R.A. (Arne) de Jong

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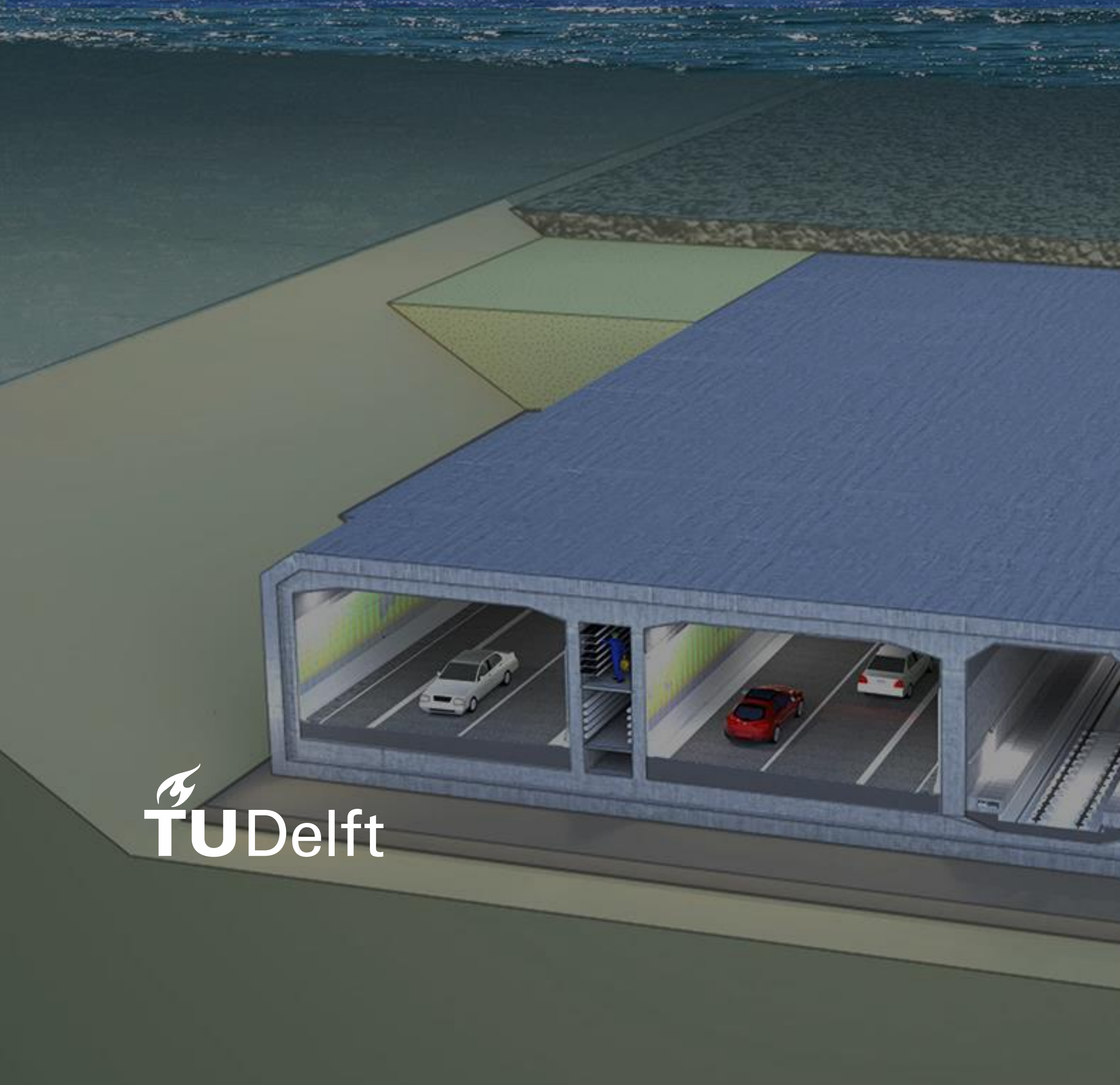

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Fehmarn Belt Fixed Link Immersed Tunnel
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RE-USE OF IMMERSED TUNNELS

An innovative method for recovering, regenerating and reusing immersed tunnels by temporarily re-floating

Master thesis

By

R.A. (Arne) de Jong

in partial fulfilment of the requirements for the degree of

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Preface

This thesis reports my findings in developing a method for lowering immersed tunnels. It is performed as graduation project to obtain my Masters' degree in Hydraulic Engineering at the Delft University of Technology at the faculty of Civil Engineering. This study was executed in collaboration with Tunnel Engineering Consultancy (TEC) and Royal HaskoningDHV.

I would like to thank my thesis committee, Bas Jonkman, Mark Voorendt, Erik-Jan Houwing and Marcel 't Hart. Thanks to Bas Jonkman for leading my committee and being enthusiastic throughout the entire process. Mark, thanks for helping with, especially, the structure of my thesis and keeping me motivated while working at home due to COVID-19. Marcel, thanks for always being able to schedule time for discussing my work and answering all my questions. Erik-Jan, thank you for providing fresh insights during my thesis and always having good advice. Also, thanks to Kristina Reinders, Gina de Rooij and Arjan Luttikholt for taking place in my committee the first few months and helping me starting the process.

Furthermore, thanks to my colleagues at RHDHV and TEC. Thanks for providing a very enjoyable working environment and for all the fun we had during lunch breaks or drinking a beer. Also, a special thanks to my roommates for the tons of coffee and being sort of my new colleagues while working at home. Finally, a special thanks to my family for supporting me during the entire journey of my study.

Arne de Jong
Delft, 31 May 2020

Abstract

The past decades the world economy grew bigger and bigger. With this growth the cargo ships grew and the ports with them. These bigger ships came with an increased draught. The increased dredging in waterways and canals is reaching its current limitation due to the presence of tunnels constructed beneath the waterways. In this thesis a method was developed for **temporarily, effectively re-floating** these immersed tunnels to be afterwards placed deeper such that waterways could be dredged.

In developing this method, the basics of constructing an immersed tunnel was discussed and as much as possible reverted. The resulting method consists of four main phases.

1. Preparation of the immersed tunnel element for the re-floating process
2. Removing a first small section in the tunnel chain, such that the main elements can be lifted
3. Re-floating the main elements and transporting them to a sheltered area
4. Preparing, adapting and re-configuring the tunnel elements and new the foundation

An analysis was made for all the new risks arising due to this method. The result was a list of measures needed for each of the risks.

To show the viability of this new method, it was applied on a case study. For this study multiple tunnels were compared. Using a multi criteria analysis the most suitable tunnel was chosen, the four criteria were available knowledge, the waterway relevancy, the tunnel relevancy and the tunnel dimension. The result was the Wijkertunnel in the North Sea Canal constructed in 1996; an concrete cross-section tunnel with a total of six tunnel elements making up a total immersed length of 600 meters. For this tunnel nearly all the design information is available, only the exact reinforcement design is lacking. A preliminary design for re-floating the Wijkertunnel was available.

For the first phase it was shown that some ballast concrete can stay due to a decrease in freeboard requirement. However, the dimensions of the bulkheads are bigger due to increased water pressures. The number of hoisting wires will be doubled from 4 up to 8, each wire will be connected with 4xM64 anchors. Regarding the prestressing a complete new lay-out was given based on external post-tensioned prestressing (inside the tunnel elements, but outside the concrete cross-section). Using MatrixFrame and optimisation was done. This resulted in wires applied over each segments joints, the number of wires per joint vary. The type of wires are post-tensioned, 7 strands per wire.

In phase two it was shown that a V-shaped opening joint with a bottom length of 6-meters long would be needed, cutting should be done using underwater cutting equipment. The soil cover should be removed using both backhoe dredgers up to 20 meters below the water level, the rest should be removed using grab dredgers.

The third phase discussed the soil adhesion, or so-called passive suction. This is the most unsecure aspect of the method. The breakout time against this adhesion was calculated assuming Darcy flow. This resulted in an expected break-out time of about 2 hours. After lifting the elements are transported to the port of Amsterdam.

From the fourth phase regarding maximum deepening the traffic requirements are governing above the joint's rotational capacity, for the Wijkertunnel the maximum deepening would be $6.62\ [m]$ in the middle of the tunnel. The new foundation will be a gravel foundation. The new approach structure should be located higher than the original. No design was given for adapted approach structure, but it was assumed to be possible. The closure joint should be constructed in-situ, similar as in the original design.

Next to these four phases also the increased pressures due to the tunnel final location are discussed. This induces higher loads on the tunnel. A solution in reducing the load is available by removing part of the soil cover. Solutions to increase in the tunnel strength are available with placing extra reinforcement steel in location possible. Also the original Wijker tunnel design was based on a simple linear calculation, it is proposed to execute a non-linear calculation for finding extra structural capacity.

The case study was finished with an evaluation of the risk's measures. For several measures the costs or increased complexity in the design were calculated. For all the risks discussed the results shows that the impact on the tunnel design was small compared to the increased safety, therefore these risks measures are beneficial.

After showing the that the new method was indeed viable for a specific case study other immersed tunnels are discussed at a more general level. First an overview was given of the different immersed tunnel and their impact on the new method. It was concluded that the method is viable for all the different immersed tunnels except those with a significant different number of tunnel elements (being six for the Wijker tunnel). The last section discussed future tunnels and how these could be adapted. The result was that for future tunnels it would be interesting to apply prefab ballast concrete compared to in-situ casted concrete.

Finally, it was concluded that this method for temporarily, effectively re-floating immersed tunnels is a viable solution for deepening immersed tunnels. It was concluded that several design aspects deviate significant from the original tunnel design or induce a high risk. Extra attention should be paid to these when applying this method, these aspects are:

- Submerged installed anchors
- External post-tensioned prestressing
- Unfavourable bulkhead location
- Soil adhesion or passive suction
- New tunnel alignment
- Soil(-cover) and the joint removal
- Increased water pressure
- Risk analysis

However, not all the possible knowledge was achieved in this thesis due to several reasons and more research is needed. For example, there was no possibility for soil samples, concrete samples or lab testing. Also, some further research would be needed since the topics were not directly in the scope of this theses. These topics are soil adhesion, a non-linear cross-sectional tunnel element analysis, concrete strength over time, re-use of the grouted prestressing, approach structures and a more detailed risk analysis for the entire re-floating process.

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1. Introduction

This thesis was made to find a new method of re-using currently immersed tunnels which did not reach their design lifetime, regarding environmental impact and the costs of construction an entire new tunnel is not preferred. This chapter explains the motivation for this thesis on immersed tunnels, followed by a more in-depth problem analysis based on the current state of immersed tunnel. The result is a problem statement. In the next section a preliminary objective of this study is defined. After defining a specific scope, the preliminary objective is translated into a main objective. The final section divided the main objective into methodological steps, all these steps are assigned to specific report chapters.

1.1 Thesis motivation

The world of tunnelling is under constant development. Since the construction of the 1st Maastunnel was finished in 1942, much changed. The tunnel dimensions grew bigger and bigger. In Turkey the construction of a 60-meter-deep tunnel is finished. Soon the construction of the longest immersed tunnel will start between Germany and Denmark. This tunnel, the Fehmarn belt connection is to be a tunnel of more than 17 km long. In the Netherlands 19 immersed tunnels were constructed in the 20th century (Gursoy & Milligen, 1993). Right now, the overall tunnelling market is growing. According to the International Tunnelling Association (ITA) an annual growth of at least 5% is expected, the increase the past years is shown in Figure 1 (STUVA, 2017).

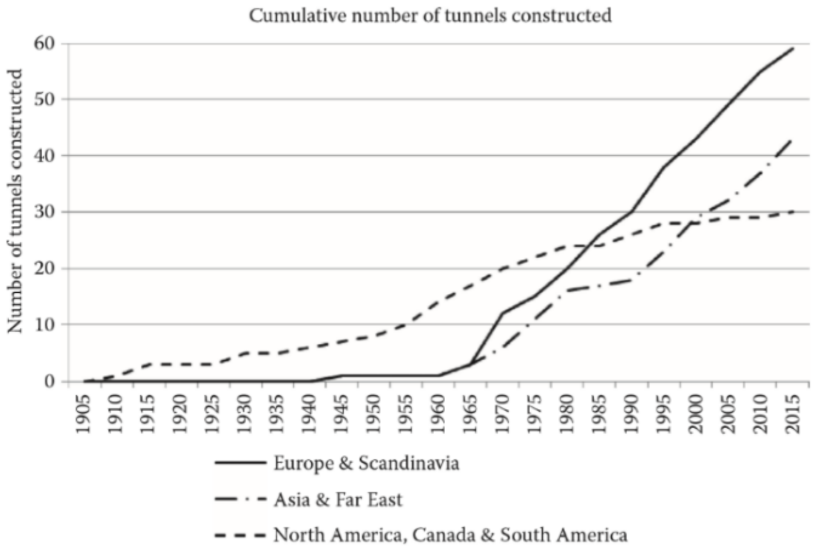


FIGURE 1, CUMULATIVE NUMBER OF TUNNELS CONSTRUCTED (STUVA, 2017)

The choice for an immersed tunnel over a bridge or another type of tunnel depends on many variables. For instance, a bridge gives a limited height for ships and big vessels. On the other hand, a tunnel is placed on the bed of a river and gives a limiting draught to the ships. A bored tunnel is constructed deeper than the level at which an immersed tunnel is placed, the bored tunnel therefore is longer and might not be possible since longer access roads are needed to reach the required depth. Also, depending on the situation one or the other may be more expensive. Currently there is tendency towards the construction of bored tunnels over immersed tunnels.

To increase the popularity of a tunnel type that has a lower impact on the surrounding compared to bored tunnels, or no limiting height compared to bridges, development of new or more efficient techniques for these immersed tunnels is needed. In addition, the adaptation and re-use of immersed tunnels offer interesting possibilities to extend their lifetime and, in this way, contribute to sustainable solutions to maintain a good working infrastructure. However, these possibilities are not common practice and require further study to determine their feasibility. This report therefore explores the possibilities for adaptation and re-use of immersed tunnels.

1.2 Problem analysis

Following from the previous section the basics of immersed tunnels is discussed. This knowledge is needed to be able to pinpoint the actual bottlenecks and problems in immersed tunnels. Below the very basic principles of constructing an immersed tunnel are discussed. The core of these tunnels are tubes made from concrete or steel. The next section discusses the state of the current research in immersed tunnels. Finally, a problem statement is defined.

1.2.1 Basic characteristics of immersed tunnels

Construction

An Immersed tunnel is constructed from tunnel elements. These so-called tunnel elements are constructed in a dry dock and are about 100 meter long. The construction in a dry dock ensures a high level of certainty in the material quality compared to constructing for example a bored tunnel beneath the surface. After the construction of these hollow concrete tubes the next process is divided in four steps; Floatation-Transportation-Immersion-Foundation (In Dutch; OTAO; Opdrijven – Transporteren – Afzinken -Opleggen)

Floatation

The first step after finishing the concrete tubes, is placing ballast tanks inside the tunnel elements. Then the elements are sealed with temporary bulkheads, these bulkheads are tested on water tightness. If everything is safe the casting basin is inundated, then controlled emptying of the ballast tank results in a floating element. When the element is afloat it can be trimmed and is ready for transport.

Transportation

The floating element is connected to tugboats and winches tow the element above the location in which it will be immersed. Depending on the route to be transported extra provisions are taken, for example extra protection against sea waves. When the element is arrived above the final location the immersion process starts

Immersion

Winches are used to position the element perfectly above the dredged trench. The element is then taken over by immersion pontoons and the exact position is monitored. If everything is correct the ballast tanks will be filled with water resulting in a negative buoyancy of the element. It is then slowly lowered on a foundation of gravel or a temporary pin & catch structure. The element is pulled horizontal against the previous element securing the first watertight seal using the GINA-profile. A second seal is installed using an Omega-profile.

Foundation

As mentioned, two types of foundations are available. The first being a prepared gravel bed on which the element is placed. The second is a catch structure connected to the previous element on one side a pin placed on a concrete tile foundation on the other side. This results in an empty space beneath the element, after perfectly vertical positioning the space is filled with a sand flow or grout installation. This is done by connecting pressure pipes to the sand flow system integrated in the tunnel concrete. Once enough sand is pumped underneath the pipes are disconnected.

Finishing works

The space between the tunnel elements is pumped empty and checked for leakage. If none is present the bulkheads can be opened and removed. The trench next to the element is backfilled and the tunnel is covered with a sand and/or a rock protection. The final tunnel piece is placed. This called the closure-joint, this is specifically designed to connect both the ends of the tunnel. If the tunnel element is constructed segmentally the prestressing wires are cut allowing higher settlement different. The ballast tanks are gradually replaced with ballast concrete securing a safe element regarding floatation. The final lining, asphalt, ventilation and other equipment is installed. The result is a fully safe and functional tunnel

Maintenance

During the lifetime of the tunnel maintenance of the tunnel is necessary. Also monitoring for leakage is executed. This keeps the element safe and functional during the design lifetime.

1.2.2 Tunnel history and available knowledge

In this section the current knowledge and developments in the world of immersed tunnels is discussed. This explains the current state of the research in tunnels in more detail and shows where possible gaps in the knowledge are.

Most commonly these immersed tunnels are designed with a 100-year lifetime. With the construction of the Maastunnel in 1942 the end of the 100-year lifetime of the tunnels is in sight. But 2042 will not be the first intervention in the constructed tunnels. Additionally, to the standard maintenance work an intervention in a tunnel might be needed.

For example, the George Massey tunnel in Vancouver was constructed in 1959 as an immersed tunnel for highway traffic. The design consisted of a 630-meter-long tunnel with 2x2 lanes. But the last 50 years the structural quality decayed and research revealed that the current state of the tunnel did not meet the safety requirements for earthquake induced loads. Additionally, the functional requirements changed during the lifetime and four lanes was just not enough. Several design solutions were explored varying from retrofitting the tunnel, construction an extra tunnel or even constructing a bridge. Sadly, before extensive research was done the budget was cut. The last years a political change took place and new plans for an immersed tunnel arose. No concrete plans are available, but the previous research showed that if would be interesting to somehow incorporate the existing tunnel in whatever the solution may be (WSP | MMM Group, 2016).

Another interesting case is the Chesapeake Bay Tunnel, this tunnel needed to be adapted. The facility is a combination of a bridge and a tunnel, the bridge part is four lanes wide and the tunnel is two lanes. To have a full-four-lane facility from end to end a new tunnel is necessary. Additionally, the thimble shoal canal is 'the eye of the needle' for the major ports in the Hampton area. An article was published by W. Grantz describing a simple method to cut this tunnel in small segments and re-using these elements when construction an extra or new tunnel. It is important to notice the Chesapeake Bay tunnel is a steel double shell tunnel which is a different type than the common method in Europe, but is also showed the possibility of recycling the old tunnel (Grantz, 2003).

A third interesting case closer to home are the Benelux tunnels, these form a vital part of the traffic infrastructure in Rotterdam, one of the biggest cities in the Netherlands. Rotterdam is also famous of having the biggest port of Europa, the port of Rotterdam. This enormous industry is under constant development resulting in bigger and more cargo transport coming in from the sea. With these bigger vessels, also comes a bigger draught. That is why the Port of Rotterdam mentioned a deepening of the Nieuwe Waterweg might be needed to hold the competitive position compared to the port of Antwerp in the future. When the dredged part is about 20 km land inwards the Benelux tunnels are reached, and a problem appears. The cover on top of the Benelux tunnels is located higher than the level until which dredging will be required.

Thomas Weeda did a case study specifically for these Benelux tunnels and explored several solutions to this problem. This resulted in two methods for deepening the tunnel. The first was digging underneath and the second was re-floating. The method of digging underneath was explored in more detail, it is shown in Figure 2. It was mentioned that it would be economically more interesting to use re-floating over the digging underneath option when more than 3.2-meter deepening is needed. (Weeda, 2015). It is important to note that in his research for the re-floating option some assumptions were made regarding rotations between the element segments which are not feasible; for example, rotations between the segments were assumed while the element is prestressed.

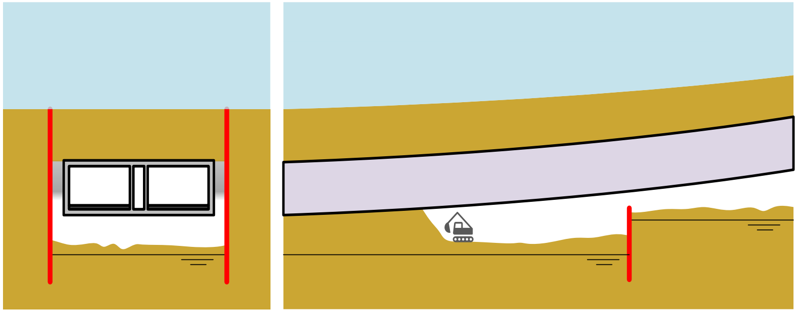


FIGURE 2, LOWERING TUNNEL BY DIGGING; SIDE VIEWS

A case in which re-floating was actually applied can be found in Texas (TxDOT, 2014). This tunnel is a concrete shell type tunnel constructed in 1953 connecting the city of Bay Town to La Porte. This connection is on the north city of the Golf of Mexico connecting the city and the port of Houston to the carribean sea and the atlantic ocean. As mentioned the tunnel design was a concrete shell, the total thickness of the combined shell was 63 centimeter and the total tunnel diameter was 11 meter. In 1970 the tunnel traffic exceeded the design capacity but it took sixteen years before in the year 1986 a contract was awarded for the construction of a cable stayed bridge. The construction of this bridge was finished in 1995 and parralel to this the tunnel was closed. Due to the increase in vessel dimension the tunnel would need to be removed and several plans were proposed, resulting in the tunnel being removed in 1998. A stage of this removal is shown in Figure 3**.**

FIGURE 3, ANIMATION STILL OF FLOATATION OF THE BAYTOWN TUNNEL (STERLINGLIBARY, 2014)

The design for this removal process was based on the fact that these elements would be decommissioned after re-floating. Simple bulkheads were installed and part of the inside of the tunnel was demolished. Concrete is removed from the shell and a cut is made in the steel shell using shape charges. Due to all the removed concrete the elements where postively buoyant and shot out of the water relative fast (about 5 hours) due to the circalur shape. After that the elements were transported and destroyed. The result was a deeper channel allowing for bigger vessels to enter the port of Houston.

As seen for several reasons it might be needed to adapt, replace or remove immersed tunnels before the 100-year lifetime mark is reached. Constructing a new tunnel is possible but the civil engineering sector already has a massive impact on global warming with the concrete production contributing a massive part. The worldwide production of concrete is responsible for 8% of the total CO_2 emission (Lehne & Preston, 2018). Another even more relevant problem is the emission of nitrogen, a division of nitrogen per sector is given in Figure 4. With the current rate of nitrogen emission exceeding the maximum allowed amount, multiple construction projects are paused or even aborted. The construction sector is responsible less than 1% in 2018 but this doesn't mean they can just keep releasing polluting gasses. (RIVM, 2019).

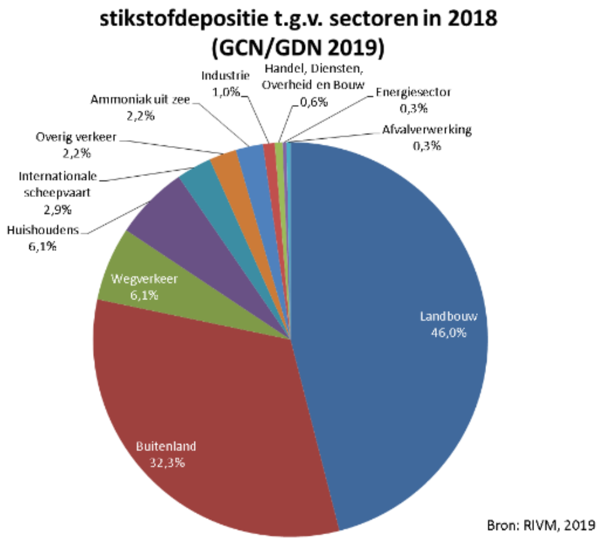


FIGURE 4, NITROGEN DEPOSIT IN 2018 IN THE NETHERLANDS (RIVM, 2019)

Keeping these two notions about emissions in mind, and the increase in global warming which has a big impact on our society but will only be bigger if not limited, raise a question. Why do we have to construct new tunnels if their ultimate lifetime is not reached? Is it possible to effectively recycle tunnels?

1.2.3 Problem statement

As stated in the previous section several reasons might rise why an immersed tunnel needs to be replaced, removed or adapted. In this thesis one specific problem is investigated in more detail.

This is the challenge in the example of the Benelux tunnels and Baytown tunnel. Here the development in the maritime transport resulted in bigger vessels passing through the waterways and canals. These bigger vessels are not only longer and wider but also have more draught. Right now, the biggest vessel a harbour can handle depends on the minimum depth of the port or the waterways leading to the ports.

The increase in vessel dimensions is not a small-scale process but worldwide. Ships all over the world are growing in dimension and so are the port. The biggest ships in the world up to 2016 are shown in Figure 5.

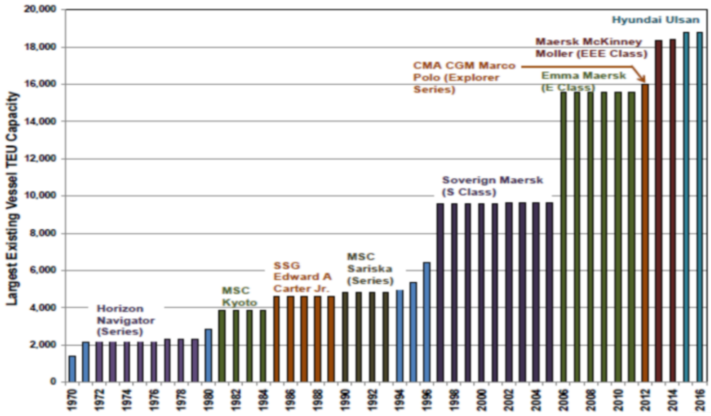


FIGURE 5, LARGEST EXISTING VESSEL TUE CAPACITY (HACEGABA, 2014)

Bigger vessels therefore imply a need of dredging in the waterways. When dredging deeper and deeper the structures in the soil beneath the waterways are exposed. And when an immersed tunnel is encountered de maximum dredging level is reached. Demolition or removal of these tunnels is possible although it is a waste to destroy a tunnel which is in a good condition. This process is summarized in the following problem statement:

Immersed tunnels, which do not yet reach their design lifetime, can be obstacle for waterways to be dredged deeper and thereby hinder the passage of vessels and thereby hinder the development of ports and the inland economics.

1.3 Thesis objective

In this section the aim of this report is defined. This is done by first setting a preliminary objective, this is then followed by a scope and results into a main objective for this study.

1.3.1 Preliminary objective

The objective of this study is to:

- Provide a feasible method to stop immersed tunnels from being an obstacle in waterways.

1.3.2 Scope

The objective above can be reached with a wide range of solutions. In this section the scope for this research is defined. This scope is needed to have a more in-depth analysis, and this maximizes the value of this thesis.

New tunnel

A complete solution to the bigger vessels in the waterways would be to destroy the existing tunnels and just construct a new tunnel. None of the current immersed tunnel have reached the end of their 100-year design lifetime. The construction of a new tunnel will also result in unnecessary extra emissions and is a waste of qualitative good material.

<u>This option will not be treated</u>

Solutions not related to the current immersed tunnels

Given the background of the researcher, solutions not directly linked to field of hydraulic engineering and not relating to the current immersed tunnels are not treated, these are for example:

- (Temporarily) increasing the water level
- (Temporarily) decreases the vessels draught
- Changes in law and/or legislation

<u>These options will not be treated.</u>

Solutions related to digging underneath the current immersed tunnels

Two possible solution directly linked to the immersed tunnels were found by Weeda in 2015. These two methods are: dredging while keeping the tunnel immersed or dredging while the elements are afloat. The second solution was briefly mentioned. The first solution was explored in more detail, but some major assumptions were made.

- the complete tunnel will be lowered simultaneously,
- No practical solution for removing and replacing the foundation was given,
- Rotation were assumed in segment joint while the elements were prestressed.

<u>This option will not be treated.</u>

Solution related to re-floating the current immersed tunnels

Weeda mentioned the first solution was more attractive compared to re-floating the elements since it was cheaper. But on the other hand, for a bigger lowering range re-floating is the best option. In this research only the re-floating option will be discussed. Several advantages of this method might be:

- Re-floating the elements is a solution to a wider range of possible problems:
    - A single element can possibly be replaced if necessary
    - Small adjustments can possibly be made to the alignment

- Re-floating is applicable for a wider lowering range

This option will be treated.

1.3.3 Main objective

The problem stated in section 1.2.3 and the scope defined above results in the following objective.

Provide a method in which immersed tunnels effectively, temporarily, can be re-floated such that the immersed tunnel won't be an obstacle when for example a waterway is to be dredged.

Note some specific words in this objective:

Effectively	Meaning that most of the tunnel is re-used when immersed again. This resulting in a high rate of circular design; lowering emission and re-using material. Disproportional use and construction of extra material and machinery is not appealing
Temporarily	Meaning that the tunnel will be immersed again and should be able to do so. The elements to be immersed are the same elements that are re-floated. No other solution like semi-submerged tunnels or bridges are considered in this report. The tunnel will be immersed in the same location, just a higher depth.
Re-floated	Meaning the elements will be buoyant again. The option of just lifting these enormous elements is not considered since special equipment needs to be designed for such enormous loads. Defying the purpose of circular design. Options like digging underneath the tunnel and then lowering are not considered.

1.4 Report approach and structure

1.4.1 Approach

In finding a valid method first the current method for constructing immersed tunnels will be described, this can then be used as a starting point for the re-floating method. After developing the re-floating method this should be validated. The validation was done by applying the general re-floating method to a specific case and check the applicability. If this general method indeed shows to be applicable to a specific case the only resulting check is whether the case tunnel is representative for the other immersed tunnels and to discuss what would differ if another case was used.

1.4.2 Methodological steps

In this section the methodological steps needed for reaching the thesis objective are defined.

1. ***Develop a general method, which is effective, temporarily and based on re-floating. Define if this new method poses extra requirements on the tunnel.***

For developing this method, the current method for constructing immersed tunnels as discussed in section 1.2.1 will be the starting point.
For the general method three basic requirements are given, similar to those for the normal construction method; a continuous horizontal balance, vertical balance and safety (regarding accessibility and water tightness).
For this general method a risk analysis is needed to check if extra measures are needed to lower any excessive risks. This analysis results in extra requirements on the tunnel regarding structural capacity due to this new general method.

2. ***Selection of a suitable case for validation of the general method and outlining the current state of this immersed tunnel.***

For analysing the current state, a case study will be used. It is important that this case study is relevant and representative for most other immersed tunnels, therefore first a multi criteria analysis is needed to select a suitable case.
This case is then analysed, two aspects are important. The first being a general analysis, this is about the tunnel size, surroundings and method of constructing. The second is a load analysis, this is about the different loads acting on the tunnel.

3. ***Validate the general method by applying it on a case study***

The general method is validated by applying it on a case study. The main criteria to show the viability is based on the relation between structural capacity and the loads on the tunnel.

4. ***Study the impact of the extra requirements, due to the new general method, on the tunnel.***

The extra risks mentioned in step 2 pose extra requirements on the tunnel. The effect of these risks is described and possibly calculated by comparing the tunnel design with or without these requirements.

5. ***Determine if this general method is viable for other currently constructed immersed tunnels, and if future tunnels can be adapted to accommodate re-floating and immersed.***

After step 3 the general method shows to be viable for a specific case study. The final check is discussing if the method is also applicable on other existing tunnels. This is done by identifying the difference between the case study and other tunnels.
Also, possible future tunnels are discussed, and if future tunnels can be adapted such that this general method can be more easily applied.

1.4.3 Report structure

In this section the different chapters are discussed. The methodological steps are assigned to a specific chapter and answered. In Figure 6 the flowchart throughout the entire report is shown more graphical.

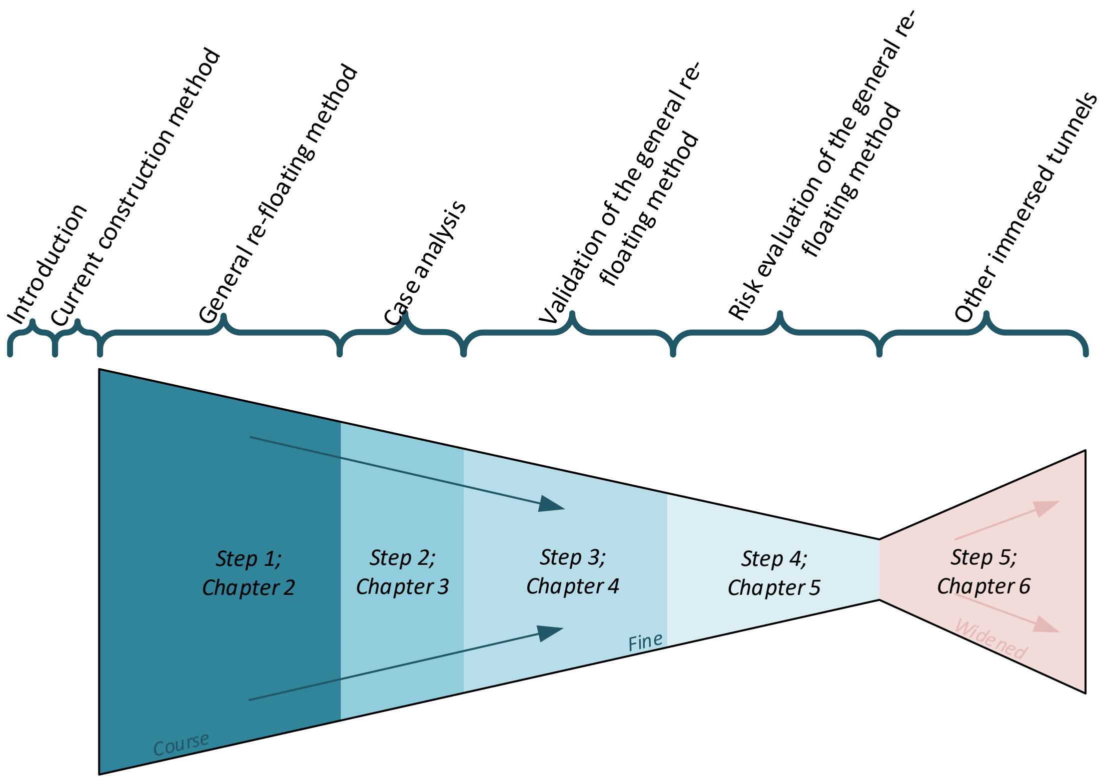


FIGURE 6, REPORT FLOWCHART

Chapter	Content
1. Introduction	This chapter is the introduction to the thesis. It explains why this research started and why it is relevant. It explains the current knowledge regarding immersed tunnels and described the method of construction such a tunnel. Also, the main objective of this thesis is stated
2. Development of a general re-floating method	In this chapter the original method of construction described above is converted in a general method for effectively, temporarily re-floating immersed tunnels. The result of this chapter is a general method which can be applied on specific cases and a risk analysis for this method. Methodological step 1 is finished after this chapter.
3. Case study	Based on the literate an overview of immersed tunnels is discussed and a relevant case is chosen. The case characteristics are then explored and described in a generic analysis and a load analysis In the generic analysis all the different aspects from the Wijkertunnel are discussed. These vary from the alignment, the type of closure joint, the type of foundation, the immersion process et cetera.

The load analysis is focused on the different loads applied on the construction and the different load combinations. These vary from self-weight, loads during transport, wave loads et cetera.
The result of this chapter is a good overview of requirements and boundaries the case sets for applying the general method. Methodological step 2 is finished in this chapter.

4. *Validation of the general method*	In this chapter the boundary conditions and requirements set above are applied on the general method, this validates the general re-floating method. For some steps the general method won't differ for this specific case and some aspects are worked out in more detail. One specific aspect, the prestressing, of the general method requires extra attention since it is fundamentally different to the original method of construction. It is discussed in more detail. The result of this chapter is a method for effectively, temporarily re-floating which is viable for a specific immersed tunnel. After this chapter the methodological step 3 is finished.
5. *Risk measures evaluation*	Some aspects in chapter 4 are designed under the addition of risk reduction measures. In this chapter the actual impact of these measures is evaluated. The result will show the increased costs and increased construction complexity due to these measures. After this chapter the methodological step 4 is finished.
6. *Applicability to other and future immersed tunnels*	As stated in the previous chapter this is only a case study for a specific situation. To increase the relevancy of this study this case is compared to other tunnels and the differences are discussed. Also is this research based on an existing tunnel but if somehow a clear, but simple, change can be made in de current design of future immersed tunnels to improve the design this should be discussed. Methodological step 5 is finished in this chapter.
7. *Conclusions and recommendations*	Here the conclusions are made, and recommendations are given. It is checked If the objective of this thesis is achieved.

2. Development of a general re-floating method

This chapter discusses a general method for temporarily re-floating method of immersed tunnels of the segmental type and returning them to a deeper location. This method is also available for monolithic tunnel, but this type is less complex since no prestressing was present during initial immersion, these method for these monolithic tunnels is mentioned in chapter 6. In the first section the approach for defining this new method is described. In the second section the construction sequence of a general method is discussed. The sections 2.3 to 2.6 are an elaboration of the categories which are stated in section 2.2 elaborated. This is then followed by providing a check on the main design criteria. Finally, a risk analysis is made.

2.1 Design approach of the general method

Requirements

The first question for the developing such a general method is what are the requirements for this method? In this chapter the requirements are summarized and collected in three groups.

The first requirements (*A.I*) for the design follows from the thesis objective posed in the previous chapter. The design should be 'temporarily', meaning that it starts with a currently existing tunnel and should end with a tunnel at the same location.
The second requirement (*A.II*) is that the design should be effective, meaning that the amount of waste should be minimized, and the amount of recycled material should be maximized.
The final requirement (*A.III*) following from the research question is that the method of removing the tunnel is by re-floating the tunnel.

The second part of the requirements follow from the constructability and operational safety (*B.I* & *B.II*), these are similar to the requirements for the original method of construction an immersed tunnel. During all the stages of the design (for example after removing the soil cover), there should be a balance of the forces. This regarding the horizontal balance and the vertical balance.
The other main aspect concerning operational feasibility is safety(*B.III*). During construction there is personnel and machinery present in the tunnel. The design rules of this are given by Eurocode, national annexes and documents such as Handboek Tunnelbouw.

The third requirement is the structural capacity of the structure (*C.I*). Each immersed tunnel is designed for a specific location with specific boundary conditions, such as depth and soil conditions. For floating and lowering of the tunnel at a deeper level these checks should be made again. The design rules of this are given by Eurocode, national annexes and documents such as Handboek Tunnelbouw.

This can be summarized as follows:

A. Requirement from thesis objective
    I. A complete start-end design
    II. Maximizing the amount of tunnel recycled
    III. While re-floating the tunnel
B. Requirement from an operational view
    I. Always a horizontal balance
    II. Always a vertical balance
    III. Always be safe with respect to water tightness and accessibility
C. Requirement from a structural view
    I. The structural capacity should not be exceeded

Approach

The design for the general method is mainly based on the first two categories of requirements (*A* & B). The third category of requirements is only important for a specific case study, however not for a general method. First a construction sequence is produced based on the requirements following from the research question. This is done in section 2.2. This sequence is based on the standard procedure for immersing tunnels as discussed in section 1.2.1. This is sequence is then reversed as far as possible. The new sequence is divided in four main parts. These parts are summarized below. The final immersing process for the adapted tunnel isn't described as this is just the standard immersion method.

A. Preparation of the element,
B. Removing a first section
C. Re-floating & transport of the main elements,
D. Preparing, adapting & reconfiguring

Based on these four main parts a solution is proposed for the different aspects which are relevant in these parts. This is the central part of this chapter and reaches from section 2.3 to section 2.6. The chapter is finished with a risk analysis for the design.

The structural requirements are discussed using a specific case and can be found in chapter 3 and further. Figure shows a flow chart of the design.

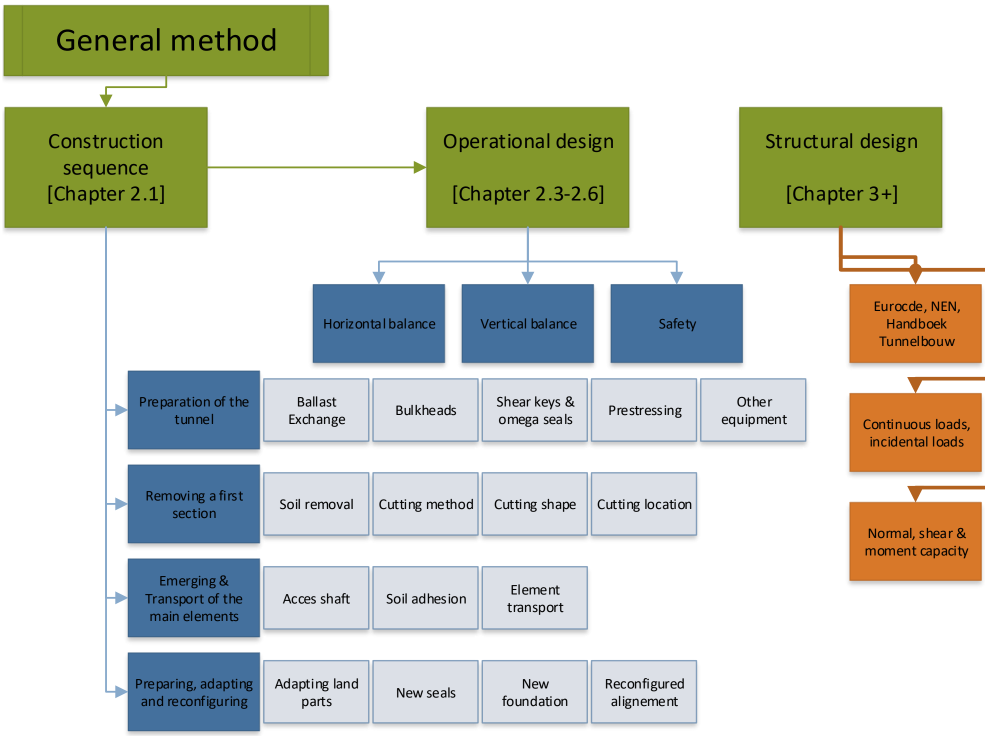


FIGURE 7, GENERAL METHOD APPROACH FLOW CHART

2.2 Construction sequence

The first step in the flow chart shown above is the formulation of the construction sequence. As mentioned, the starting position when making the construction sequence is the sequence used in the original construction method but reversed. For every step keeping in mind the requirements posed above. The standard method is already described in section 1.2.1, the following paragraphs are in the reversed order of the original method described.

