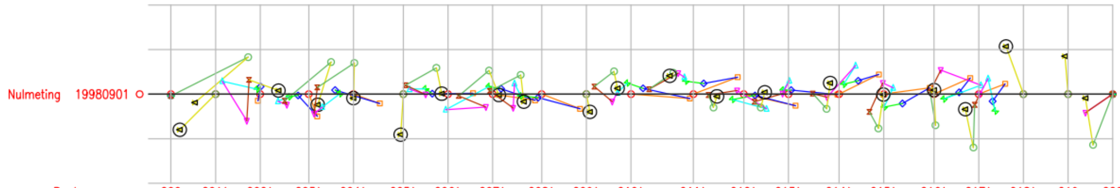


Puntnummers		200	BT20	201	202	203	204	205	206	207	208	209	210	211	212	213	214	215	216	217	218	219	220
15e herhaling	19961209	n.g.	0.0	−6.6	−19.5	−11.6	−15.6	−7.1	n.g.	−12.1	−7.2	−6.0	n.g.	−4.9	−5.5	−14.5	−12.1	−8.2	−9.9	−14.4	−4.8	0.0	n.g.
16e herhaling	19971210	n.g.	0.0	−6.2	−19.8	−12.4	−15.9	−7.7	n.g.	−13.1	−8.3	−7.2	n.g.	−4.8	−6.0	−15.3	−13.0	−8.8	−9.7	−14.3	−4.1	0.0	n.g.
17e herhaling	19981215	n.g.	0.0	−7.1	−22.5	−14.4	−18.4	−10.0	n.g.	−16.9	−11.1	−10.0	n.g.	−7.1	−8.0	−18.0	−15.1	−11.6	−11.5	−15.6	−5.1	0.0	n.g.
18e herhaling	19990214	n.g.	0.4	−8.9	−24.6	−15.6	−20.8	−11.2	n.g.	−20.0	−12.7	−11.8	n.g.	−8.8	−9.5	−20.4	−16.7	−13.1	−14.5	−19.5	−8.3	−0.2	n.g.
19e herhaling	19990615	n.g.	0.0	−5.8	−17.2	−13.0	−13.9	−9.0	n.g.	−11.6	−9.3	−8.6	n.g.	−4.4	−6.9	−15.2	−15.5	−14.9	−16.3	−22.2	−9.4	0.0	n.g.
20e herhaling	19991004	n.g.	0.0	−4.7	−15.3	−12.7	−13.9	−7.3	n.g.	−8.9	−5.4	−4.0	n.g.	1.2	−4.2	−12.1	−13.2	−11.8	−12.9	−18.2	−4.2	0.0	n.g.
21e herhaling	19991215	n.g.	0.0	−7.2	−20.9	−13.1	−15.7	−8.2	n.g.	−12.0	−6.2	−4.4	n.g.	−0.7	−3.7	−13.7	−13.6	−11.4	−12.2	−17.9	−6.3	0.0	n.g.



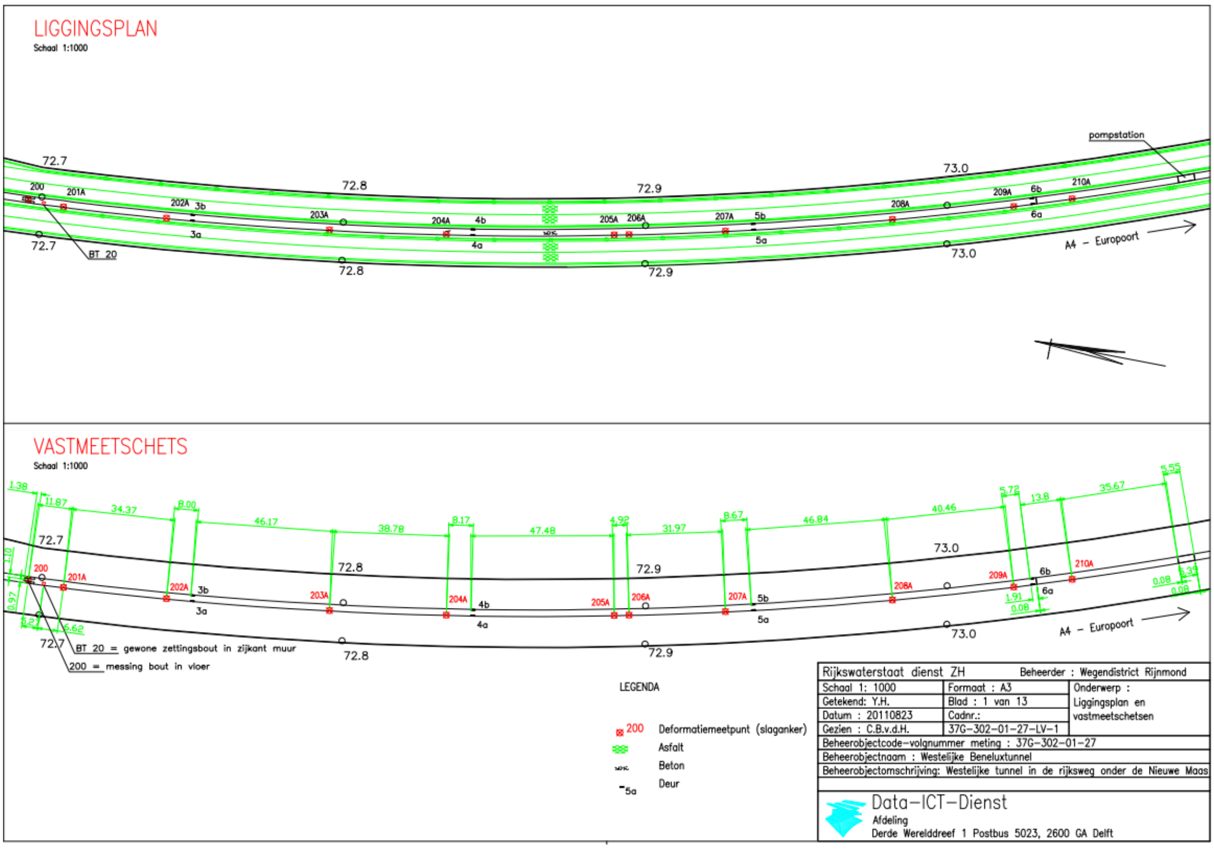
Puntnummers		200	BT20	201A	202A	203A	204A	205A	206A	207A	208A	209A	210A	211A	212A	213A	214A	215A	216A	217A	218A	219	220
22e Herhalingsmeting	20000315	n.g.	0.0	−8.3	−23.6	−15.3	−19.9	−10.9	n.g.	−17.2	−9.2	−7.7	n.g.	−4.5	−7.4	−19.7	−19.0	−16.4	−16.4	−22.3	−12.3	0.0	n.g.
23e Herhalingsmeting	20000928	n.g.	−5.0	−9.6	−20.1	−17.8	−18.6	−11.5	n.g.	−13.3	−9.3	−7.5	n.g.	−3.0	−7.6	−15.7	−16.1	−14.4	−12.3	−17.7	−4.8	0.0	n.g.
24e Herhalingsmeting	20001015	n.g.	0.0	−4.8	−15.6	−13.7	−14.7	−7.9	n.g.	−10.0	−6.3	−4.8	n.g.	−0.6	−5.5	−13.9	−14.7	−13.2	−11.4	−17.2	−4.6	0.0	n.g.
25e Herhalingsmeting	20010921	n.g.	−5.7	−10.9	−23.5	−21.2	−21.1	−15.6	n.g.	−17.1	−12.8	−9.9	n.g.	−5.8	−9.8	−19.7	−17.6	−16.3	−13.1	−19.6	−4.8	0.0	n.g.
26e Herhalingsmeting	20081008	nulm	−6.7	−12.6	n.g.	−21.7	−22.6	−14.0	nulm	−17.9	−11.7	−9.8	0.0	−4.6	−10.5	−22.8	−20.1	−17.8	−15.2	−22.0	−6.6	0.0	nulm
27e Herhalingsmeting	20110816	3.0	−4.3	−10.4	−23.9	−20.0	−18.5	−9.8	4.2	−13.3	−8.3	−6.5	3.5	0.8	−7.4	−22.0	−17.9	−16.5	−15.1	−21.4	−6.0	0.0	2.1
27e Herhalingsmeting	20110823	3.0	−4.3	NULM	NULM	NULM	NULM	NULM	NULM	NULM	NULM	NULM	NULM	NULM	NULM	NULM	NULM	NULM	NULM	NULM	NULM	0.0	2.1