Finally these steps are gathered in four main construction phases. The result of this is also shown in figures in appendix A.

2.2.1 Reversed stages

Tunnel lifetime | *in original method: Maintenance*

This design is only regarding re-floating, adapting and replacing the tunnel. Since the duration of this is in the order of months maintenance is not important. After the tunnel is in its new location a new maintenance scheme should be made. The life expectancy of the tunnel may be increased due to the process of re-floating, adapting and replacing.

Preparation works | *in original method: Finishing works*

All the finishing works can be reverted. Instead of finishing it is now the starting works in preparing the element for the rest of the process. For removing the electrical equipment, it is important to be careful such that most of the material can be recycled and installed in the new tunnel. Also, the top asphalt layer and cladding is removed.

The second part of the finishing work is installing the ballast concrete. With the reversed process this means placing new water tanks, filling them with water and removing the ballast concrete. Since removal of the concrete means a shift in element weight the bulkheads should be in place for safety regarding leakage, this does however reduce the accessibility into the element. When the bulkheads are placed, and the ballast concrete is replaced with ballast water the soil cover can be removed. This soil cover was also acting as an extra safety barrier against floatation of the element. Instead of cutting the post tensioned prestressing wires new wires should be installed in a early stage.

Adhesion | *In original method: Foundation*

Regarding the foundation not much is happening. The main problem is the adhesion of the element, also known as passive suction. This can be explained as the element sticking to the foundation. This occurs since it takes much of time for the water the pass through the soil pores and diminish the negative pressure due to the lifting forces on the element.

The amount of adhesion and time to diminish are an unknown factor. For gravel foundation this amount is to be expected to be lower compared to sand due to the grain size. Measures are needed to reduce the uncertainties.

Re-floating | *In original method: Immersion*

The element is completely prepared for re-floating, but first the closure joint needs to be removed. In the original process this was discussed in the finishing works but now needs more attention. Instead of the closure joint an opening joint is needed to 'break the chain' of the tunnel elements. This is completely new since it is not needed in the standard installation of immersed tunnel. The exact method for this is discussed in section 2.4. After removal of the opening joint the main elements can be removed. Before re-floating the space between the bulkheads is inundated. Lifting pontoons are placed directly above the element and the lifting cables are tensioned. The adhesion will be (partly) destroyed and the elements are disconnected with some sort of jacking element. The final step is re-floating the element.

Transportation | *In original method: Transportation*

When the element reaches the surface extra ballast water is pumped out and the elements can float. Based on the loads during transport the same freeboard is applied as during initial transport or other measures are taken (for example transport over sea is not possible in the winter months due to the higher waves. If transport over sea is needed a more detailed planning in which months transport occurs is needed). Tugboats are then connected, and the elements are transported to a safe location.

Safe location | *In original method: Floatation*

The safe location for the elements does not need to be a dry dock such as in the original situation. This means the floatation step does not need to be reversed. It might even be impossible to replace the curved elements on a safe dry dock foundation.

Re-configuration | *In original method: Construction*

In the safe location the tunnel element is inspected and if needed repaired. Also new GINA-seals are installed, this is needed since the old seals are damaged or bigger seals are needed for the new alignment.

At the original location of the tunnel dredging takes place such that the tunnel is re-placed at a deeper location. Also, a new foundation is installed. If needed the land structure are adapted. In another location a new closure joint is produced, this joint does not necessarily have the same shape as the original closure joint. When everything is prepared the standard immersion process can be used to immerse the adapted tunnel in the new trench.

2.2.2 New construction phases

Summarizing all these steps the new construction method for re-floating, adapting and replacing the tunnel can be divided in four main phases:

A. Preparation of the element
B. Removing a first section
C. Re-floating & Transport of the main elements
D. Preparing, adapting & reconfiguration

The tunnel lifetime and first preparing works together form the first step; **A. Preparation of the element.** The adhesion, re-floating and transportation are translated in the **B. Removing a first section** and **C. Re-floating & Transport of the main elements**. The final step is the re-configuration, this is translated into **D. Preparing, adapting & reconfiguration**. This is for both the tunnel elements and the tunnel new location.

In appendix A.**Error! Reference source not found.** this method is divided in 45 smaller steps, including clear images showing the situation of the tunnel during these steps.

2.3 Preparation of the element

The first step in the general method is about preparing the tunnel elements for re-floating. This section 'Preparation of the element' is from the start of the project up to the moment the continuous chain of element is broken by removing the first section.

While the tunnel is accessible for traffic the worksite is prepared and temporarily access roads are constructed. When all this is finished the tunnel is shut down and the decommissioning works starts. Light and ventilation is installed for this and backbone equipment, asphalt surfacing, and cladding is removed. This process is divided in several important topics discussed in the separate sections. In Figure 8, Typical immersed tunnel element an overview is given of a typical immersed tunnel element.

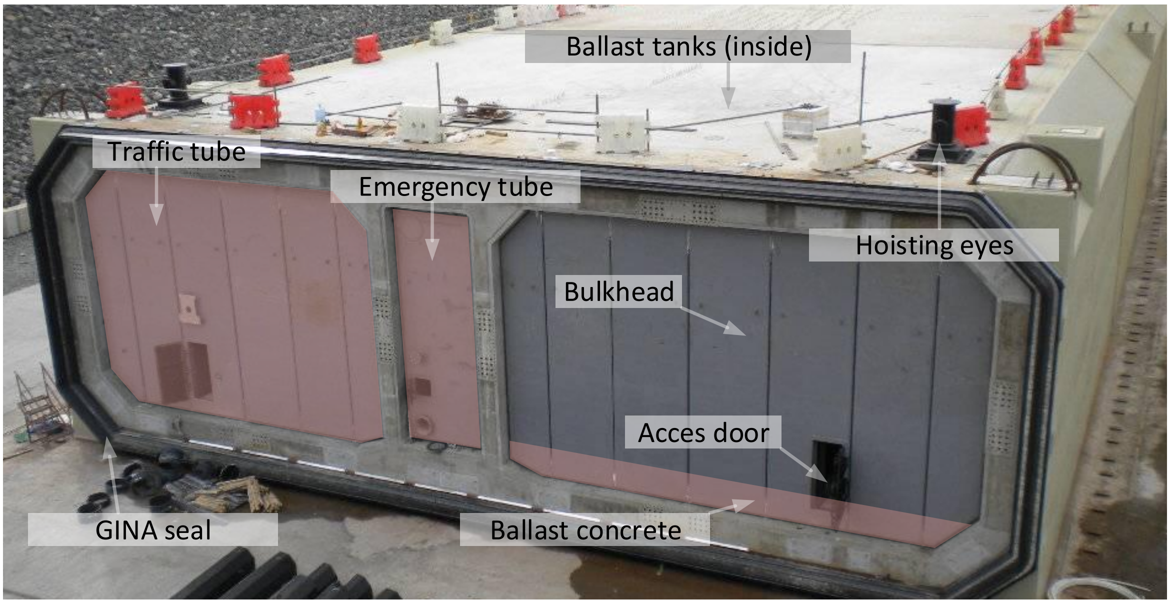


FIGURE 8, TYPICAL IMMERSED TUNNEL ELEMENT (ECPLAZA, 2020)

2.3.1 Placing ballast tanks and removing concrete

The ballast tank and ballast concrete have a vital position in maintaining the vertical balance throughout the complete process of immersing, re-floating and transporting the elements. If the element should float, then no ballast concrete is present, and the tanks are emptied. When the elements are resting on the foundation ballast concrete is installed and the tanks can be removed.

This process of interaction between the ballast concrete and ballast tanks is a well-known process used in the standard procedure of immersing the tunnels. For re-floating some aspects are the same as when the element was installed but for a few the circumstances are changed.

Construction method of tanks

For example, the material of which the tanks are construction are still the same. The tanks are constructed from a PVC liner backed with plywood panels which then are supported by steel columns. This method is relative cheap, and materials can be recycled for future tanks. A design choice must be made regarding the dimension and position of these tanks.

The construction will be mainly done by personnel with light equipment since everything should fit through the doors in the bulkheads.

Removal method of concrete

With the normal construction the ballast concrete can be placed relatively easy since the concrete is in a liquid form. For removing this concrete, the solid material should be cut, drilled or blasted in smaller pieces. This should be done by personnel since in this stage the bulkheads are already installed, and no big equipment is at hand.

The thickness of this ballast concrete layer can reach up to 1 meter. In the original design of immersed tunnel, the removal of the elements is not considered, and the design is optimized in such a way that the least amount of ballast concrete is needed. To be able to gain the original freeboard all the ballast should therefore be removed.

The precision of this process should be in the orders of centimetres. Special care should be taken to not remove any of the structural concrete since this can compromise the structural capacity of the elements.

Dimensions of the tanks

In the 'average' immersed tunnel four ballast tanks are present during the immersion process. These tanks are located close beneath the hoisting eyes. This location is preferred since it results in the lowest shear forces and moments in the structure during transport. The width of the tanks can reach from wall to wall and a small bridge is constructed over the tanks.

The main reason why the water is not let in freely in the elements is for stability reasons. Smaller compartments result in less sloshing which have a negative impact on the stability. For these reasons the length of the tanks is limited to about 25-30 meters, when the tanks are longer the sloshing of the water can become too much and the element might become instable.

The height of the tanks is limited by two factors. The first is if both walls are used as part of the tanks, if this is the case there should be about two meters free height above the tanks. This height is needed for personnel to walk over the tanks. If only one wall is used, a path with a minimal width of about two meter is needed next to the tanks. The height of the tanks is then limited by the space needed to install water pumps and pipes, but also inspection of the tanks should be possible. An example of a ballast tank constructed with using 1 wall as part of the tank is shown in Figure 9, Installed ballast tanks.

FIGURE 9, INSTALLED BALLAST TANKS (BABER & LUNISS, 2013)

Positioning of tanks

As mentioned before, in an average tunnel the tanks are located beneath the hoisting lines resulting in the lowest forces. These lines are connected at about ¼ and ¾ of the tunnel length. When re-floating the elements, this longitudinal position can be remained. For the decision if either one or two walls are implemented (or even none) in the ballast tanks an extra factor becomes important compared to the standard procedure.

The prestressing of the elements, which was previously located in the concrete cross-section, is destroyed after initial placement. This prestressing should be applied somewhere else in the tunnel cross-section. This results in space needed for these prestressing wires. The location is to be determined when considering the technical requirements. In the original design the strands are in the roof and the floor of the cross section. Therefore, it would be logical to apply them in the same location. Construction them on the floor will be preferred since it is easier accessible when constructing.

This results in three basic design options which can differ based the specific cross-section of the tunnel. These options are shown in Figure 10, Ballast tank design lay-outs, using a cross section with two traffic tubes and a middle escape tube. Option A is constructing the tanks next to the inner walls, this leaves space for the prestressing wires to be put closer to the outer fibre of the element. This is more favourable regarding the prestressing but is less favourable for the stability when lifting the element.

Option B is comparable with option A but then the tanks located near the outer walls. Option C is with tanks stretching over the entire width of the elements. This means all the prestressing should be applied to the roof or be in the escape tube.

In none of these variants the ballast tanks are placed in the escape tubes. Firstly, the escape tube is perfect access route through the tunnel for both materials and personnel. Secondly not all the immersed tunnels have such an escape tube and this design is made such that it is applicable to most of the currently constructed immersed tunnels.

A final decision for which lay-out to use can be made if more is known about the prestressing and other aspect. This is done after chapter 4.

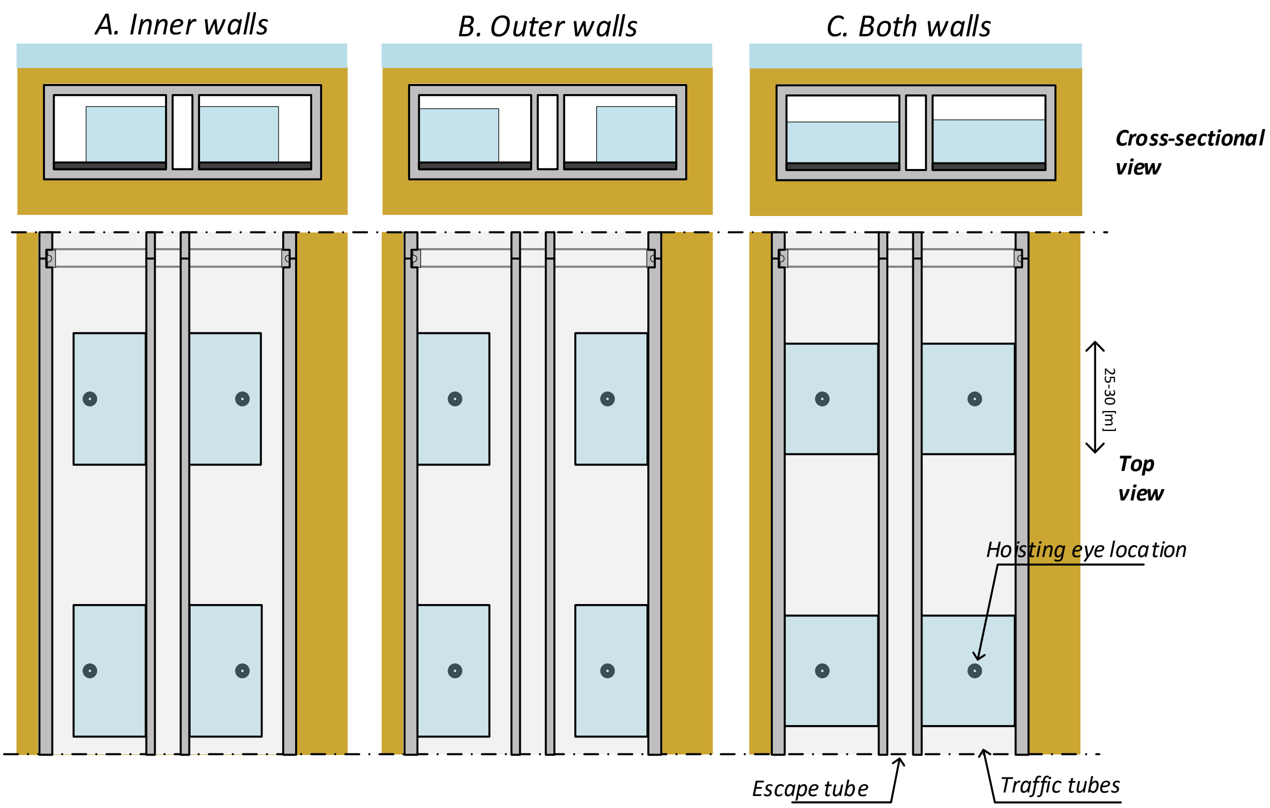


FIGURE 10, BALLAST TANK DESIGN LAY-OUTS

Phasing

As mentioned above the primary function of these ballast tanks and the ballast concrete is to regulate the weight of the element and determine the positive or negative buoyancy. During the construction of these ballast tanks the element should stay immersed with a certain factor of safety.

When all the ballast concrete is removed at once and no ballast tanks are constructed the element is too light and will start floating. But constructing all the ballast tanks when the concrete is still in complicates the removal of concrete since only a limited amount of space is available. This process will therefore be carried out in phases.

- A ballast tank will be constructed and filled. The water will be pumped in through valves in the bulkheads, enough water is present in the river and this will be used
- Part of the concrete is removed
- A ballast tank is constructed on the free space
- More concrete is removed
- More ballast tanks are constructed
- Et cetera

Since the element is designed with just enough freeboard without any ballast concrete all the concrete should be removed to be able to float again. The result will be an element without any ballast concrete and a controllable buoyancy made possible with ballast tanks.

Safety

The ballast tanks form a vital position in the stability and even the possibility of floating the tunnel elements. In case of a partial failure of these ballast tanks the water shouldn't be flow freely in the element. To prevent this a water leakage collection point is constructed close to the location of the bulkheads. This leakage is collected and siphoned back into the tanks by a pumping system. Also, camera's and pressure sensors are installed the measure the water level in the tanks.

2.3.2 Installing bulkheads

To close the tunnel tubes, construct a box and be able to float the element, bulkheads are installed on the end of the element.

Construction of bulkheads

The conventional method of constructing bulkheads is in a drydock in near perfect circumstances. Most of the bulkheads are a combination of concrete and reinforcing steel. When the bulkheads are constructed in immersed state first the steel columns are placed, followed by the formwork and the concrete is cast, as shown in Figure 11. To enable access to the closed element, hatches are installed in the lowered part of the bulkhead, these can be shut watertight. Also, valves are installed for electricity, water transport and ventilation. A special valve connection is constructed for the access shaft, this shaft is discussed in chapter 2.5.1.

The dimensions of this bulkhead depend on the forces during transport. The concrete and steel combination bulkheads installed in the drydock are in the order of 30 centimetres. In order to have enough space to work between the bulkheads and place the Omega-seals, the bulkheads are placed at an indent of about 1 meter into the element.

Compared to constructing in a dry dock the bulkheads are not installed on a clean flat part of structural concrete. Now they are placed partly on the structural concrete but also on the rests of the ballast concrete. To ensure water tightness some sort of epoxy material or rubber is needed to seal the edges.

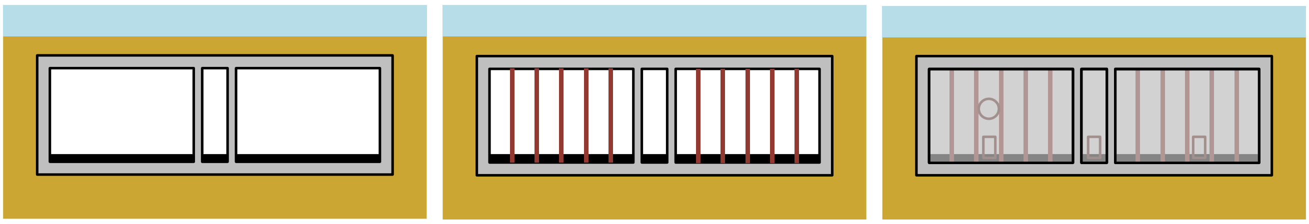


FIGURE 11, BULKHEAD CONSTRUCTION PHASING; CROSS-SECTIONAL VIEW

Phasing

The bulkheads are the biggest obstacle in the tunnel elements, when placed there is no possibility for big equipment to be used since everything should fit through the small door in the bulkheads. Therefore, the construction of the bulkheads is delayed as much as possible. But on the other hand, they form a vital part in the safety for personnel working in the element, this is discussed in the next paragraph. The construction will take place after the first few ballast tanks are placed and most of the material is in the element. At this time nearly none of the ballast concrete is removed.

Safety

The design code mentions that during every step in the process, a double water barrier should be present. During operation of the tunnel this is the GINA-seal and the Omega-seal. But when the ballast concrete is removed and ballast tanks are placed the element might shift a little bit, this will result in leakage in the existing seals. Therefore, an extra barrier is needed, this will be the bulkheads. It is important to construct these bulkheads with care such that they are waterproof.

To secure this test will be carried out. In the drydock these tests are carried out by raising the water level and checking if there are any leaks. In the worst case the bulkheads will collapse, and the elements are flooded. The amount of damage will be minor since the water levels are then lowered and new bulkheads are installed.

With the constructing being underwater there is no possibility of having such a controlled test and if a failure occurs constructing new bulkheads will be massively complex and have high costs or it might even lead to irreversible damage. Another type of test is needed; while the elements are immersed the valves used for pumping water through the bulkheads into the ballast tanks are already present. Through these valves water will be pumped into the space between the heads. This will show is there are any leaks. The disadvantage of this option is that the pressure achieved by pumping the water in is much lower than the pressure of a 20-meter water column. The structural capacity of the bulkheads is therefore still a risk. The method described above is shown in Figure 12, Bulkhead watertight testing phasing.

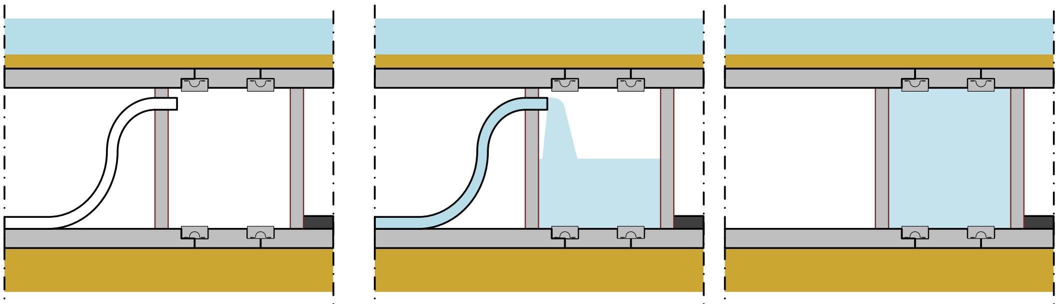


FIGURE 12, BULKHEAD WATERTIGHT TESTING PHASING; LONGITUDINAL VIEW

With these safety measures in place there is still a possibility of the bulkheads failing. In case this happens the flooding of the complete tunnel should be prevented. This is done by constructing more than 1 bulkheads at a time. During work in the elements 3 bulkheads are installed. The first one is the primary bulkhead and the end of the element, the second is the secondary bulkheads blocking the water. The third bulkhead is the primary bulkhead of the element to be floated next. This is shown in Figure 13, Bulkhead failure safety. The probability of double failure of the bulkheads (i.e. failure of the 2nd secondary bulkhead) is assumes to be acceptable.

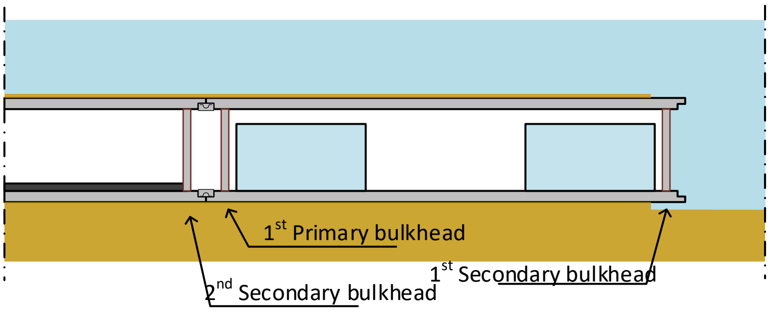


FIGURE 13, BULKHEAD FAILURE SAFETY, LONGITUDINAL VIEW

2.3.3 Reconnecting hoisting eyes

For the lowering of the element it is connected by wires to pontoons floating above the element. After placing on the foundation, the hoisting eyes and other equipment are removed. These are a risk regarding ship anchors which might hook behind one of these elements.

For lifting the elements these eyes should be installed again. The previously used locations won't be available since the bolts are removed and have been exposed to the water for a few decades. The new anchor connecting the hoisting eyes with the concrete are installed by divers.

To be able to install these connections the top gravel or sand layers should already be removed. Also, regarding the previously mentioned safety reasons the connections should be installed as late as possible. Resulting in the lowest risk of a ship anchor attaching itself to the tunnel. An example of a hoisting eye is shown in Figure 14.

FIGURE 14, HOISTING EYE ON TUNNEL ELEMENT (BABER & LUNISS, 2013)

2.3.4 Removal of shear keys & omega seal

Shear keys

If each segment is lifted on its own, then depending on the type of tunnel shear keys may be present. If the tunnel uses dowel and/or steel shear keys these can be removed. If a half-joint or discrete shear key is used the shear key function is removed when the element is horizontally moved away from the other elements.

Omega seals

Once a safe water barrier is supplied by the installed bulkheads and this barrier is tested the Omega-seal can be removed. These seals are bolted on the tunnel and then covered by infill concrete. This removal is done by hand, Figure 15, Space between bulkhead after placing elements (Courtesy of Kent County Council, BAM Nuttall/ Carillion/Philip Lane) shows the small space between the bulkheads, where the Omega seals are located.

There is no need to hurry the removal of the Omega-seal since it provides an extra barrier for the water. Therefore, removal of the Omega-seals is one of the final steps before moving on to the next 'chapter' in the process.

FIGURE 15, SPACE BETWEEN BULKHEAD AFTER PLACING ELEMENTS (BABER & LUNISS, 2013)

2.3.5 Applying prestressing

Depending on the type of tunnel the tunnel elements are prestressed together during transport. For a segmental tunnel post tensioned prestressing is applied in the concrete cross-section. These wires are stressed in the drydock. The segments are then clamped together, transported, immersed and when the tunnel is in final position the prestressing wires are cut for allowing higher differences in settlements per segment. For monolithic tunnel this prestressing is not installed.

As mentioned above the method described in this chapter is for a segmental tunnel. For the re-floating process the obvious steps would be to reverse the steps for the immersion process. But since the wires are cut there is no possibility to use the previous installed prestressing.

Variants for the original prestressing

Several solutions are possible to this problem. The first question to be answered is if these elements even need to be lifted all together or can they be lifted all separately? Secondly is there a possibility the lift all these segments simultaneously without prestressing. This leads to the following four variants, note that the segment joints as drawn are not representative of the original segment joints and no ballast tanks are drawn.

A. Lifting each segment separate

In this variant each segment is lifted as single segment, as shown in *Figure 16, Lifting single segment*. To execute this method loads of bulkheads need to be installed. For each segment a pair of bulkheads is installed, all these need to be tested and made sure to be watertight. Also, for each segment hoisting point need to be installed by divers.

The main advantage of this is that there is no need for a renewed prestressing. This saves much time and materials and therefore money. This also eases the process of installing the ballast tanks again since no space is needed for the prestressing cables.

The main disadvantage of lifting these segments one by one is that the watertight segment joints also need to be destroyed. In most cases this would be a W9U-i seal. For destroying these seals, the same method for destroying the initial prestressing can be used.

But when considering the complete process of not only re-floating, but also the immersing process and a new tunnel this leads to significant problems. Somehow these elements should be connected to each other in a watertight manner. The W9U-i profile is incorporated in the concrete when casted, so this method is not available. The solution might be to handles the segment faces like element faces and install GINA-profiles and Omega profiles et cetera.

Moreover, when the elements are casted in the drydock, they are directly next to each other. This results in practically no slack between the segments. This results in risk of destroying the segment face, especially when a half-joint type of shear key is applied.

Also, a quick calculation by hand shows that only for certain tunnels it is possible the gain a high enough buoyancy for such a small element. When considering the disadvantages and risks above the method 'A. Lifting each segment separate' is such a complicated process that this option is very unfavourable for re-floating an existing tunnel.

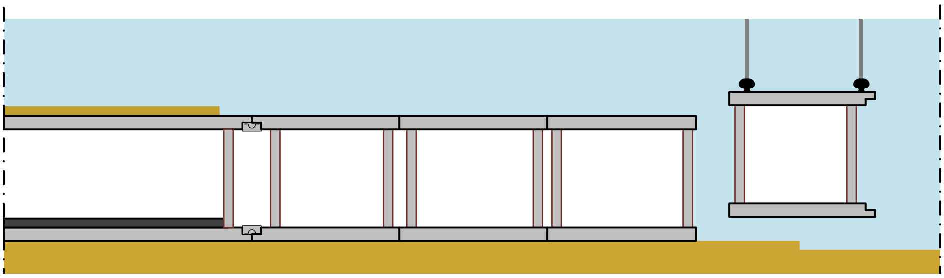


FIGURE 16, LIFTING SINGLE SEGMENT; LONGITUDINAL VIEW

B. Lifting segments together, hoisting per segment

In this variant the segments are not disconnected and lifted all together (Figure 17Figure 17, Lifting multiple segments together**)**. Compared to the previous method this method does not require instalment of bulkheads at each segment, this leads to less security checks and a lower risk of failure of the bulkheads. Also, the watertight segment joints can stay in place since the complete element is lifted at once.

To compress the elements no external prestressing is used but the forces of the horizontal water pressure are utilized to push the elements together. When the elements reach the water level the bottom of the element is still under a pressure of about 0.1 MPa (assuming a 10-meter-high element). To prevent an element from 'dropping' out between the elements the fluctuation in water pressure due to current and waves is not safe. The solution would be to lift all the segment simultaneously, this requires a set of hoisting eyes to be constructed on each element. This is much work for the divers but is not impossible.

This principle is only possible if any longitudinal movement between the segment is blocked. This can be done by applying prestressing inside, this is a completely different method and is described below. A source of this longitudinal movement might be any movement in the lifting pontoons, this can be solved by connecting all these pontoons and thereby blocking any relative movement of the pontoons to each other.

Another essential requirement for this principle is the uniformity in vertical position. Measures should be taken to make sure that each of the lifting pontoon used the same lifting rate. If this uniformity is not assured forces might act on the half-joint segment joint or the W9U-i profile for which it was not designed. This can lead to rupture and leakage. When this occurs, the complete method falls apart since the horizontal water pressure is not only present outside the elements. The element is flooded, and buoyancy is gone. This is such a high risk that method 'B. Lifting segments together, hoisting per segment' is not favourable without using prestressing forces the secure the movement.

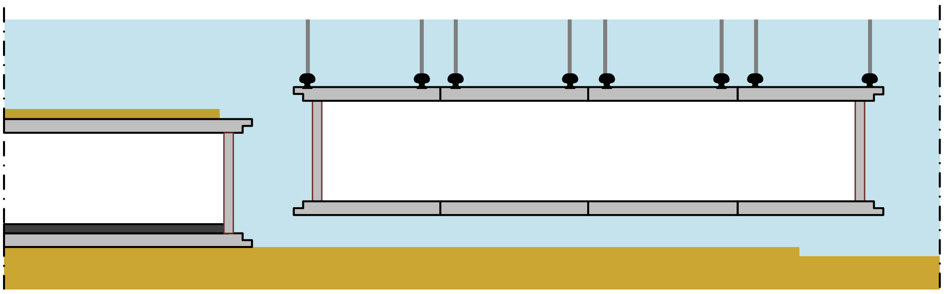


FIGURE 17, LIFTING MULTIPLE SEGMENTS TOGETHER; LONGITUDINAL VIEW

C. Lifting segments together, hoisting per element, segments prestressed

This variant is the closest to the reversed process of a normal immersion. When the element is still on its sand or gravel foundation prestressing is applied. In the normal method this is applied in the concrete cross-section but as mentioned before this is not possible in this situation.

The solution would be to apply external prestressing (external meaning outside the concrete cross-section but might still be inside the tunnel element) (see Figure 18). This is an established method in bridge design, especially in concrete box bridges. The main difference compared to the design in bridges is that this process is underwater and is only temporarily. While the construction of a bridge is on a clean construction site but is designed for a complete tunnel lifetime. Installation of external prestressing in already existing structures is also common practice to strengthen structures such as bridges. So, installing in an already used tunnel is not completely new.

The advantage of this method that only one set of hoisting eyes need to be installed and only one pair of bulkheads is used to make a complete watertight element. This method also does not destroy any of the segment joints, so they don't need any repair or adjustment while the element is prepared for a new immersion.

A disadvantage is that in the design of the ballast tanks space is needed for the prestressing. This might be averted if the prestressing is installed outside of the element. But this is a not a common method since it is not the same as installing on the outside of a box girder bridge due to the presence of water.

The amount of prestressing needed is case sensitive and differs for every tunnel and location. Also, the route and location of transport play a role in this. For transport over sea higher loads are induced on the element which leads to more prestressing required.

To show the feasibility of this method for a certain case a more detailed calculation is needed. This should for example include the moment at which the prestressing is applied since a maximum foundation pressure is given by SATO for which the strands can be stressed. Also, settlements between the segments might have occurred resulting is small gaps between the segments, these gaps can't be closed by the prestressing wires while it is on the foundation due to the friction of the elements. These gaps are closed when the elements are lifted but this results in a reduction of the prestressing force. This more detailed design is made in section 4.1.5.

Summarizing this shows that method 'C. Lifting segments together, hoisting per element, segments prestressed' is a viable option. Further on, more details of this method are discussed.

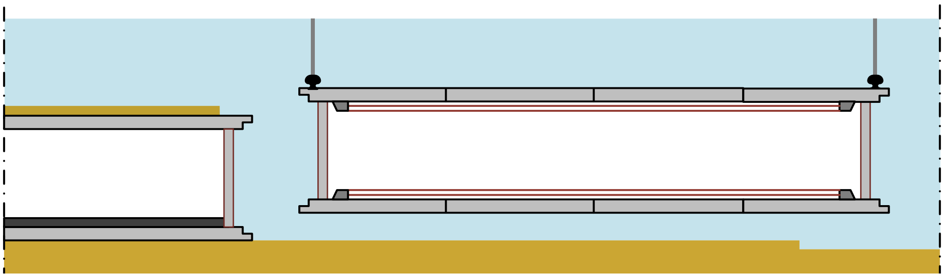


FIGURE 18, LIFTING SINGLE ELEMENT, INCLUDING PRESTRESSING; LONGITUDINAL VIEW

D. Lifting segment together, more than one element

This method is using the same principle as method C, but with a small difference. In option C the original element length is chosen to be floated at once. But if the prestressing forces allow for longer element to be lifted this might be advantageous (see Figure 19*).*

Lifting longer elements results in fewer lifting operations, and therefore using less bulkheads and hoisting eyes. This then might lead to a lower cost. A disadvantage is that a segment joint is opened or even a segment is cut in half. As mentioned in option A this leads to complication during the construction of the new tunnel.

Another option would be to lift two complete elements at once. This new element would have a length up to 200 meters and the possibility of this depends on which case and the local conditions. The advantage of this is than an extra immersion joint of GINA-profile and Omega-seal is still intact. The disadvantage is that no other type of GINA can be installed (which might be useful, allowing for bigger immersion joint rotations). It is assumed option D only attractive if for a specific case option C does not cover a complete solution to the re-floating method

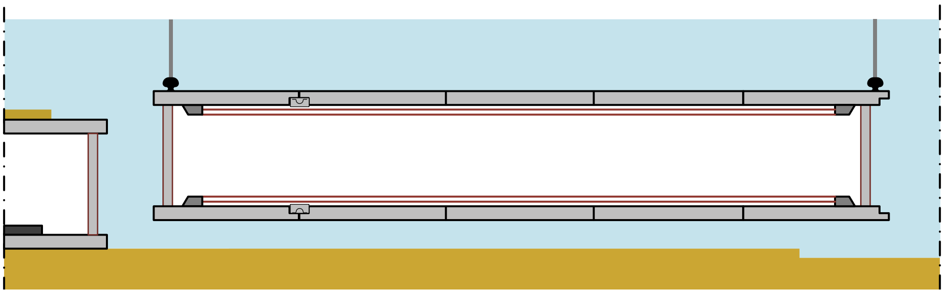


FIGURE 19, LIFTING MULTIPLE ELEMENTS, INCLUDING PRESTRESSING; LONGITUDINAL VIEW

In Table 1 an overview of these variants is given.

Variant	Advantage	Disadvantage	Conclusion
A. Hoisting segment	No prestressing	Destroying the W9U-I profiles Collision damage Lots of bulkheads	Not favourable
B. Hoisting element, per segment	No prestressing Limited bulkheads	Destroying the W9U-I profiles High risks of collapse	Not viable
C. Hoisting element	No destruction Limited bulkheads	Prestressing	Most favourable
D. Hoisting element+	No destruction Possible less lifting	Prestressing Partly destruction	Viable

TABLE 1, PRESTRESSING VARIANTS OVERVIEW

Details for new prestressing design

The result of the above 4 variants is that the most favourable option for having prestressing for lifting the elements is method C. This method used external prestressing, as mentioned option D might also be applicable. Below several design variants are shown and explained.

Connecting wires to the concrete

Two options main options are available with respect to the location on which the prestressing wires are connected to the elements. The first would be to connect the wires the two most outer segments clamping the other segments between them. The other option with be to connect the wires at each element and clamping every joint.

The advantage of the first is a lower amount of connections between the wires and the concrete, the location of these connections in then close to the location of the hoisting eyes, if this in an advantage is not yet known and will be checked. Also, the longer wires result in a lower relative lowering in the prestressing forces due to the mentioned gaps in the segment joints due to the settlements.

The advantage of the second variant is that a lower amount of space is needed for installing the prestressing. Also, a different amount of prestressing can be applied depending on the location of the element. The third argument would be that the material costs for prestressing wires is very high and this solution might therefore be cheaper. Both are shown in *Figure 20, Prestressing design variant (1/2)*.

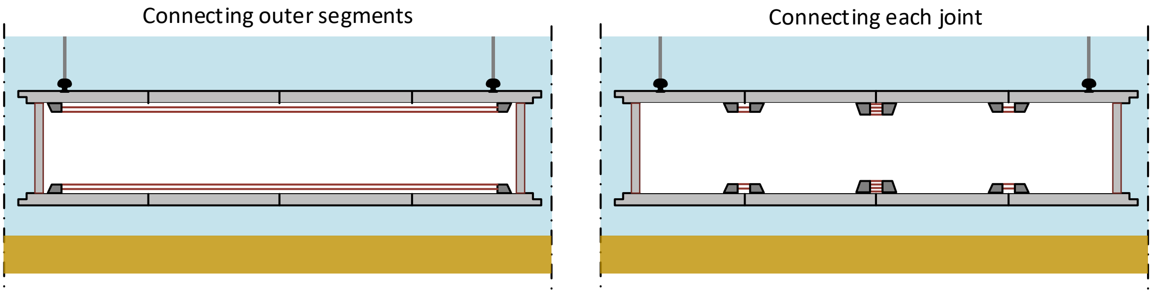


FIGURE 20, PRESTRESSING DESIGN VARIANT (1/2) ; LONGITUDINAL VIEW

Tendon shape and location

When the prestressing is initially applied it is in the roof and floor of the element. In most designs the amount of prestressing in the roof is higher than the amount in the deck. This is needed since more tension is to be expected in the roof than in the deck. (*Figure 21, Prestressing diagram in the Wijker tunnel (left is the roof, right is the deck)*)

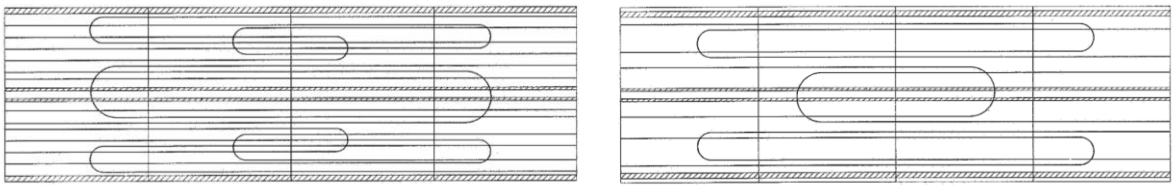


FIGURE 21, PRESTRESSING DIAGRAM IN THE WIJKER TUNNEL (LEFT IS THE ROOF, RIGHT IS THE DECK)

In the design of an external prestressing this distribution can also be applied. It should be noted that installing prestressing equipment on the roof is complicated compared than installation on the floor.

Another variant with more tension in the roof is to apply curved or bend tendons instead of straight tendons. These can be connected to the walls or to the floor and the roof close to the walls.

If the amount of prestressing is too high or there is no location to be applied a combination of the variant B and C might also be possible, the use of an extra pair of hoisting eyes is less complication than hoisting eyes for each segment joint. The exact location and impact of lay-out is calculated further on but the concept is shown in ***Figure 22, Prestressing design variants (2/2)***.

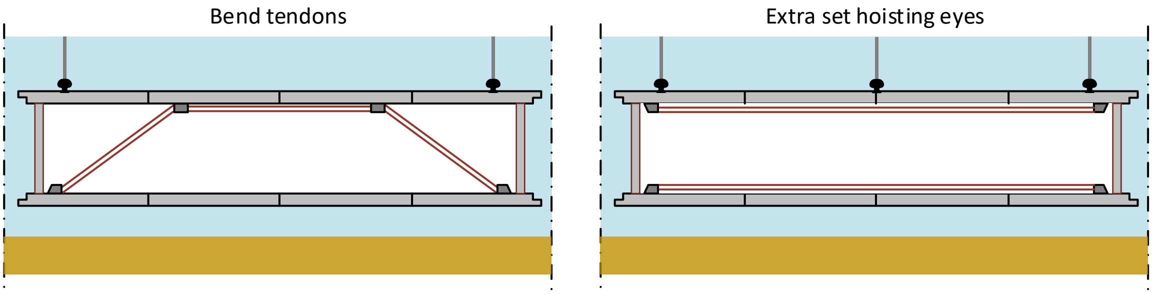


FIGURE 22, PRESTRESSING DESIGN VARIANTS (2/2) ; LONGITUDINAL VIEW

Also the placement of the ballast tanks might be a limiting factor to where the prestressing wires can be placed and connected to the tunnel element. As mentioned above the cables will be placed inside the cross-section and is placing near the deck easier. Regarding the structural design it is more useful to place the prestressing closer to the outer fibre.

Regarding the choice which variant is the best two main criteria area available. Firstly the chosen lay-out should be constructible, meaning a minimal or maximum amount of wires (depending on the tunnel dimension and ballast tank lay-out). Secondly the variant with the lowest cost is the most attractable.

2.3.6 Installing other equipment

Furthermore, some equipment is needed for monitoring and operational safety. The last step before closing the elements is installation of camera's and monitoring equipment, an example to monitor is if there is any leakage through the bulkheads. Also as mentioned above, the ballast tanks can be leaking, this is not only monitored with camera's but also using water pressure sensors in the tanks. Finally monitoring is needed in the essential valves responsible for operating the ballast system during re-floating and floating.

When re-floating the elements, they will be disconnected from the other elements and for a small period they are disconnected from any power supply. Power packs are installed to provide the element with power during the removal operations.

Then the final steps before the cutting process starts are taken. The power supply to the outer banks of the tunnel is disconnected and the installed power packs take over. All personnel and equipment leave the elements and the bulkheads are shut.

2.4 Removing the first section

Similar to the specially design closure joint in the original design a specially designed opening joint is required when removing a first section of the continuous tunnel. Removing this section breaks the chain and depletes the normal forces due to the closure joint and replaces it with water pressure.

This section is a description of the possible and preferred methods of removing such a first section. In the previous chapter the preparation of the tunnel is already described.

2.4.1 Removing soil cover and backfill

The inside of the tunnel is accessible through the bulkheads. But for removing a first section, the section should also be accessible from the outside. On top of the tunnel is a cover consisting of soil or gravel, this needs to be removed. Also, the trench next to the element is backfilled after placement of the tunnel and needs to be removed.

The cheapest method of removal is with cranes from land, this is only possible at the closure joint, if close to shore. If this is not possible the equipment can be floated in with pontoons and the soil is dumped in barges. Two main types of equipment are available. A backhoe dredger or a grab dredger. The first having the advantage of a very controlled removal and a limiting depth of 20-25 meters depending on the crane arm (TEC, 2017). The grab dredger has the advantage of being able to remove bigger rocks but is less precise. Figure 23 shows both. A choice for which equipment to use depends on the actual situation.

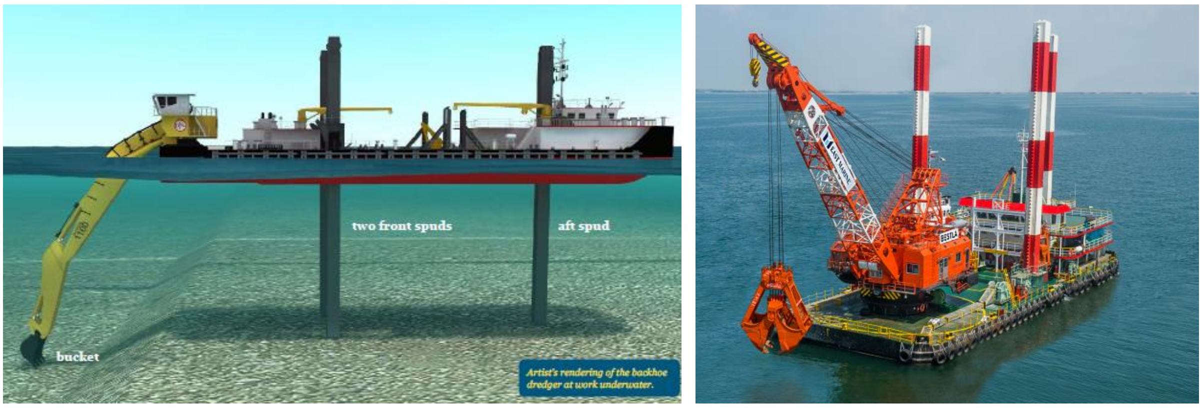


FIGURE 23, DREDGING EQUIPMENT (TEC, 2017)

Depending on the type of cutting method even removal underneath the tunnel might be necessary. This can be done by sucking pipes underneath the element. It is not possible to let divers beneath the element since this is too risky.

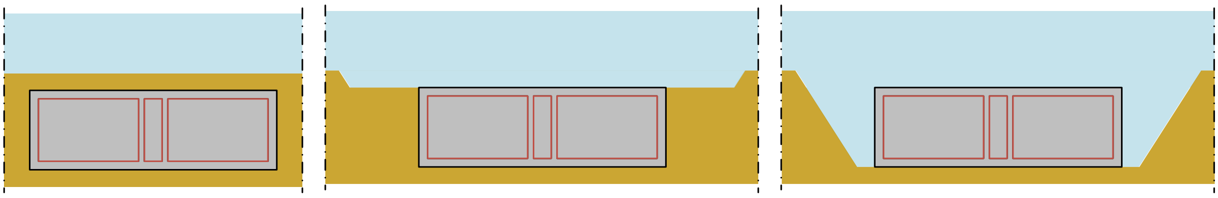


FIGURE 24, SOIL REMOVAL PHASING; LONGITUDINAL VIEW

The order in which the removing is done is shown in Figure 24Figure 24, Soil removal phasing. When the soil layer is removed the first divers can installed equipment on top of the element if needed. For example, hoisting eyes but this depends on the cutting method.

2.4.2 Determination of cut location & opening joint width

As mentioned, special care goes to the construction of the closure joint. When looking for a location to start the cut, it seems the obvious answer to start at the closure joint. When deciding which location to start the following aspects should be considered.

The order of construction combined with the type of nose & catch structure or discrete joints might result in having only one way of removing elements. When the elements are slightly tilted during lifting only the 'inside' of such a joint can rotate since the outside is pushed against the foundation. If this type of joint is the case the cut should start at the closure joint.

Since the closure joint is constructed in situ and the fact that during cutting some of the material might be damaged the exact parameters of the materials is not known. Therefore, the closure joint material won't be recycled in the constructed of a new closure joint (however the concrete debris might be recycled for other purposes).

The length of the so called 'opening joint' depends on other requirements for the initial gap. For example, an access shaft will be mounted on the bulkhead. This access shaft will be discussed in chapter ***2.5.1*** but has a minimal diameter of about 2 meters.

Also depending on the longitudinal movement of the elements before lifting extra space is needed. It is assumed that this distance is maximum 50 centimetres. The final requirement for the opening joint width is that it should have enough space for unexpected movement during lifting without bumping into the other element. This spacing should be present at both sides.

A minimal opening joint width of 5 meters is set, leaving just above 1 meter of slack between the elements when lifting.

If the original closure joint does not have the required length a part of the first element should be cut as well. This leads to the first element being a bit shorter when lifting. For the immersion process later, this means that a very big closure joint should be designed or even an extra short element is needed. This element can also be designed to handle the bigger rotations in the joints as discussed in chapter ***2.6.1***.

2.4.3 Determination of cutting method & shape

The actual cutting of the concrete can be executed with several techniques. These can be separated in roughly two categories, the demolition of concrete or the cutting of concrete.

The demolition of concrete is achieved by drilling or blasting. Drilling in the concrete is a very intensive process and loads of holes are needed. Blasting is the result of high energy impact loads which rupture the concrete. This method is useful for removing large parts of concrete but is less precise. For the removal of steel tunnels such as the Baytown tunnel in Texas shape charges are a good alternative for blasting.

The other method involves cutting mechanisms. These are sawing, water jetting or wire cutting. All these options have a very local impact on the construction. The advantage of this is that the concrete next to the cutting line is not affected. A disadvantage is that sawing an element of 30 meters wide and 8 meters high is much work. Another disadvantage is that for example wire cutting is not available for cutting internal concrete, such as the half-joint, without damaging the external part of the half-joint. A combination of these methods might be the best solution. Using the strong aspects from each method an and supplementing each other weaknesses.

With both blasting, cutting or a combination of these two there is friction to be expected between the pieces when lifting. Especially when the block starts to tilt and leans against the elements which stays in place. To ease the removal of the block two type of measures can be taken. First some kind of guide structure or rail can be installed, this should be done by divers. Depending on the type of cutting these guidance structures are installed before or after cutting. Another solution would be to change the angle on which the element is cut. If the opening joint is cut in a V-wedge shape would this ease the removal (as shown in Figure 25Figure 25, Opening joint design shape variants). The opening joints is relatively small and will be hoisted without installing bulkheads. Note that wire cutting from a pontoon combined with a V-wedge is a very unfavourable option due to the angle of the wire.

In a further design the method of removing concrete, the structural impact and the shape of the piece to be removed should be investigated in more detail.

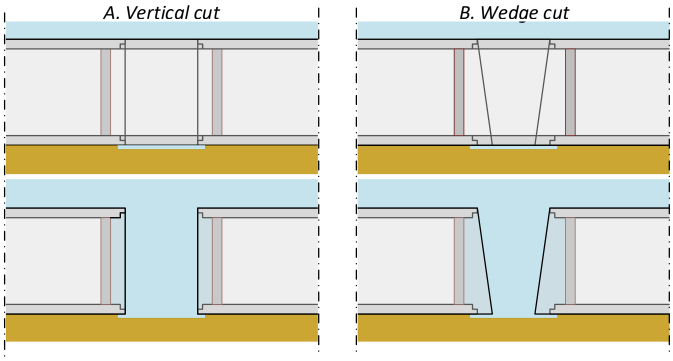


FIGURE 25, OPENING JOINT DESIGN SHAPE VARIANTS; ; LONGITUDINAL VIEW

2.4.4 Determination of settlement effects

When disconnection the chain of elements there is a loss of friction between the elements and the normal forces are taken over by the water. This loss of vertical friction can result in an increase in settlements. While cutting the element this jump in settlements might occur suddenly. If these settlements are expected measures should be taken to prevent an instant jump inducing high loads on certain parts of the elements.

During the operation stage of the tunnel the sum of the forces results in a downwards force of about 6% of the buoyancy of the element. (Baber & Luniss, 2013). A quick calculated is made to check to compare the effective soil pressure at the bottom of the structure before digging the trench and after placement of the element. There is no change in water level, so the effective stress increases the same as the total soil pressure. ($\sigma_{total} = \sigma_w + \sigma'$)

For a tunnel with elements of 100-meter-long, 8-meter-high and 30 meters the total buoyancy is about $240\ 000\ [kN]$ (not considering indents of the bulkhead and differences in water pressure.). When resting on the riverbed the downward force due to the element is 6% of this buoyancy (not considering the cover layer). The resulting downward force is $14\ 400\ [kN]$.

In the original situation the location of the element was filled with sand, a block of the same size consisting of wet, immersed sand has a weight of: $(\rho_{sand,wet} - \rho_{water}) * w * h * l = (19.2 - 10) * 30 * 8 * 100 = 220\ 800\ [kN]$. Realizing the weight of the original soil was much higher than the submerged weight of the element is it valid to assume no significant settlement differentiations are present.

2.5 Re-floating & Transport of main element

When the first element is cut the bulk part of the process can start, re-floating and transporting multiple, about 100 meters long, concrete floating boxes. In this chapter this process is explained starting at the prepared element and an already removed opening joint.

2.5.1 Securing access to element

During the complete process of re-floating and transport it is from the upmost importance to have access to the elements. The design is made such that no access to the element is needed but the consequences of a small failure inside the element combined with no access are enormous.

For example, leaking ballast tanks can be fixed by personnel entering the element. But also, the pipes transporting water, data cables for monitoring and power cables for the pump should have access to the element. In a standard immersion process such an access shaft is mounted on top of the element. After placement of the elements and full access is gained from within the tunnel the shaft is unbolted and removed, the hole is then permanently sealed.

Compared to re-floating is the process of installing an access shaft on the roof different. The element is under water and the shaft is installed by divers. A new hole should be cut and new locations for the bolts are prepared. This is a complex process and easier solutions are available. The solution would be integrating the access shaft in the bulkhead design. As mentioned in section 2.3.2 during the bulkhead design a special valve is installed on which an access shaft can be mounted. After removing the opening joint or after removing an element the next shaft is placed (as shown in Figure 26Figure 26, Access shaft placed on bulkhead).

The diameter of the shaft is determined by the materials and personnel needed to fit through the shaft. A detailed design of this will be made in a later stage but for now at least a diameter of 2 meter is assumed (allowing for a stretcher to fit through horizontally)

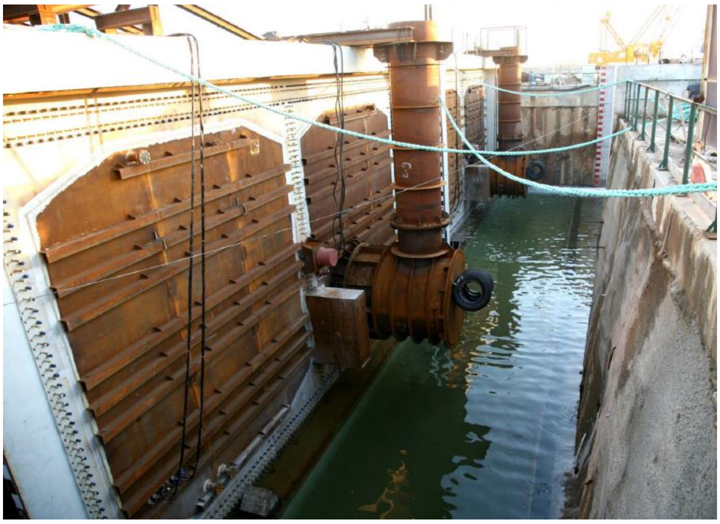


FIGURE 26, ACCESS SHAFT PLACED ON BULKHEAD (TEC, 2017)

2.5.2 Defining buoyancy approach

Regarding buoyancy two option can be distinguished. Being positive and negative buoyancy.

In the case of positive buoyancy, the amount of ballast removed results in an element that start floating from itself without hoisting pontoons. This process is not allowed since it causes enormous uncertainties in when the element starts floating (due to the soil adhesion discussed in chapter ***2.5.3***) and the elements in uncontrollable when starting to float.

The design is made such that the element has negative buoyancy. In an average design the four hoisting eyes are connected to the pontoons floating above and these lines are each preloaded with about $1000\ [kN]$, resulting in a total lifting force of $400\ [tonnes]$. When the element is lifted the ballast, tanks are gradually emptied resulting in a floating tunnel element with about $15 - 25\ [cm]$ freeboard and in the case of transport over sea the freeboard is a bit more.

2.5.3 Breaking the soil adhesion

When the hoisting wires are connected to the pontoons and start tensioning the immersed element an extra downward force develops. This is the element sticking to the foundation, the lifting of the element results in a under pressure in the water beneath the element. The amount of this adhesion force is unknown and will dissolve over time.

A quick conservative calculation is made showing the expected duration if no measures are taken. It is assumed that with a layer of water of 5 mm thick no adhesion forces are present (TEC, 2017). This duration is calculated assuming Darcy flow through the sand foundation. For the element dimension, the same standard element is used as in section *2.3.5*. This results in an expected flow duration of 84 days. This value is an extreme upper boundary since this process is progressive, but the total duration is still expected to be in the order of tens of hours, maybe days. The complete calculation is given in appendix I.

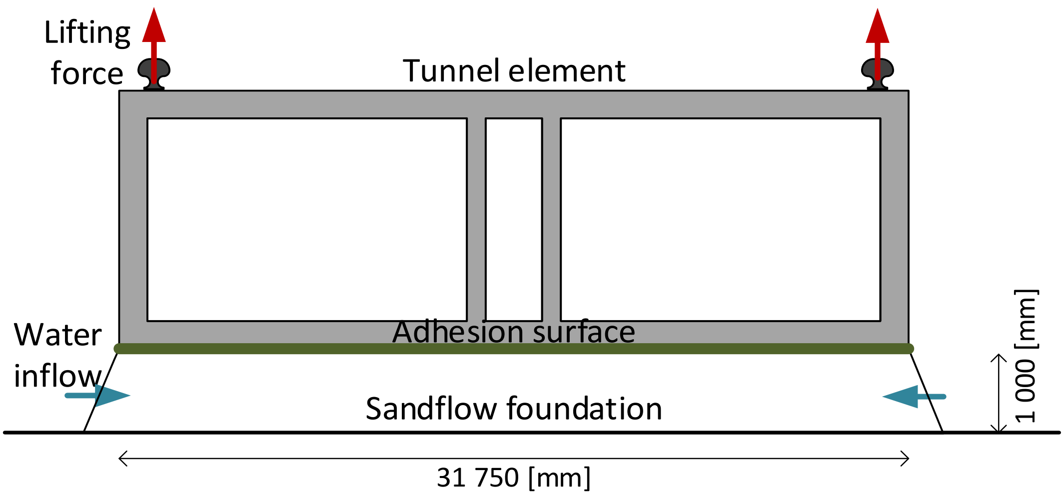


FIGURE 27, SOIL ADHESION SITUATION SKETCH

In the section mentioned above, an assumption was made regarding a minimal water layer needed, this was set at 5 mm. A master thesis by den Hertog regarding passive suction under mud mats did small scale test, this test loaded a small circular plate under tension and measured the displacement of the plate. These resulted in a breakout displacement of about 2 mm, as shown in Figure 28**Error! Reference source not found.**. (Hertog, 2017). Another research regarding lifting large object from a porous seabed also concluded a breakout distance of 2 mm. This is based on a 2D analysis and a Darcy flow without the element being on some sort of hill. Figure 29 shows the breakout in the lines being vertical at the dimensionless displacement ($\log_{10}\sigma$) reaching 0. (Mei, Yeung, & Liu, 1995).

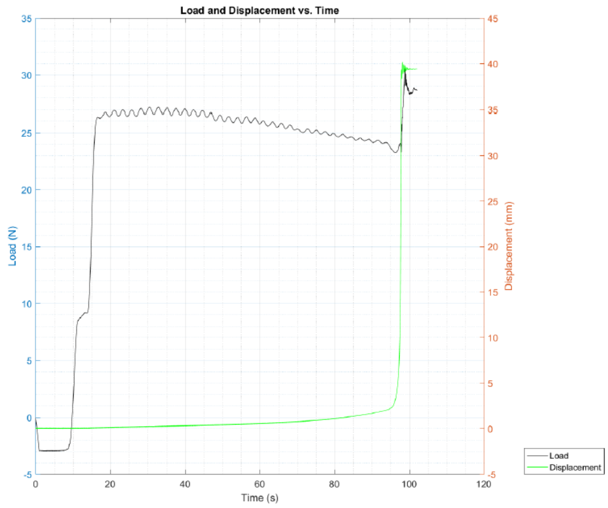


FIGURE 28, LOAD-DISPLACEMENT DIAGRAM (HERTOG, 2017)

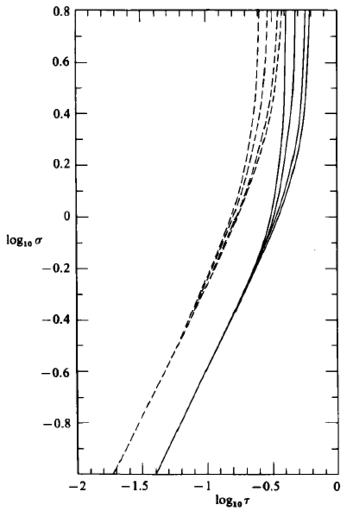


FIGURE 29, DIMENSIONLESS LOAD-DISPLACEMENT DIAGRAM (MEI, YEUNG, & LIU, 1995)

This second research also showed the difference between vertical lifting and lifting at a single side, these two methods are shown in Figure 30. The effect of this is in the order of 35-40% faster breakout when the same force is applied compared to a complete vertical lifting. This is however not applicable to this method since to be able the have the same lifting force the amount of wires attached to a single side should be doubled, this negates the 30-40% effect. Also, the exact location of the hoisting wires isn't exactly on the outer wall but partly inwards, reducing the effect even more.

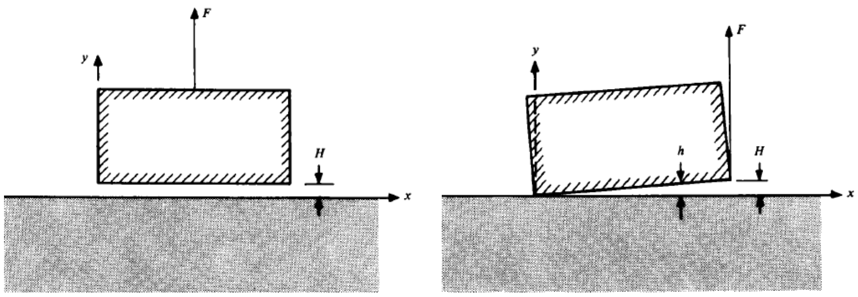


FIGURE 30, BREAKOUT CALCULATION SCHEME (MEI, YEUNG, & LIU, 1995)

These two methods described above both give very different results. Using the calculations from den Hertog the breakout is expected to be in the order of seconds, this is assumed to be not representative for the actual situation. Using the calculations from Mei the breakout is expected to also be in the order of weeks. For now, the same simple Darcy approach used in first is assumed to be representative.

Reducing waiting time measures

During all these hours or days pontoons are placed in the middle of the waterway and the navigation channel is blocked. Measures should be taken to minimize this blockage. In this chapter the possibilities are discussed. In a further detailed design, more calculation is needed for which variant to use. This calculation should also make clear which variant are preferred.

A. Higher lifting force

The assumed lifting force was set at $1000\ [kN]$. This number is based on standard operations during immersion process. But for the re-floating bigger equipment can be used (see *Figure 31, Breaking soil adhesion variants (1/2)*). In the Darcy the duration scales linear with the lifting force. It is expected that a double lifting force will not result in a half the duration, this due the progressive characteristics of the process. The disadvantage of this method is that bigger equipment should be available, and the structural capacity of the element should be checked on higher tension forces.

B. Removing soil underneath

Another solution would be to remove the soil underneath the foundation. This can be done from the water level using sucking pipes. It is expected that up to 3 meters can be sucked away underneath the elements. This results in a smaller flow length and in less water volume needed. The problem with this method is the complete removal of the foundation. After removal there might not be enough foundation to lower the element on if needed for whatever safety reason (for example failure of one of the hoisting lines). Another disadvantage of this method is that only a small part of the foundation is removed (up to 20% for a 30-meter-wide element), a better alternative would be the next variant.

C. Water jetting from sides

In this variant water is jetted underneath the element. The principle is the same as variant B, removing soil and removing the amount of water needed. The difference however is that this method allows for larger parts to be removed, up to 10 meters from both sides. (see *Figure 31, Breaking soil adhesion variants (1/2)*). The notion regarding the minimal foundation width staying in place is also important in this variant.

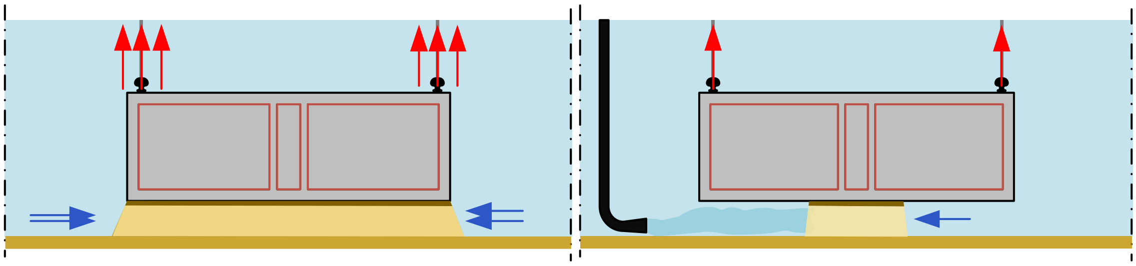


FIGURE 31, BREAKING SOIL ADHESION VARIANTS (1/2) ; LONGITUDINAL VIEW

D. Using previous systems

The fourth variant makes used of the already placed pipes. These pipes are used during the placement of the foundation. These pipes are connected to a sand flow installation and this sand is jetted underneath the element. Divers check when the sand is flowing out underneath the tunnel and the pumping can be stopped. These pipes are left filled with sand. Before using these pipes again special care should be taken regarding removing the soil from these pipes.

E. From inside the tunnel

The final option would be to drill holes in the floor of the existing elements and start pumping. These holes should be constructed in such a manner that no water flows into the elements. The advantage of this method that no use is made of divers and the holes can be installed relatively controlled in a dry condition. A disadvantage is that these holes should be constructed before placing the ballast tanks and/or prestressing. Both variant D and E are shown in Figure 32.

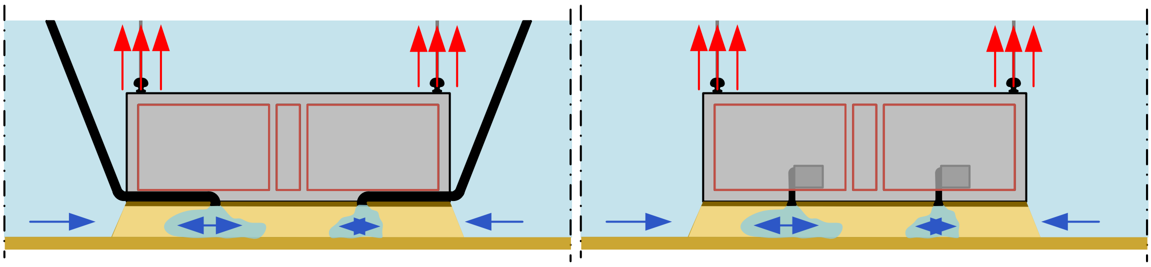


FIGURE 32, BREAKING SOIL ADHESION VARIANTS (2/2) ; LONGITUDINAL VIEW

As mentioned before more detailed calculation are needed to make better decision on which method should be used. This method may also differ based on each tunnel.

2.5.4 Transportation of the element

When the element is in floating stage it is ready to be transported. In the current position the elements are in the middle of the water and the time the waterway is unnavigable should be minimized. Since weeks, months or even more time is needed for adapting the bed and the new alignment. And since the preparation of the elements for renewed immersion is needed, the element should be transported to a safe location. The requirements for such a location would be that there is a safe possibility for personnel to work on the element, and that the hindrance in the navigation channel is minimal. Three options are available

A. The original drydocks

The elements can be transported to where they came from at first, the drydock. The idea is that the dock is inundated, and the elements are floated in, placed on a bed and the water level is lowered. It is very hard to lower the element simultaneously on the bed without load concentrations and deformations of the bed.

Also, the location of this drydock might be far away from the location where the elements are emerged, or the drydock is even removed. In the Netherland the only permanent drydock is in Barendrecht close to Rotterdam. This means that elements coming for example from the surroundings of Amsterdam need to be transported over the North Sea. In the original design the elements are designed for these higher loads due to waves and wind by applying extra prestressing. In the reversed transporting this is more difficult and transport over sea is not the preferred option

B. Next to the waterway, on shore

Another option would be to moor the elements very close by to the shoreline. The advantage of this option would be that nearly no transport is needed. A disadvantage is that this option blocks part of the waterway. The biggest problem is however the safety of the personnel working on the elements and installing new profiles and seals. On both the faces of the element is a temporarily construction mounted supplying a dry and watertight area to work in. However, such a construction is not designed to withstand the impact of a ship collision. So extra guidance structures preventing a ship collision should be installed (see *Figure 33, Tunnel element protected*). This is very expensive and therefore location the elements close to the original location is not preferred.

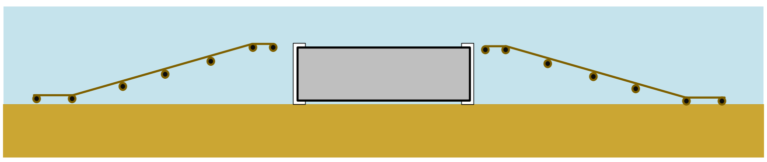


FIGURE 33, TUNNEL ELEMENT PROTECTED; TOP VIEW

C. In a nearby harbour, connected to a quay wall.

The final option would be to transport the element to a nearby port. It is assumed in most of the cases a port is relatively close by since one of the reasons to remove the elements was to deepen the waterway for ships to access a port. These ports have enough capacity and safety measures to handle loads of vessels each year. With average dimensions of 100-meter-long and 30-meter-wide the elements can be handled just like ships and moored to a quay wall at a safe location. This option does not block the waterway and is safe for personnel and is therefore the preferred option.

2.6 Preparing, adapting & reconfiguration

When the elements are safely docked away the preparation of the new trench and the new alignment can be started. Also, the emerged elements need to be prepared for re-immersing, maybe even adapting the shape of the elements is needed. In this chapter these preparations and adaptions are discussed.

2.6.1 Calculating new alignment

The problem stated in chapter 1 asked for a deeper waterway. This is the goal of the temporarily re-floating of the elements. This chapter is not aimed to provide an exact design of this deeper waterway, but this waterway does have an impact on the alignment on the new tunnel. In the study by Weeda it was assumed that the new alignment of the tunnel was made possible using rotations in both the immersion and the segment joints. It was not mentioned how to rotate the segment joints while the complete element is prestressed. This seems not feasible and such all the rotations in the new alignment should take place in the immersion joints.

The main two factors in the new alignment are the requirements regarding the capacity of the tunnel to adapt to rotations. The second requirement comes from the traffic, for example the maximum radii of the tunnel arcs compared to the traffic speed.

Rotational capacity

In this paragraph the possibilities regarding the rotations and joints are investigated based on an average tunnel with 6 elements and a length per element of 100 meters. The elements are 8-meter-high and for ease of calculations it is assumed than the GINA-profiles are located 200 millimetres below the outer fibre. This average tunnel is needed to create possible variants and show the advantages of the variants.

A very simply line model is constructed showing the maximum deepening. For the currently installed GINA-profiles an elongation of 60 mm is assumed. When the elements are prepared for re-immersing new GINA-seals can be installed. In earthquake sensitive countries larger type of seals or installed, already compensating for the expected movement of the tunnel. These new seals are expected to have a higher elongation of 90 mm. The rotation of the element is calculated by dividing the expansion in the joint by the distance to the rotation point. This point is assumed to be the other GINA-profile (securing no compression in the other GINA-profile). (See Figure 34)

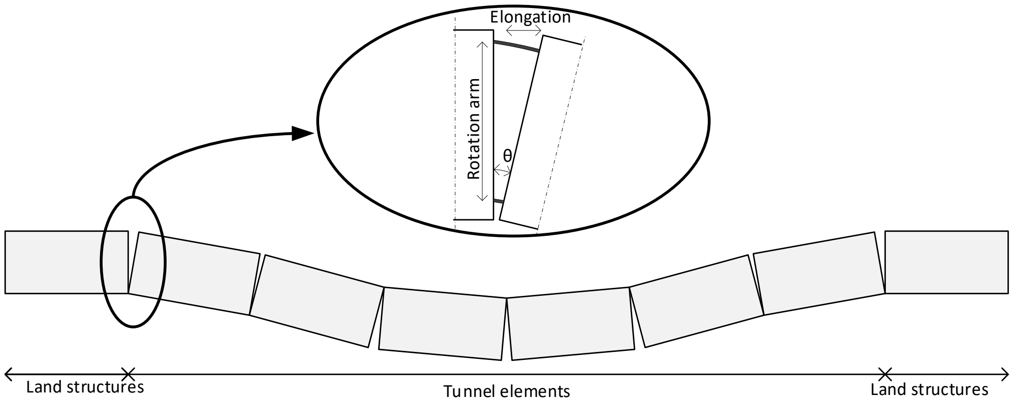


FIGURE 34, IMMERSION JOINT ROTATION SITUATION SKETCH

Extra rotation can be acquired in using two methods. Note that the option of completely lowering the land parts is not considered, this is discussed in section *2.6.4*. The first would be to construct a special type of closure joint, this joint can then be constructed in such a shape already compensating for the rotation in the dimensions of the concrete. This would result in one side having the configuration with possibly a bigger GINA profile or an adapted shape resulting in a rotation higher than the capacity due to the increased seal. This option is the same as using the same closure joint but adapting only one land head, but since a new closure joint already needs to be redesigned is this not preferred. This option is further on mentioned as the option with adapting ONE land head.

The other option would to install some sort of sheet pile construction around the other land head and adapt such that the element can already be placed at an angle. In such a case the maximum amount of rotation at the land heads is determined by the number of elements available to compensate for the rotation. For a tunnel with 6 elements this would result in a rotation of 2.5x the GINA-rotation-capacity at both land heads. This option is further mentioned as adapting TWO joints.

The complete calculation of these variants is discussed in appendix C. For the assumed average tunnel the results are shown in

	Variant 1; NO adaptions	**Variant 2; ONE adaption**	**Variant 3; TWO adaptions**
60 expansion	2.37	2.84	3.55
90 expansion	3.55	4.26	5.33

Table 2. It is important to note that these values are not based on a real tunnel and are chosen to be an average tunnel. The maximum expansion of the actual immersion joint depends on the actual profiles used and are case specific, also the number and dimensions of the elements might differ. Also note that this model is very simplified. It can be concluded that regarding rotational capacity a new alignment is possibly for the standard tunnel dimension but requires adaptions in the land head and/or closure joint.

	Variant 1; NO adaptions	**Variant 2; ONE adaption**	**Variant 3; TWO adaptions**
60 expansion	2.37	2.84	3.55
90 expansion	3.55	4.26	5.33

TABLE 2, MAXIMUM LOWERING FOR EACH DESIGN VARIANT

Traffic requirements

For the traffic requirements the boundaries are divided in the vertical and horizontal alignment. With deepening the tunnel there is no change in the horizontal alignment. The design for the tunnel vertical alignment is based on two parameters (Nes, Wiggenraad, & Lint, 2014). First, for longer slopes (+20 meters) the safety is governing, this safety is based on a difference in velocity between trucks and regular traffic. The reduction in truck velocity is shown in Figure 35 for different slopes. For highways this reduction is limited to 20 $[km/h]$.

The second parameter is the arc of the slopes. The maximum convex arcs are defined by the minimal amount of horizontal view needed. The view needed for various scenarios are known and for each speed this results in a maximum arc, the design values are given below. For the concave arcs the vertical acceleration is governing, this results in arcs in the order 10% compared to convex arcs.

Design speed $[km/h]$	**120**	**100**	**80**	**50**
Road course	12 400	8 300	5 000	900
Stopping	11 000	4 700	1 800	300
Dodging	12 300	8 100	4 700	1 100

TABLE 3, MAXIMUM CONVEX ARCS

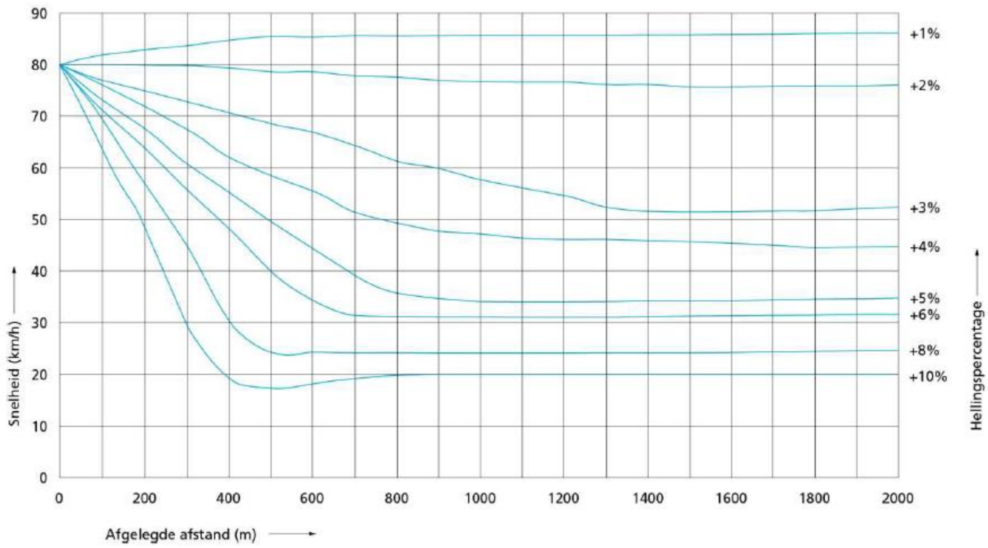


FIGURE 35, TRAFFIC REQUIREMENTS DESIGN GUIDELINES

2.6.2 Preparing element face and seals

During the lifetime of the tunnel several loads are applied on the front face of the elements and the GINA profiles attached to it. For the GINA profiles this might result in plastic deformations or a lowering in watertight properties. Also, during the lifting of the elements, the front face and profiles might get damages by concrete rubble from the opening joint, cables running through the bulkheads or other dangers present.

With the element afloat again, the front face should be inspected for damage before lowering again. Also, the (possibly damaged) GINA profiles need replacement. As mentioned, another reason for replacing the GINA profile might be the need of a different type of seal. This type might differ in height or rubber properties to allow for bigger rotation in the immersion joints needed for the new alignment. Also, a new catch or nose & chin structure can be installed if needed for the re-immersing process, such a catch structure is shown in Figure 36, Tunnel element face including nose.

FIGURE 36, TUNNEL ELEMENT FACE INCLUDING NOSE (BABER & LUNISS, 2013)

To be able to execute all this work a temporarily dry dock is constructed. When the element is moored to the quay walls it is safe to construct a dry working dock on the element faces. It is not necessary to have the complete element in a dry dock since the bulk part of the roof, floor and walls are not in need of change. A design for such a small dry dock is shown in Figure 37, Temporary safe dry dock**.**

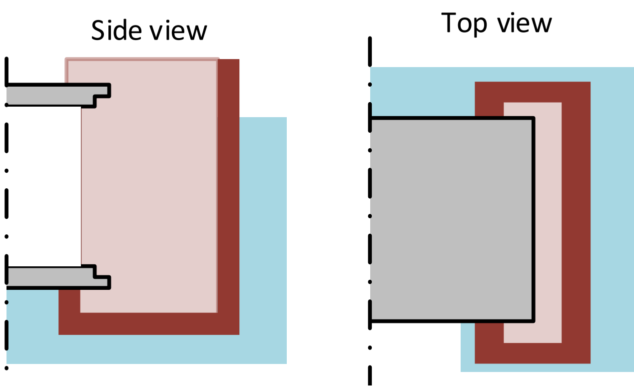


FIGURE 37, TEMPORARY SAFE DRY DOCK

2.6.3 Construction new foundation

During the preparation of the tunnel a new foundation is build. The two main foundation types are a gravel bad, or a catch and pin structure followed by underflowing of sand. In the case of a gravel bed the gravel can be removed after re-floating the tunnel. The gravel layer is than dug out, transported away and filtered. After that the necessary amount of soil is removed and a new gravel bed can be placed using the standard method for a gravel bed and the recycled gravel material.

In the case a catch & pin structure is used, the new foundation is more complex. When the elements are removed part of the soil is removed, uncovering a concrete tile. This tile was used as a foundation of the pin during installation and before the underflowing of the sand took place. When this tile is removed the rest of the soil is removed. It would still be possible to install new tiles and have a new foundation with the same underflow method.

But also, during the initial process of placing the element a jack and ram are used to locate the element in the exact height. At the secondary end a pin is shoved out underneath the element and rests on the foundation tiles. The primary end then rests on the catch structure of the previous element and the pin which is jacked (as shown in Figure 38, Jack and ram structure) . After placing the sand foundation this ram arrangement is casted into the ballast concrete. The result of this is a watertight, inaccessible pin which can't be used again. This is a problem for the re-immersing process.

Special equipment should be designed to locate the element on the exact height when lowered on a new catch & pin structure. A better solution would be to construct a gravel foundation instead. Concluding that for re-immersion an already before immersed tunnel always a gravel bed should be used.

FIGURE 38, JACK AND RAM STRUCTURE (BABER & LUNISS, 2013)

2.6.4 Adapting approach structure

Regarding the land structure two options are available. The first would be to entirely lower the land part, the second is only adapting a small part of the land head. Both these methods are proposed as solutions by Weeda. (Weeda, 2015)

The standard construction of a land part is in de dry, in an area enclosed by for example sheet piles. The complete lowering of this part is nearly as difficult as constructing a new one or at least as expensive. The lowered part would also include a longer access route to the tunnel since it is positioned lower. For these reasons the options of completely lowering a land part is not discussed anymore.

The other option is a local adaption of the land part. This local adaption includes a possibly adaption of the face of the land part (allowing for bigger rotations mentioned in chapter *2.6.1*). The other part will be adapting the floor and the asphalt. This should be done in such a manner that there is no significant kink in the road when the steeper elements are placed. Both are shown in *Figure 39, Adapted land head*.

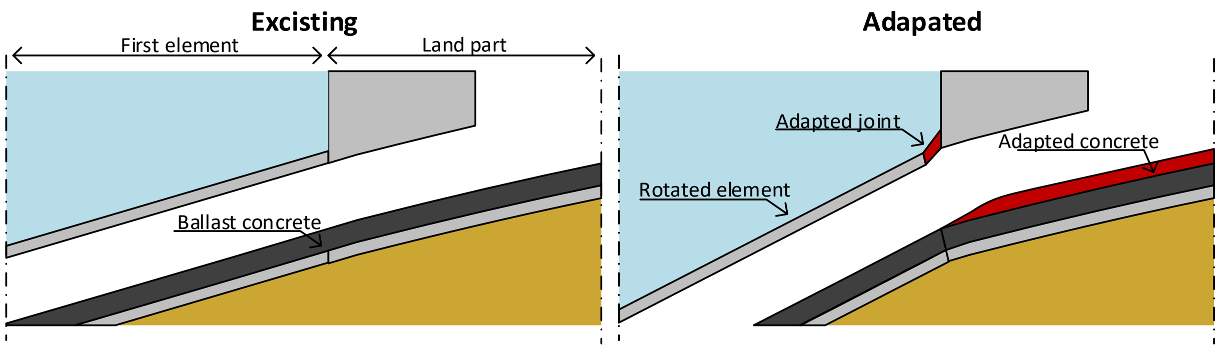


FIGURE 39, ADAPTED LAND HEAD; LONGITUDINAL VIEW

2.6.5 Adapting the new closure joint

In the new tunnel alignment, the closure joint is different compared to the original designed closure joint. This original closure joint was destroyed during the cutting and lifting operation. The new closure joint has 3 main functions, these are discussed below.

Connecting the elements The closure joint connects both sides of the tunnel and makes it a continuous link. The water pressures pressing on the GINA seals are removed and this longitudinal pressure is carried by the joint

Adapting higher rotations As discussed in 2.6.1 the new closure joint might be needed to adapt for extra rotations. If this is the case the shape of the new closure joint should compensate for this.

Adapting for length lost The opening joined discussed before is longer than the original closure joint. This new closure joints compensates for this by being a longer joint than the original.

Construction method

In an original design of a closure joint several options are available. Four options are shown below. (Baber & Luniss, 2013)

Concrete in-situ joint This is the most basic option. A formwork is constructed around the joint, temporarily wedges are placed and the space between is dewatered. Then an in-situ concrete joint is constructed. The method is shown in Figure 40.