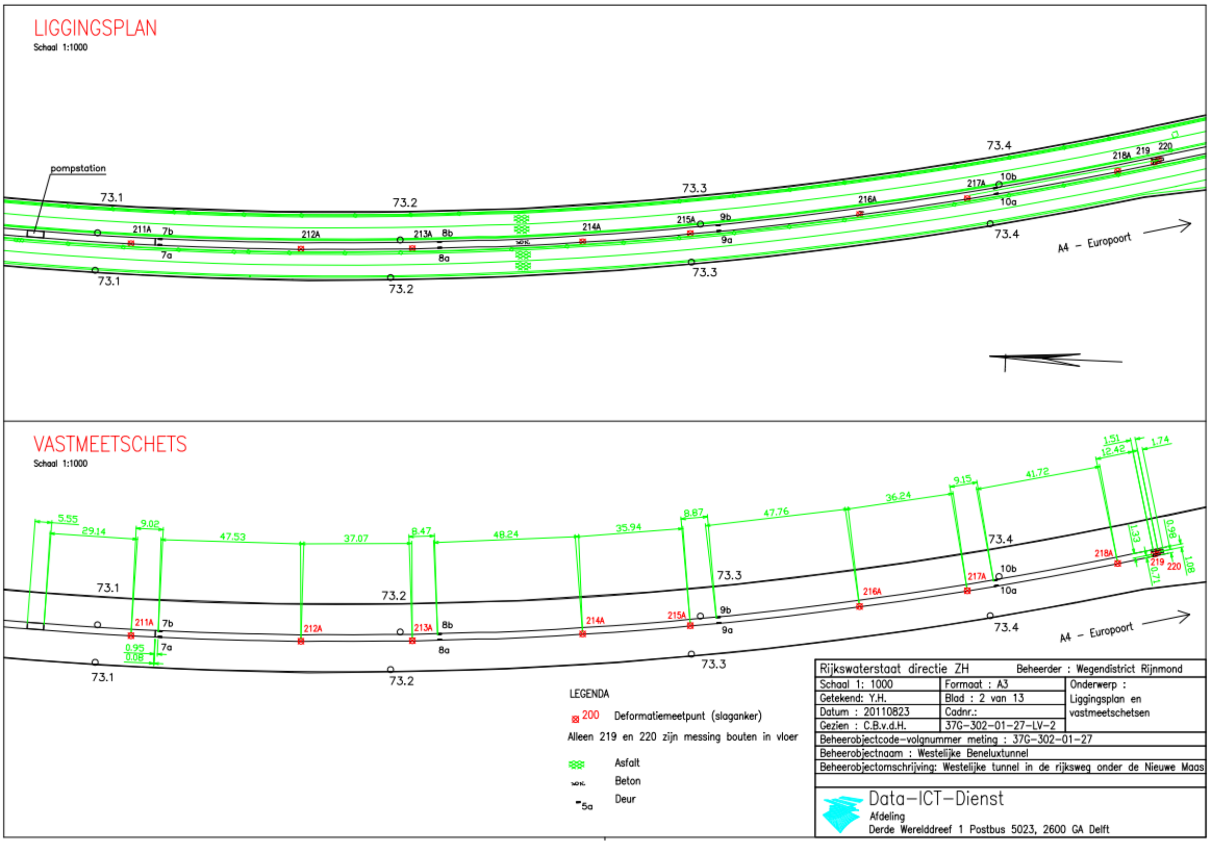
X/Y-axis [mm]



Puntnummers		200	201A	202A	203A	204A	205A	206A	207A	208A	209A	210A
1e Herhalingsmeting	19990214	0/0	n.g.	−1/−1	2/−5	6/−2	n.g.	6/0	10/−1	9/−3	n.g.	13/−1
2e Herhalingsmeting	19990615	0/0	n.g.	0/2	−2/0	−4/1	n.g.	−4/0	2/1	−1/−1	n.g.	3/1
3e Herhalingsmeting	19991015	0/0	n.g.	−1/1	−4/−1	−3/0	n.g.	−5/1	−1/2	−5/0	n.g.	−1/2
4e Herhalingsmeting	19991215	0/0	n.g.	−8/3	−7/−2	−7/−3	n.g.	−9/1	−11/−4	−6/2	n.g.	−3/0
5e Herhalingsmeting	20000315	0/0	n.g.	−3/−6	−5/−2	−9/−4	n.g.	−5/0	−2/−3	−6/−3	n.g.	−4/−2
6e Herhalingsmeting	20001015	0/0	n.g.	−2/3	−5/−1	−8/2	n.g.	−10/2	−8/0	−12/1	n.g.	−8/2
7e Herhalingsmeting	20081007	17/8	nulm	n.g.	5/7	0/7	nulm	−3/6	−1/5	−5/4	nulm	−4/5
8e Herhalingsmeting	20110816	5/−2	−8/−8	4/1	2/−2	0/−1	−1/−9	−1/0	1/0	−4/−2	1/−4	−3/1
8e Herhalingsmeting	20110823	5/−2	NULM	NULM	NULM	NULM	NULM	NULM	NULM	NULM	NULM	NULM