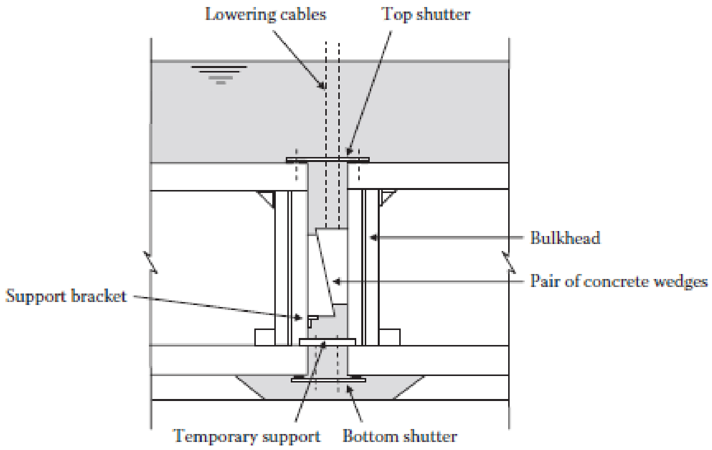


Figure 40, Concrete in-situ closure joint (Baber & Luniss, 2013)

Prestressed segment

It might be that the length of the closure joint exceeds the capacity of the in-situ construction (in for example longitudinal flexibility, or the sheet piles). If this is the case an extra small prestressed segment can be connected to the final element to be immersed. After immersion this prestressing is cut, and the segments acts as part of the closure joint. This is for example done on the Bjørvika tunnel in Oslo. The method is shown in Figure 41.

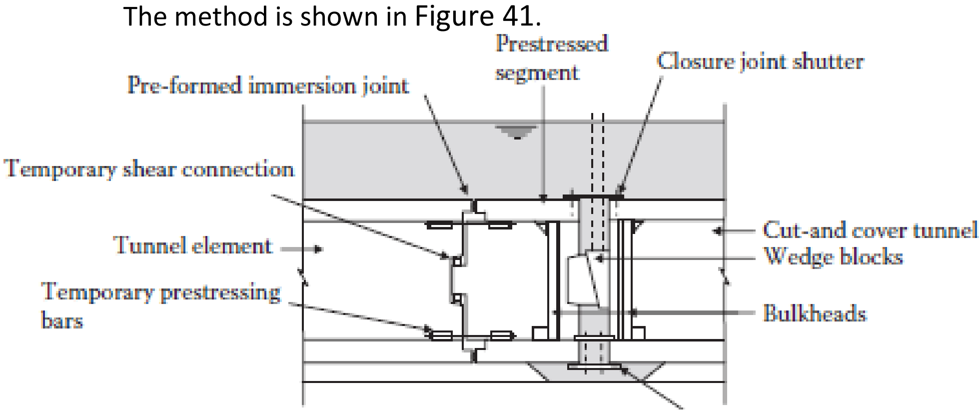


Figure 41, Prestressed segment closure joint (Baber & Luniss, 2013)

Terminal block

This option is based on a sleeve and spigot system. In the approach structure a terminal block is constructed which will be acting as a spigot. Once the final element is placed this terminal block is jacked outwards and a GINA seals makes the connection watertight. This option is however complex regarding the tolerances of the original terminal block construction. The method is shown in Figure 42.

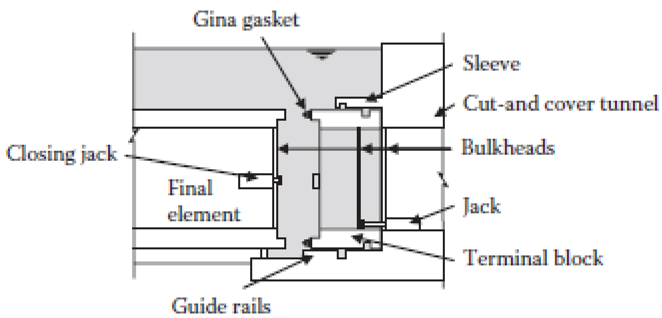


Figure 42, Terminal block closure joint (Baber & Luniss, 2013)

V-wedge

This method is used on the Naha tunnel in Japan and is a relatively new method. It is a joint constructed off-site and is shape in a V (having a longer roof then floor).

This shape utilizes the difference in static water pressure between the roof and the floor. The main advantage of this method is that no formwork is needed and less diver actions. The method is shown in Figure 43.

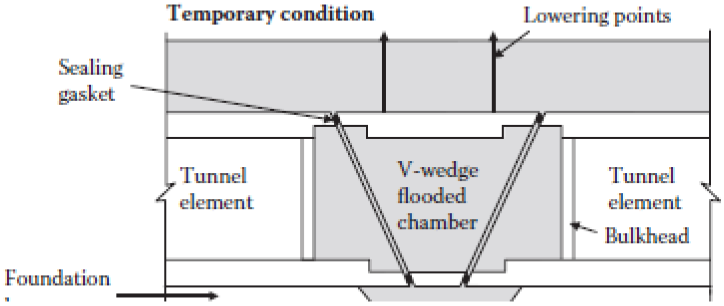


FIGURE 43, V-WEDGE CLOSURE JOINT (BABER & LUNISS, 2013)

However, this new closure joint is way longer than the 0.5-2-meter joints usually used, as mentioned a minimal of 5 meters is required. Therefore, not all the options mentioned above are viable. The in-situ joint is possible if the closure joint doesn't become have an excessive length, this is depending on each tunnel but for example a joint longer than the tunnel segments isn't a good option.

The prestressed element can be used; however, this extra segment should have a minimal length since the tunnel was not designed on extra loads due to this segment. Also, the connection of this segments might be difficult depending on the location where the element is repaired.

The terminal block option is only viable if the original closure joint was a terminal block and the new joint is no longer. If this is not the case a completely new approach structure is needed to act as a sleeve for this longer block.

During a normal V-wedge construction the tunnel elements are used as a negative mould. This secures an exact fit of the wedge. With an immersed tunnel it is impossible to have such a mould and the required tolerance for a V-wedge wouldn't be achieved. This option is not viable.

2.7 Loads and safety for the general method

To verify the feasibility of the second category of requirements posed in the beginning of this chapter all the existing forces and safety measures during all the possible stages are summarized. The forces are divided into two groups, forces for horizontal balance and for vertical balance.

2.7.1 Loads inventarisation

A summary of all these forces is shown in Figure 44, Longitudinal view including forces and Figure 45, Cross-section view including loads. For clearer figures no forces are mentioned in both the cross-sectional and longitudinal view although they might act in both multiple directions. A more detailed explanation of each force is given in appendix B.

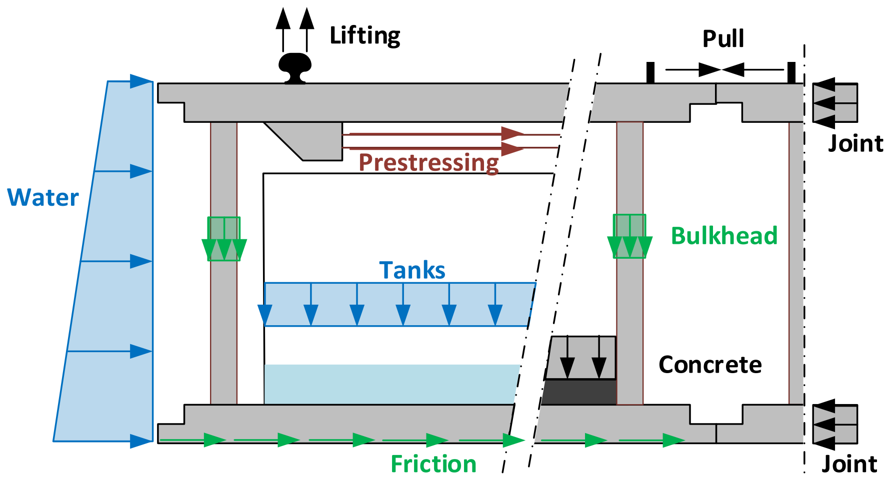


FIGURE 44, LONGITUDINAL VIEW INCLUDING FORCES

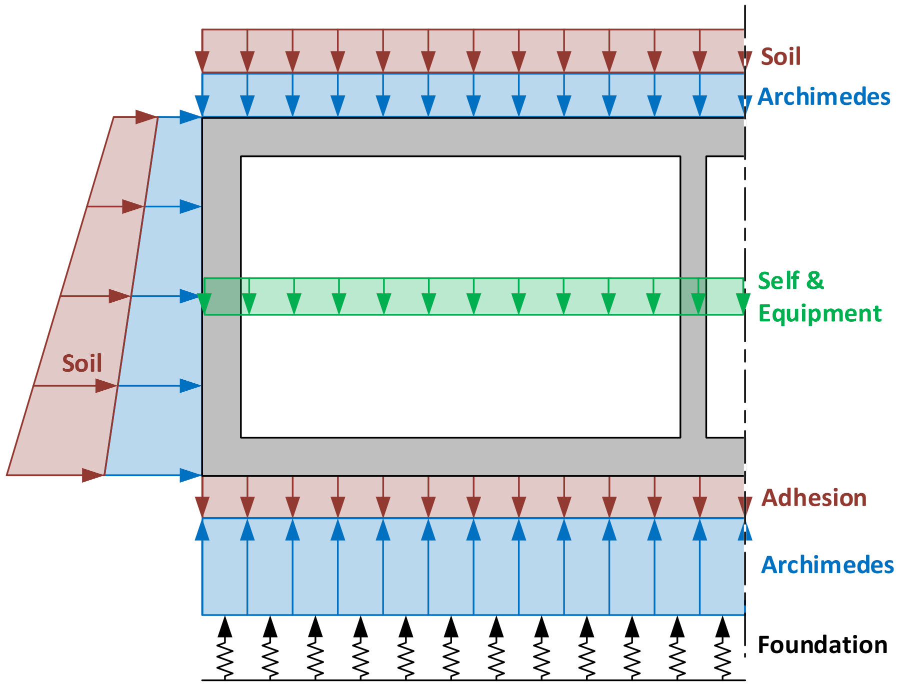


FIGURE 45, CROSS-SECTION VIEW INCLUDING LOADS

2.7.2 Load cases definition

In this section the possible loads defined above and the construction stages defined in section **2.2** are combined into several load cases. Each of these load cases then can be checked on the design requirements regarding the balances and safety. All the different forces are discussed above, graphical lay-out with the acting loads depending on which construction phase is given in appendix B

Horizontal balance

The only forces in transversal direction during all the stages are the soil pressures and the water pressures. During all stages these forces are mirrored to both sides. The following load cases on horizontal balance are regarding longitudinal direction.

Scenario	Phases	Balancing forces
H-I	A.1 – A.8	$F_{joint} = \pm F_{friction} + F_{elements}$
H-II	A.9– B.2	$F_{joint} = \pm F_{friction} + F_{elements} \pm F_{prestressing}$
H-III	B.3 – C.5	$F_{water,sec} = \pm F_{friction} + F_{elements} \pm F_{prestressing}$
H-IV	C.6 – C.9	$F_{watersec} = \pm F_{friction} + F_{prestressing} + F_{water,prim}$
H-V	C.10 →	$F_{watersec} = \pm F_{prestressing} + F_{water,prim}$

Vertical balance

For the vertical balance the same method is used as for the horizontal balance

Scenario	Phases	Occurring forces
V-I	A.1 – A.4	$F_{w,self} + F_{w,soil} + F_{w,concrete} = F_{archimedes} + F_{foundation}$
V-II	A.5	$F_{w,self} + F_{w,soil} + F_{w,concrete} + F_{w,bulkhead} = F_{archimedes} + F_{foundation}$
V-III	A.6	$F_{w,self} + F_{w,soil} + F_{w,concrete} + F_{w,tanks} + F_{w,bulkhead} = F_{archimedes} + F_{foundation}$
V-IV	A.7 – A.13	$F_{w,self} + F_{w,soil} + F_{w,tanks} + F_{w,bulkhead} = F_{archimedes} + F_{foundation}$
V-V	A.14 – C.8	$F_{w,self} + F_{w,tanks} + F_{w,bulkhead} = F_{archimedes} + F_{foundation}$
V-VI	C.9 – C.10	$F_{w,self} + F_{w,tanks} + F_{w,bulkhead} + F_{adhesion} = F_{archimedes} + F_{lift}$
V-VII	C.10 – C.11	$F_{w,self} + F_{w,tanks} + F_{w,bulkhead} = F_{archimedes} + F_{lift}$
V-VIII	C.12	$F_{w,self} + F_{w,tanks} + F_{w,bulkhead} = F_{archimedes}$
V-IX	C.13 →	$F_{w,self} + F_{w,tanks} + F_{w,bulkhead} = F_{archimedes} \pm F_{waves}$

Results

For each of these load cases a specific criterion is governing, these are gathered from the Eurocode. The result is a variable which needs to be checked and the criteria on which to be checked. A numerical value can be achieved by applying this general method to a specific case.

Load case	Check for	
V-I	Min/max foundation pressure	f.o.s. against uplift >1.02
V-II	Min/max foundation pressure	f.o.s. against uplift >1.02
V-III	Min/max foundation pressure	f.o.s. against uplift >1.02
V-IV	Min/max foundation pressure	f.o.s. against uplift >1.02
V-V	Min/max foundation pressure	f.o.s. against uplift >1.02
V-VI	Min/max lifting force	Force = 4000 kN
V-VII	Min/max lifting force	Force = 4000 kN
V-VIII	Min/max freeboard	0.15 < freeboard < 0.25
V-IX	Min/max freeboard	0.15 < freeboard < 0.25

2.7.3 Safety

The safety is about two topics. How is the construction watertight and how is the tunnel accessible during construction?

Water tightness

During the complete lifetime of the tunnel, starting from the original drydock until the final de-construction, it should be watertight. This is secured with several design features discussed below, in all stages two of these features should be present as explained in section 2.2.

GINA-seal	In the immersion joint a GINA-seal is present, this provides the initial seal. This seal is essential during the installation of the tunnel
Omega-seal	The Omega-seal is constructed on the inside of the tunnel cross-section. It covers the GINA-seal and applies a second barrier during tunnel lifetime.
W9U-i profile	This seal is located between the tunnel segments, it is casted in the tunnel while it was in the drydock. This seal can only handle small deformations.
Bulkheads	During transport and immersion (and re-floating) bulkheads are installed on the end of the elements. These make the tunnel elements watertight 'boxes'
Concrete	The main part of the element. If the cracks in the element grow too big the tunnel might start leaking. Extra coatings could prevent this. This is especially needed for monolithic tunnels

- After the tunnel is constructed in the drydock the element is watertight due to:
  - W9U-I profiles
  - Bulkheads
- The tunnel is immersed and connected to the other elements, watertight due to:
  - W9U-I profiles
  - Bulkheads
  - GINA-seal
- The tunnel is finished and is operational, watertight due to
  - W9U-I profiles
  - GINA-seal
  - Omega-seal
- After preparation for immersion the tunnel is watertight due to
  - W9U-I profiles
  - GINA-seal
  - Bulkheads
- When floating and transported the element is watertight due to
  - W9U-I profiles
  - Bulkheads

Accessibility

During the process three stages of accessibility are available. First, access through the tunnel standard access route and access to the element is possible with big machinery since no bulkhead is present. The second stage is access through the tunnel standard access route, access into the element is limited due to the smaller doors in the bulkhead.

The third stage is access through the access shaft; this is limited due to the small dimension of the access shaft. This is only needed in case of emergencies.

2.8 Risk analysis for the general method

In this section a risk analysis is made for the procedure proposed in the previous chapters. It is important to notice that a big part of the risk occurring in the re-floating and re-immersion process are also occurring in the standard immersion process. These risks will not be handled. A complete overview of all the failures, probabilities, consequences and measures types is given in appendix D.

A quantative approach is chosen to define the probabilities and the consequences. The main source for this information is the expert judgement from people at RHDHV and TEC-tunnels.

2.8.1 Probabilities & consequences

The total risk for each of the possible failures is defined by a combination of the probability and the consequence of the failures. The probabilities of failure during lifetime are divided in 4 categories:

1. Very, very unlikely $p < 1\%$
2. Very unlikely $1\% < p \leq 10\%$
3. Unlikely $10\% < p \leq 25\%$
4. Likely $p > 25\%$

The consequences of a failure can occur in different fields. For example, failure of the jack system results in delay and high costs but has no consequence for the tunnel quality or the environment. The consequence total score is a summation of the scores from the five aspects. The following aspects are considered:

Safety

0. None
1. Extra monitoring needed
2. Evacuation of construction site
3. Completely unsafe; evacuation of surrounding area

Environment / surroundings

0. None
1. Minor impact
2. Big impact
3. Irreversible impact

Quality

0. None
1. No deviation from final requirements, repairable
2. Not repairable, maintenance during lifetime
3. Permanent damage, lower performance than required

Time [weeks]

0. t = 0
1. 0 < t ≤ 4
2. 4 < t ≤ 26
3. t > 26

Costs

0. € = 0
1. 0 < € ≤ 250.000
2. 250.0000 < € ≤ 2.500.000
3. € > 2.500.000

In the next chapter each of the failures is be combined with a probability and a consequence. The result in a complete overview of the failures, and the respecting probabilities of occurring and failures. This big overview is shown in appendix D

2.8.2 Failure modes

As mentioned in the introduction to this chapter only failures specific to the method defined in section 2.2. These are divided into the four main construction steps. For each of the risks a small explanation is given describing the cause and the effect of the risk. Also, for each risk the measures type is given; accepting the risk, corrective measures or preventive measures. The actual measures are discussed in section 2.9.

Preparation of the element

Unexpected floatation

The first failure is unexpected floatation of the element; this failure is caused by the removal of too much ballast concrete. As much as possible ballast concrete will be removed before placing the bulkheads. These bulkheads from a blockade for the bigger equipment and the rest of the removal will be done by hand or small machinery. During lifetime the FoS against uplift is 1.06 while during construction this is 1.02, this 4% ballast concrete can be removed. When immersing the element, the exact weight of the element can be calculated using all the densities of the concrete and the amount of reinforcement. This information might be lost over the years or is only available for the contractor who immersed the tunnel.

Due to the very big consequences this risk should be lowered, and preventive measures are needed.

Prestressing

The prestressing installed during the preparation of the element can fail. Depending on the degree of failure the element might be damaged or even completely collapse. The damage of the element means a new element will need to be construction; total collapse of the element will also result in blocking the waterway which leads to extra costs. Two main causes for this failure arise. The first is failure of the equipment, for example slipping of the prestressing wires at the connection point to the concrete, or lower strength than expected. The other cause might be a design error and the applied stress level is too low.

The risk for a partial failure is low but when this failure occurs there is no possibility for a corrective measure since the element is closed off. Regarding complete failure preventive measures are needed to lower the probability of occurring.

Bulkhead

The failure of the bulkheads can be divided in two categories, total failure and partial failure. Partial failure occurs when the connection between the bulkhead and structural concrete is not completely watertight. This results in water leakage. If this is a small amount it can be pumped, if it is a substantial amount some sort of epoxy should be applied to the bulkhead to prevent more leakage. The total failure of the bulkhead is structural failure. This will occur when the element is on at the deepest location and the element next is removed.

Partial failure of these bulkhead is allowed, and corrective measures can be put in place. The consequence of complete failure of the bulkhead is too high and preventive measures should be put in place

Removing a first section

Damage to the other elements

When cutting to prepare the opening joint also some other part of the tunnel may be accidentally damaged. Another cause for damage to the rest of the structure may be the opening joint crumbling while lifting. The damage to the construction might need to be repaired or hinders the work of the divers in future operations.

If the opening of the first joint is made with care and precision only corrective measures are needed if the tunnel does indeed get damaged. The consequence of this failure is quite easily solved.

Element stuck

When lifting the opening joint out of the gap it should be strictly vertical. Any rotations in the opening joint can result in the element being stuck. If possible, the element should be lowered again, if not extra cuts are needed which results in a delay in the planning.

If no extra measures are in place the probability of this failure occurring is likely. Preventive measures should be in place to lower the probability of this risk.

Re-floating & Transport of the main elements

Access shaft

After removal of the opening joint the access shaft is mounted. For the re-floating method a specific failure is important, leaking of the access shaft. Compared to the standard operation the shaft is now installed while under water. Leaking of this shaft may be possible due to leaking at the connection point between the bulkhead and shaft. Also due to the unconventional shape of the shaft high loads may lead to deformation or failure. Complete structural failure of the access shaft denies access to the element, but the bulkhead might still be watertight.

A partial failure of the access shaft has a minor consequence and a minor probability of occurring, therefore this risk is accepted. A complete failure of the access shaft also has a relatively small consequence so only a corrective measure should be put in place.

Damage to element

The element which is to be emerged and transported can be damaged. Two main causes for this are present. The first is the removal of the rock cover and rock fill, if the material for this is not stable it may damage the element roof or side. The other reason are the other elements, if not enough horizontal distance is present before initializing the vertical movement a collision may occur damaging the element face or even worse.

This risk is comparable to damage to the other element while opening the first joint. A corrective measure suffices.

Element won't float

When all the ballast concrete is removed, and the tanks are partly emptied the pontoons should be able to lift the tunnel element. If the element wont won't start floating this might be to two reasons. The first is the actual weight of the element. Since the element is constructed decades ago the actual weight might be unknown, and due to safety reasons not enough ballast concrete was removed. The other reason might be that the adhesion is higher than expected.

When this failure occurs, there is no possibility for a corrective measure and the consequence is very high. Therefore, a preventive measure should be in place.

Preparing, adapting & reconfiguring

Failure of habitat

In the safe location the element is checked, and a new GINA is installed. During this failure the habitat may fail. The first reason for this might be the connection between the element and the habitat, the result of this is leakage. The solution is like the solution for partial failure of the bulkheads, some sort of epoxy material is injected to secure a watertight habitat.

The second type of failure is complete failure. This may be due to a collision or other type of accident. The result is a collision of the habitat. This is the most unsafe scenario for the personnel present in the habitat. The risk of this is mitigated by choosing a very safe location for the tunnel and guidance structures if the location is not completely safe.

A partial failure of the habitat has a low probability of occurring and a low consequence, this risk is accepted. A complete failure of the habitat has an even lower probability of occurring and is also accepted.

Leakage in joints

When the new tunnel is constructed it is important to notice that the element has undergone a process which is not standard for the tunnel elements. This might result in leaking of the old W9U-i profiles, the element might have deformed or the rubber in these elements are not watertight due to corrosion.

The other failure might be at the location of the new GINA profiles. These are made of a different type than the initial design. If this leaking is too much the water pressure between the bulkheads can't be reduced. Leakage in the joint has a big impact during the resting lifetime of the tunnel. A preventive measure is put in place, this measure differs for both the GINA and W9Ui

Figure 46 shows a small overview of the different risks occurring and the type of measure needed. As mentioned, all these risks are summarized in appendix D. In the appendix also the probabilities and consequences mentioned in section 2.8.1 are assigned to each risk.

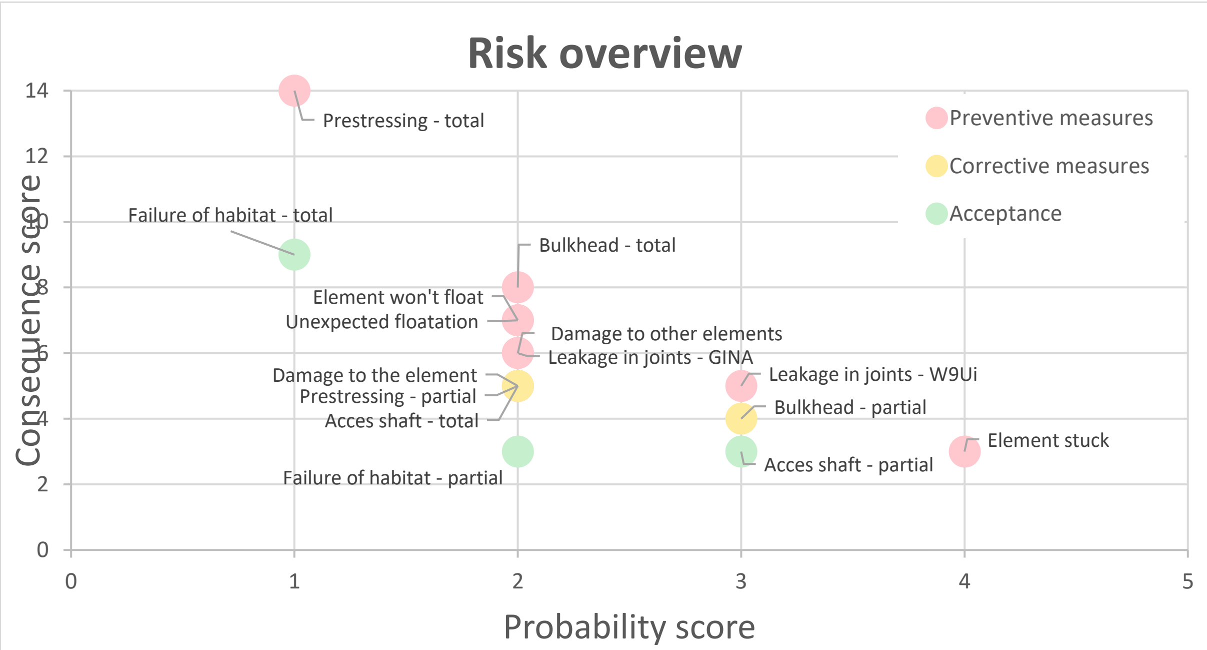


FIGURE 46, RISK OVERVIEW, INCL MEASURE TYPE

2.9 Measures regarding specific risks

For each of the risks mentioned in the previous chapter a type of reaction is chosen: Acceptance, preventive measures or corrective measures. For the risks which can be accepted no more explanation is needed. In this chapter the corrective and preventive measures are discussed in more detail.

2.9.1 Corrective measures

Bulkhead – partial

After construction the bulkheads will be tested as described in section 2.3.2. This test will show if partial failure occurs. If this failure, e.g. leaking, extra epoxy material is applied to close off the leaks.

Access shaft – total

If the access shaft fails, the access to the tunnel element is blocked. A new access shaft can be mounted regaining access.

Structural failure – partial

The partial structural failure will be shown in more cracks than expected, this will lead to more leaking in the element. These leaks can be closed off using epoxy or an extra pumping capacity is needed during the lifetime of the tunnel

2.9.2 Preventive measures

Unexpected floatation

The standard safety against uplift is a 2% of the weight of the element. This is increased with 20% resulting in a safety factor of 1.024 instead of 1.020

Prestressing – partial & total

The amount of prestressing needed is based on the minimum pressure (or maximum tension) in the tunnel element. Depending on the loads during transport. The codes set a minimum pressure in the outer fibre of the tunnel at $\sigma = 0.2\ N/mm^2$ or $\sigma = 0.3\ N/mm^2$. For normal transport the first is governing, for transport over sea the latter is governing.

It is decided that for extra safety an increase of 10% is needed. Resulting in a minimum pressure of respectively $0.22\ N/mm^2$ or $0.33\ N/mm^2$

Bulkhead – total

To compensate for the risk the loads on the bulkheads are increase with a 10% safety factor. Other measures, such as testing during construction is also used. The method is as mentioned in chapter 2.3.1.

Damage to elements

The damage to the element can be prevented by applying a protective layer on the other elements. During construction this layer is not present, during removal the soil cover can act as such a protective layer. Only the soil cover from a single element is removed, after this element is re-floated and transported another soil cover is removed. This will result in a longer, but safer construction.

Element stuck

To prevent the element from getting stuck while lifted two types of measures are needed. The first is the application of guidance structures. These rails are installed before cutting the element. This is an underwater operation and will be done using divers. These divers will also do extensive monitoring during lifting.

Element won't float

The actual adhesion is very unknown. The calculations for the durations until a so-called 'breakout' are all depended on the actual lifting force. To prevent the risk of not lifting the effective lifting force is reduced by 20%. Note that is the effective lifting force regarding adhesion, regarding punching shear not the resulting 80% is used but 100% of the applied force.

Another preventive measure would be to reduce the uncertainty in soil parameters, this could be done by taking soil samples and lab tests. This is however not possible to execute during this thesis but should be executed in an actual design.

Leakage in joints -GINA

Leakage in the new GINA joint can't be prevented by taking a safety factor regarding the loads on the seal into account. This is because the seals can start leaking when they aren't compressed enough, but rupture in the seal can occur if they are compressed too much.

The failure of the seals can be partly prevented by protecting the GINA-seals during transport. For this the same procedure is applied as in the original immersion method.

Leakage in joints -W9Ui

The leakage in the W9Ui profile can be prevented by limiting deformations. These deformations can be limited by increased the prestressing force. This measure is already taken regarding the failure of the prestressing. It is assumed the 10% applied regarding prestressing failure is also the measures for leakage in the joint due to deformations.

2.10 Concluding remarks

In this chapter the standard constructing sequence for immersed tunnels is reverted in a construction sequence for temporarily re-floating an immersed tunnel, adapting it and re-placing it in a deeper location. All the different aspects of this new construction sequence are discussed followed by an analysis regarding the horizontal balance, vertical balance and safety during these stages.

Finally, a risk analysis is made showing the weak points in the construction sequence. For some of these risks, measures are needed.

In the upcoming chapters this construction sequence is applied on a specific case study and general method can be tested on the structural requirements.

3. Case study | The Wijker tunnel

In this chapter several possible case tunnels applicable for a case study are gathered. From a multi criteria analysis the best case is chosen, and this is case will be applied on the general method to show its feasibility in chapter 4. This tunnel is then analysed in two ways. First a general analysis is made followed by a load analysis.

3.1 Case determination | Multi Criteria Analysis

As mentioned, loads of immersed tunnels are constructed all over the world. One of these will be used as a case study for the rest of the calculations. In appendix E this case is determined; this is done using a multi criteria analysis. First the weight of the criteria's is determined and then checked for the possible cases. Four of these possible cases are checked.

The result from the MCA is shown in Table 4, MCA Case study result. The four criteria in this analysis are the available knowledge, the waterway relevancy, the tunnel relevancy and the tunnel dimension. The result is that the Wijkertunnel located in the North Sea Canal will be used as case.

Tunnel	**Score**
Liefkenshoektunnel	6,77
Wijkertunnel	7,31
Benelux tunnels	7,23
Busan-Geoje tunnel	6,38

TABLE 4, MCA CASE STUDY RESULT

3.2 General analysis

In 1957 the first high capacity tunnel was constructed under the North Sea Canal. The Velsertunnel connected the north and south embankments of the canal with a tunnel. This tunnel had 2x2 traffic lanes. Quite soon the traffic intensity increased, and an extension was needed. A completely new highway, the A22, was constructed resulting in more than doubling the traffic capacity. With this new highway also came a new crossing of the North Sea canal.

Again, a tunnel was preferred instead of a bridge. This new tunnel, the Wijker tunnel, has 2x2 lanes and an emergency tube between de double lanes. The exact location of the tunnel elements is given relative to the location of the start of the highway, these are the coordinates of the middle axis of the tunnel elements. These are shown in Figure 47, Overview tunnel elements (closure joint on the right)

3.2.1 Tunnel location and dimensions

Location

In Table 5, Tunnel element location the tunnel location shown. The tunnel elements are symmetric around the middle of the tunnel (1=6, 2=5, 3=4). Since the immersed tunnel is part of a complete tunnel design including long approach structures, the location of the tunnel elements starts at about 2,6 kilometres.

For the calculations the water pressures, the entire element depth is assumed to be equal to the pressure at the deeper end of the element (bold text). The depth shown in the table is the bottom of structure depth.

FIGURE 47, OVERVIEW TUNNEL ELEMENTS (CLOSURE JOINT ON THE RIGHT)

TE	Location [mm]	X general	Y General	X local	Y local	-NAP [m]
0	2 630 389	106 220 048	496 628 636	0	0	**16.023**
						17.098
						18.174
						19.250
1	2 726 059	106 245 029	496 720 987	24 981	92 351	**20.327**
						21.403
						22.479
						23.557
2	2 821 729	106 270 009	496 813 338	49 961	184 702	**24.576**
						25.377
						25.949
						26.293
3	2 917 399	106 294 990	496 905 689	74 942	277 053	**26.407**
4	3 013 069	106 319 970	496 998 040	99 922	369 404	
5	3 108 739	106 345 497	497 090 240	125 449	461 604	
6	3 204 409	106 372 743	497 181 948	152 695	553 312	

TABLE 5, TUNNEL ELEMENT LOCATION

Cross-sectional parameters

In a report from TNO regarding reparability of tunnels the exact dimension of the Wijkertunnel are given. Based on these dimensions the cross-sectional parameters are defined (using the Shoelace Algorithm) (Burggraaf, Overbeek, & Vervuurt, 2007).

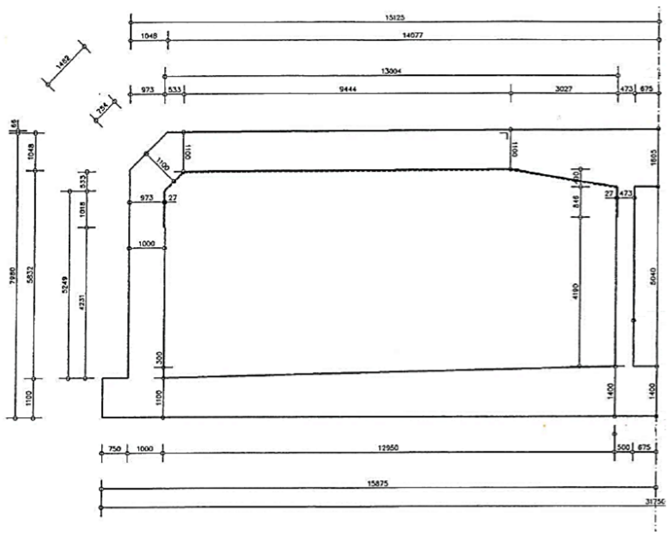


FIGURE 48, TUNNEL CROSS SECTION DIMENSIONS

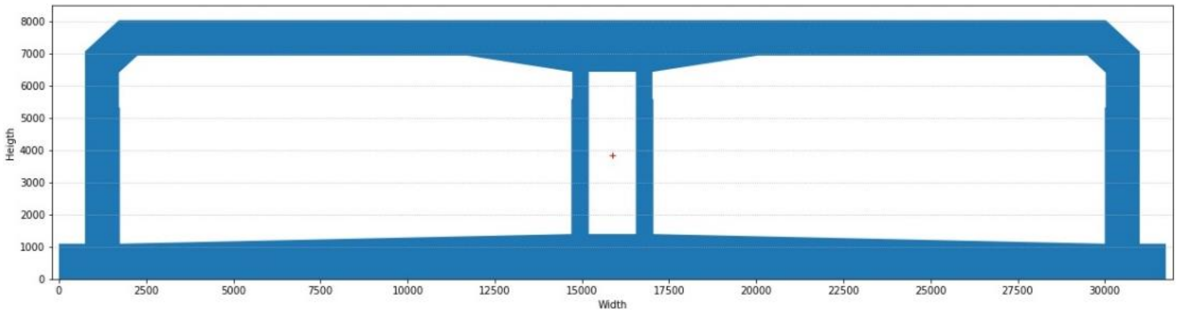


FIGURE 49, COMPLETE TUNNEL CROSS SECTION

Parameter	Value	Unit	Formula	Description
$\mathbf{A_{concrete}}$	91.43	m^2	(Burggraaf, Overbeek, & Vervuurt, 2007)	Cross-section total area
$\mathbf{A_{face}}$	244.07	m^2	"	Cross-section concrete area
$\mathbf{A_{bulkhead}}$	152.64	m^2	"	Cross-section bulkhead area
$\mathbf{C_x}$	15.875	m	"	Centre of gravity x-direction
$\mathbf{C_y}$	3.86	m	"	enter of gravity x-direction
$\mathbf{I_{xx}}$	915.81	m^4	"	2nd moment inertia xx
$\mathbf{I_{yy}}$	7970.17	m^4	"	2nd moment inertia *yy*
$\boldsymbol{l_e}$	95 600	mm	(TEC, 1993)	Element length
$\boldsymbol{h_e}$	8 045	mm	"	Element height
$\boldsymbol{w_e}$	31 750	mm	"	Element width
$\boldsymbol{l_s}$	23 900	mm	"	Segment length

TABLE 6, TUNNEL CROSS-SECTIONAL PARAMETERS

3.2.2 Alignment

Horizontal alignment

The arches are infinite from segment 1.D to segment 5.D, the other elements (5.C-6.A) have a $R_h = 5\ 000\ [m]$ in upward direction

Vertical alignment

The tunnel is designed with a maximum slope of 4.5% and a maximum velocity of $v = 120\ [km/h]$. At the land part a top arch is constructed with a radius of $R_{v,top} = 10\ 000\ [m]$.

The first and last seven segments have no vertical curvature ($R_v = \infty$). The middle ten segments have a curvature of $R_{v,bottom} = 2\ 500\ [m]$.

3.2.3 Foundation

During placement the elements rest on the previous element and a strut and jack structure. The final foundation is a sand foundation placed with the underflow method.

The stiffness of the sand foundation is to be expected to develop during the lifetime of the tunnel. During the initial stages this stiffness is an order of magnitude smaller. It is important to note that the original design already mentioned that this change in soil stiffness won't have a significant impact on the force distribution in the cross section.

Parameter	Value	Unit	Formula	Description
$\mathbf{K_{max}}$	30 000	kN/m^3	(TEC, 1993)	Soil stiffness maximum
$\mathbf{K_{min}}$	3 000	kN/m^3	"	Soil stiffness maximum

TABLE 7, TUNNEL FOUNDATION PARAMETERS

3.2.4 Soil cover

The limiting minimal soil cover is calculated at 65 centimetres. But verbal agreement during the design set a soil cover at 1.0-meter-thick till a depth of the tunnel roof of NAP -16.00 meter. If the tunnel roof is deeper the soil cover is filled up to a depth of NAP -16.00 meter.

Parameter	Value	Unit	Formula	Description
$t_{s,min}$	1 000	mm	(TEC, 1993)	Minimum soil cover
$t_{s,max}$	2 362	mm	"	Maximum soil cover

TABLE 8, TUNNEL SOIL COVER PARAMETERS

3.2.5 Water(way) levels

The water levels are checked for a 1 in 1000-year requirement. The average water depth NAP -0,40 meter.

Parameter	Value	Unit	Formula	Description
$h_{water,min}$	- 0.70	m -NAP	(TEC, 1993)	1 in 1000-year min water level
$h_{water,ave}$	- 0.40	m -NAP	"	Average water level
$h_{water,max}$	+ 0.40	m -NAP	"	1 in 1000-year max water level

TABLE 9, TUNNEL WATER LEVEL PARAMETERS

Waterway parameters

As mentioned the bottom of structure in the middle of the tunnel is 26.407 $[m]$. Adding the structure height and the soil cover the water depth equals 15.6 $[m]$ at an average water level. According to Rijkwaterstaat the waterway dimensions are a width of 270 $[m]$ and a depth of 15.1 $[m]$.

Transport

The elements were originally constructed in the drydock in Barendrecht. To transport these elements to the building site the elements were transported over sea. During this transport a minimum and maximum freeboard was set. This amount is higher compared to standard transport due to the higher loads on sea. During this transport the minimal stress in the outer fibre of the element can't be drop below 0.3 $[kN/m^2]$, for standard transport this value is set at 0.2 $[kN/m^2]$

FIGURE 50, TUNNEL TRANSPORT ROUTE

Parameter	Value	Unit	Formula	Description
FB_{min}	0.40	m	(TEC, 1993)	Maximal transport freeboard
FB_{max}	0.15	m	"	Minimal transport freeboard
σ_{outer}	0.3	kN/m^2	"	Minimal stress outer fibre sea

TABLE 10, TUNNEL FREEBOARD PARAMETERS

3.2.6 Steel wiring

Concrete reinforcement

The exact amount of steel reinforcement in the tunnel elements is unknown. The values based on moments in the initial stage of the design are known and shown in Figure 51, Tunnel cross-sectional reinforcement (preliminary). In this design stage, four different situations were considered based on the environment the tunnel was in (being salt water). The figure shows the situation with the most reinforcement. The loads for this situation are discussed in 3.3. The actual amount of reinforcement is shown in Figure 52, Tunnel cross-sectional reinforcement (final), but this image is not readable. Is it mentioned that extra reinforcement is located near the lifting boulders) (Camerik & Leeuw, Cement, 1994)

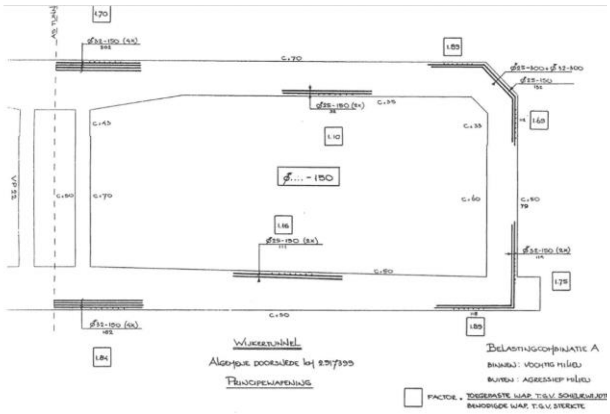


FIGURE 51, TUNNEL CROSS-SECTIONAL REINFORCEMENT (PRELIMINARY)

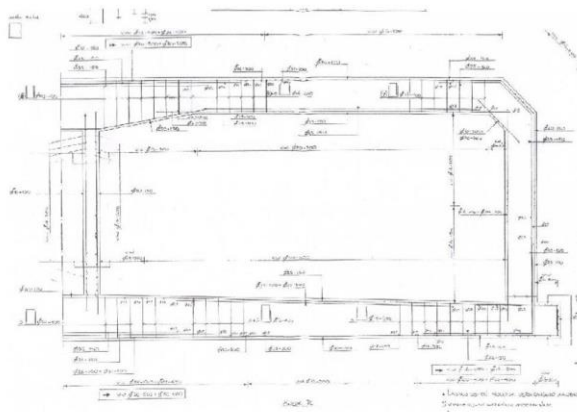


FIGURE 52, TUNNEL CROSS-SECTIONAL REINFORCEMENT (FINAL)

The actual shape and reinforcement in the segment joints is also unknown. It is assumed that a standard shear key is used, and these keys have a capacity of 3 meganewtons each (Lagen, 2016). These are applied in both outer walls and a half in each inner wall (since these are thinner are smaller). The result is a total shear capacity of 9 MN.

Prestressing

The actual amount of prestressing is unknown. The only available knowledge is that for calculation the amount of prestressing the transport over sea was governing (Groot, Kerk, & Roelands, 1994). The amount of wires can be seen in Figure 53, Prestressing diagram in the Wijker tunnel, the amount of strand per wire is unknown (left is the roof, right is the deck).