Puntnummers		211A	212A	213A	214A	215A	216A	217A	218A	219	220
1e Herhalingsmeting	19990214	10/4	12/−3	11/0	9/4	10/1	8/4	6/2	n.g.	n.g.	0/0
2e Herhalingsmeting	19990615	2/2	3/0	1/1	4/3	4/−2	6/0	3/−2	n.g.	n.g.	0/0
3e Herhalingsmeting	19991015	−2/3	−2/−1	−1/1	1/2	0/−3	2/−1	4/−4	n.g.	n.g.	0/0
4e Herhalingsmeting	19991215	−2/4	5/−3	0/3	4/6	2/1	11/2	2/4	n.g.	n.g.	0/0
5e Herhalingsmeting	20000315	−3/5	−2/1	−1/0	−3/−1	0/2	2/5	0/0	n.g.	n.g.	−6/−4
6e Herhalingsmeting	20001015	−10/1	−8/0	−8/−2	−6/0	−3/−4	−1/1	−1/−2	n.g.	n.g.	0/0
7e Herhalingsmeting	20081007	−5/5	−7/−3	−6/−3	−3/−3	−1/−8	0/−7	−1/−12	nulm	nulm	−4/−11
8e Herhalingsmeting	20110816	−5/4	−6/0	−5/0	−2/2	0/0	0/1	−3/−3	−4/11	−1/8	−6/−1
8e Herhalingsmeting	20110823	NULM	NULM	NULM	NULM	NULM	NULM	NULM	NULM	−1/8	−6/−1

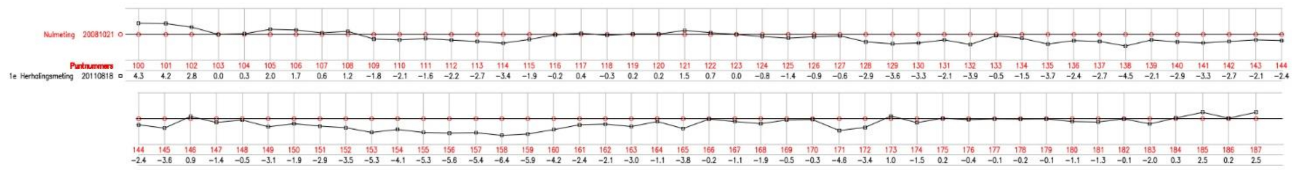




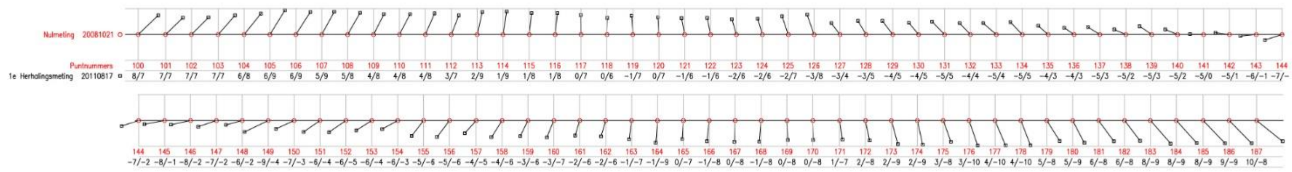
LIGGINGSPLAN
Schaal 1:1000
pompstation
73.1
73.2
73.3
73.4
211A
7b
7a
212A
213A
8b
8a
214A
215A
9b
9a
216A
217A
10b
10a
218A
219
220
A4 – Europoort
VASTMEETSCHETS
Schaal 1:1000
LEGENDA
200 Deformatiemeetpunt (slaganker)
Alleen 219 en 220 zijn messing bouten in vloer
Asfalt
Beton
5a Deur
Rijkswaterstaat directie ZH
Beheerder : Wegendistrict Rijnmond
Schaal 1: 1000
Formaat : A3
Onderwerp : Liggingsplan en vastmeetschetsen
Getekend: Y.H.
Blad : 2 van 13
Datum : 20110823
Cadnr.:
Gezien : C.B.v.d.H.
37G-302-01-27-LV-2
Beheerobjectcode-volgnummer meting : 37G-302-01-27
Beheerobjectnaam : Westelijke Beneluxtunnel
Beheerobjectomschrijving: Westelijke tunnel in de rijksweg onder de Nieuwe Maas
Data-ICT-Dienst
Afdeling
Derde Werelddreef 1 Postbus 5023, 2600 GA Delft

2nd Beneluxtunnel[76]

Z-axis [mm]



X/Y-axis [mm]



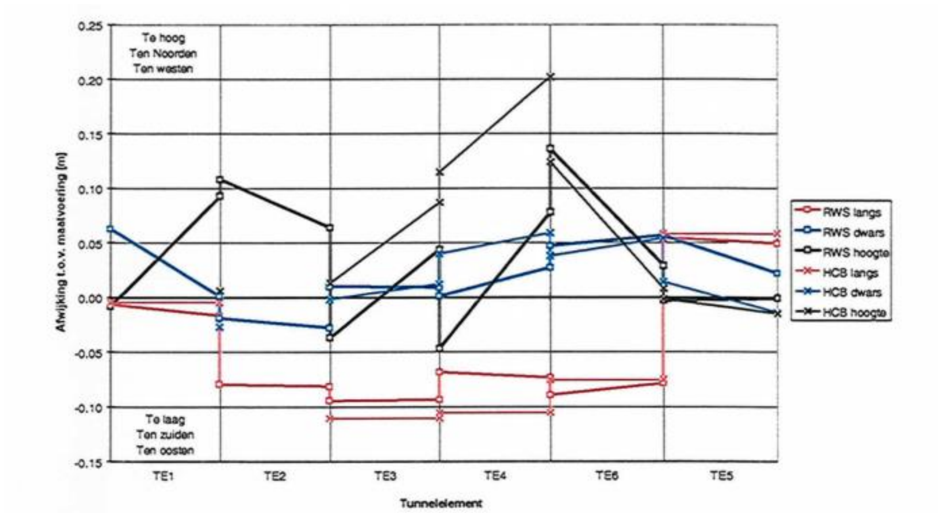


Figure E.1: zettingsgrafiek na afzinken[38]

Appendix F: Model

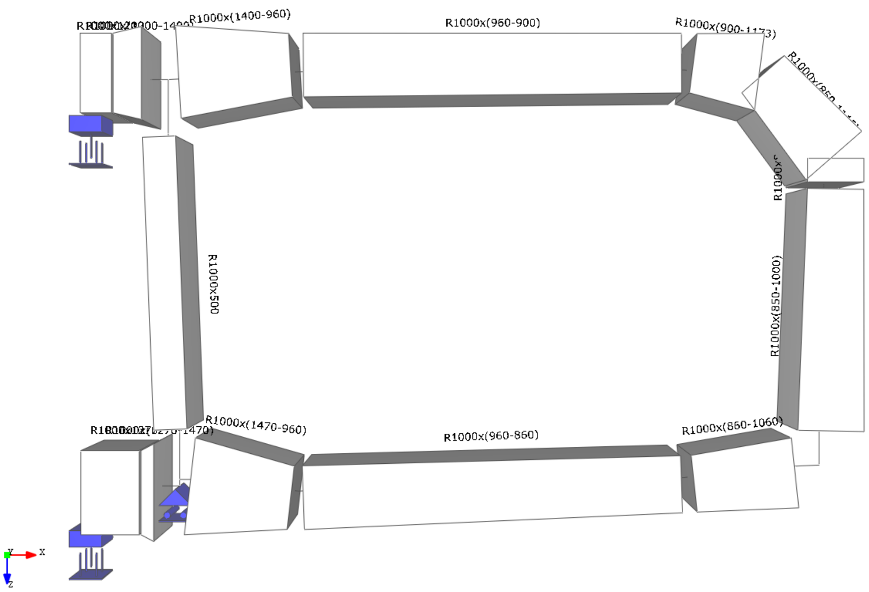
Matrixframe

Matrixframe is a frame analysis programme that uses the displacement method to determine the internal forces.

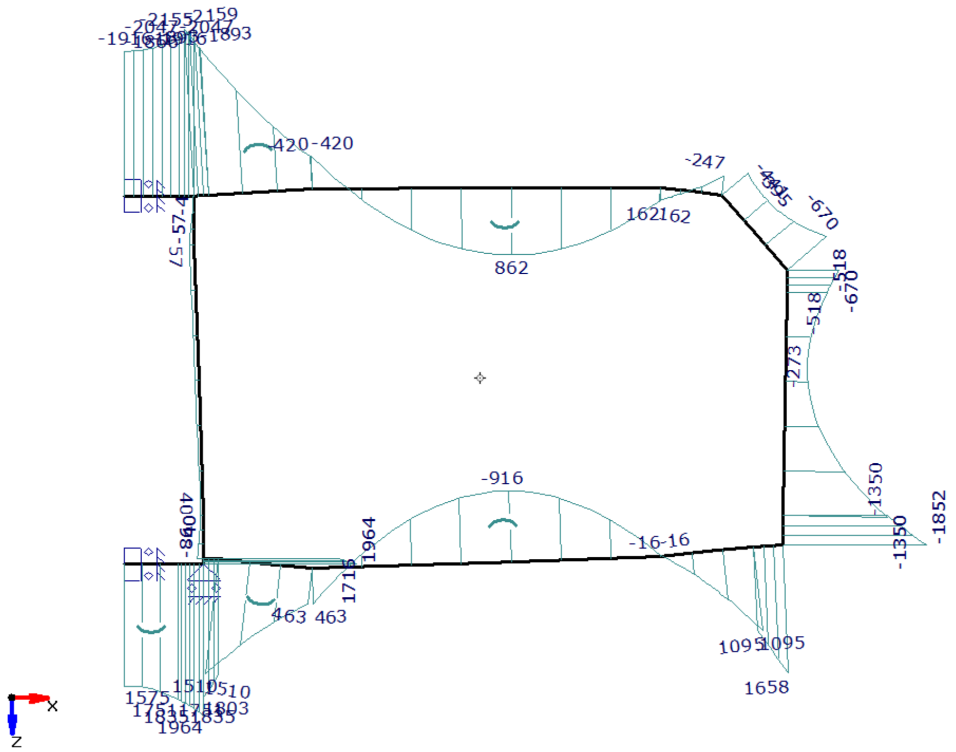
The loads are inserted in the model as described in 6.2.1. Some comments are given regarding the input of the loads:

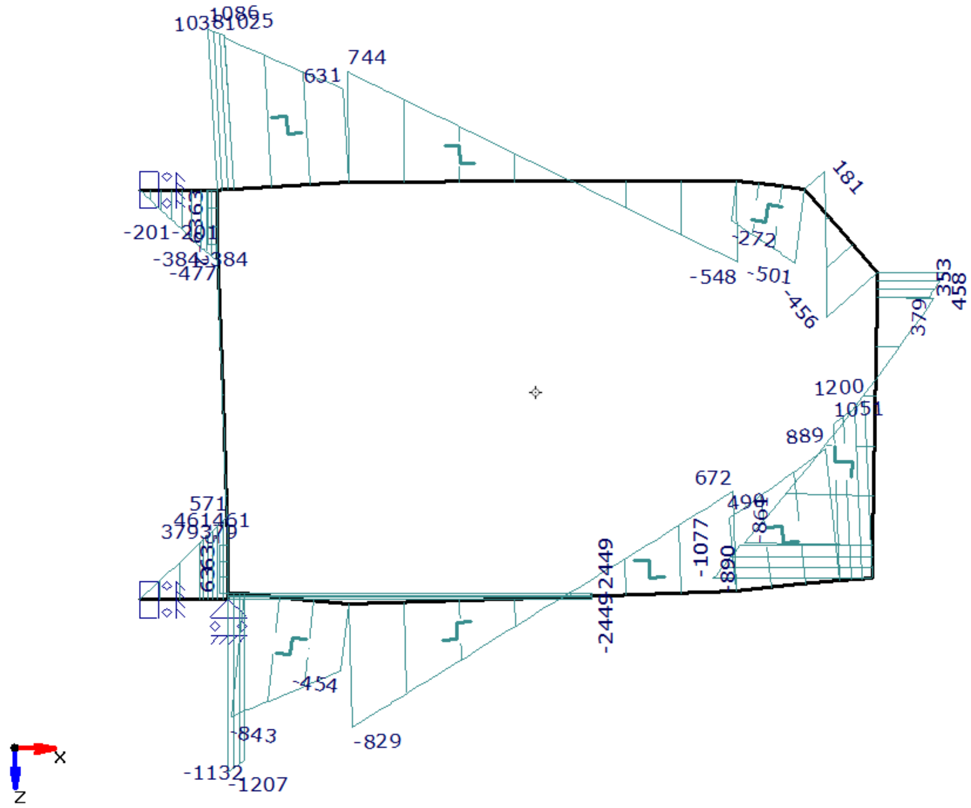
- verschil overspanning etc
- dikte
- Puntlasten hoeken
- Punt+Koppel voorspanning
- Soil resultant (33)
- q