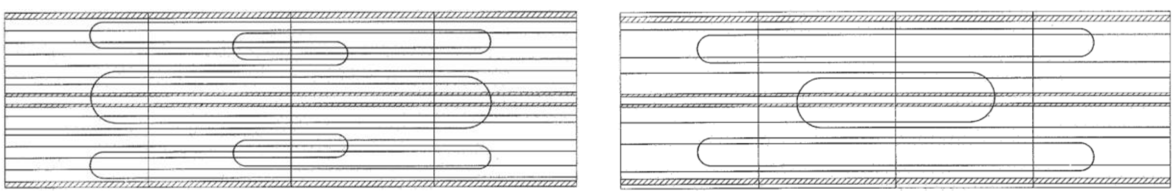


FIGURE 53, PRESTRESSING DIAGRAM IN THE WIJKER TUNNEL

3.2.7 Material properties

To be able to define the different loads for each load case the materials densities are defined with an upper and lower boundary.

Parameter	Value	Unit	Formula	Description
ϕ_{sand}	30	°	(TEC, 1993)	

$k_{ad,wall}$	0.58	-	$\tan(\phi)$]	Wall-soil adhesion
$\rho_{sand,min}$	19.00	kN/m^3	(TEC, 1993)	Density wet sand, minimum
$\rho_{sand,max}$	20.00	kN/m^3	"	Density wet sand, maximum
$\rho_{water,min}$	10.00	kN/m^3	"	Density water, minimum
$\rho_{water,max}$	10.25	kN/m^3	"	Density water, maximum
$Concrete\ class$	C30/37	-	"	Concrete class
$E_{concrete}$	32 000	N/mm^2	"	Concrete young's modulus
$\rho_{concrete,min}$	24.214	kN/m^3	"	Density concrete, minimum (130 kg/m^3 reinforcement)
$\rho_{concrete,max}$	24.914	kN/m^3	"	Density concrete, maximum (130 kg/m^3 reinforcement)
$\rho_{ballast}$	23.000	kN/m^3	"	
$\rho_{bulkhead,min}$	23.863	kN/m^3	"	Density bulkhead, minimum
$\rho_{bulkhead,max}$	24.555	kN/m^3	"	Density bulkhead, maximum

TABLE 11, TUNNEL MATERIAL PARAMETERS

3.3 Load analysis

In this chapter the different loads acting on the tunnel are discussed. This is based on the report with a preliminary design regarding the Wijker Tunnel design (TEC, 1993).

3.3.1 Load combinations

In this design fifteen unit-loads were defined resulting in several load combinations.

Number	Type	Description
I	Self-weight concrete	This load is based on the amount of concrete multiplied with the density of concrete (here 24 $[kN/m^3]$ is used.
II	Permanent water load	Water pressure when the element just is immersed. The real density is incorporated with multiplying the load with $\gamma_{water}/10$, with γ_{water} is the real density
III	Ballast tanks	The location and actual load of the ballast tanks depends on the placement of the tanks and the degree to which they are filled
IV	Self-weight interior	The interior load is built up with fire resistance layer, New Jersey profiles, ballast concrete, mid-tunnel prefab concrete elements, cables, fire extinguisher etc.
V	Permanent soil load	These are the vertical and horizontal pressure due to the soil next to the element. This should be multiplied with: $\frac{\gamma_{wet}(\gamma_{wet}-\gamma_{water})}{10}$
VI	Variable soil load	These are the vertical and horizontal pressure due to the soil on top of the element. This should be multiplied with: $\frac{h*(\gamma_{wet}-\gamma_{water})}{10}$, for horizontal pressures $k_n = 0.5$
VII	Variable water load	Water pressure due to the water on top of the element. This should be multiplied with $\frac{H*\gamma_{water}}{10}$
VIII	Soil adhesion	This load acts on the ears of the element in vertical direction. It is based on the soil above the element roof, and is calculated using $\tan(\phi)$.
IX	Soil adhesion	This load acts on the ears of the element in vertical direction. It is based on the soil beneath the element roof, and is calculated using $\tan(\phi)$.

X	Temperature T1	This is the temperature load based on an inside temperature of $-10°$ and a outside temperature of $+5°$
XI	Temperature T2	This is the temperature load based on an inside temperature of +15° and a outside temperature of $+5°$
XII.1 **XII.2**	Explosion	This is a load of $100\ [kN/m2]$, active on all wall of a single tube. For modelling this is split in a contra- and co-symmetric load (resulting in a pressure of 100 and 0 per tube)
XIII	Traffic (single tube)	Load due traffic in a single tube. Not significant so dismissed.
XIV	Traffic (double tube)	Load due traffic in a double tube. Not significant so dismissed.
XV	Wave	Wave loads on the element during transport

TABLE 12, ORIGINAL TUNNEL UNIT-LOADS

With these loads several load combinations were made for all the phases in the construction of the element. Regarding this research only the load combinations which differ from the original load cases are interesting. These are compared to the load cases important for this design (as stated in chapter 2.7.2)

During the re-floating process nothing differs from loads for which it was designed. These loads do not have to be reconsidered regarding strength and stability.

During lifting load cases V-VII and during transport the load cases V-VIII & V-IX are relevant. Regarding structural capacity the loads are expected to be smaller than during the initial design since it was transported over sea. Regarding the prestressing capacity these loads are interesting since new, possibly less effective, prestressing is applied

During the re-immersing process nothing differs from the loads for which it is designed until the moment the element reaches its original depth. In the original design three 'phases' occur. The first being placement on its pins is not relevant in this case since it will be founded on a gravel bed. The second was ballasting with ballast concrete and this was mentioned to be not governing over the next phase so was not considered (TEC, 1993). The only relevant phase is the final situation of the tunnel element.

3.3.2 Relevant loads

From the previous chapter it follows that three special load cases are important for this design, for these load cases more information is gathered.

Final position |For lowering

For the final position eight different load cases were calculated in the original design, these load cases are shown below

Comb	Description	Load
A	High water, maximum soil pressure, adhesion above and below the roof	$I + III + IV + (II + VII)\dfrac{\gamma_{water}}{10.2}$ $+(V + VI)\dfrac{h * (\gamma_{wet} - \gamma_{water})}{10} * \gamma_3$ $+(VIII + IX) * \gamma_3$
B	Comb. A + Temperature T1	$Comb. A + \ X * \gamma_4$
C	Comb. A + Temperature T2	$Comb. A + \ XI * \gamma_4$
D	Comb. A + Explosion load	$Comb. A + (XII.1 + XII.2) * \gamma_2$
E	Low water, no soil pressure, minimum adhesion	
F	Comb. E + Temperature T1	

G	Comb. E + Temperature T2	
H	Comb. E + Explosion load	

TABLE 13, FINAL SITUATION LOAD CASES

With:
$\gamma_{water} = 10.2\ [kN/m^3]$
$\gamma_{wet} = 20.0\ [kN/m^3]$

Regarding the second four cases only the combination with explosion is governing, only for the tube in which the explosion occurs. Deepening of the tunnel has a positive impact on this load since it is acting in the opposite direction. It can be concluded that the final four load combinations are not relevant for this research.

For the structural capacity four load cases need considering, combined with a range of the foundation pressure of $3\ 000\ to\ 30\ 000\ [kN/m^3]$.

Lifting & transport |For prestressing

During transport no temperature gradient or explosion loads are considered. The only variable is an absence of waves, positive wave loads or negative wave loads.

Comb	Description	Load
A	General lifting (deepest location)	$Comb. C + Lifting$
B	General lifting + suction	$Comb. C + Lifting + Suction$
C	General lifting (shallow location)	$Comb. C + Lifting$
D	Floating element & Standard transport of elements	$I + IV + (II + VII)\dfrac{\gamma_{water}}{10.2} + III$
E	Comb. C + Waves	$Comb. C - XV$
F	Comb. C - Waves	$Comb. C + XV$

TABLE 14, LIFTING & TRANSPORT LOAD CASES

3.4 Concluding remarks

This chapter gives an outline of the necessary parameters of the original design and the material properties. Combined with the different load combinations acting on the element almost every aspect is known to apply this case on the general method.

The only lacking knowledge at this moment is the actual amount of applied prestressing in the element and in the segment joints. For this research this information was not accessible, while doing a contracted design such information is assumed to be available.

4. Validation of the general method

In this chapter the method described in chapter 2 will be tested with the case study defined in chapter 3. This will be done for each of the steps described in section 2.2.

4.1 Preparing the element

4.1.1 Placing ballast tanks and removing concrete

In the Wijkertunnel ballast concrete is in both the tunnel tubes, no ballast concrete is present in the emergency shaft between the elements. The thickness of this layer varies in the transverse direction of the element. Near the outer walls the thickness of $400\ [mm]$, near the inner walls the thickness is $659\ [mm]$. This results in a total ballast volume of $13.71 * l_{element} = 1310.7\ [m^3]$.

Two factors should be changed compared to the initial design:

- For the Wijkertunnel a factor of safety against uplift during lifetime of 1.06 is used during construction this FoS should have a minimal of 1.02. Since unexpected floatation is identified as a big risk, and the exact weight of the element is unknown a higher FoS during construction is applied. This is set to 1.024.
- In the initial design the element was transported over sea (implying a minimal freeboard of $400\ [mm]$. During this process the elements won't be transported over sea (as discussed in section 2.5.4). So, a freeboard of $150\ [mm]$ suffices.

Ballast concrete

The element is designed on the limit, so to achieve the initial freeboard all the ballast concrete should be removed. However, this is not needed since a reduction in freeboard is allowed. The vertical balance calculation shows that a minimum of 6.1% of the ballast should stay in place due to the increased FoS against uplift. A maximum of 18.0% of the ballast can stay in place due to the decrease in required freeboard. A complete overview of these calculations is given in appendix F.

The risk for unexpected floatation is already covered in the increase of the factor of safety. Therefore, the maximum amount of concrete can be removed. This 18% staying results in a total of $1074.8\ [m^3]$ to be removed.

For ease of construction the ballast concrete to stay in place is located beneath the ballast tanks. This does however reduce the effective height of the ballast tanks. The area covered by the ballast tanks equals 26% of the floor area in the tubes (based on the dimensions discussed further on). The remaining height below these tanks is then:

$$t_{concrete;remaining} = \frac{V_{concrete;remaing}}{A_{tanks}} = \frac{18\% * 1310.7}{16 * 10 * 4} = 369\ [mm]$$

Ballast tanks

The Wijker tunnel is a relative compact tunnel. The maximum inner free height is about 5metres. If the tanks spreads over the entire length the maximum effective height of the tanks is only 3 metres , leaving enough space the walk over the tanks. For this reason, the tanks are only connected to a single wall.

The width of the tanks then becomes 10 metres (leaving 3 meters of space next to the elements. For the height of the tanks a maximum of 1 meter above the tanks for construction and pump installation is needed, 0.4-meter extra height for the tanks to prevent water spilling over the edges and 0.4 meter regarding the ballast concrete beneath. The free height in the element equals 5.3 meters. The resulting height for ballast water is then 3.50 meters. A height of 3.44 is used.

From the vertical balance calculations (shown in appendix F) it follows that a length of the ballast tanks of 16 meters is needed (this is less than the maximum of 20 meters posed in section 2.3.1).

This lay-out results in the use of only 4 tanks, which is the most effective solution considering the fact that only 4 lifting eyes are used (as mentioned in section 4.1.3). A graphical demonstration of the lay-out is shown in Figure 54, Actual ballast tanks lay-out.

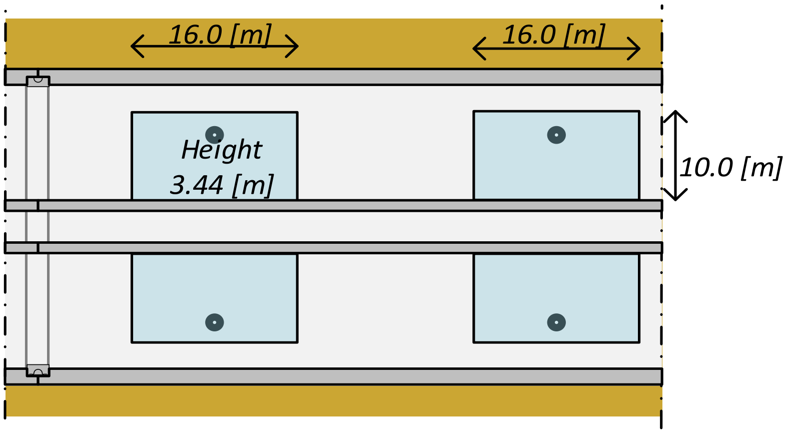


FIGURE 54, ACTUAL BALLAST TANKS LAY-OUT; TOP VIEW

Phasing

Nowhere during the entire construction process the FoS against uplift should drop below 1.024. To achieve this the soil cover on top of the element can be used. The ballast tanks are constructed while the soil still on top (the minimal cover layer thickness is used; $1000\ [mm]$).

This is an extra downwards load slightly bigger than the weight of the entire ballast concrete. Using this order all the concrete can be removed before the ballast tanks are constructed.

4.1.2 Installing bulkheads

The bulkheads are designed making use of both steel and concrete. In horizontal direction the load will be carried by the reinforced concrete. The loads on the concrete is the load averaged over height. The minimal concrete reinforcement is applied.

The load in vertical direction is carried by the steel HEB-profiles with steel quality S235, the beams are calculated on two supports with an increasing q-load over depth. The length of the longest beam is about 5.8 meters long, the maximum water density is used. The distance between the beams is $1300\ [mm]$ resulting in 20 beams used per element per joint, this distance is based on standard distances in other tunnel and a quick cost analysis showed this is the optimal value.

Based on the risk of failure of the bulkhead the 10% is added to the design loads. Unity checks are calculated for the cross-section, buckling is not considering since the beams are supported with concrete. The concrete calculation is both for the design moment and the design shear forces.

These calculations are made for each joint (which differs in water depth and therefore differs in the applied load). The result for each joint is shown in Table 15, Bulkhead results. The complete calculations are shown in appendix 0.

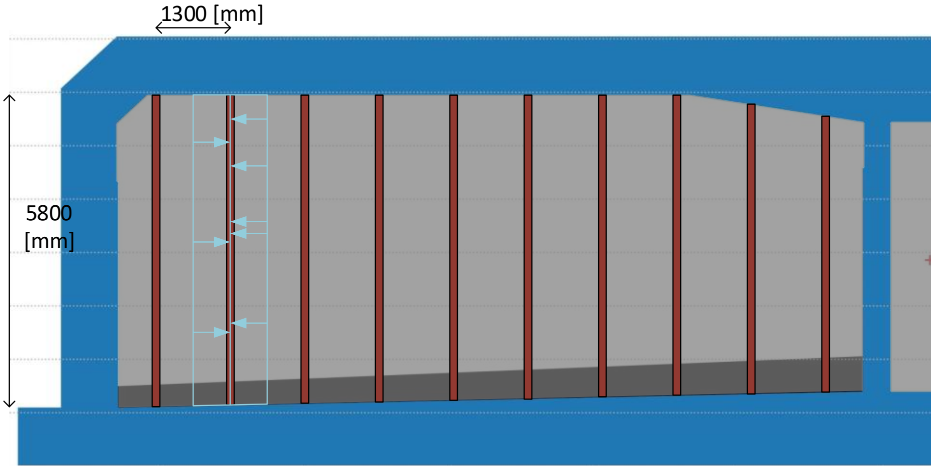


FIGURE 55, LOAD DISTRIBUTION ON BULKHEADS, CROSS-SECTION VIEW

	Joint 1&7	Joint 2&6	Joint 3&5	Joint 4
Beam type	HEB500	HEB600	HEB650	HEB700
Maximum deepening [m]	All	7+	2.28	2.97
Beam type			HEB700	HEB800
Maximum deepening [m]			7+	7+
Concrete thickness	300	340	370	380
Maximum deepening [m]	All	3.7	1.6	0.7
Concrete thickness		350	380	390
Maximum deepening [m]		7+	3.8	2.2
Concrete thickness			390	400
Maximum deepening [m]			7+	3.7
Concrete thickness				410
Maximum deepening [m]				5.3

TABLE 15, BULKHEAD RESULTS

The type of beam used can directly be translated to an amount of steel used in kilogram per running meter beam. The total beam length is known, and therefore the amount of kilograms of steel is known. Regarding the concrete, using the thickness and the total area the total volume is calculated. Note that per tunnel element 2 bulkheads are present and a total of 6 element for the complete tunnel. The result is shown in Figure 56, both for the concrete and steel usage.

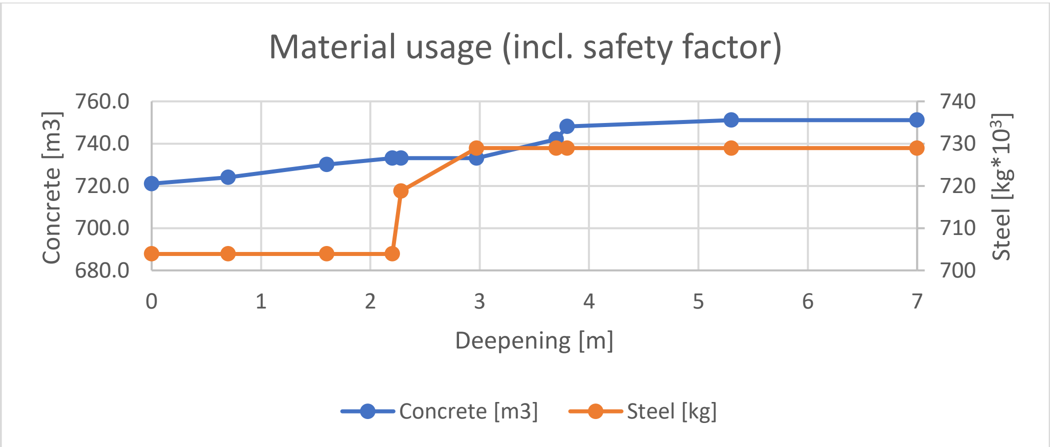


FIGURE 56, MATERIAL USAGE BULKHEADS

4.1.3 Reconnecting hoisting eyes

The force regarding buoyancy is mentioned in section 4.1.1. For calculating the total lifting force the adhesion calculated is needed, this calculation belongs to phase **C. Re-floating & Transport of the main** element, it is defined in section 4.3.1. The risk factor is defined in section 2.9. The resulting total lifting force on the hoisting eyes is a combination of these forces and equals $F_{lifting,total} = 12000\ [kN]$.

This results in enormous lifting forces per location of $F_{lifting,single,4wires} = 3000\ [kN]$. Due to the installation underwater, it is not possible to apply supplementary reinforcing. To reduce the force on the connection to the concrete four extra hoisting locations are constructed. The complete overview is given in Figure 57. These locations are only used for breaking the adhesion forces, after the element breakout the standard lifting procedure using only four wires is used. The resulting force on each of the 8 hoisting wires is $F_{lifting,single,8wires} = 1500\ [kN]$

Anchor connection

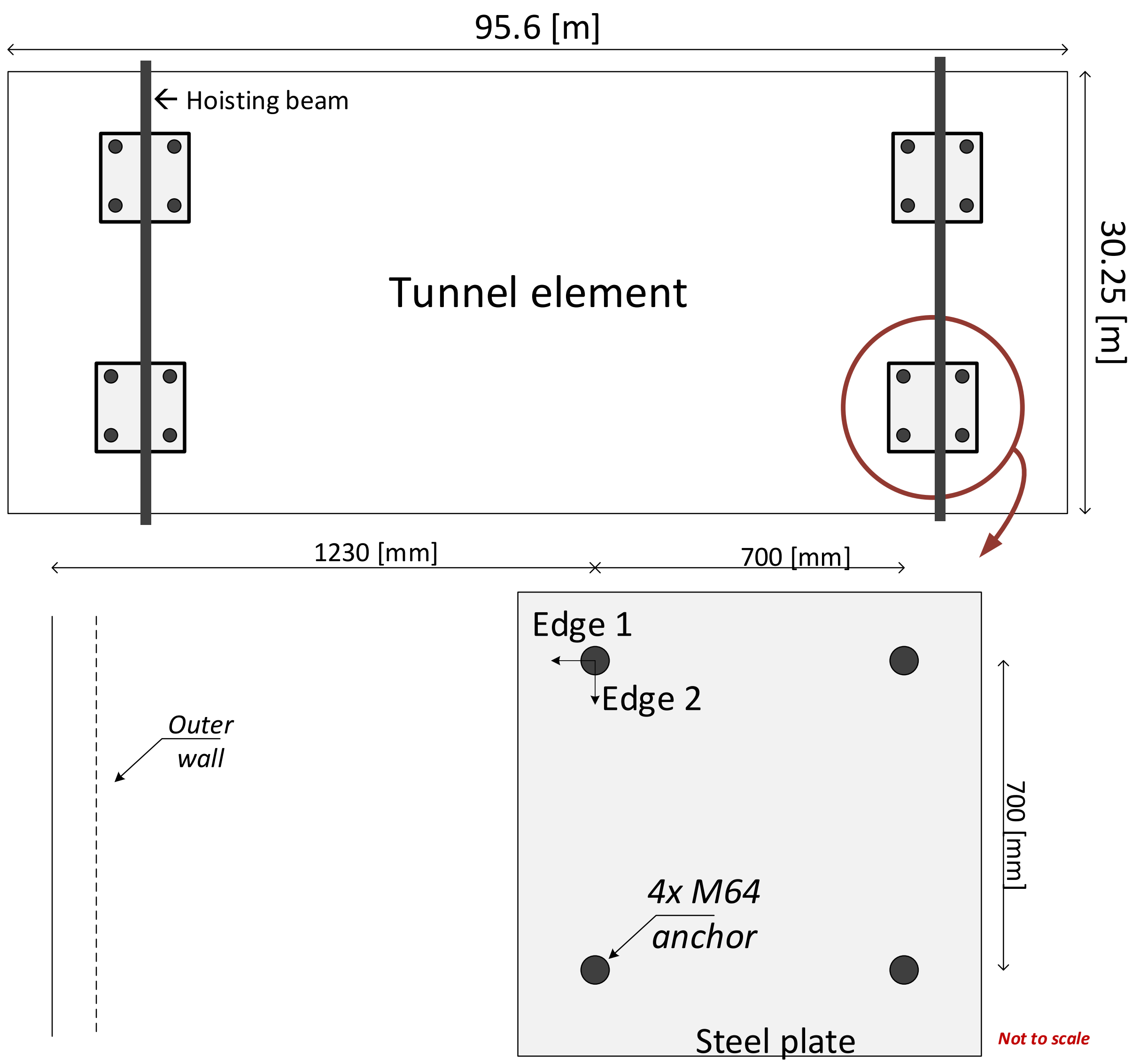


FIGURE 57, ANCHOR LAYOUT TUNNEL ROOF; TOP VIEW

A shear force is expected to occur due to water flow and waves; this is set at 10%. The design loads on the anchors then are: $N_{Ed} = 1500\ [kN]$ and $V_{Ed} = 150\ [kN]$

The design resistance of this anchor set-up is calculated according to NEN-EN 1992-4. The complete calculation is shown in appendix H and is divided in tension failure, shear failure and a combination. The resulting unity checks are shown below

Tension		Shear		Combination	
Failure	*U,C,*	*Failure*	*U.C.*	*Failure*	*U.C.*
a) Steel	0.371	a) Steel with lever arm	0.062	Steel	0.144
b) Concrete cone	0.970	c) Concrete pry-out	0.049	Other than steel	0.971
c) Pull-out	0.437	d) Concrete edge	0.062		
e) concrete splitting	Pass				
f) Concrete blow-out	Pass				

TABLE 16, U.C. ANCHOR FAILURE

Hoisting beam

On the water are floating pontoons with the winches, these drive the wires during lifting and immersion. Between the pontoons a beam is located connecting the pontoons and positioning the wire. The distance between these pontoons is about the width of the element. It is estimated at $31.0\ [m]$, leaving $37.5\ [cm]$ between the pontoon and the element. The beam between the elements is loaded with two point loads, located $1955\ [mm]$ from the end. A HEB800 profile, quality S355 is applied. The resulting cross sectional unity check is:

$$U.C._{cross} = \frac{M_{Ed}}{M_{Rd}} = \frac{1{,}955 * 1500}{3620} = 0.81$$

The governing failure mode shows to be lateral buckling. The buckling length of $31.0\ [m]$ could be reduced by applied buckling supports in the beam. When placed each $3.1\ [m]$, the unity checks become $U.C._{lat.buck} = 0.99$. Other solutions to the buckling would be to increase to a higher class profile or adapt the current HEB or IPE profile.

This adaption would be to weld extra parts to the current profile. Two options would increase the buckling capacity, see Figure 58. The red area shows welding plates on the sides of the profile making it a hollow cross section. The green area shows welding extra plates to the bottom flange, increase the moment of inertia in the horizontal direction.

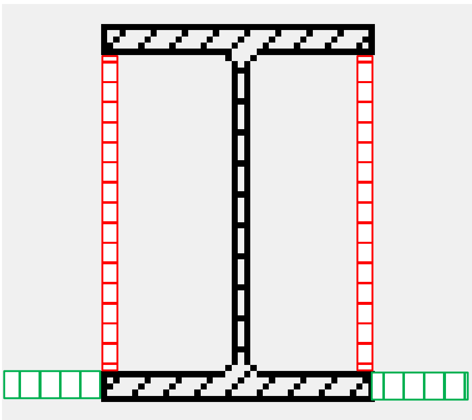


FIGURE 58, HEB-PROFILE WELDING ADAPTIONS

Moment in cross section

The hoisting force induces also a moment force in cross-sectional direction. The force spreads equally between the four anchors, these are located $700\ [mm]$ from each other (both in cross-sectional direction as in longitudinal direction). The MatrixFrame cross-section as defined in section 4.5 is used to evaluate the impact. Due to spreading in longitudinal direction and two anchors in cross-direction the loads become:

$$F_{lifting,cross\ sectional\ spreaded} = \frac{F_{lifting,single,8wires}}{l_{spread} * w_{spread}} = \frac{1500}{2 * (2 * .7)} = 536\ [kN]$$

The resulting moments due to these forces are shown in Figure 59. For some parts this loads has a positive impact, other a negative. The red area is responsible for a 20% increase in moment in the cross-section. Note that this moment only occurs during the initial lifting stage, until breakout occurs. The increase in force at this location after placing the element in a deeper location are higher and therefore governing, this is shown in section 4.5.3.

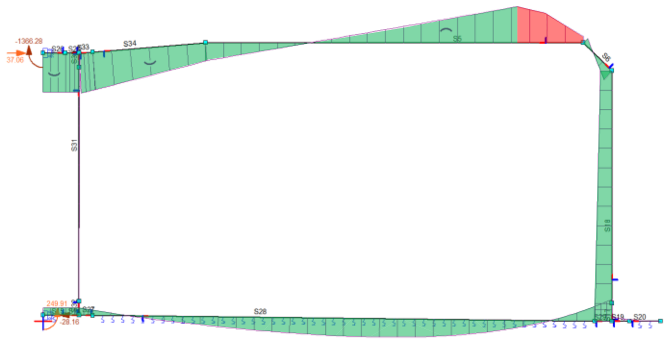


FIGURE 59, HOISTING MOMENTS IN CROSS-SECTION

4.1.4 Shear keys, omega seals, prestressing & other equipment

The four segments making up a single element are lifted together. The shear forces are transferred through the segments joint by shear keys. These are located in the walls and are shown in Figure 58. As already mention in section 3.2.6 the exact capacity is unknown, this is assumed at 9 $[MN]$ for all the keys combined

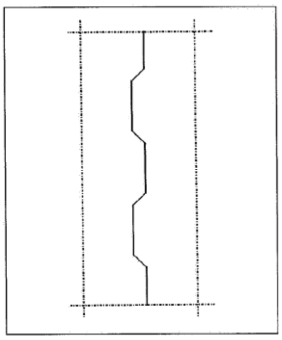


FIGURE 60, WIJKER TUNNEL SHEAR KEYS (CAMERIK & LEEUW, CEMENT, 1994)

The removal of the Omega seal is does not need extra calculation and is as described in section 2.3.4
The prestressing is a complex design and is discussed in detail in section 4.1.5.

4.1.5 Applying the prestressing

In this section the prestressing for the Wijker Tunnel is designed. This is done by producing a model which represent the tunnel. The results from this model are the internal forces regarding shear forces, normal forces and moments acting on the tunnel. These internal forces then are checked on the requirements posed above.

This is done for each of the variants mentioned in section 2.3.5. The chapter is finished by choosing the optimal variant. All the calculations are shown in appendix L.

Model setup

The Wijkertunnel as described in chapter 3 is a nearly prismatic tunnel in the longitudinal direction. Therefore, a simple 1D-beam model suffices. Due to the expected amount of load-combinations and load-cases some sort of software is desirable compared to a excel sheet. Due to the researchers experience the MatrixFrame software is used.

For the model two separate cases are considered. The first being the element under tension of the lifting forces, the second being the element floating. The element is modelled as a beam, the dimensions of this beam are as stated in section 3.2.1

During lifting operations, the beam is supported by two supports in z-direction, these represent the lifting wires. The weight of the ballast tanks is such that the reaction force in the supports equals the lifting force (being $2000\ kN$ per set of wires). The other forces on the element are:

- Weight of the element
- Weight of the ballast tanks
- Weight of the bulkheads
- Buoyancy force (note that this force starts working at the location of the bulkhead)

During transport operations, the beam is supported by springs in z-direction. The stiffness of these springs equals $K = w_e * \rho_{water}$. The other forces on the element are the same as during lifting operations, excluding the buoyancy force, but including wave loads. The Matrix Frame model is shown in the appendix.

To validate if this model represents the real situation some knowledge of initial prestressing and the loads during initial transport is needed. These are not available. However, this model is intensively discussed with experienced people in immersed tunnel design at TEC and RHDHV and is assumed to be valid.

Structural behaviour of the governing element

The governing element regarding longitudinal loads is the deepest element (due to the highest loads and the option just below the water level is already considered). The depth of this governing element is shown in chapter 3, adding the 1-in-1000-water level and a lowering of 4 meters (this is a conservative approach, more lowering would result in extra normal forces) results in a water column on the element of $22.76\ [m]$. Note that the depth of the lowered element is only important for lase case A.1 to B.2.

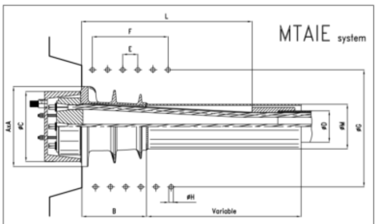
First all the general loads acting on the structure are gathered in section 2.7.1. For these, the exact parameters regarding material densities is unknown. Therefore, an upper and lower bound is created. This is followed by the loads created by applying the prestressing, this is done for each of the variants. The complete calculations and models are found in appendix L.

Prestressing wires model input

For all the variants the 'prestressing is preloading' concept is applied. This results in 4 extra load types on the structure. These prestressing wires are applied as shown in Figure 61 and are attached at 0.30 $[m]$ for the wall and/or floor.

FIGURE 61, PRESTRESSING WIRES IN BOX GIRDER BRIDGE (TENSACCIAI, 2007)

For the wires data is used from Tensacciai (TENSACCIAI, 2007). The maximum prestressing force applied is based on EN 1992-1-1, paragraph 5.10.2 and 5.10.3. The MTAIE Anchorage is used (applicable for external anchorage). The wires were assumed they have a centre of gravity 0.3 meters from the inner fibre of roof and floor. Steel with a characteristic tensile strength of $f_{pk} = 1860$ $[MPa]$ and a strand diameter of 15.2 $[mm]$ is used.



Type	A [mm]	B [mm]	C [mm]	D [mm]	E [mm]	F [mm]	G [mm]	H [mm]	L [mm]	M [mm]
4 MTAIE 15	150	100	135	63/3.6	50	300	170	10	310	82.5
7 MTAIE 15	180	120	160	75/4.5	60	360	220	12	360	101.6
9 MTAIE 15	200	180	177	90/5.4	60	360	250	12	430	127.0
12 MTAIE 15	220	190	195	110/6.6	60	420	310	14	450	139.7
15 MTAIE 15	250	208	210	110/6.6	65	455	350	14	520	152.4
19 MTAIE 15	280	225	245	125/7.4	70	490	400	16	600	168.3
22 MTAIE 15	300	240	265	125/7.4	70	560	430	16	660	193.7
27 MTAIE 15	325	250	295	140/8.3	80	640	470	20	700	203.0
31 MTAIE 15	350	300	330	160/9.5	80	680	530	20	750	229.0
37 MTAIE 15	400	360	330	160/9.5	80	720	580	20	800	229.0

Strand diameter 15,2 mm – nominal cross section area 140 mm^2 – nominal mass 1093 g/m										
Number of strands	4	7	9	12	15	19	22	27	31	37
Characteristic tensile strength f_{pk} = 1.860 MPa										
Maximum prestressing force $P_{0,\,max}$ [kN]	806	1.411	1.814	2.419	3.024	3.830	4.435	5.443	6.250	7.459
Maximum overstressing force $P_{0,\,max\text{-}ov}$ [kN]	851	1.490	1.915	2.554	3.192	4.043	4.682	5.746	6.597	7.874

FIGURE 62, MTAIE PRESTRESSING DIMENSIONS

Variant 1; Connecting outer segments

The first variant is most close to the original lay-out of the prestressing. Two layers are placed in the element, one near the roof and one near the floor. These wires are attached to the two outer segments, 4 meters from the segment joint, the length of the wires results in 55.8 $[m]$

Between these 56 meters three segment joints are located. If somehow longitudinal displacement occurred it is assumed a 10 $[mm]$ gap is present, this is however a conservative assumption. After lifting the element this gap is closed and a part of the prestressing capacity is lost. This amount equal $\frac{10*3}{55\ 800} = 0.027\%$

The roof thickness is 1100 $[mm]$ thick over the main part. The floor thickness increases linear from 1100 $[mm]$ to 1400 $[mm]$, a thickness of 1250 $[mm]$ is used. This is summarized in below.

Part	Distance from 0 [mm]	Eccentricity [mm]
Outside roof	8045	4185
Inside roof	6945	3085
Roof wires	6645	2785
Centre of gravity	3860	-
Floor wires	1550	2310
Inside floor	1250	2610
Outside floor	0	3860

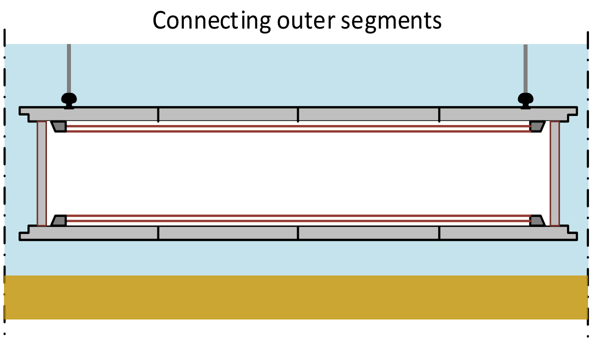


FIGURE 63, PRESTRESSING VARIANT 1 LAYOUT

The introduction of the prestressing forces introduces load acting on the element at the location mentioned above, 4 meters from the outer edges.

Variant 2; Connecting each joint

The distance to the roof is the same in this variant as in variant 1. The difference is that for each segment joint a different amount of prestressing can be applied. The element consists of 4 segments, with 1 middle joint and 2 outer joints. Due to the symmetric nature of the loads the prestressing in the two outer joints is equal.

The loss of prestressing due to current displacements between the segments is calculated the same as for variant 1. The only difference is a wire length of 8 meters and a single segment joint. $P_{loss} = \frac{10}{8\,000} = 0.063\%$. The prestressing is assumed to be connected 4 meters away from each joint. This is based on a shear crack angle of $\theta = 26.6°$ and a tunnel height of about 8 meters.

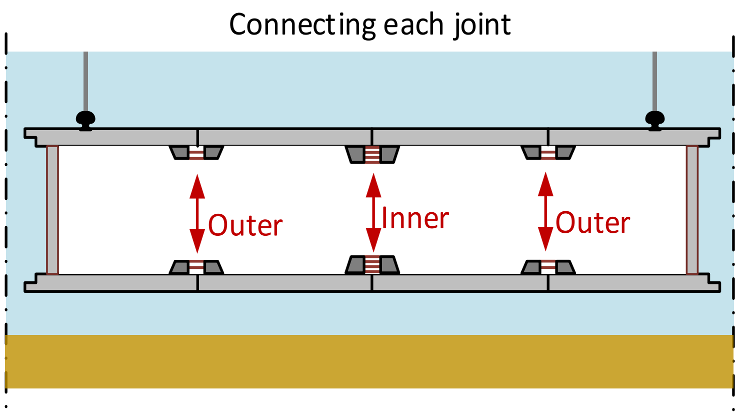


FIGURE 64, PRESTRESSING VARIANT 2 LAYOUT; LONGITUDINAL VIEW

Variant 3; Bending the tendons

For this variant it is assumed the tendon connections are located at the same eccentricity as before. The location of connections to the element are variable. The angle under which these tendons are placed is $\alpha = \tan^{-1}\left(\frac{5095}{Longitudinal\ distance}\right)$. At the end a normal force is present

The relaxation due to the displacements is similar as in variant 1. Based on the different lay-outs and amount of segment joints the prestressing loss is about 0.02%

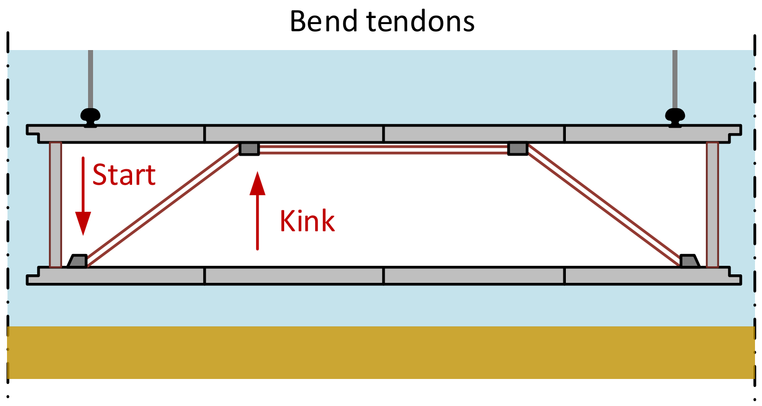


FIGURE 65, PRESTRESSING VARIANT 3 LAYOUT; LONGITUDINAL VIEW

Variant 4; Extra hoisting eye

The variant 1 showed that the critical load case is during transport and the impact of waves. An extra set of hoisting eyes would only have effect during load case A.1-B.2.

This variant is therefore definitely not the most effective solution and won't be discussed further.

Variant optimization

For optimization, the resulting length of all the wires together should be minimal. The model in MatrixFrame is written such that an output file is created with the governing moments and shear forces for the different locations of the ballast tank (with a step size of 10 centimetres, $x = 0$ means the centre of the ballast tanks is above the outer segment joints.).

The requirement is a minimum of $\sigma_{joint,req} = 0.22\ [N/mm^2] = 220\ [kN/m^2]$ pressure in joint. This should be in the outer fibre of the element floor and roof. From the MatrixFrame output the stresses can be calculated using the following formula (using the MatrixFrame positive and negative definitions (positive moment = tension in bottom fibre) a minus is needed regarding the normal force).:

$$\sigma_{Rd} = \frac{-N}{A} + \frac{Mz}{I} > 220$$

In the third variant also a shear force is introduced. The maximum shear force capacity of the joint is unknown. An average shear key is assumed having a capacity of 3 MN. These are in the outer walls; the inner walls are so thin that these together are assumed to represent half a shear key each. Resulting in a maximum shear force of $V_{Rd} = 9\ [MN]$.

With these requirements and the variables for each variant the optimal solution can be found. This is done for all the available wire types mentioned above. This result in a location of the ballast tanks, locations of the hoisting eyes and the amount of wires needed.

<u>Variant 1; Connecting outer segments</u>

As mentioned for variant 1 only two layers of prestressing are applied, one in the roof and one in the floor. The prestressing force is divided into an eccentric part P1 and a co-centric part P2. P1 is chosen such that the stresses in roof and floor are equal, then P2 is chosen such that the stresses are above the required level of $\sigma_{joint,req}$. The complete calculation is shown in appendix H. The result is shown in the table below, in this table the most outer positions of the ballast tanks are mentioned.

Wire type [MTAIE 15]	Ballast tanks from x= [m]	Wires roof [-]	Wires floor [-]	Total wire length [m]
4	-0.3 to 0.0	34	6	2232
	6.4 to 6.7	29	11	
7	-0.8 to -0.7	20	3	1283
	5.2 to 6.4	17	6	
9	-0.2 to 0.7	15	3	1004
	6.0 to 6.8	13	5	
12	-2.7 to -2.6	13	1	781
	4.7 to 7.2	10	4	
15	-2	10	1	614
	4.7 to 6.8	8	3	

FIGURE 66, PRESTRESSING VARIANT 1 WIRE OPTIONS

<u>Variant 2; Connecting each joint</u>

The approach for this variant is mostly like the approach of variant 1. The only difference is that this optimization using P1 and P2 is done for the outer and inner joint separately. The total amount of wires needed is a summation of the two outer and the one inner joint. The complete calculation is shown in appendix H.

Wire type	Wires roof [-]	Wires floor [-]

[MTAIE 15]	Ballast tanks from x= [m]	Outer	Inner	Outer	Inner	Total wire length [m]
4	0.4	23	33	6	0	728
7	-3.3 to -3.1	14	23	1	0	
	1.2 to 1.4	13	19	4	0	424
9	-3.5 to -2.8	11	18	1	0	
	-1.2 to -1.0	11	16	2	0	336
12	-2.4 to -2.2	8	13	1	0	
	0.3 to 0.4	8	11	2	0	248
15	-2 to -1.7	7	10	1	0	208

FIGURE 67, PRESTRESSING VARIANT 2 WIRE OPTIONS

Variant 3; Bending the tendons

In this variant only a single 'layer' of prestressing is applied, this is however not a horizontal layer. This results also in a shear force acting on the tunnel element. The wires are assumed to start the earliest at 2 meters from the start of the bulkheads. The kink in the cable can anywhere, even in the middle.