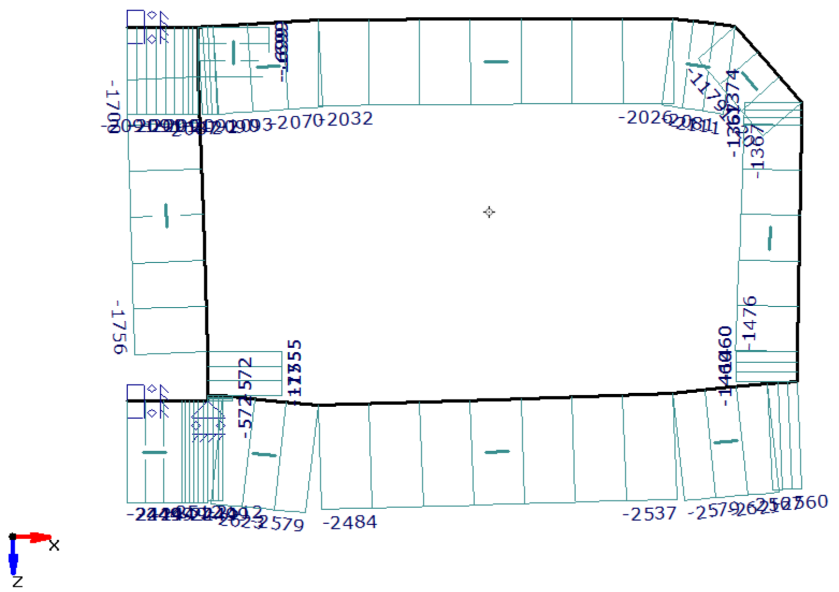
Model info



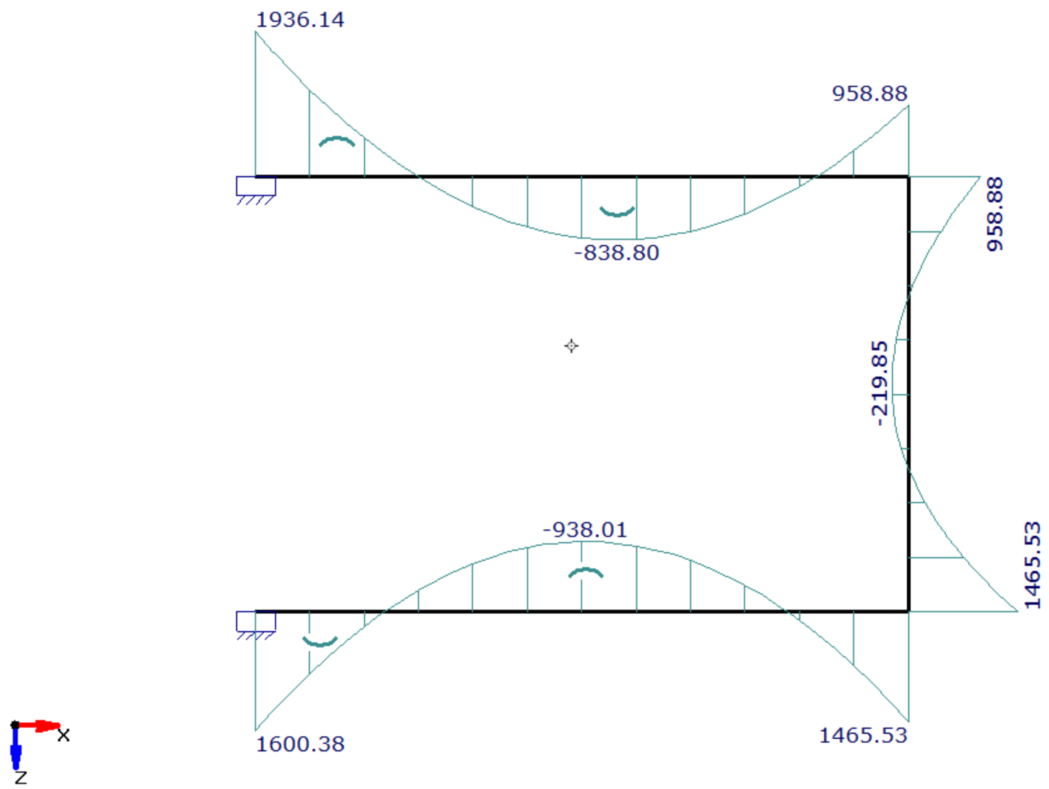
Results







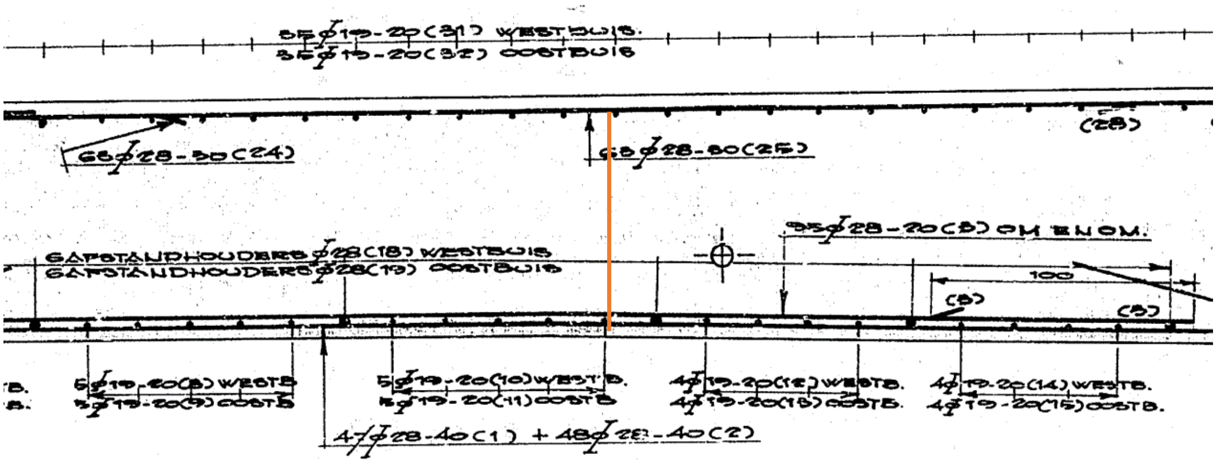
Validation model



Appendix G: Cross-section capacity

Example calculation: Point 4

important parameters:	A	Z	f.max	ε.max	E		load	
height 900 mm	mm^2	mm	N/mm^2		N/mm^2			
concrete	61309522	(X.e)	44.43333	0.001869	23774.65	N	-2038000	N
reinforcement	6157.522	827.75	341.2174	0.001625	210000	S	0	N
prestressing	1120.78	700	244.60	0.001254	195000	M	861000000	Nmm



Subject: centre roof

input

field or support (f/s)	f				
height	900	mm			
reinforcement	amount	diameter	distance	(nr)	
top.up.1					
top.up.2					
split					
top.low.1					
top.low.2					
bot.up.1	95	25	20		
bot.up.2					
split		28			
bot.low.1	47	28	40		
bot.low.2	48	28	40		
Prestress					
height from top	700	mm	(0 if not in tension zone)		

Loads					
	N	-2038	kN		
	S	0	kN		
	M	861	kNm		
Generate		N=0	X.e		
	concrete	0	297.7637		
	reinforcement	0.000205	331.3166	*use solver	
	prestressing				
Output					safety
	M.max	**2276.877**	kNm		2.644457
	V.max	**803**	kN		#DIV/0!