Wire type [MTAIE 15]	Ballast tanks from x= [m]	Wires [-]	Connection [m] Floor	Roof	Total wire length [m]
4	1.2	33	4.53	32.21	2887
7	1.4	19	4.76	32.4	1653
9	1.4	15	4.76	32.8	1305
12	1.4	11	4.75	32.2	957
15	1.4	9	4.76	32.8	783

FIGURE 68, PRESTRESSING VARIANT 3 WIRE OPTIONS

Variant selection

The decision on which variant is the best is made on two criteria:

Constructability

The constructability is mainly about the amount of wires applied. This should be low enough to fit in the cross section of the tunnel which also includes the ballast tanks and locations to walk. It is concluded that a maximum of 20 wires can be applied, 10 per tunnel tube.

But on the other hand, not too low that all the prestressing is based on a single cable, or that all the prestressing force is located at a single point. It is concluded that at least 4 wires are needed, 2 per tunnel tube.

Costs

The lower the cost, the more attractable the solution is. The total wire length is a good indication of the cost of the solution. Another indicator is the amount of connections to the element to be constructed.

The cost is directly linked to the number of kilograms of prestressing steel used. based on oral communication at RHDHV the price of prestressing steel is set at $6\ €/kg$, including construction a standard factor of 1.5 is added resulting in a total price of $9\ €/kg$ (incl. VAT). This is 50% increase in construction cost.

Due to the high amount of connection to the concrete the second variant will have a higher factor regarding construction cost. Roughly three times more connections to the concrete are needed (two outer and one inner joint, compared to a single prestressing wire). For this a 2.5 factor is used (a 150%

increase) resulting in 15 $€/kg$. In Table 17, Prestressing cost summary the total cost per variant is shown.

Wire type	Variant 1			Variant 2			Variant 3		
Strands	Length [m]	Weight [kg]	Cost [x10^3 €]	Length [m]	Weight [kg]	Cost [x10^3 €]	Length [m]	Weight [kg]	Cost [x10^3 €]
4	2232	9754	**87.8**	728	3181	**47.7**	2887	12616	**113.5**
7	1283	9815	**88.3**	424	3244	**48.7**	1653	12645	**113.8**
9	1004	9879	**88.9**	336	3306	**49.6**	1305	12841	**115.6**
12	781	10247	**92.2**	248	3254	**48.8**	957	12556	**113.0**
15	614	10070	**90.6**	208	3411	**51.2**	783	12841	**115.6**

TABLE 17, PRESTRESSING COST SUMMARY

Based on the cost and how complex lay-out of the tendons the third variant is not the optimal solution. Based on the criteria regarding constructability the only good solutions are:

- Variant1; Wire type: *7 MTAIE15*
- Variant2; Wire type: *7 MTAIE15*

Table 17 shows that variant two is cheaper, **therefore the variant 2 is chosen.**

Selected variant verification

In this chapter a final verification is made to check if the applied amount of prestressing results in allowable stresses in the concrete regarding normal forces, shear forces and moments. This check is done both for the forces located at the segment joints and the forces located anywhere in the construction. The calculation for this section is done in chapter M.

- From previous chapter it is concluded that 7 MTAIE15 wires are used.
- In the roof 13 wires are located at the outer joint, 19 wires are in the inner joint.
- In the floor 4 wires are located at the outer joint, 0 wires are in the inner joint.
- The middle of the ballast tank is 1.2 to the middle from the outer segment joints.
- The hoisting eyes are placed as much as possible outwards, without make load case A.1 to C.2 governing. This is favourable regarding stability. The distance from the outer end is 18,9 $[m]$.
- The relaxation due to displacements results in a 0.125% reduction in maximum prestressing force.

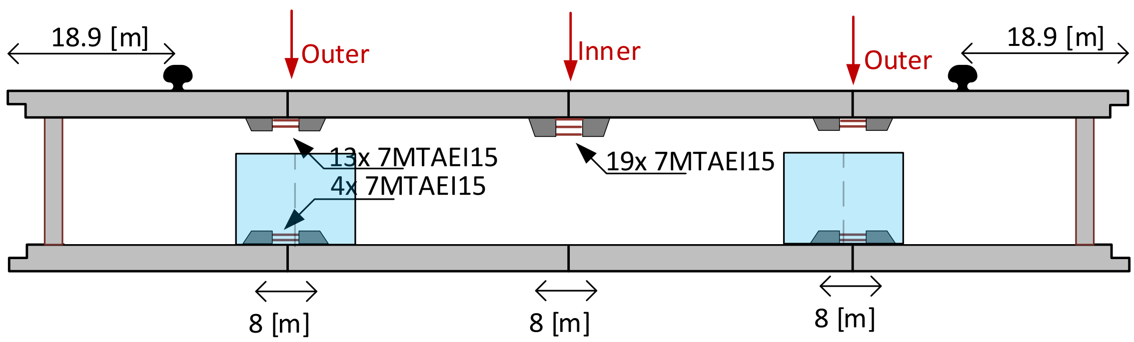


FIGURE 69, POST-TENSIONED PRESTRESSING LAY-OUT; LONGITUDINAL VIEW

<u>Segment joints</u>

First the forces in the segments joint are discussed. The internal forces are calculated using the MatrixFrame model, sadly not a single governing moment line can be created since different models are used, based on which load case is relevant. The governing design loads are gathered in an 8-meter influence area around the exact location of the joint. This value is based on a shear crack angle of $\theta = 26.6°$ and the element height. The results are shown in Table 18

Design loads	**Load case A.1 - C.2**		**Load case D.1 – F.2**	
	Outer	Inner	Outer	Inner
Moment roof [kNm]	-56 368	-98 296	-69 799	-109 197
Moment floor [kNm]	-6 067	-35 559	-4 486	-34 920
Normal force [kN]	-9 940	-9 940	-9 404	-9 404
Shear force [kN]	2 083	737	3101	1079

TABLE 18, MATRIXFRAME INTERNAL FORCES SEGMENTS

As mentioned, the prestressing induces extra loads. With the wires prescribed and loading them to full capacity these loads become:

The resulting unity checks are given in Table 19. All values are below the critical value of 1.

Unity check	**Load case A.1 - C.2**	**Load case D.1 – F.2**

	Outer	Inner	Outer	Inner
Roof stresses	0.766	0.749	0.999	0.925
Floor stresses	0.931	0.928	0.984	0.962
Shear forces	0.231	0.082	0.345	0.120

TABLE 19, VERIFICATION U.C. SEGMENT JOINT

<u>Entire element</u>
The highest possible loads anywhere in the construction are shown in Table 20.

Governing load	**Load case A.1 - C.2**	**Load case D.1 – F.2**
Moment roof [kNm]	-98 296	-109 196
Moment floor [kNm]	-6 933	-4 486
Normal force [kN]	-9 940	-9 404
Shear force [kN]	5 992	4 193

TABLE 20, MATRIXFRAME INTERNAL FORCES ELEMENT

This table shows that maximum loads occurred near the segment joints. In the rest of the element the resistance capacity regarding moments and normal force is higher than near the segment's joints (since these joints can't even handle tensile forces). Therefore, it can be concluded that the unity checks are lower than 1.

Regarding shear resistance the same method of calculating the shear resistance in the bulkhead is used. This method is discussed in appendix 0. The result is a $V_{Rds;min} = 20\ 402\ [kN]$. This results in a unity checks lower than 1.

It can be concluded that this solution regarding prestressing is a valid solution, nowhere in the construction the loads exceeds the resistance.

4.2 Removing the first section

4.2.1 Removing soil cover and backfill

Removal of the soil will be done in the phasing as discussed in section 2.4.1. The determination for which equipment to use is based on two criteria, the amount of precision during the removal and the depth until which to be removed is needed.

A high amount of precision is desired for two main reasons. First it is needed for lowering the load from the soil still acting on the element while the re-floating process should start. This is specifically important for the first tunnel to be re-floated, since not much knowledge is available regarding the soil adhesion. The other reason is that the element should not be damaged by the dredging equipment. This was already identified as a risk in section 2.8. Corrective measurements are enough but still the amount of failures should be limited to a minimum.

Therefore, a combination of the backhoe dredger and grab dredger is used. Making use of an average backhoe dredged a maximum depth of 20 meters is applied. Based on the average water level this reached up till $-20.40\ [m\ NAP]$. Until the ultimate depth a grab dredger is used. Figure 70 shows the dredging with respect to the equipment used.

From the adhesion calculations and the original foundation thickness in section 3.2 it follows 1 meter of free space is required below the element. The slope of the trench is set at 1:3, just as in the original design.

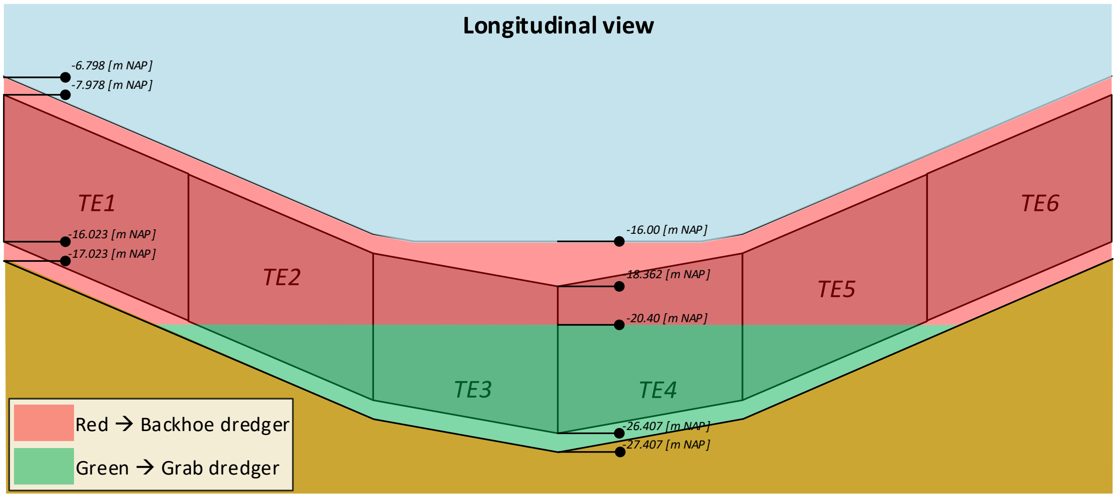


FIGURE 70, LONGITUDINAL DREDGING SCHEME; LONGITUDINAL VIEW

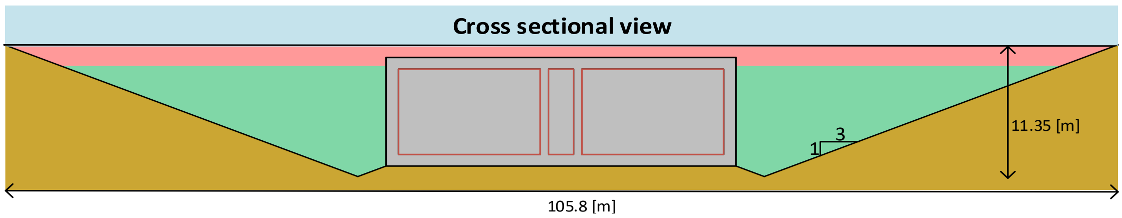


FIGURE 71 CROSS-SECTIONAL DREDGING SCHEME; CROSS-SECTION VIEW

The volumes of soil to be removed can be calculated rounded at cubic meters. And are the following:

	TE1	TE2	TE3	TE4	TE5	TE6	Total

Backhoe dredger $[m^3]$	37 396	32 014	29 378	29 378	32 014	37 396	197 576
Grab dredger $[m^3]$	53	5 435	14 404	14 404	5 435	53	39 784

TABLE 21, DREDGING VOLUME SUMMARY

4.2.2 Determining opening joint

As discussed in section 2.4.2 the minimal required length of the opening joint is 5 meters. The original closure joint in the Wijkertunnel had a width of 2700 $[mm]$, and is located after tunnel element 6. The joint is in-situ casted against the land head and the tunnel element

For the opening joint of 5 meters two cuts are needed. These cuts are slightly tilted such that the opening joint has a V-wedge, about a 1:8 angle is applied. The first cut start at the left of the original closure joint. The second start 4.3 meters into the final segment of the final tunnel element. As mentioned in the risk measures in section 2.9 guidance rails are installed near the cuts. The final lay-out is shown in Figure 72. The red area shows the opening joint, the red lines show the location of the guidance rails.

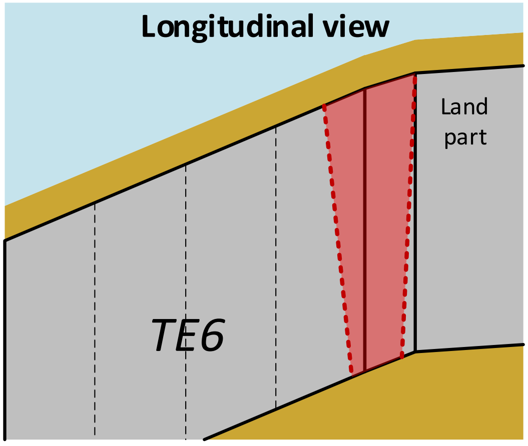


FIGURE 72, OPENING JOINT; SIDE VIEW

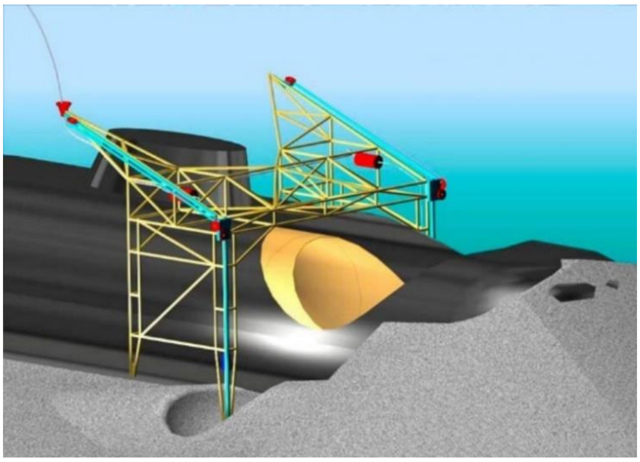


FIGURE 73, CUTTING EQUIPMENT (CUT, 15)

Regarding the cutting method; since the Wijkertunnel is a concrete tunnel and damage to adjacent parts should be minimal the blasting solution will not be treated. For cutting the Cutting Underwater Technology Limited company has much experience regarding submerged cutting in for example concrete (CUT, 15). They can design special diamond wire cutting machines (DWCM). A cutter for a specific shipwreck was already designed (being 25-meter-wide, compared to the 31 required for the Wijkertunnel), it is shown in Figure 73.

4.3 Re-floating & transport of main element

Securing access to element & defining buoyancy

The access to the element is discussed in section 2.5.1, this method is applied and does not need further explanation.
The buoyancy is already discussed when calculation the vertical balance in section 4.1.1 and appendix F.

4.3.1 Breaking the soil adhesion

In section 2.5.3 the soil adhesion was already discussed for a general tunnel. In this section the adhesion for the Wijkertunnel is discussed. The complete calculations can be found in appendix I.

Improving breakout layer thickness

Due to multiple researchers pointing out that the breakout thickness is 2 millimetre this value is used for the Wijkertunnel. Compared to the value's used in section 2.5.3, also the water density and the element dimensions are changed. This results in an expected breakout duration of 39.2 days.

Improving soil permeability

However, this is still based on a very conservative approach regarding foundation permeability. The Wijkertunnel foundation is placed with 'underflow pancakes' beneath the element and the actual permeability of the sand used should be known. During this research this value is unknown, but a good assumption can be made. Baber and Luniss mention the standard sand flow used has a $D_{50} = 150 - 500\ [\mu m]$ and $D_{10} > 100\ [\mu m]$ (Baber & Luniss, 2013). Making use of Hazen's formula $K = C * D_{10}^2$, applicable from a effective diameter from 0.1 to 30 mm the expected soil permeability is $0.010\ [cm/s] = 8.64\ [m/day]$, based on an empirical coefficient set at the average value of 1. A conservative approach is used setting the permeability at 2 $[m/day]$. The calculated breakout time results in a duration of 0.98 days.

An important notion should be made that if this project is executed a test sample of the foundation soil should be taken. This sample can then be tested, and the actual permeability is defined.

Increasing tension

It is concluded that extra measures are needed. The first is increasing the force in the hoisting eyes. This results in a higher pulling force in the winches on the pontoon or an increase in number of hoisting eyes. Regarding the buoyancy already a force of 100 ton is required in in each winch, a quick search shows that several winches up to 300 ton are available (Ellsen Marine Winches, 2020). The force for breaking the adhesion is set at a total force of 8000 [kN], considering the risk measures this is an effective force of 6400 $[kN]$. The increased force results in a breakout time of 0.61 days, or 14.7 hours.

Reducing effective width

The final measure to be taken is a lowering of the element effective width. From the options described in section 2.5.3 the *option C. Water jetting from sides* is an effective solution with a reach of the jets of 10 meters. In case of a calamity not the complete foundation can be removed since the element should be able to be lowered on its foundation. It is assumed 50% of the foundation should stay in place, this results in a foundation allowed to be removed of about 8 meters at each side. The expected breakout time then becomes 1.79 hours, this is an acceptable value. As mentioned, an overview of these calculations is shown in appendix I.

4.3.2 Transportation of the element

As mentioned in section 2.6.2 three options are available regarding the location in which the element can be repaired and prepared for the new immersion process. The first option, being the original drydock will result in transport over sea. This induces higher loads and higher requirements regarding freeboard. This option will only be used if no other option is available.

The second option is sheltering in the current waterway, this also includes constructing protective measures regarding ship collisions. The current waterway is the North Sea canal which is intensively used. Blocking part of this waterway will result in a lower capacity of the canal and therefore sheltering in the current waterway is not viable.

The third option is sheltering in a nearby harbour. This option is a good option since the port of Amsterdam is close by. In this harbour several quay walls can be used to dock the elements. Also, several ship repair docks are available if complicated maintenance is needed. Figure 74 shows the location of the Wijkertunnel and the location of the Damen Shipyards Drydocks (this dock does have enough dimensions) (DAMEN, 2020). Note that only a single tunnel element can be prepared in the dock, other element should be sheltered elsewhere. In the coming years it might also be possible to use the old Ijmuiden locks as shelter.

FIGURE 74, WIJKER TUNNEL AREA MAP

4.4 Preparing, adapting & reconfiguration

4.4.1 Calculating new alignment

Regarding the new alignment two aspects need attention. As discussed in section 2.6.1 these are the rotational capacity and the traffic requirements. This analysis is made with a varying increase in depth, note that this is only the lowering in the middle of the tunnel. The increase in the effective depth of the waterway is less since it is not defined on the depth in the middle but also the width.

Traffic requirements

Based on the tunnel parameters in section 3.2.2 the tunnel is divided in three parts. Two symmetric straight parts (no vertical curvature) having a total length of 7 tunnel segments ($\approx 168\ meters$) and a middle concave curved part of 10 segments ($\approx 239\ meters$). The convex curve is in the land parts. From section 2.6.1 it is known that the truck speed reduction is governing and that this maximum speed reduction is 20 $[km/h]$. Combined with the given slope length a maximum slope is defined in Figure 75. The resulting maximum slope is 8.1%, since the slope partly continues in the concave curve a conservative maximum of 7.5% is applied.

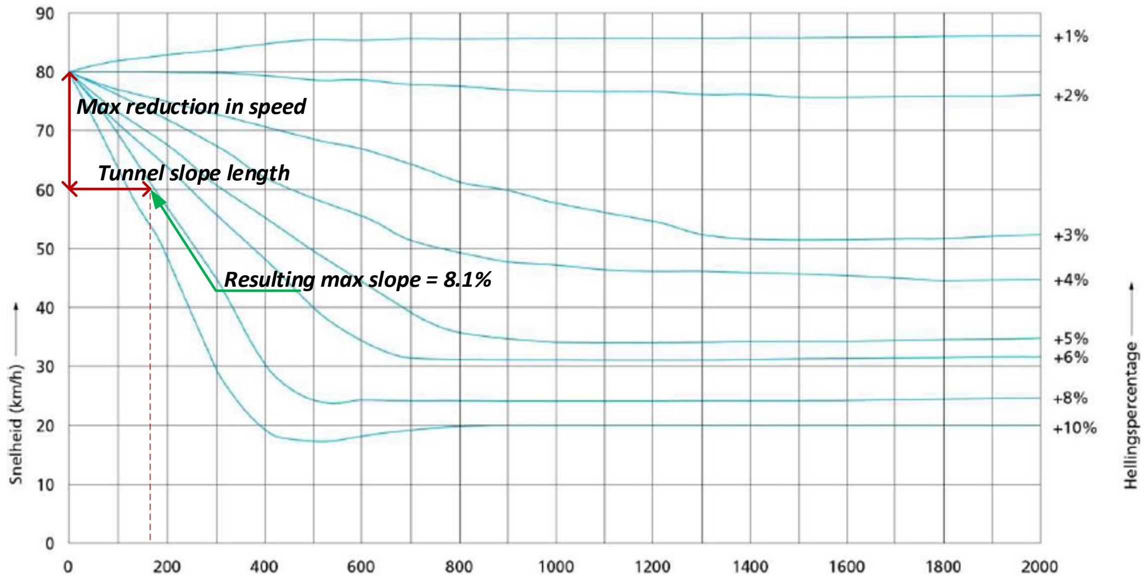


FIGURE 75, WIJKER TUNNEL SPEED REDUCTION

In calculation the curves a parabola shape is used. The required curvature is then $R = 100 * \frac{x}{i} = 100 * \frac{119.5}{7.5} = 1593\ [m]$. This is higher than the minimum set in Table 3 for a speed of 120 $[km/h]$. The total change in height over the length of the tunnel is then $7.5\% * 168 + \frac{119.5^2}{2*1593} = 17.02\ [m]$. In the original design this distance was $10.4\ [m]$. The maximum deepening in the middle of the tunnel is $6.62\ [m]$

Rotational capacity

Regarding rotational capacity in section 2.6.1 three options are discussed. NO adaption to the land parts, ONE side adapted, or TWO sides adapted. In all these options rotations are captured in the GINA seals. The design of is divided in three parts. The complete calculation for the GINA design is shown in appendix X.

First, for the faces of the elements four different depth must be calculated (for four different element faces), this is done for both the original situation and the situation with maximum deepening based on the traffic requirements. For these depths a total resulting water pressure of the element is calculated, this results in an average compression force over the entire seal.

Secondly, extra compression or relaxation occurs in the seal (Smitt, 2000). This occurs due to several reasons, first being an uncertainty in construction of the steel IPE profiles on which the seals are applied. Secondly a deviation occurs due to misplacement. Thirdly an extra rotation due to settlements. Fourthly a relaxation due to the placement of the closure joint and the release of water pressures. The final factor is an expansion in the concrete due to changing temperatures.

Thirdly a difference in compression over the height of the element. This compression is not present in a standard immersed tunnel. The big worldwide producer of GINA and OMEGA seals, Trelleborg, was contacted and asked for the possibilities in constructing a GINA seal with a difference compression over height. This type of seal is possible to construct and already applied before. Note that this type of seal changes for example its width over the height of the element, this results in different compressions over the height.

Based on these three values a GINA seal can be chosen. Figure 76 shows an example seal with a force-compression diagram, it shows two joints (voeg4 and voeg7)

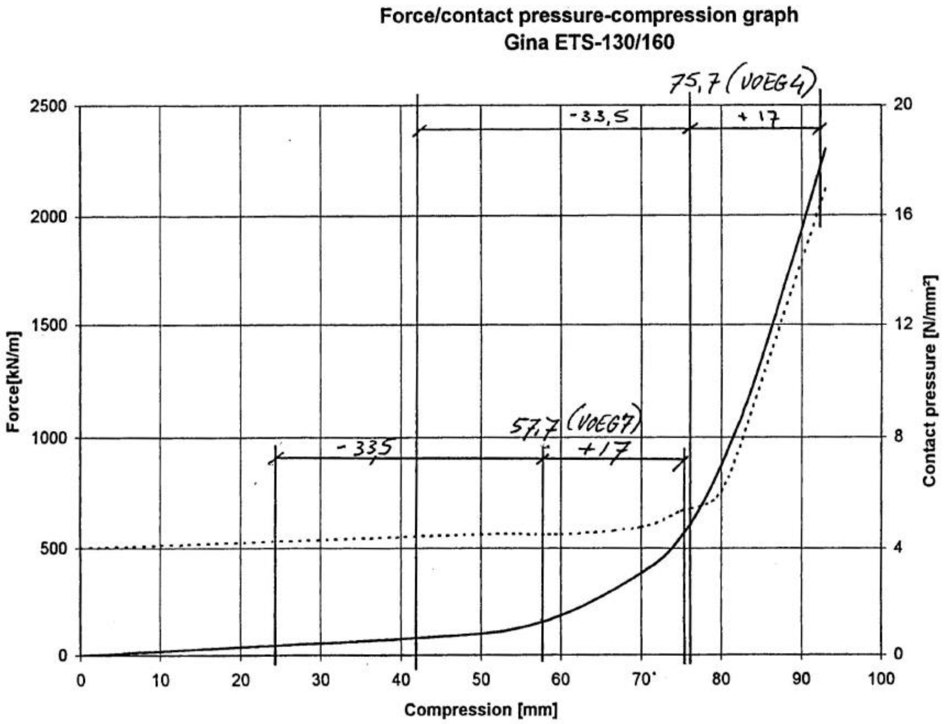


FIGURE 76, CALANDTUNNEL GINA-SEAL (SMITT, 2000)

The remaining question then is the amount of rotation required in between the elements. For this rotation three options in section 2.6.1 are discussed. The calculation made in appendix J is executed with a variable expansion for the GINA seals. The result is a maximum deepening in meters depending on the variant chosen in the middle of the tunnel. These values are expressed by the following formula's, all units in millimetres.

$$Deepening_{NO\ adaption} = 38.0 * Expansion_{GINA}\ [mm]$$

$$Deepening_{ONE\ adaption} = 45.6 * Expansion_{GINA}\ [mm]$$

$$Deepening_{TWO\ adaption} = 57.0 * Expansion_{GINA}\ [mm]$$

Since in this design the actual amount of deepening is kept as free as possible no exact design can be made for the GINA seal. However, with the values from the GINA design all the parameters are available. These can be provided to a producer such as Trelleborg, which guaranteed to be able to provide such a type of seals.

4.4.2 Preparing element face and seals

The method of preparing the element regarding the new GINA seals and new nose & catch-structure is the same as discussed in section 2.6.2. The dimensions of this new GINA seal are discussed in section 4.4.1

4.4.3 Construction new foundation

As already discussed in section 2.6.3 the best solution would be a gravel foundation since for a new sand foundation a new jack and pin structure should be incorporated in the design. For the new foundation a standard gravel foundation will be applied.

Before placing the new foundation, the old foundation should be removed, this is done using a grab dredger as discussed in section 2.6.3. The new gravel bed is placed from a floating scraping pontoon. From this pontoon a pipe is lowered and places the foundation, the thickness of this foundation is 1 meter. A plan for such a foundation is shown in Figure 77.

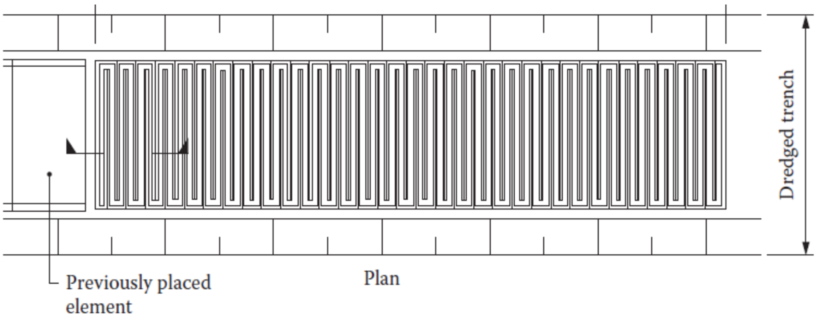


FIGURE 77, GRAVEL FOUNDATION

4.4.4 Adapted approach structure

For the approach structure the two main options available are discussed before. The most attractive solution would be only local adjustments. If this is not possible general lowering is needed.

In the original design the approach structures vertical alignment was shaped in a convex curvature with $R_{top} = 10\ 000\ [m]$. The horizontal length of this curve is $i_{old} * 0.01R_{top} = 450$ [m]. However, in the new tunnel the slope is increases from 4.5% to 7.5%.

Figure 78 shows to original design of the approach structure, the left side being the normal level, on the right the first element is placed. It shows that before the location of the element about 140 meters is at the slope of 4.5%, this distance can be incorporated in the new curve.

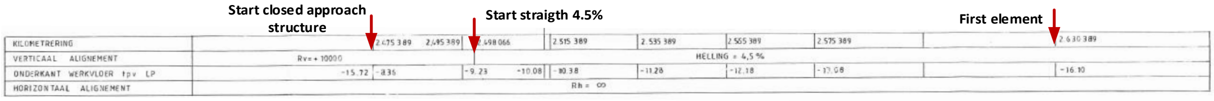


FIGURE 78, WIJKERTUNNEL APPROACH STRUCTURE VERTICAL ALIGNMENT

To reach the 7.5% slope a curvature of $R_{new} = \frac{x}{i} * 100 = \frac{450+140}{7.5} 7867\ [m]$ is needed.

Relative to the first tunnel element the original change in height over the complete approach structure is:

$$h_{old} = h_{straight} + h_{curved} = 140 * 4.5\% + \frac{450^2}{2 * 10000} = 5.94 + 10.125 = 16.07\ [m]$$

$$h_{new} = h_{straight} + h_{curved} = 0 + \frac{(450 + 140)^2}{2 * 7867} = 22.12\ [m]$$

This shows the start of the new approach structure is placed 6 meters higher than the current location. In the open part of the approach structure the asphalt level is quite easy to increase. In the 160 meters of covered approach structure this is a more complicated. The exact method for this is not described in this research, it is assumed to be possible, note that this increase in height is only up to 2.51 $[m]$.

A complete overview is given in Figure 80, the location of the approach structure is given in Figure 79.

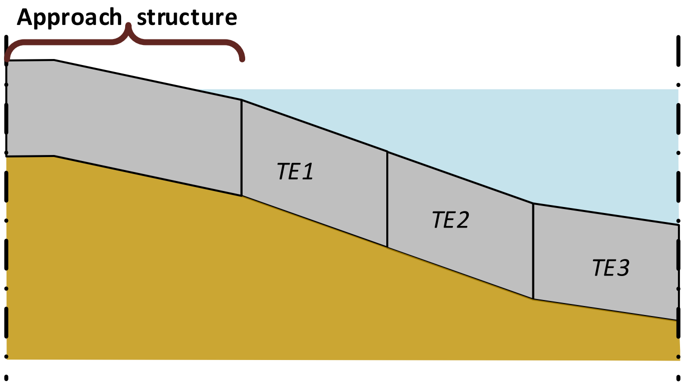


FIGURE 79, APPROACH STRUCTURE LOCATION; LONGITUDINAL VIEW

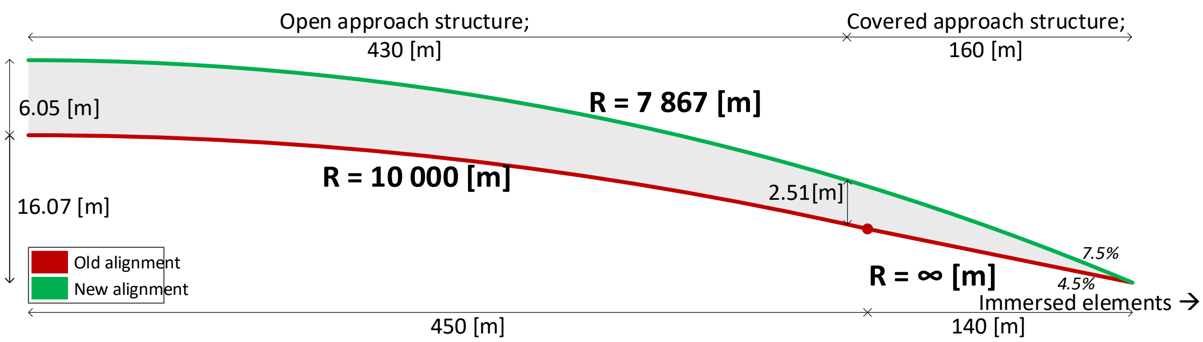


FIGURE 80, APPROACH STRUCTURE OVERVIEW

4.4.5 Adapted closure joint

In section 4.2.2 it was already discussed that the opening joint had a width of 5 meters at the bottom, this joint is shaped in a V. Regarding the new closure joints the options were discussed in section2.6.5.

The in-situ joint is quite a good solution here. The only problem being that the temporarily wedges are placed at an angle (due to the angled cut), this lowers the effectiveness of these wedges.

The prestressed segment method is not viable since the new elements are subjected to another type of prestressing than normally used in this method. Also, the construction of this segment is not possible in a dry dock. The third option, a terminal block is not possible.

4.5 Final situation

This part of the method is not discussed in section 2.2 because it is not part of the 'new' method. However, it is important to take in mind. Due to the increase in depth there is an equal increase in water pressure on the tunnel, see Figure 81 for a graphical representation. In this chapter the impact of this increase on the tunnel is discussed.

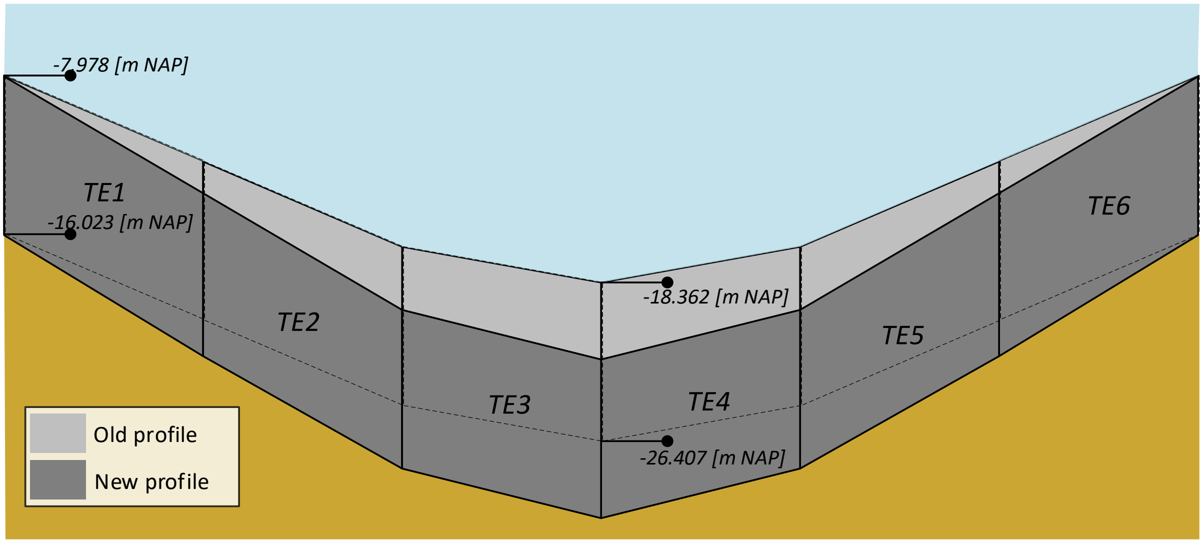


FIGURE 81, NEW TUNNEL ALIGNMENT; LONGITUDINAL VIEW

4.5.1 Approach

First a governing tunnel element should is chosen. This is one of the deepest elements since the extra deepening at this location is the highest. The two middle elements are both as deep.

The cross-section of the tunnel has a horizontal symmetry around the escape tube. This divides the cross-section in two similar parts. However, the thickness of the concrete in the roof and floor is variable. This results in complex hand calculations. To be able to make calculations this cross-section is modelled in MatrixFrame software. For this the dimensions mentioned in section 3.2.1 are used.

This model than is validated using the report with the starting points regarding the Wijkertunnel design (TEC, 1993), this is all discussed in section 4.5.2. The validated model is then put under an increasing load due to an increasing depth in section 4.5.3. Possible measures are taken if these loads are higher than resistance of the elements, this is discussed in the final section. For all this the calculation is also shown in appendix H.

4.5.2 Tunnel cross-section

MatrixFrame model

As mentioned, the cross-section is based on the design report. A few aspects need further consideration.

Some parts of the element are very thick and won't rotate, for these the stiffness is set at unlimited (node 2,8 and 10 in Figure 82, Original design Wijker tunnel). These are the T-shape connections in on the left and lower right. The thickness of the other parts is equal to the original design.

The soil is represented by a spring with $K = 3\ 000\ [kN/m]$, in the design also a stiffness of $K_{max} = 30000$ is also considered. However, the impact of this is minimal on the cross-section.

The left part of the cross-section is simulated with a horizontal support at node K3 and K4, also the rotation is fixed.

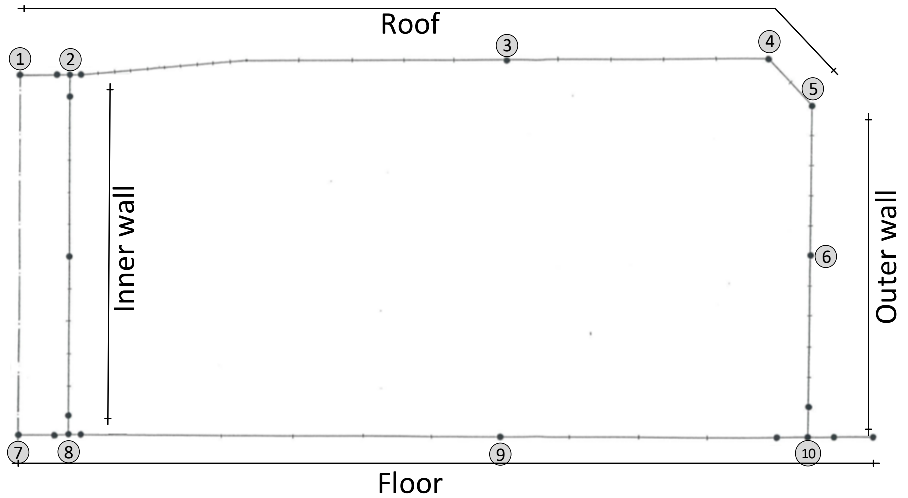


FIGURE 82, ORIGINAL DESIGN WIJKER TUNNEL

Validation

To validate the element the loads as described in section **2.7.2** are applied. This results in a moment-line. These moments can be compared to the line from the original design (TEC, 1993) for each of the nodes. The absolute result is shown in **Table 22, Cross-sectional validation**.

Node	Location	Original [kNm]	MatrixFrame [kNm]	Difference [%]
1	Roof middle	5154	5322	3.3%
2	Roof inner wall	5240	5452	4.0%
3	Roof span	1975	1941	1.7%
4	Roof corner	1147	1083	5.6%
5	Roof corner	1625	1556	4.2%
6	Outer wall span	743	666	10.4%
7	Floor middle	4195	4248	1.3%
8	Floor inner wall	4405	4441	0.8%
9	Floor span	2235	2087	6.6%
10	Floor corner	2320	2356	1.6%

TABLE 22, CROSS-SECTIONAL VALIDATION

From this comparison follows that the model is a good representative of the real tunnel. The only location having a significant (bigger than 10%) difference is in the inner wall. However, this is mainly because the moment here is very low, the absolute value of the difference is still small.

4.5.3 Impact of lowering the governing element

In this section the impact of the increased water pressure is evaluated. Below the moments and shear forces due to the design loads according to load combination A as defined in section 3.3.1 are shown.

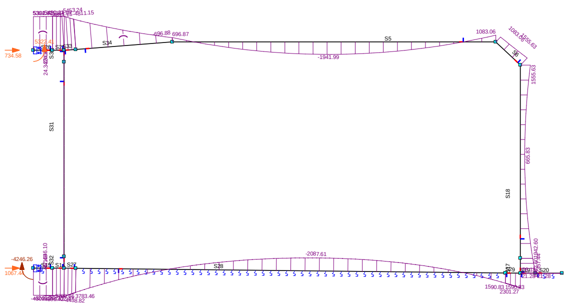


FIGURE 83, MOMENT LOAD COMB. A

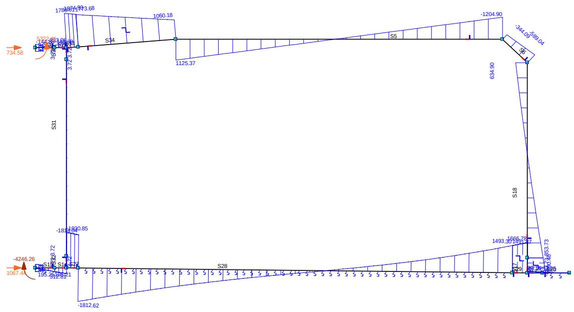


FIGURE 84, SHEAR LOAD COMB. A

The only increase in loads due to the lowering is the increase in variable water pressure as defined in section 3.3. A decrease in loads can be achieved by a decrease in soil cover thickness. For both loads the moment changes linearly to the amount of load applied. In appendix K the change for both 1-meter water pressure increase, and 1-meter soil thickness decrease is shown. Table 23 shows a summary of the values regarding moments, the water column increases the absolute value, the soil cover decreases the absolute value.

Node	Location	MatrixFrame [kNm]	Water increase [kNm/m]	Cover decrease [kNm/m]
1	Roof middle	5322	211	213
2	Roof inner wall	5452	216	218
3	Roof span	1941	81.5	82.2
4	Roof corner	1083	56.3	43.2
5	Roof corner	1556	73.5	68.9
6	Outer wall span	666	23.0	36.9
7	Floor middle	4248	170	169
8	Floor inner wall	4441	178	177
9	Floor span	2087	75.0	75.0
10	Floor corner	2356	75.0	57.0

TABLE 23, LOAD CHANGE DUE TO WATER AND SOIL

4.5.4 Measures against increased load

This section discussed the measures needed to still have an enough unity check. The first category is lowering the loads on the tunnels, the second is increasing the design strength, for the latter two methods are available.

Lower design loads

As mentioned in section 3.2 a surplus of soil cover is present. The thickness of this cover can be reduced such that it compensated for the increase in water level. Considering the minimal soil cover thicknesses, a reduction of 1.36 meter can be applied.