Input: reinforcement

	amount	diameter	distance	total dist	A.s/bar	A.s/m	z(top)	z(bot)
		mm	cm	m	mm^2/bar	mm^2/m	mm	
top.up.1	0	0	0	0	0	0	45	855
top.up.2	0	0	0	0	0	0	45	855
		0						
top.low.1	0	0	0	0	0	0	45	855
top.low.2	0	0	0	0	0	0	45	855
bot.up.1	95	28	20	19	615.7522	3078.761	785	115
bot.up.2	0	0	0	0	0	0	785	115
		28						
bot.low.1	47	28	40	18.8	615.7522	1539.38	841	59
bot.low.2	48	28	40	19.2	615.7522	1539.38	841	59
cover	45	mm						

Effective reinforcement					z(top)	z(bot)
	A.s.top	0	mm^2	/m	#DIV/0!	#DIV/0!
	A.s.bot	6157.522	mm^2	/m	827.75	72.25
	A.s.shear	6157.522	mm^2	/m		
Prestressing					z(top)	z(bot)
	A.p	1120.78	mm^2	/m	700	200

Material properties

concrete				
f.cd	44.43333	N/mm^2		
E.cme*	23774.65	N/mm^3	* E.cm elastic is less than E.cm	
ε.c3	0.001869			
reinf.				
f.yd	341.2174	N/mm^2		
E	210000	N/mm^2		
ε.smax	0.001625			
prestress				
f.pd	1107.638			
Δf.pd	244.60			
E	195000			
ε.pmax	0.00568			
Δε.pmax	0.001254			
bond factor	0.5			

Loads

N	-2038	kN	/m
	-2038000	N	/m
S	0	kN	/m
	0	N	/m
M	861	kNm	/m
	8.61E+08	Nmm	/m
width	1000	mm	

1 a

Maximum elastic deformation of concrete:

ε.c3	0.001869

Horizontal equilibrium, find x.e

a	22216.67	
b	582915.7	
c	-2.1E+09	
Xe	-324.001	
Xe	**297.7637**	mm

so:

ε.c	0.001869
ε.s	0.003327

$\Delta\varepsilon.p$	0.002525	

and

$\sigma.c$	44.4	N/mm^2
$\sigma.s$	698.6	N/mm^2
$\Delta.\sigma.p$	492.3	N/mm^2

N.c	6615317	N
N.s	4301430	
dN.p	275886.8	
Horizontal equilibrium	0	

calculation using solver:

N	2038000
Nc	6615316.85
Ns	4301430.49
ΔNp	275886.84
H	0.48193413
$\varepsilon.c$	0.00186894
$\varepsilon.s$	0.0033265
$\Delta\varepsilon.p$	0.00252467
$\sigma.c$	44.4333333
$\sigma.s$	698.56523
$\Delta.\sigma.p$	492.310275
Xe	297.763699

$\sigma.s$ > f.yd   Thus, concrete not governing

2 a

Maximum elastic deformation of steel

ε.smax	0.001625
f.yd	341.2174

Horizontal equilibrium, find x.e

N	2038000
Nc	4270919
Ns	2101053
ΔNp	131865.5
H	0.000205
ε.c	0.001084
ε.s	0.001625
Δε.p	0.001207
σ.c	25.7815
σ.s	341.2174
Δ.σ.p	235.3093
Xe	331.3166

Steel governing

Moment resistance

Z.c	340	mm
Z.s	377.75	mm
Z.p	250	mm
M.r	2.28E+09	Nmm
	2276.877	kNm

Shear:

h	900	
V.rd.c	802724.9	N
	803	kN
V.rd.c.min	685322.3	N
k	1.491547	
p.l	0.007439	
A.sl	6157.522	mm^2/m
d	828	mm
b.w	1000	mm
sig.cp	2.264444	N/mm^2
N.ed	2038000	N
A.c	900000	mm^2
f.cd	44.43333	N/mm^2
f.ck	58.65	N/mm^2
v.min	0.488267	
C.rd.c	0.12	
k.1	0.15	

Appendix H: Costs

Cost factors

	value			year	Jan-15	source
Economic factors						
Value of time	€ 9.00	per	hour	2010	€ 9.77	De maatschappelijke waarde van betrouwbaarheid - herdruk_tcm174-347760
Traffic hindrance						
Trucks	€ 85.00	per	hour	2014	€ 85.00	B.vd.Meer (Gemeente Rotterdam)
Cars	€ 11.72	per	hour	2014	€ 11.72	based on 1.2 person/car
Dredging costs						
Suction						
shallow	€ 16.00	per	m^3	2014	€ 16.00	P. Koman (Van Oord)/B.vd.Meer (Gemeente Rotterdam)
deep	€ 20.00	per	m^3	2014	€ 20.00	P. Koman (Van Oord)/B.vd.Meer (Gemeente Rotterdam)
bottom protection	€ 25.00	per	m^3	2014	€ 25.00	B.vd.Meer (Gemeente Rotterdam)
underneath elements	€ 45.00	per	m^3	2014	€ 45.00	P. Koman (Van Oord)
Maeslantkering	€ 419.34			1997	€ 597.58	
Maeslantkering	€ 401.10			1988	€ 714.59	
sheet pile						
Material	€ 1,000.00	per	ton	2014	€ 1,000.00	B.vd.Meer (Gemeente Rotterdam)
driving	€ 3,000.00	per	piece	2014	€ 3,000.00	B.vd.Meer (Gemeente Rotterdam)
anchors	€ 125.00	per	m	2014	€ 125.00	B.vd.Meer (Gemeente Rotterdam)
Demolition						
normal concrete	€ 100.00	per	m^3	2014	€ 100.00	B.vd.Meer (Gemeente Rotterdam)
underwater concrete	€ 200.00	per	m^3	2014	€ 200.00	B.vd.Meer (Gemeente Rotterdam)
concrete						
concrete	€ 150.00	per	m^3	2014	€ 150.00	B.vd.Meer (Gemeente Rotterdam)
reinforcement	€ 140.00	per	m^3	2014	€ 140.00	B.vd.Meer (Gemeente Rotterdam)
formwork	€ 75.00	per	m^2	2014	€ 75.00	B.vd.Meer (Gemeente Rotterdam)
total						
Finishing						

road	€ 35.00	per	m^2	2014	€ 35.00	B.vd.Meer (Gemeente Rotterdam)
tiles	€ 150.00	per	m^2	2014	€ 150.00	B.vd.Meer (Gemeente Rotterdam)
diver (+crew)	€ 240.00	per	hour	2014	€ 240.00	B.vd.Meer (Gemeente Rotterdam)
tugboat	€ 300.00	per	hour	2014	€ 300.00	B.vd.Meer (Gemeente Rotterdam)
steel						Breedveld staal
round	€ 1,300.00	per	t	2014	€ 1,300.00	
bulkheads						
material+constructing	€ 4,000.00	per	t	2014	€ 4,000.00	B.vd.Meer (Gemeente Rotterdam)
apply	€ 10,000.00	per	t	2014	€ 10,000.00	B.vd.Meer (Gemeente Rotterdam)
dock						
rent	€ 50.00	per	m^2/day	2014	€ 50.00	B.vd.Meer (Gemeente Rotterdam)