Without an increase in total load in any node the water level can be raised by 1.03 meters. The outside bottom corner is governing in this case. Note that also the loads are based on the most extreme unfavourable tunnel location. Each meter moved away from this location the water column on top decreases by 7.5 $[cm]$.

Increase design strength

The original design for the Wijkertunnel is based on a simple beam-model and the governing failure mode is the cracking width. Based on oral communications at RHDHV it is assumed more strength could be find if a non-linear calculation is made. The exact percentage in extra strength is unknown but cases up to 70% are known, a more realistic value would be 20%-25%.

Also, extra strength could be gained by placing extra reinforcement or concrete. This is preferably done inside the tunnel cross-section. Placing material on the outside of the tunnel is only possible if the area is dry (and therefore only being the roof during floatation) and it is applied over the entire width, securing a flat surface such that a ship anchor which fails won't hook onto the tunnel.

Regarding the roof, Figure 85 shows free space is located just below the roof. This space can be used. Regarding the floor, as discussed in section 4.1.1, 18% of the original ballast concrete can stay in place. During floatation this ballast concrete could be replace with structural concrete.
Regarding the inner walls, the increase in loads is negligible. Regarding the outer walls, no space is available on the inside of the tunnel.

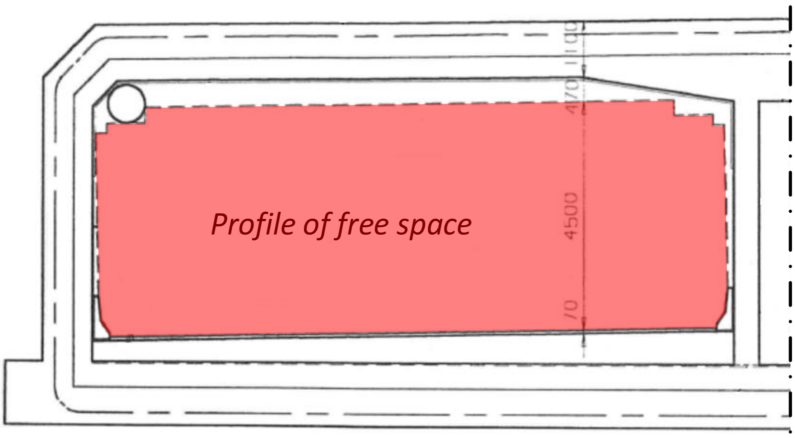


FIGURE 85, WIJKERTUNNEL PROFILE OF FREE SPACE; CROSS-SECTION VIEW

Results

It is concluded that for deepening the Wijkertunnel up to an extra depth of 1.03 no measures other than a reduction in soil cover is needed. For lowering deeper, the percentual increase in moment and shear forces can be calculated for each node, these are shown in Table 24.

Moment increase in cross-section [%]																								
	Deepening [m]																							
Node	1.0		1.5		2.0		2.5		3.0		3.5		4.0		4.5		5.0		5.5		6.0		6.5	
1	0	0	1	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21
2	0	0	1	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21
3	0	0	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	22	23
4	0	1	2	4	5	6	8	9	10	11	13	14	15	17	18	19	21	22	23	24	26	27	28	30
5	0	0	1	2	3	5	6	7	8	9	11	12	13	14	15	16	18	19	20	21	22	24	25	26
6	0	0	0	0	0	0	1	2	3	4	5	5	6	7	8	9	10	11	11	12	13	14	15	16
7	0	0	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22
8	0	0	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22
9	0	0	1	1	2	3	4	5	6	7	8	9	9	10	11	12	13	14	15	16	17	18	18	19
10	0	1	1	2	3	4	5	5	6	7	8	9	9	10	11	12	13	13	14	15	16	17	17	18
Shear increase in cross-section [%]																								
1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
2	0	0	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22
3	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
4	0	0	1	2	3	4	5	6	8	9	10	11	12	13	14	15	16	17	19	20	21	22	23	24
5	0	0	1	3	4	5	7	8	10	11	12	14	15	16	18	19	20	22	23	24	26	27	29	30
6	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
8	0	0	1	2	3	4	5	6	7	8	9	9	10	11	12	13	14	15	16	17	18	19	20	21
9	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
10	0	0	1	2	3	3	4	5	6	7	8	8	9	10	11	12	13	13	14	15	16	17	18	19

TABLE 24, PERCENTUAL CROSS-SECTIONAL LOAD INCREASE

This table also shows the most critical cross-sections. Especially the outside top corner seems to be critical (node 4 and 5). When a high amount of deepening is needed a FEM analysis is required with for example DIANA software. This thesis does not cover a complete 3D-analysis of the Wijkertunnel since it was not the direct objective stated in section 1.3 and does take much work.

If this method does not provide the required strength the increase in the first 5 nodes can be carried by installing more reinforcement. Node 4 and 5 require moving the current ventilation equipment or replacement with more modern (and possibly smaller) equipment.

5. Risk measures evaluation

In this chapter the risks measures defined in section 2.9 are evaluated. This is only possible for the measures which have a clear quantitative measure in for example loads increase. This is done to have a clearer view on the impact of these measures. The method of evaluation is mainly based on the increase in costs for the materials. The measures to be evaluated are shown in Table 25.

Risk number	Risk description	Measurement
1	Unexpected floatation	20% increased uplift factor
11	Element won't float	20% reduction in lifting force
5	Bulkhead total failure	10% increased bulkhead loads
2,3	Prestressing failure	20% increased minimal prestressing

TABLE 25, RISK MEASURE EVALUATION OVERVIEW

The cost of materials in based on information gathered at Royal HaskoningDHV and information from the Hydraulic Structure lecture notes (Molenaar & Voorendt, 2016). The following prices are including constructing factor et cetera. For concrete a standard reinforced concrete are used.

$$Reinforcing\ steel: 5\ €/kg$$

$$Prestressing\ steel: 9\ €/kg$$

$$Concrete: 350\ €/m^3$$

5.1 Increased uplift factor

The factor of safety against unexpected uplift is only important during load case *V-V*, this situation discusses the tunnel element completely prepared for re-floating and even the soil cover is removed. The lower factor of safety gives freedom in two design variables. To minimal amount of ballast concrete staying in place and the volume of the ballast tanks.

Ballast concrete

First a lower minimal amount of concrete should stay in place. The 20% higher factor of safety was responsible for a 3.1 percent-point increase in minimal ballast staying in place.

Ballast tanks volume

Secondly, since the design was not designed at the minimal amount of concrete staying in place, but at the maximum. The volume in ballast tanks can be adapted (since this maximum was not yet reached in load case *V-VI* and *V-VII*). Compared to a standard FoS, the increased FoS resulted in an increase in ballast tanks volume of 4.5%

These tanks have an impact for two reasons.

- The volume of ballast water to be pumped is increased, this results in more time with closed waterways during lifting operations.
- The ballast tanks were important in the prestressing design. The less safe FoS results in a water level reduction in the tanks of 14 $[cm]$, which equals a load of about $q_{reduction} \approx 0.14 * W_{tanks} * 2 * \rho_w \approx 28\ [kN/m]$. Making use of the calculations in chapter 4.1.5, a moment reduction in the most disadvantageous load case is calculated for both joints. This is compared to the values in the current prestressing design.

$$M_{outer;FoS} = 1911\ [kNm] \rightarrow 2.7\%\ reduction$$
$$M_{inner;FoS} = 379\ [kNm] \rightarrow 0.35\%\ reduction$$

5.2 Reduction in lifting force

For the calculation of the adhesion the effective lifting force was reduced with 20%. This reduction has effect on two aspects, the breakout time as discussed in section 4.3.1 and the loads on the hoisting eyes as discussed in section 4.1.3.

Breakout time

Figure 86 shows the breakout time using the final input parameters as discussed in section 4.3.1, also a trendline is plotted.

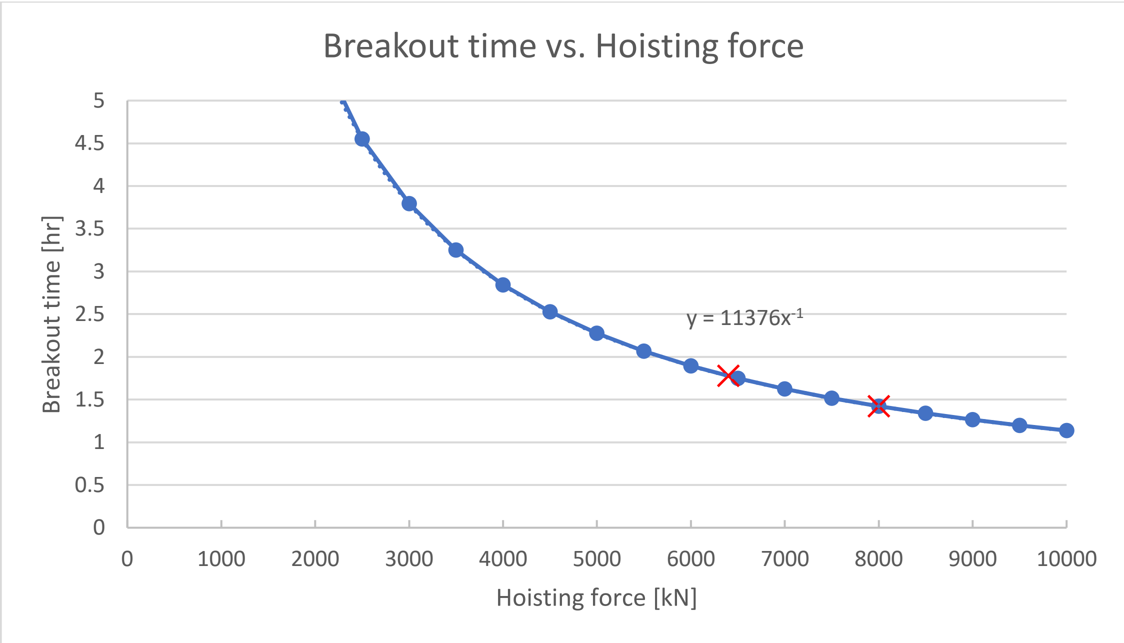


FIGURE 86, BREAKOUT TIME VS HOISTING FORCE

If the same equipment was used but the tunnel behaves as expected without measure, the breakout time would further reduce to 1.44 hour. It shows the measure is responsible for only an increase in time of about 20 minutes.

It might also be an option to reduce another aspect of the design. Without the measure and resulting in the same breakout time, the jetting width is 7.6 meters. The measure is responsible for only a 5% increase in jetting 'depth'.

Hoisting eyes

Without applying the reduction factor the total lifting force would reduce from $12\,000\ [kN]$ to $10\,400\ [kN]$. The main costs in the anchor constructing is the amount of diving needed; c.q. the number of anchors required per hoisting eye and the number of hoisting eyes.

- The number of anchors is currently 2x2 per wire, this could be reduced to 2x1 not to 2x2. The reduction to 2x1 was however already available for the $12\,000\ [kN]$ load but was not preferably due to the highly asymmetric nature.
- The amount of hoisting wires could be reduced from 8 to 7, without exceeding the previous force in the wires, $\frac{10400}{7} = 1486\ [kN]$. This shows the safety measure resulted in an increase of 14% in anchors. Note that the asymmetry of the hoisting wires is less important since these are already located far away from each other.

5.3 Increased bulkhead loads

Like as in section 4.1.2 the dimensions can be calculated depending on the depth. This is done using the same method. The exact dimensions for each depth are not given but Figure 87 shows the material usage with respect to the amount deepened.

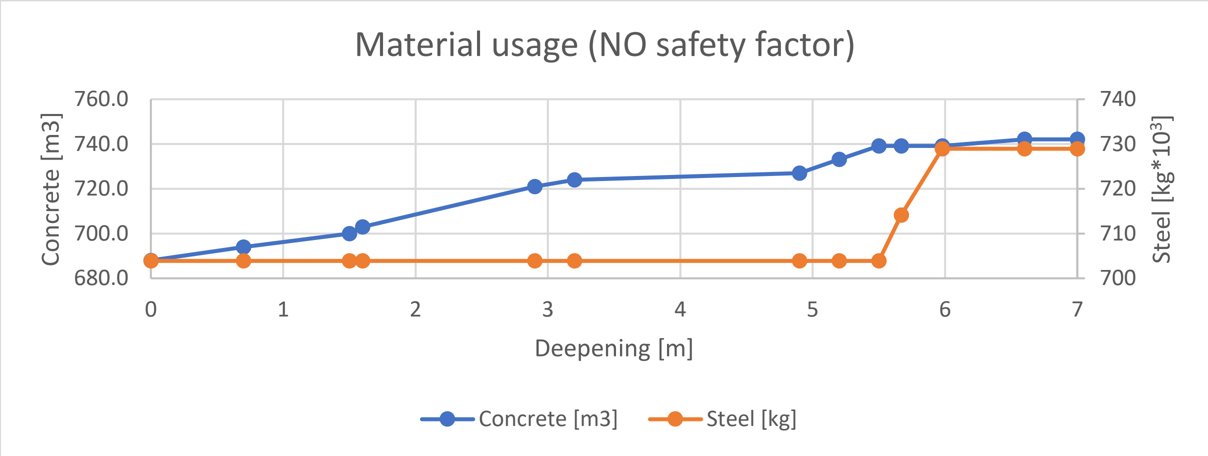


FIGURE 87, MATERIAL USAGE WITHOUT SAFETY FACTORS

These values are then compared to the values with safety measure and according to the costs mentioned above translated to materials prices. This is shown in Figure 88 for both the concrete, steel and total costs.

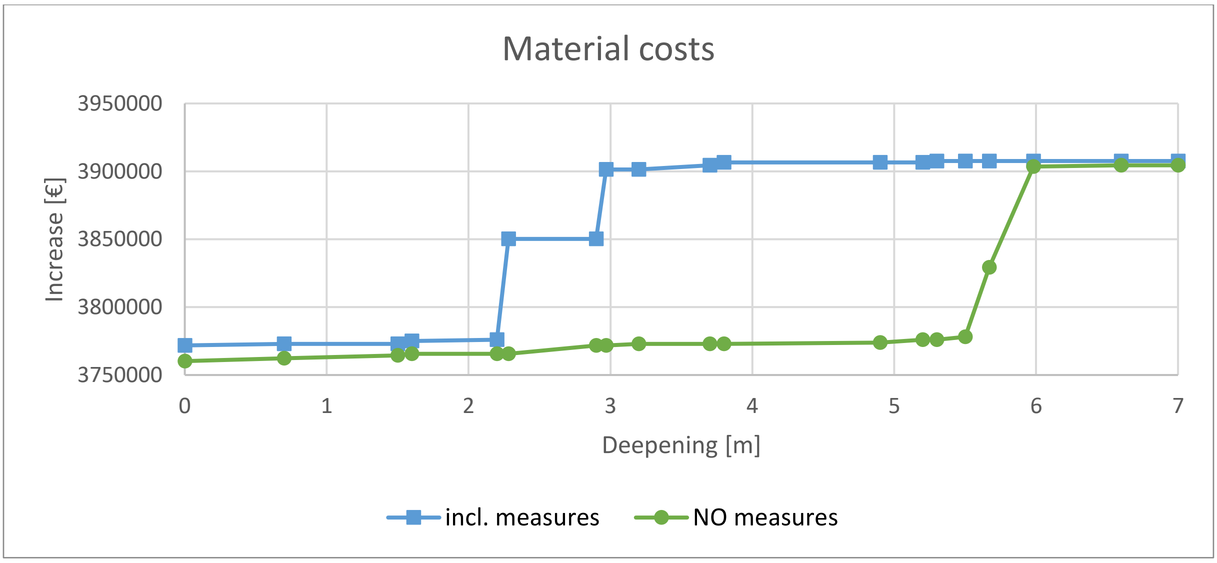


FIGURE 88, MATERIAL COSTS COMPARISON

5.4 Increased minimal prestressing

The prestressing requirement was set at $220\ [kN/m^2]$, without the risk measures this would be set at $200\ [kN/m^2]$. The same optimisation as discussed in chapter 5.3 is done. The optimal solution is when the middle of the ballast tanks is placed $1.00\ [m]$ to the outside from the segment joint. The hoisting eyes are located $16.0\ [m]$ from the outer end.

Again, the clear winner regarding to costs is variant 2. The total wire length is $448\ [m]$. Using the same construction factor regarding costs the result is shown in Table 26. The increase in costs due to the safety measure is $\frac{2.7}{46} = 5.9\%$

	Length [m]	Weight [kg]	Cost [x10^3 €]
Incl. safety measure	424	3244	**48.7**
NO safety measure	400	3061	**46.0**
Difference	24	183	**2.7**

TABLE 26, PRESTRESSING SAFETY MEASURE COST

5.5 Conclusion

For each of the risk measures the impact on the design is discussed and decided if this is indeed a valid extra risk measure.

Uplift factor

- Regarding the minimal ballast concrete requirement, the increase in factor of safety against uplift has no impact on the design.
- Regarding the ballast tank volume, extra pumping is needed (only 4.5% compared to the 20% safety factor increase). The reduction in moment for the prestressing design are very low (less than 3% compared to the 20% safety factor increase).
- In the risk analysis the costs of an unexpected uplift were estimated up to $250\ [* 10^3 €]$, and extra maintenance during the entire lifetime.

Based on these considerations it is concluded that this measurement is <u>good</u> for lowering the risk against unexpected floatation.

Effective lifting force

- Regarding break-out time, the extra duration is only 20 minutes. This time can also be exchanged in a reduction in jetting depth of 5%. Another important notion is that this calculation is based on a very rough estimation regarding soil permeability. The range of actual permeability is higher than the impact of this measure
- Regarding hoisting eyes, the safety measure resulted in an increase of 14% in anchor usage. This is a significant amount especially since these need to be installed by divers.
- In the risk analysis the costs of no floatation were estimated up to $2\ 500\ [* 10^3 €]$

Based on these considerations it can be concluded that this measurement is <u>good</u> for lowering the risk that no break-out occurs.

Bulkhead loads

- Regarding the increase in cost only the material cost is relevant. Figure 88 shows a maximum absolute value in increase of $134\ [* 10^3 €]$, this is about 4%. This is a representative value since the expect maximum deepening is probably somewhere between the 3 and 5 meters.
- In the risk analysis the costs of a partial failure were estimated up to $250\ [* 10^3 €]$, and the costs of a complete failure even an order bigger.

Based on these considerations it can be concluded that this measure is <u>mostly good</u> for lowering the risk for partial and complete failure of the bulkheads since it does induce significantly more costs and manhours of construction.

Prestressing

- Regarding the cost increase the 20% safety measure only induces a cost increase of 5.9%

- In the risk analysis the costs of a total failure of the prestressing is projected to be over 2.5 million euros.
- The increase in stresses also limits the deformations. This has a direct positive effect on the risk identified regarding failure of the W9Ui failure.

Based on these considerations it can be concluded that this measure is <u>good</u> for lowering the risk of prestressing failure and the risk of failure of the W9Ui profiles.

6. Applicability to other and future immersed tunnels

This chapter is aimed at increasing the relevancy of this thesis. This is done by widening the scope by not only discussing a single case study but also making a comparison between this case study and other existing immersed tunnels. Also, a short section about the impacts of local surroundings on the tunnel is given.

The second part of this chapter is about future tunnels. Chapter showed that sometime complex solutions are needed and these probably could be prevented if in future tunnel design a possibly re-floating situation is incorporated.

6.1 Other existing tunnels

In this section the other existing tunnels are compared to the case study tunnel, the Wijker tunnel. The general method showed so far to be viable for the Wijkertunnel. A square, concrete, segmental tunnel, made up of six immersed elements with the dimensions of roughly 100x30x8 meters (l x b x h).

The final question then is, what differs in other immersed tunnel compared to the Wijkertunnel, and could the general method still be applied to these tunnels. A complete overview comparing each aspect to a change in tunnel design is in Table 27. Each column shows a change in a specific design aspect. The rows represent each of the steps in the general method. A marked box means the change in design aspect has an influence on the specific steps in the method.

An actual example of another immersed tunnel possibly being in need of lowering is the Liefkenshoektunnel. This tunnel was already discussed during the case selection in chapter 3. It is also segmental, has eight instead of six elements and is a bit longer elements. This tunnel is located in de Westerscheld, just before the tunnel of Antwerp. The average local water depth is actually less than the Wijkertunnel while the port is bigger compared to Amsterdam. The current solution for big vessels is a detour through the new world biggest dock, the Deurganckdok or using the high tides. However lowering the Liefkenshoektunnel could be a future solution if the capacity of this dock is exceeded.

Construction aspects	**Difference compared to the Wijker tunnel**				
	Monolithic element	Steel shell tunnels	Circular tunnels	More/less elements	Wider tunnels
2.3 Preparation of the element					
Ballast tanks and concrete					
Installing bulkheads					
Reconnecting hoisting eyes					
Removal of shear keys & omega seal					
Prestressing					
Other equipment					
2.4 Removing the first section					
Removing soil					
Cut location & length					
Cutting method & shape					
Settlements					
2.5 Re-floating & Transport of main element					
Access to element					
Buoyancy					
Soil adhesion					
Transport of the element					

2.6 Preparing, adapting & reconfiguration	
New alignment	
Preparing element	
New Foundation	
Adapted land structure	
Adapted closure joint	

TABLE 27, CHANGES PER TUNNEL TYPE

Monolithic tunnels

The Wijker tunnel consists of six element which each are build up from four segments. These segments were required to compensate for possible settlement differences and lower the crack width and therefore lower the leakage through the concrete. The other option would be monolithic tunnels. Such tunnels consist of an element with only a single segment up to 100 meters long. To ensure water tightness an extra coating or layer is applied around the tunnel.

The main difference regarding monolithic tunnels is that no segment joints are present compared to the Wijkertunnel. This makes the design much easier since no extra prestressing is required. The prestressing or reinforcing steel is just as designed and no prestressing wires were cut after placing.

An aspect which is very important in the design of monolithic tunnels is the placement of the foundation. Since these tunnel segments are long and very stiff it is difficult to adapt the tunnel shape to the settlements. Therefore, the accuracy in placing the foundation is extra important, placing a new foundation does need extra care.

Regarding the prestressing the Wijkertunnel is a more complex tunnel. With the general method being viable for this tunnel it is expected to be viable for monolithic tunnels. Regarding the foundation it is just an important notion since for placing this very accurate foundation the standard method used for construction a monolithic tunnel foundation can be used

Steel shell tunnels

The Wijker tunnel cross-section is constructed from concrete. However, mostly outside of Europa, other types of immersed tunnels are used. These are for example single of double steel shell type tunnels. An example of destroying a steel tunnel was the removal of the Baytown tunnel mentioned in section 1.2.2.

Two differences are identified regarding the steel shell tunnels. The first is that cutting a steel shell tunnel is easier. The concrete layer is removed from the inside and shape charges are used to make the final cut; this method is already proven to be viable during the removal of the Baytown tunnel. The other difference is the construction of the anchors to the tunnel. Compared to the concrete tunnel the anchor should somehow be welded to the tunnel, this welding occurs while the tunnel is immersed.

Regarding the cutting a concrete tunnel is more complex. The immersed welding of the anchor is difficult but not ground-breaking, since for both options divers are needed there is not much of a difference. With the general method being valid for this concrete tunnel it is expected to be viable for steel shell tunnels.

Tunnel shape

The Wijker tunnel is a box shape, however some tunnels have a circular shape. This circular shape is expected to have a positive impact on the adhesion and breakout time since it is easier for the water to 'creep' beneath the element.

An aspect which does not become more complex, but different, is the removal of the top soil layer. The backhoe dredger is only capable the removing horizontal layers of soil, so a grab dredger is needed for removing soil from a circular shape.

Regarding the closure joint, new solutions are available for especially steel circular tunnels. A common closure joint is a tremie joint.

With these three changes the Wijkertunnel is the more disadvantageous tunnel. With the general method being viable it is expected to be viable for circular tunnels.

Tunnels with different number of elements

The Wijker tunnel consists of six tunnel elements. However, the number of elements used for an immersed tunnel differs from a single element to tens of elements. A disadvantage of the general method is that each of the elements is removed in series since it starts from the opening joint, parallel removal is not available.

For tunnels with much elements it is important to notice it is not safe to start with immersing reconfigured elements while others are still getting re-floated. Therefore, much area is needed to store and adapt all these elements.

Another aspect in the general method was the rotational capacity of the joints. This capacity was responsible for the maximum amount of lowering available. With only a single element no lowering is possible without adapting the approach structures, tunnels with 2-4 elements have a very limited deepening due to the low amount of immersion joints.

Most of the immersed tunnels have six to eight elements. However, for tunnels which have significant more or less tunnel elements the general method becomes less viable or even impossible.

Wider tunnels

The Wijker tunnel consists of two tubes with double lanes and an emergency tube. The total width of the tunnel is about $32\ [m]$. Other tunnels might be wider, such as the 2nd Benelux tunnel with a width of $45\ [m]$.

The width of the tunnels firstly has an impact on the adhesion calculation. In this calculation the breakout time is depended on the width to the third order. Therefore, an increase in width of 50% increases the breakout time with 338%.

Taking this into account the general method might not be viable for tunnels much wider than the Wijkertunnel. An important notion however is that this adhesion calculation is based on big assumptions and that a test on the actual soils is expected to have a positive impact.

Local conditions

Some aspects of the general method are not depended on the type of size of the tunnel but on the local surroundings. These local conditions could impose extra requirements on the method of re-floating

Tidal area	The Wijkertunnel is sheltered from the tide by the Ijmuiden locks. This is however not the case for every immersed tunnel. In a tidal area the design should be based on the governing water level, this is probably the highest level (and therefore highest loads). Regarding transport or lifting a minimum water depth is needed. If this minimum is exceeded

during low tide a time window in which the element could be transported is calculated. During this time window the adhesion should be broken and extra research in this adhesion is needed to guarantee the element starts floating in this window.

Highly changing water density In tidal area's an extra problem could be the change between fresh and salt water. This change is about a 2.5% in water density. If this is the case extra care should go to defining the freeboard required and the safety against unexpected uplift.

High currents In areas with high currents the factor of safety against uplift might need to be increases since these currents induce a shear stress on top of the tunnels. Also, the design shear loads on the wires should be increased.

Sediment transport Some areas, mostly tidal areas, are subjected to high intensities of sediment transport. If this is the case extra care should go to removing the top soil layer and even an extra dredging phase should be implemented just before lifting.

6.2 Future tunnels

Some aspects in the general method needed a creative solution which for example required a high number of divers or resulted in extra risks. This section discusses possible adaptions in the future immersed tunnels design such that, in case this is needed, re-floatation is easier. In important notion to make is that in it may be assumed that in 50-years-time the technique is evolved forward. It might have evolved such that for example diving work might be easier due to the use of small submarines.

Anchor protection

In section 4.1.3 a new anchor design is proposed, this design make use of divers installing many anchors. The most critical aspect of this design was concrete cone failure due to tension. This failure mode could be solved by applying supplementary reinforcement; however, the element being immersed makes this nearly impossible.

The anchor in the original design however could be installed using supplementary reinforcement since the element is in a drydock. Over the lifetime of the tunnel these initial anchors deteriorated and couldn't be re-used.

A solution to this problem would be to protect these anchors from eroding, or even keep the hoisting eyes in place. These anchor then could be dimensioned such that higher loads can be applied then during the initial immersion. Such an anchor protecting should be designed such that no sharp edges could hook an accidental ship anchor to the tunnel.

The challenge is how to keep these anchors free from any eroding or leaking. If this occurs, it did not only cost extra money to install the stronger anchors but the advantage of this is lost. It is assumed that while lowering the risk of failure the costs would increase such that installing the anchor when re-floating is needed is cheaper.

Easy ballast removal

The current ballast concrete is just casted in-situ. Removing this is very intensive work in a small enclosed area. A solution to this would be to design the ballast concrete not as a single continuous slab. For example, prefab concrete slabs could be used and stacked on top of each other.

In this new ballast concrete design, it is important to consider that the ballast layer is the foundation of the asphalt layer and that any horizontal movement of this prefab concrete should be restricted.

Replaceable bulkheads

One of the risks identified in section 2.8 was a partial or complete failure of the bulkheads. This partial failure was due to leakage between the element and the bulkhead. A solution to this would be to not completely remove the connection between the initial bulkhead and the element. This connection could be covered by a casing, which then is covered by ballast concrete.

Access shaft

In the original design the access shaft is location on the roof of the element. After finishing the tunnel this connection is removed and filled with gravel and a layer of concrete. This location was not suitable for a new access shaft.

At first sight it might seem possible to keep this access point intact like the anchor proposal. However, the impact of leaking in this location is enormous. The leakage is a direct opening from the outside of the tunnel to the inside. Also repairing this leak is very complex. This adaption should not be considered for future tunnels.

7. Conclusions & Recommendations

7.1 Conclusions

After executing the first methodological step, a method is available for temporarily, effectively re-floating immersed tunnels. This method is divided in four phases, all the important aspects during these phases are discussed. For each of thesis aspects one or more suitable solutions are available. It is concluded that reversing the normal construction for immersed tunnels is a good basis for developing this new method. However, some aspects deviate significantly from this initial design. These are added to Table 28, an overview of the most important aspects for this new method.

This method does however come with new risks which do not occur during the standard construction of immersed tunnels. It is concluded that these risks can be divided in three categories; accepted risks, risks with preventive measures or risks with corrective measures. With the knowledge during this stage of the research in re-using immersed tunnels it is concluded that the risks are lowered to an acceptable consequence or probability of failure.

From the second methodological step it is concluded that this method is indeed relevant for several tunnels and that the Wijkertunnel in the North Sea Canal is a good case study. After analysis of this case enough knowledge was available to apply the general method on this case.

After applying this new method on the case study, it showed that for each aspect indeed at least one solution is viable. However, some solutions are more insecure than other. The resulting maximum amount of deepening available for the Wijkertunnel is 6.62 $[m]$, note that the increase in waterway depth is significantly lower.

After the fourth step it is concluded that for the risk measures which were quantitively defined the impact is relatively small and these risks measures are indeed a good option in lowering the consequence or probability of failure. However not all measures were evaluated so it is difficult to predict the entire impact of these measures.

The final step is comparing the Wijker tunnel to other existing tunnels. It is concluded that this method is viable for nearly all other immersed tunnel types. Only tunnel with a high (10+) or a very low (1-3) number of tunnel elements are impossible. Regarding future tunnel it is concluded that making use of prefab ballast concrete would increase the ease of removal of this concrete, using Replaceble bulkheads might reduce the risk of leaking.

The original thesis objective was stated in the first chapter as:

Immersed tunnels, which do not yet reach their design lifetime, can be obstacle for waterways to be dredged deeper and thereby hinder the passage of vessels and thereby hinder the development of ports and the inland economics.

It is concluded that the general method described in this thesis is a valid solution to this problem for most tunnels, with this method new risks arise, and some design aspects need special attention.

Important aspects

The eight most important design aspects or increased risks are mentioned below in Table 28. These aspects require special attention when applying this method to an immersed tunnel

Aspect	Importance
Submerged installed anchors for hoisting wires	These anchors are now installed submerged but with an increased load. Take care the required capacity is reached.
External post-tensioned prestressing	This prestressing is critical for the entire horizontal balance throughout the entire process. Take care the prestressing force is reached and preserved throughout the entire process. Also, other variants then applied at the Wijkertunnel might be better solutions for another tunnel
Unfavourable bulkhead location	These bulkheads are now installed on a used surface instead of a new tunnel. Testing of these bulkheads is limited so make sure these are properly designed to ensure buoyancy throughout the process.
Soil adhesion or passive suction	The amount of adhesion is not exactly known, the type of foundation might also differ for other tunnels. Wrong calculations and assumptions might lead to excessive waterway blocking or even unexpected floatation. Also, other variants then applied at the Wijkertunnel might be better solutions for another tunnel
New tunnel alignment	Regarding the new alignment several aspects can be governing. For the Wijkertunnel the traffic requirements where governing (above structural requirements). The GINA-seal elongation is assumed not to be limited but with extreme elongations extra failure modes could be introduced. Make sure to check the governing aspect for another tunnel.
Soil (cover) and the opening joint removal	The soil cover is placed for safety against uplift but also against falling anchor or other equipment. Make sure not to damage the element while removing the soil cover and the opening joint.
Increased water pressure	Due to the increase in depth the water pressures have increased. For the Wijkertunnel this increase could probably be compensated since it was designed decades ago using simple software. For tunnels designed more on the limit this might not be the case and it is important to take care the tunnel has the capacity to bear these loads.
Risk analysis	The current risk analysis for the method is limited to the current stage of the design. In a further design stage extra risks might occur. Also take care to evaluate the impact of all other risks which were not quantitively defined in this thesis.

TABLE 28, CONCLUDING IMPORTANT DESIGN ASPECTS

7.2 Recommendations

In this section the recommendations following from this thesis are shown. These recommendations are based on information discovered during the thesis or assumptions in need of more research.

Soil adhesion

As mentioned multiple times in section 2.5.3 and 4.3.1 the current adhesion calculation is based on the Darcy equation. Based on the current literature this is a valid assumption, but this method does not cover the entire concept.

For a next design stage more research in lifting large immersed object more knowledge is needed regarding the soil adhesion. Aspect in which research is needed are:

- The break-out distance is set at 2 $[mm]$. It is assumed that the water flow is constant before this distance, and unlimited after this distance. However, an exponential development of this flow speed is expected.
- For now, the calculation is made for 2D (in longitudinal direction). However due to a 3D effect the break-out could occur sooner.
- It is assumed the duration which the element is submerged has no impact. However, consolidation is expected to influence the soil permeability. However, this effect is unknown regarding submerged objects.

Non-linear structural analysis

As mentioned, an increase in cross-sectional loads is expected due to the lowering. The original Wijkertunnel design was simple and a non-linear analysis might show an increase in structural capacity. This non-linear cross-sectional structural analysis is needed not only for the Wijkertunnel but each case for which this re-floating method is applied.

Concrete strength

The concrete in this re-floating method is assumed to be similar to the concrete during initial placement. Depending on the type of concrete the strength might have increase. Current codes describe possible increases in concrete classes over time but also an increase in safety factors. Especially for submerged concrete extra research is needed to show which values could be applied in this method.

Re-use of post tensioned grouted prestressing

Chapter 5 discussed the use of prestressing wires. This both an expensive and complex part of the re-floating method. For now, no possibility of re-using the previously installed prestressing could be found. Extra research might uncover possibilities of re-using, this research is not only relevant for the re-floating method but also for other post-tensioned structures.

Approach structures

For now, it was assumed that is was possible to raise the approach structures a few meters. Especially for the covered part of the approach structure research is needed if this is possible.

More detailed risk-analysis

The current risk analysis made in chapter 2 defines some measures on a quantitively and some measures on a qualitative scale. Extra research is needed resulting in a quantitively impact of each measure. This would result in a more detailed evaluated of the risks measures.

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Appendices

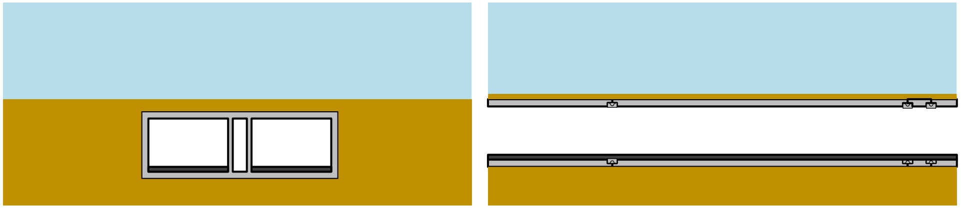
A. Re-floating sequence

Below the complete sequence for temporarily re-floating of immersed tunnels is shown. This is done for the preparation, re-floating, replacing and immersing process. Including images for both a cross-sectional and a longitudinal view.

A Preparation the element

A.1 Install lighting and ventilation for decommissioning and removal works

A.2 Remove backbone components such as M&E equipment, asphalt surfacing, cladding.

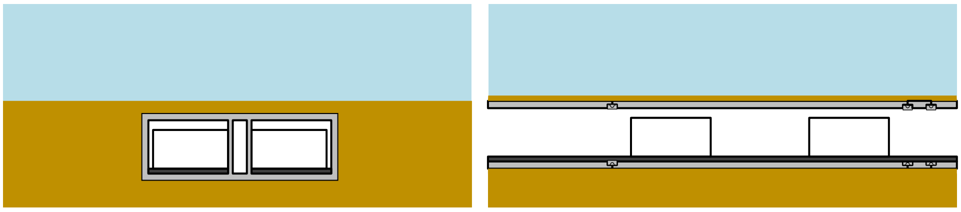


A.3 Install ballast water tanks

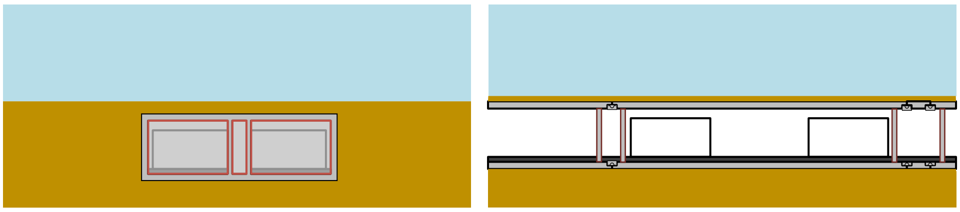
A.3.1 Create a water leakage collection behind location of the future bulkheads

A.3.2 Position the pumps to pump leakage water into ballast tanks

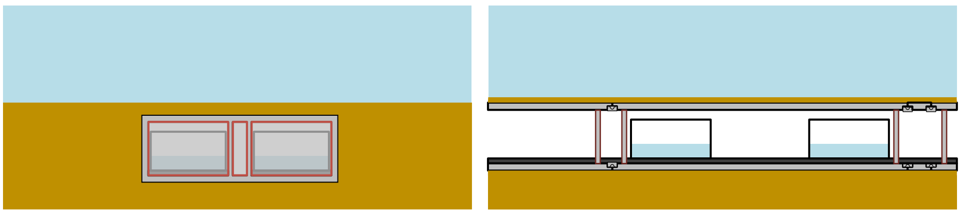
A.4 Install power packs / batteries for power supply in the element during the removal operations



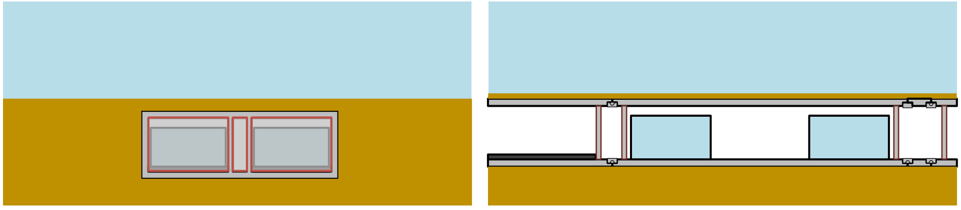
A.5 Install bulkheads; on one end the bulkhead may be accommodated with a docking station to connect an access shaft (after cutting)



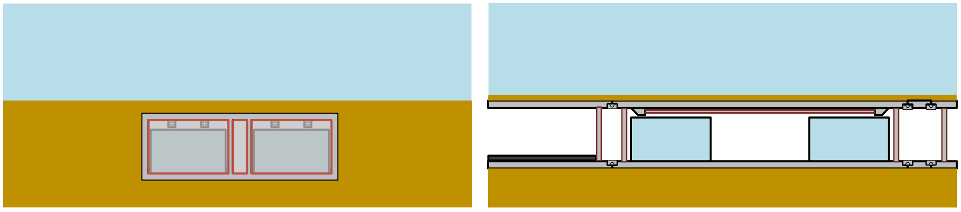
A.6 Pump in ballast water to a base level resulting from the buoyancy calculations. The ballast system must be controlled remotely by pneumatic or hydraulic valves.



A.7 Remove ballast concrete; depending on the design step 6&7 are carried out in multiple phases.



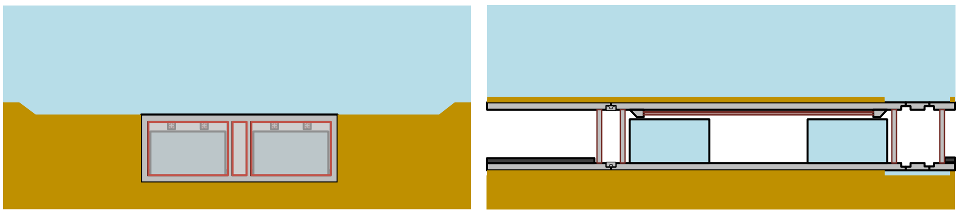
A.8 Install external prestressing equipment (if needed, discussed further on)
A.9 Apply prestressing force
A.10 Install cameras to monitor
A.10.1 any leakage through bulkheads
A.10.2 water levels in the ballast tanks (including pressure sensors)
A.10.3 essential valves for the operation of the ballast water system



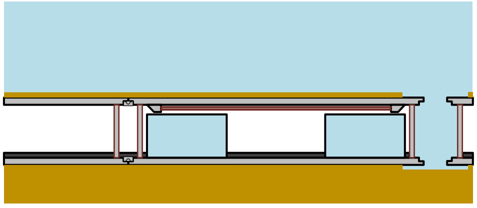
A.11 Remove the Omega-seals
A.12 Remove the shear keys if needed (based on which elements are lifted together)
A.13 Prepare element for cutting operations
A.13.1 Disconnect element from power supply from the banks
A.13.2 Connect to power packs
A.13.3 Close bulkhead doors
A.14 Remove the rock protection and back fill of the tunnel in the areas where the cuts are made

B Removing a first section

B.1 Removal of all rock fill and ground fill on top and against the section.