Time vs speed

length trajectory

BT1

BT2

mean

Value of time (costs VVU)

Design speed	km/h	120	100	80	50
	m/s	33.33333333	27.7778	22.22222222	13.88888889
time to pass	s	57	68.4	85.5	136.8
delay	s	-11.4	0	17.1	68.4
	h	-0.00316667	0	0.00475	0.019
expected intensity	vhc/day	200000	200000	200000	200000
Of which: cars	%	85	85	85	85
trucks	%	15	15	15	15
cars	/day	170000	170000	170000	170000
trucks	/day	30000	30000	30000	30000
cost/car		-€ 0.04	€ -	€ 0.06	€ 0.22

cost/truck		-€ 0.27	€ -	€ 0.40	€ 1.62
cost/day cars		-€ 6,311.61	€ -	€ 9,467.41	€ 37,869.65
cost/day trucks		-€ 8,075.00	€ -	€ 12,112.50	€ 48,450.00
cost/day	€	-€ 14,386.61	€ -	€ 21,579.91	€ 86,319.65
costs/yr	€	-5251111.97	0	7876667.949	31506671.8
costs/lifetime	€	-262555598	0	393833397.4	1575333590
costs/lifetime	million €	-262.555598	0	393.8333974	1575.33359
environment	%	5		-5	-10
intensity change	%	5		-5	-10
traffic jam sensitivity	%	5		-5	-10
	%	15		-15	-30
corrected costs/lifetime		-223.172259	0	334.7583878	1102.733513

Traffic hindrance

distance			
	normal trajectory	5.4	km
	Blankenburg trajectory	20.6	km
	Maastunnel trajectory	17.5	km
	Van Brienenoordbridge trajectory	34.4	km
Mean "part of total route"			
	Blankenburg trajectory	80	%
	Maastunnel trajectory	80	%
	Van Brienenoordbridge trajectory	60	%
mean additional travel distance			
	Blankenburg trajectory	11.08	km
	Maastunnel trajectory	8.6	km
	Van Brienenoordbridge trajectory	15.24	km
Mean max speed			
	Blankenburg trajectory	100	km/h
	Maastunnel trajectory	60	km/h
	Van Brienenoordbridge trajectory	90	km/h
Congestion factor			
	Blankenburg trajectory	1.1	
	Maastunnel trajectory	2	

Van Brienenoordbridge trajectory	1.3	
VVU's/mean user		
Blankenburg trajectory	0.12188	hour
Maastunnel trajectory	0.286666667	hour
Van Brienenoordbridge trajectory	0.220133333	hour
Expected participation		
Blankenburg trajectory	40	%
Maastunnel trajectory	40	%
Van Brienenoordbridge trajectory	20	%
intensity	160000	vhc/day
VVU's/day	33191.25333	hour/day
costs/day	€ 324,288.21	€
costs/week	2270017.476	€
costs/week	2.270017476	million €

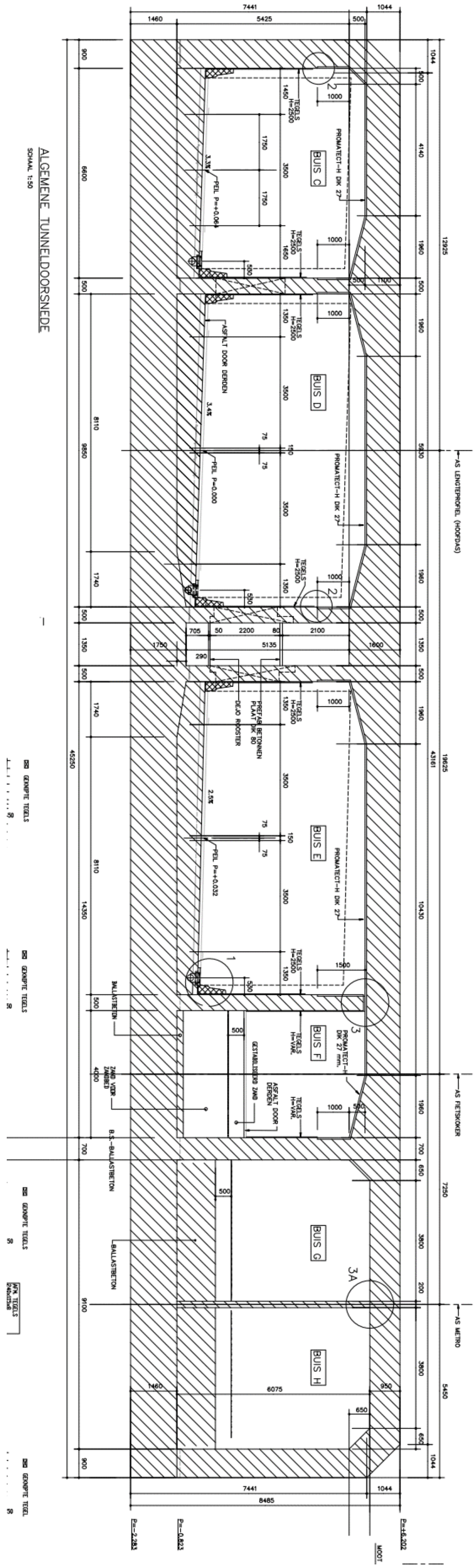
Appendix I: Drawings

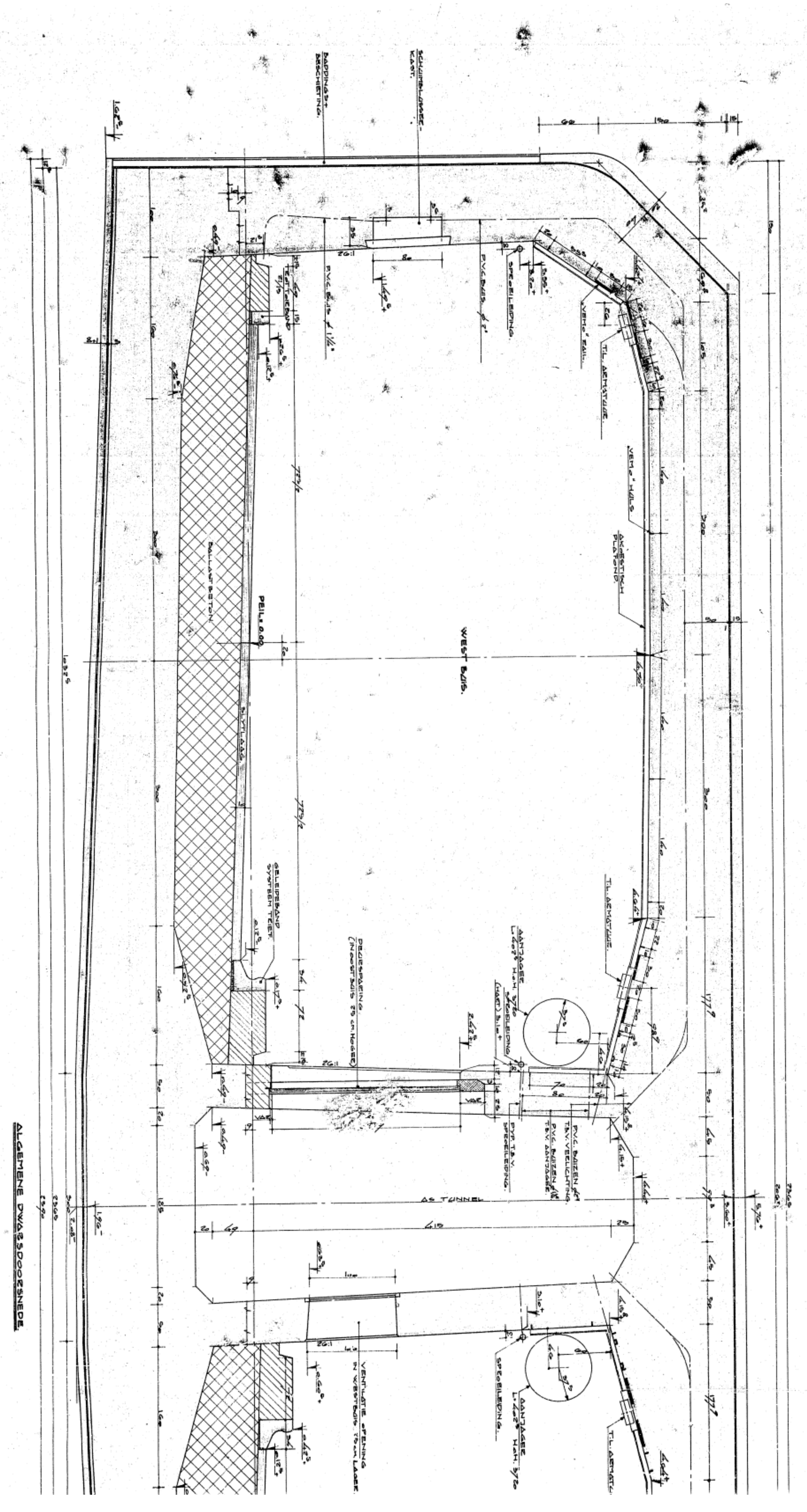
Cross-section 2nd Beneluxtunnel

Cross-section 1st Beneluxtunnel

Longitudinal section 2nd Beneluxtunnel

Longitudinal section 1st Beneluxtunnel

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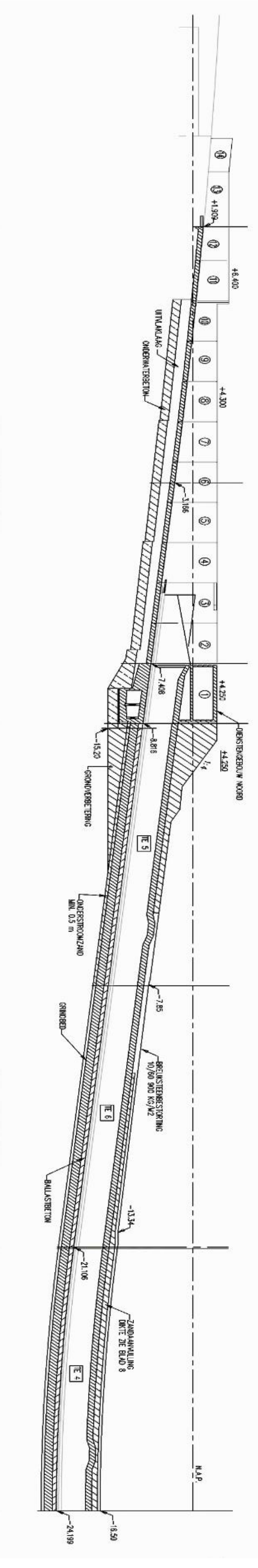
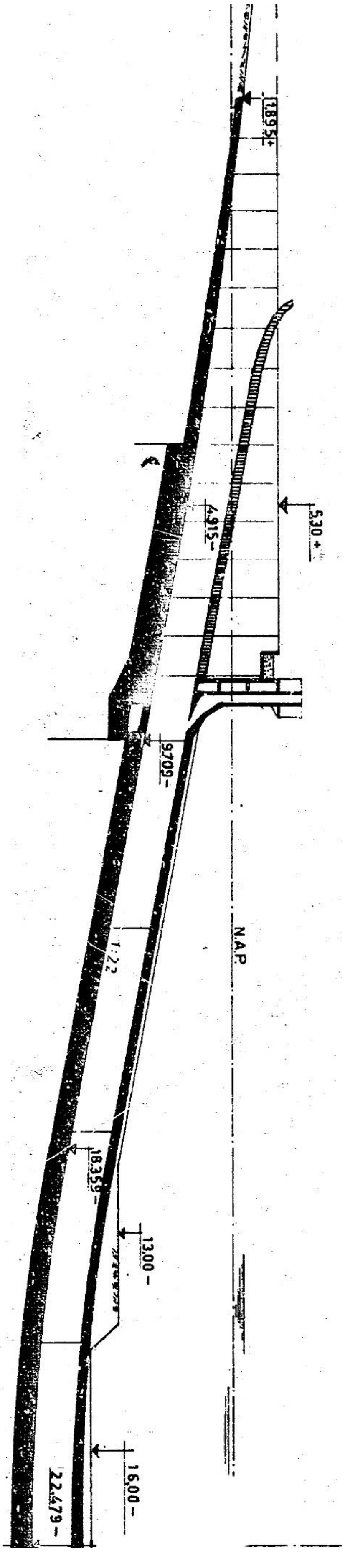
WEST BUIS
AS TUNNEL
BALLASTBETON
PEIL ± 0.00

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