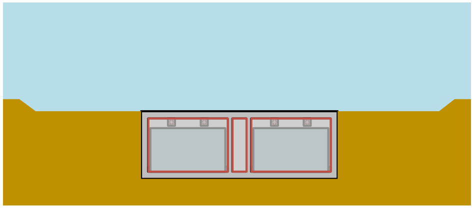


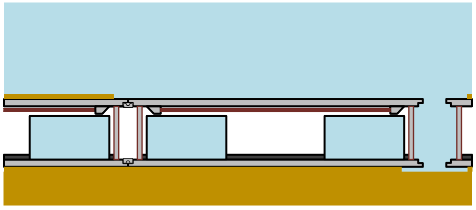
B.2 Install vertical guidance structures at the location of the cuts if needed
B.3 Disconnect the joint
B.3.1 Make a first cut.
B.3.2 Make a second cut.
B.4 Hoist the concrete block out of the water



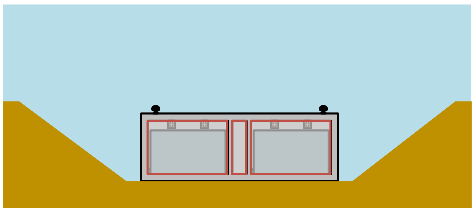
C Re-floating & Transport of the main elements

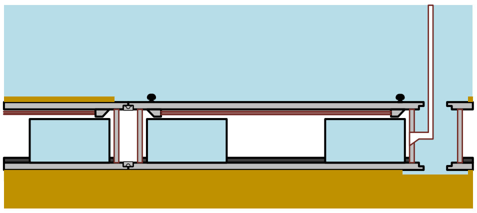
C.1 The ballast tank levels have been filled to a level that the tunnel element will remain on the riverbed after removal of all fill.
C.2 Removal of all rock fill and ground fill on top and against the tunnel element.



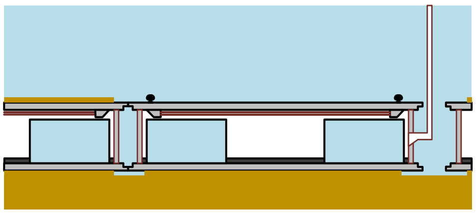


C.3 Mount temporary structures to assist removal operation, such as
- C.3.1 Access tower against the bulkhead
- C.3.2 Survey tower on top of the roof/ on the bulkhead
- C.3.3 survey pitch/roll sensors within the element
- C.3.4 Bollards to accommodate any winches that may be needed
- C.3.5 Lifting lugs if needed
- C.3.6 Guide structures that may ease the removal of the element and make sure that collision between remaining part of the tunnel and the removed section will not happen (reverse catch principle).

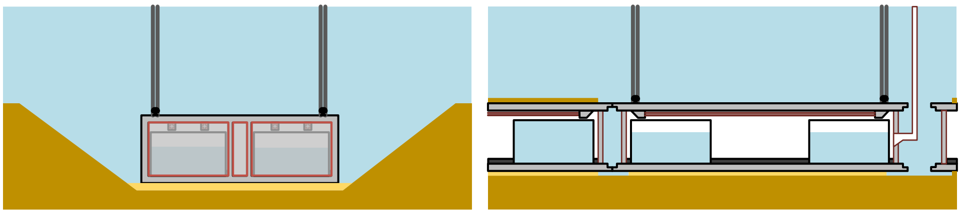




C.4 Airlift the perimeter of the tunnel
C.5 Inundate space between the bulkheads
C.6 Cut away part of the GINA seal

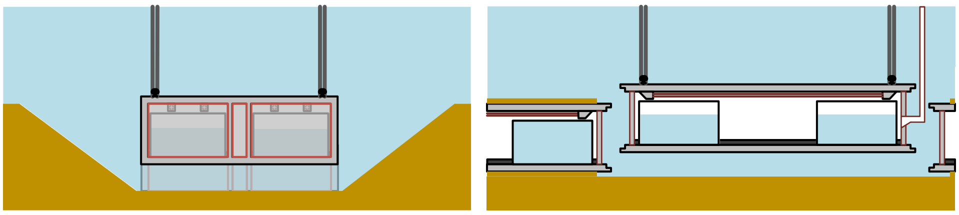


C.7 Float catamaran pontoons above the tunnel element and connect lifting lines.
C.8 Connect any other lines using for winching / warping the element.
C.9 Preload the lifting cables to a level related to a slight overweight percentage.



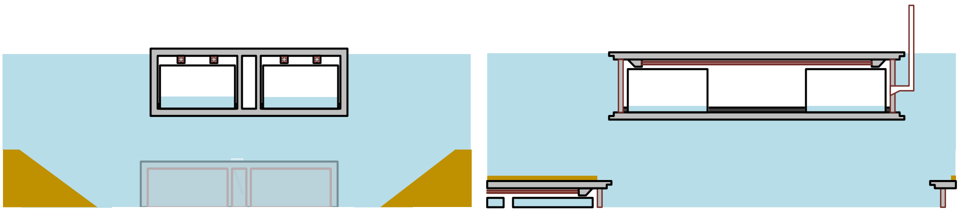
C.10 Destroy the adhesion

C.11 The adhesion will be destroyed, and the element will be carried by the pontoons.



C.12 When the element approaches the water level gradually the ballast water tanks are emptied.

C.13 The elements are capable to float on their own



C.14 Transport to a safe location for impacts of waves but also for personal to work

C.15 Repeat this process for the other elements

D Preparing, adapting & reconfiguration

- D.1 Construct a safe dry dock to work in
  - D.1.1 Replace the GINA-profiles
  - D.1.2 Check and possibly install a new catch-structure
- D.2 Remove the original tunnel foundation
- D.3 Remove the concrete tiles
- D.4 Dredge the waterway until the desired depth
- D.5 Place a new foundation
- D.6 Construct a safe dry dock around the land heads
- D.7 Adapt the land heads, fitting the new tunnel alignment
- D.8 Use the conventional method for immersing the elements
  - D.8.1 Place the new designed closure joint

B. Loads & Load cases

In this appendix the loads as shown in section 2.7.1 are discussed, these are divided in the forces relevant for the horizontal and the vertical balance.

Then all these forces are combined with the construction sequence into different load combinations.

Horizontal balance

Prestressing $F_{prestressing}$	During the initial transport longitudinal prestressing is applied to keep the segments together as one element. This force is than negated by cutting the wires when the element is resting on its foundation. This force might be present when transport after re-floating.
Water pressure F_{water}	Horizontal water pressures push on the sides of the elements; these pressures are equal on both sides and are neglected with regards to stability. The pressures on the secondary end pushes it against the other elements when the water at the primary end is pumped out.
Pulling F_{pull}	When the secondary end is placed next to the previous element the initial connection between the GINA-seals is made by pulling the elements together. Note that the material for this is removed during finishing the element.
Bottom friction $F_{friction}$	When the element is placed on its foundation friction occurs between the element and its foundation. This force is also important when re-applying the prestressing.
Closure joint F_{joint}	The closure joint is placed between the final element. Several designs are available for this joint. The horizontal pressure which was first secured by the water pressure is now secured by the closure joint between the elements.
Elements $F_{element}$	When the element rest against the next elements longitudinal forces can be transferred through the elements.
Soil fill F_{soil}	Since the element is embedded the soil fill next to the element is acting on the element. This is also a transversal force.

Vertical balance

Buoyancy $F_{archimedes}$	The Archimedes force is in upward direction and is equal to the weight of the total amount of water displaced by the element. This force therefore depends on the exact dimensions of the element (e.g. the indent for the bulkheads) and the density of the water in which it is immersed (being fresh or salt water). Note that having the construction partly emerged from the water lowers the amount of water displaced and therefore lowers the upward force.
Self-weight $F_{w,self}$	This total force depends on the density of concrete. While the fluctuations in concrete density are not that big, with these vast volumes of concrete the variation in self-weight is still significant. Note that for re-floating the elements have been weight in before, so the exact weight should be known.
Soil-cover $F_{w,soil}$	The soil cover on the tunnel is for protection against falling anchor, the thickness of this is about 1 meter. Note that the exact thickness is difficult to measure due to sediment transport. Also, the density is an unknown factor.
Bulkhead $F_{w,bulkhead}$	When the bulkheads are placed either from concrete or steal an extra load is applied at the end of the elements
Ballast tanks $F_{w,tanks}$	During the process of re-floating and immersing ballast tank are placed inside the elements. These tanks are filled with water and can be relatively easy to adapt.
Ballast concrete $F_{w,concrete}$	After placing the elements in their 'final' position the ballast tanks are gradually replaced with ballast concrete on the base of the elements. This weight is more permanent than that of the ballast tanks. The spread in density of ballast concrete is lowered compared to structural concrete. Note that the unit weight of the ballast concrete is in the order of 2.5 times higher than of water. Thus, the volume of the ballast tanks is substantially bigger than the volume of ballast concrete. Also note that if possible, not all the ballast concrete needs to be removed.
Equipment $F_{w;\,equipment}$	During the process M&E equipment is placed. Many vehicles, machinery and building materials go in and out the element. The weight of all this is neglected.
Adhesion $F_{adhesion}$	When the elements are lifted form the foundation bed water needs to fill the space beneath the elements. This results in the elements 'sticking' to the foundation. This adhesion force can be mitigated by time or other Measures
Lifting F_{lift}	A lifting force is applied to lift the element out of the water. This is done by placing pontoons above the immersed elements.
Foundation pressure $F_{foundation}$	The elements rest on a foundation. Depending on the type of tunnel this varies between sand, gravel or piles. This force makes the equilibrium between upwards and downward forces when the element is immersed
Waves F_{waves}	During transport the elements encounters waves. These can have a positive or negative impact on the vertical balance

Horizontal load cases

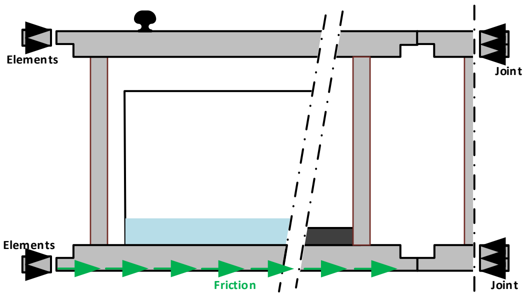


FIGURE 89, LOAD CASE H-I

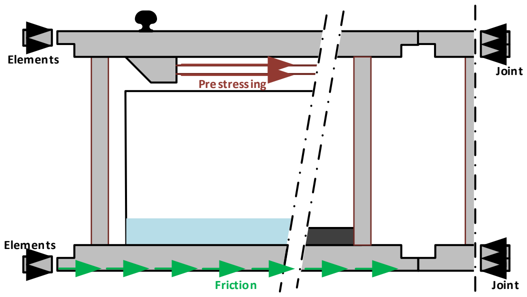


FIGURE 90, LOAD CASE H-II

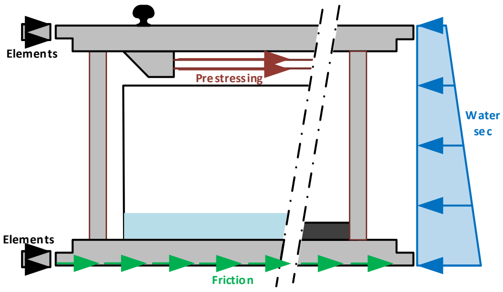


FIGURE 91, LOAD CASE H-III

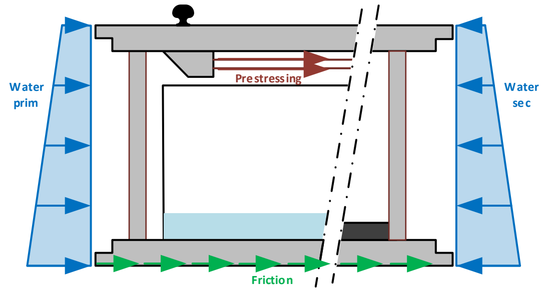


FIGURE 92, LOAD CASE H-IV

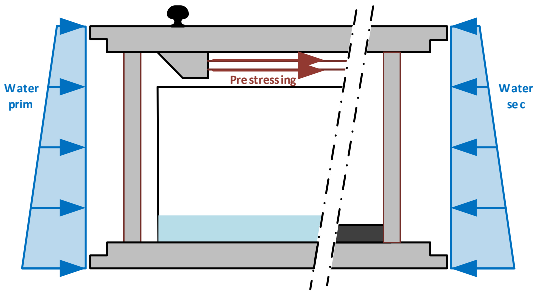


FIGURE 93, LOAD CASE H-V

Vertical load cases

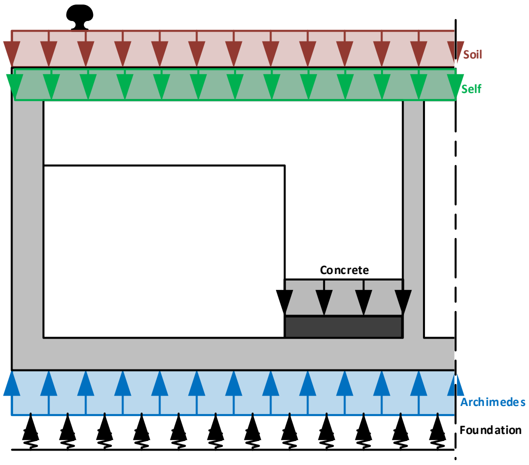
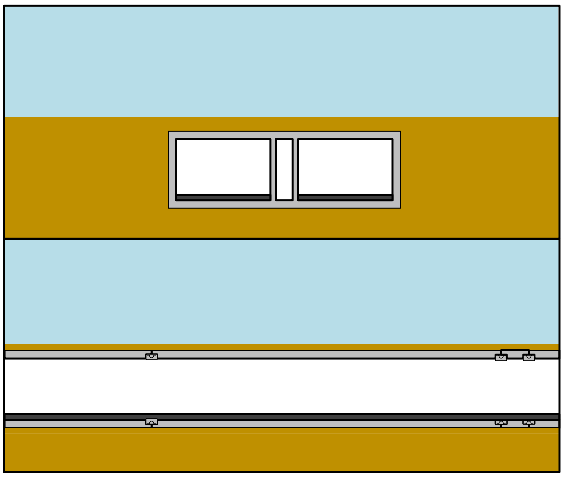


FIGURE 94, LOAD CASE V-I



CONSTRUCTION PHASE A.1-A.4

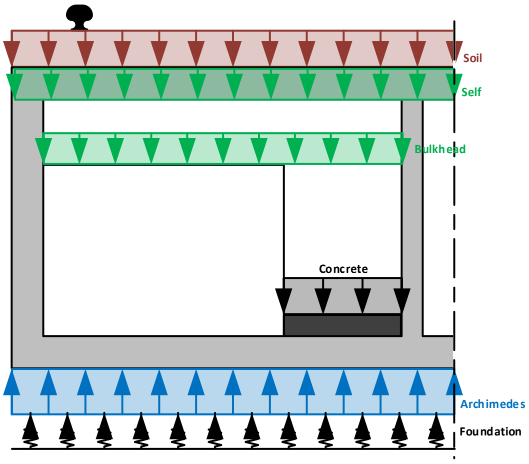
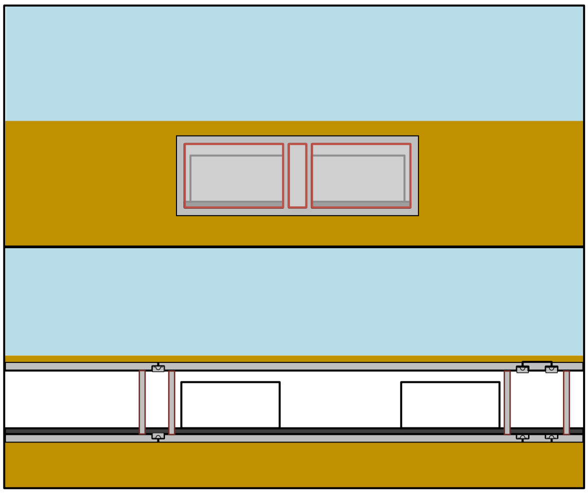


FIGURE 95, LOAD CASE V-II



CONSTRUCTION PHASE A.5

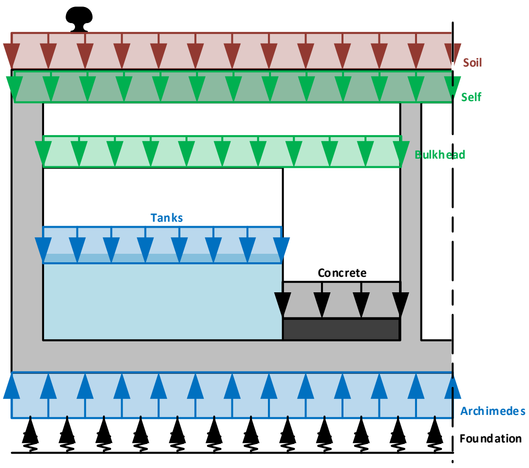
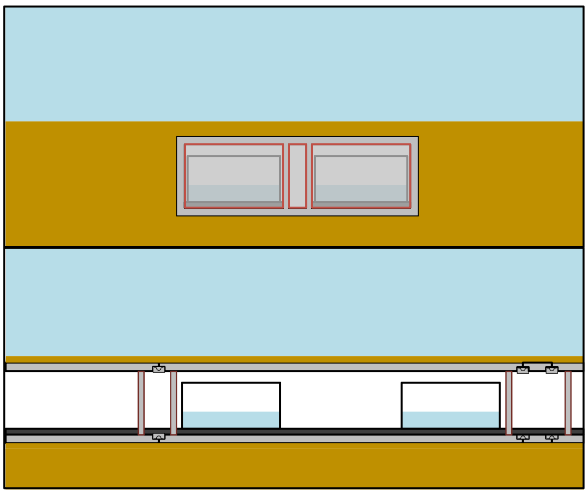


FIGURE 96, LOAD CASE V-III



CONSTRUCTION PHASE A.6

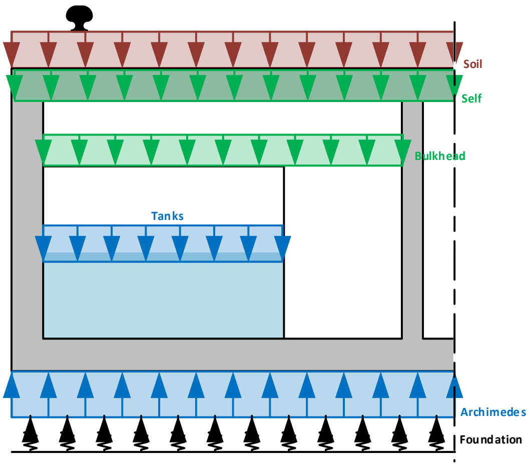
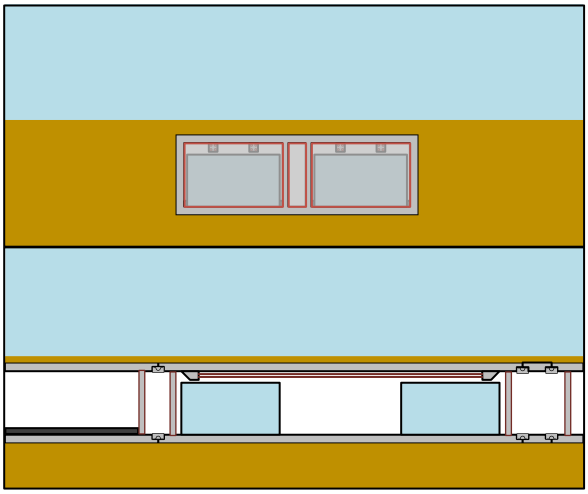


FIGURE 97, LOAD CASE V-IV


CONSTRUCTION PHASE A.7-A.13

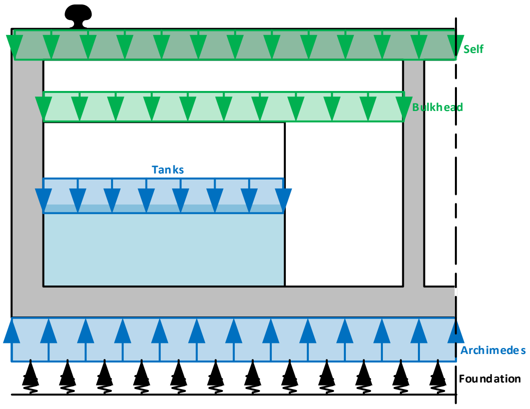
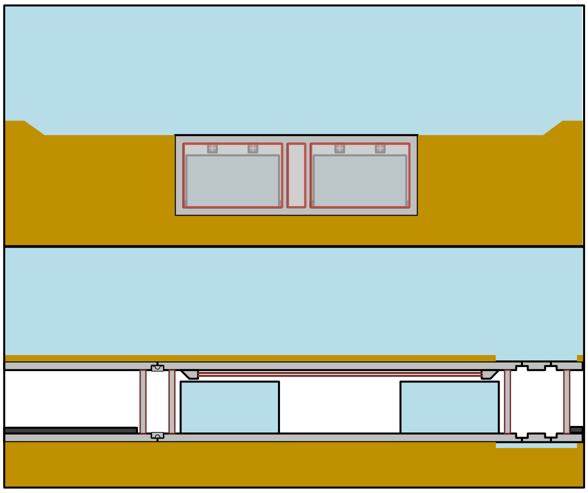


FIGURE 98, LOAD CASE V-V


CONSTRUCTION PHASE A.14-C.8

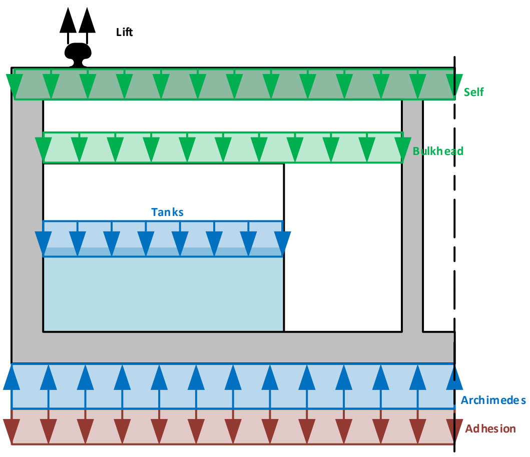
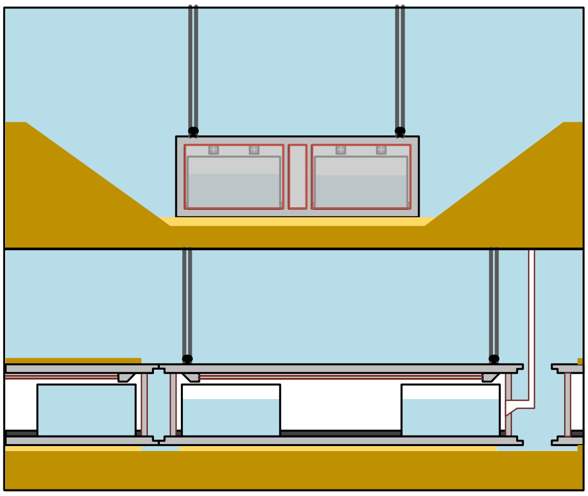


FIGURE 99, LOAD CASE V-VI


CONSTRUCTION PHASE C.9-C.10

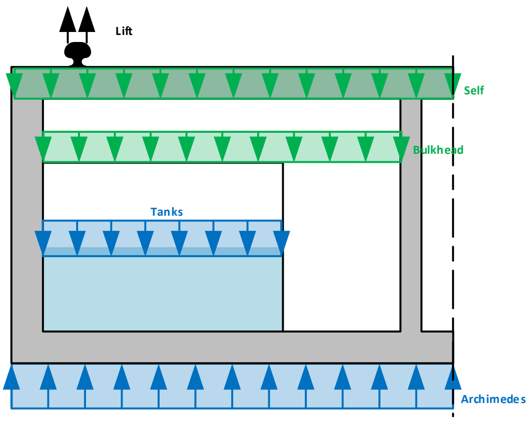
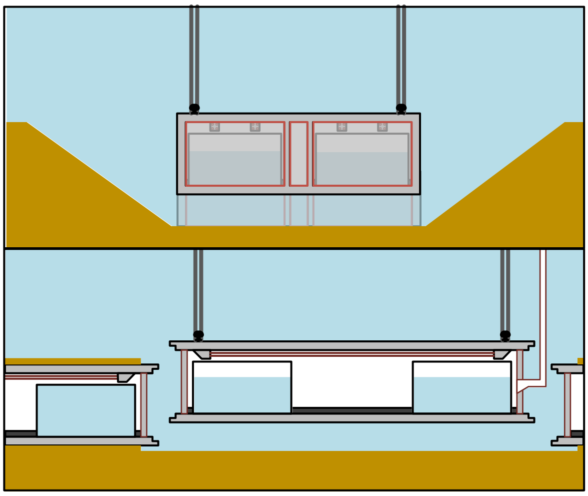


FIGURE 100, LOAD CASE V-VII



CONSTRUCTION PHASE C.10-C.11

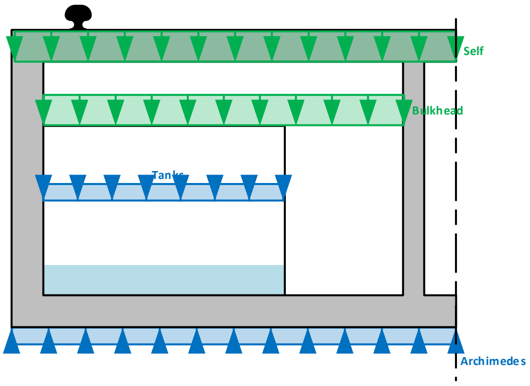
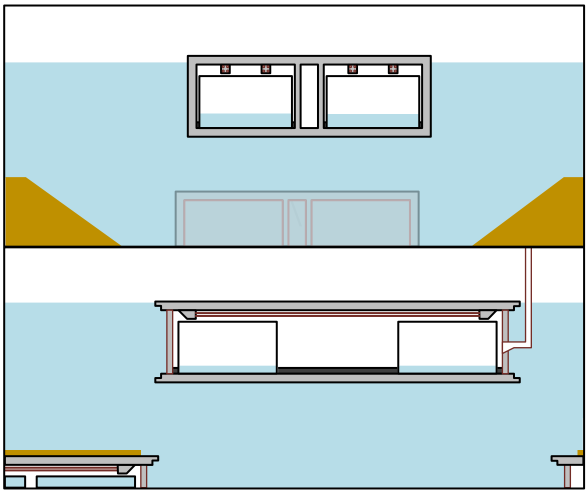


FIGURE 101, LOAD CASE V-VIII



CONSTRUCTION PHASE C.12

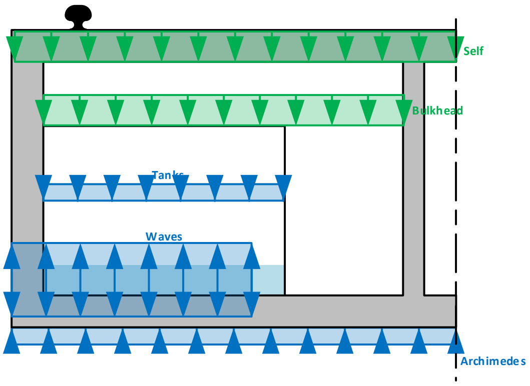


FIGURE 102, LOAD CASE V-IX

C. General rotational capacity

In this appendix a calculation is made showing the maximum rotation and the maximum change in the alignment, based on a given GINA seal elongation. In Figure 103 a simplified situation is shown.

SITUATION SKETCH

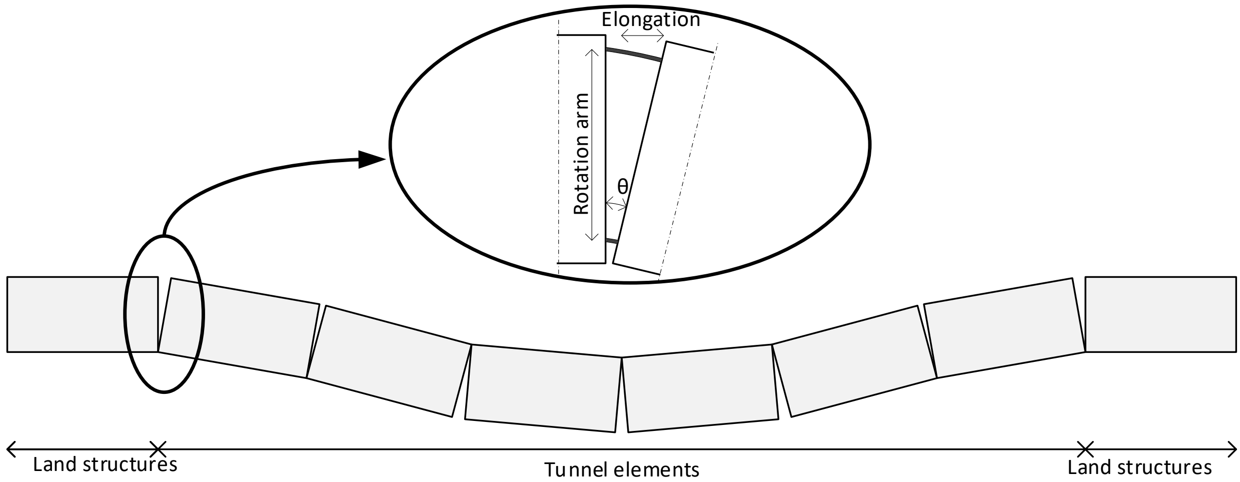


FIGURE 103, GENERAL ROTATIONAL CAPACITY; SITUATION SKETCH

CALCULATION METHOD

The tunnel is modelled as a simple line model, each tunnel element represents a part of the line. The rotation between the lines or elements; θ is calculated. This is directly linked to the rotation arm which represent de distance between the GINA seals, and the top seal elongation (relative to the lower seal elongation):

$$\theta = \frac{elongation}{rotation\ arm}$$

To arrive at a horizontal land structure not all the rotations are at maximum capacity. This value depends on the amount of elements and the variant chosen (regarding adaptions of the land structures)

INPUT PARAMETERS

Parameter	Value	Unit	Source
$Elongation_{normal}$	60	mm	Assumption
$Elongation_{bigger}$	90	mm	Assumption
Element height	8000	mm	Case tunnel
Rotation arm	7600	mm	Case tunnel
Tunnel elements	6	-	Case tunnel
Element length	100	m	Case tunnel

TABLE 29, GENERAL ROTATIONAL CAPACITY; INPUT PARAMETERS

RESULTS

In Table 30 the maximum percentage of rotation is given with respect to the variants regarding each of the immersion joints, negative being a clockwise rotation.

	Variant 1; NO adaptions	Variant 2; ONE adaption	Variant 3; TWO adaptions
Joint land-1	-1	-1	-2.5
Joint 1-2	-0.5	-0.8	1
Joint 2-3	1	1	1
Joint 3-4	1	1	1
Joint 4-5	1	1	1
Joint 5-6	-0.5	1	1
Joint 6-land	-1	-2.2	-2.5

TABLE 30, GENERAL ROTATIONAL CAPACITY; RESULTING JOINT ROTATIONS

Figure 104 shows the new tunnel alignment for the two given GINA seal expansions and the case tunnel dimensions. The maximum deepening in the middle of the tunnel is given in Table 31

	Variant 1; NO adaptions	Variant 2; ONE adaption	Variant 3; TWO adaptions
60 expansion	2.37	2.84	3.55
90 expansion	3.55	4.26	5.33

TABLE 31, GENERAL ROTATIONAL CAPACITY; RESULTING MAXIMUM DEEPENING

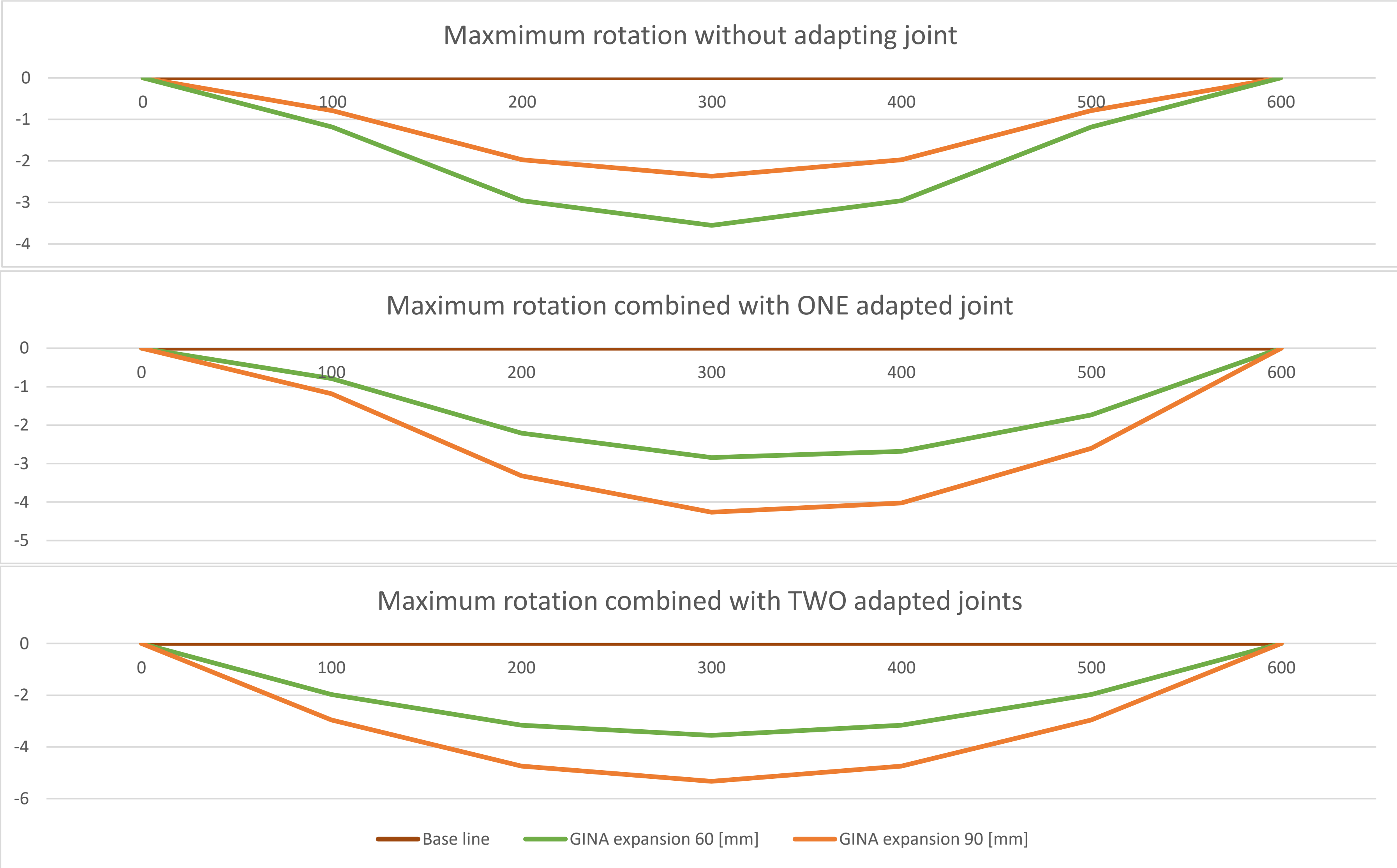


FIGURE 104, GENERAL ROTATION CAPACITY; RESULTING ALIGNMENT

D. Risk analysis

In this appendix a summary is given of the risk analysis made in section 2.8. Also, the types of measures are mentioned.

No.	Failure	Category	Possible causes	Effect	Consequence	Probability	Risk score	action type	-action
1	Unexpected floatation	A. Preparation of the element	Removal of too much concrete + unknown element weight	Random element in the waterway	3. Completely unsafe; evacuation of surrounding area 0. None 2. Not repairable, maintenance during lifetime 1. 0 < t ≤ 4 1. 0 < € ≤ 250.000	2. Very unlikely	14	Preventive	Higher safety factor, gather more information
2	Prestressing - partial	A. Preparation of the element	Slipping in connection with concrete	Deformations during transport, possible damage to structure	1. Extra monitoring needed 0. None 2. Not repairable, maintenance during lifetime 0. t = 0 2. 250.0000 < € ≤ 2.500.000	2. Very unlikely	10	Preventive	Carefully constructing
3	Prestressing - total	A. Preparation of the element	Not enough prestressing force	Element collapsing in the middle of waterway	3. Completely unsafe; evacuation of surrounding area 2. Big impact 3. Permanent damage, lower performance than required 3. t > 26 3. € > 2.500.000	1. Very very unlikely	14	Preventive	Higher safety factor
4	Bulkhead - partial	A. Preparation of the element	Bad connection between bulkhead and construction	Leaking of the element	1. Extra monitoring needed 0. None 1. No deviation from final requirements, repairable 1. 0 < t ≤ 4 1. 0 < € ≤ 250.000	3. Unlikely	12	Corrective	Testing while immersed
5	Bulkhead - total	A. Preparation of the element	Not enough structural capacity, higher loads than expected	Flooding of the element	2. Evacuation of construction site 0. None 2. Not repairable, maintenance during lifetime 2. 4 < t ≤ 26 2. 250.0000 < € ≤ 2.500.000	2. Very unlikely	16	Preventive	Higher safety factor, gradually increasing pressure
6	Damage to other elements	B. Removing a first section	Falling concrete rumble, cut at the wrong location	Damaged element	1. Extra monitoring needed 1. Minor impact 1. No deviation from final requirements, repairable 1. 0 < t ≤ 4 2. 250.0000 < € ≤ 2.500.000	2. Very unlikely	12	Preventive	Preliminary monitoring, limited removal of soil
7	Element stuck	B. Removing a first section	Uneven lifting of the element	new cuts needed, lowering if possible	0. None 0. None 1. No deviation from final requirements, repairable 1. 0 < t ≤ 4 1. 0 < € ≤ 250.000	4. Likely	12	Preventive	Guidance structures, extra monitoring

8	Access shaft - partial	C. Emerging & Transport of the main elements	Bad connection between shaft and construction	Leaking into the element	1. Extra monitoring needed 0. None 0. None 1. 0 < t ≤ 4 1. 0 < € ≤ 250.000	3. Unlikely	9	Acceptance	Testing while immersed
9	Access shaft - total	C. Emerging & Transport of the main elements	Not enough structural capacity, higher loads than expected	No access to tunnel element	2. Evacuation of construction site 0. None 1. No deviation from final requirements, repairable 1. 0 < t ≤ 4 1. 0 < € ≤ 250.000	2. Very unlikely	10	Corrective	Careful use of access shaft
10	Damage to the element	C. Emerging & Transport of the main elements	Collision during rock removal, collision with other elements	Damaged element	1. Extra monitoring needed 1. Minor impact 1. No deviation from final requirements, repairable 1. 0 < t ≤ 4 1. 0 < € ≤ 250.000	2. Very unlikely	10	Preventive	Limited removal of soil, extensive monitoring
11	Element won't float	C. Emerging & Transport of the main elements	Element heavier than expected, Adhesion stronger than expected	Element won't float	0. None 0. None 1. No deviation from final requirements, repairable 3. t > 26 3. € > 2.500.000	2. Very unlikely	14	Preventive	Destroying more adhesion, lab tests, safety factors
12	Failure of habitat - partial	D. Preparing, adapting & Reconfiguration	Bad connection between habitat and construction	Leaking into the habitat	1. Extra monitoring needed 0. None 0. None 1. 0 < t ≤ 4 1. 0 < € ≤ 250.000	2. Very unlikely	6	Acceptance	Monitoring
13	Failure of habitat - total	D. Preparing, adapting & Reconfiguration	Collision or accident	Flooding of the habitat	2. Evacuation of construction site 1. Minor impact 2. Not repairable, maintenance during lifetime 2. 4 < t ≤ 26 2. 250.0000 < € ≤ 2.500.000	1. Very very unlikely	9	Acceptance	Safe location, otherwise, guidance structures
14	Leakage in joints - GINA	D. Preparing, adapting & Reconfiguration	Uneven compression of seals, transport	Leakage into the element	1. Extra monitoring needed 0. None 2. Not repairable, maintenance during lifetime 1. 0 < t ≤ 4 2. 250.0000 < € ≤ 2.500.000	2. Very unlikely	12	Preventive	Protect seals during transport
15	Leakage in joints - W9Ui	D. Preparing, adapting & Reconfiguration	Damage to concrete, too much deformations in early stage	Leakage into the element	1. Extra monitoring needed 0. None 2. Not repairable, maintenance during lifetime 0. t = 0 2. 250.0000 < € ≤ 2.500.000	3. Unlikely	15	Preventive	Limit deformations

TABLE 32, RISK ANALYSIS

E. Case selection

In this appendix the a multi criteria analysis is made on in which a case is selected to apply on the general method.

Criteria

Available knowledge For this research it important to know as much as possible about the tunnel design, with the most details about the current tunnel the design will be the most specific. This information is about dimension, concrete reinforcement, design loads et cetera.

Waterway relevancy As mentioned in the introduction the main reason for re-floating the tunnels is too be able to dredge the waterway. It is worth to check if already some plans are available for dredging the waterway the tunnel is located in. This can also be seen in the future development plans for the port close to the tunnel location.

Tunnel relevancy Another aspect regarding relevancy is if the tunnel should not be replaced, since for example the design-lifetime is reached. If that is the case the temporarily re-floating is not attractable. Also, if there is for example an enormous increase in traffic through the tunnel there is a possibility a completely new tunnel is needed.

Tunnel dimensions Each immersed tunnel is designed for its own specific requirements; therefore, all the tunnels have different dimension. For this research it is interesting to choose one of the most complex tunnel designs for this method. If is then is possible the adaptions for other tunnels are less complex. The complexity of the tunnels is shown in the depth of the tunnel location, size of the elements and number of elements (at least 4).

In the following table the relative importance of each of the categories is shown. As seen the category 'waterway relevancy' is nowhere more important than another, therefore all the scores are doubles and 'waterway relevancy' gets a '1' score.

	Available knowledge	Waterway relevancy	Tunnel relevancy	Tunnel dimensions	Score	Weight
Available knowledge		1	1	0	4 (2)	31%
Waterway relevancy	0		0	0	1 (0)	8%
Tunnel relevancy	0	1		0	2 (1)	15%
Tunnel dimension	1	1	1		6 (3)	46%

TABLE 33, MCA CRITERIA IMPORTANCE

Possible tunnels

The complete list of all the immersed tunnel reaches over 100 tunnels constructed in the last 40 years. Not all these tunnels can be checked with the multi criteria analysis. Combined with the literature study done earlier a short-list is made with four tunnels. The requirements for these tunnels are:

- It should be segmental tunnels, not monolithic.
- It should have a concrete cross-section. No single- or double shell type tunnels.

The four tunnels are discussed in short below:

Liefkenshoektunnel (ITA, 2019)

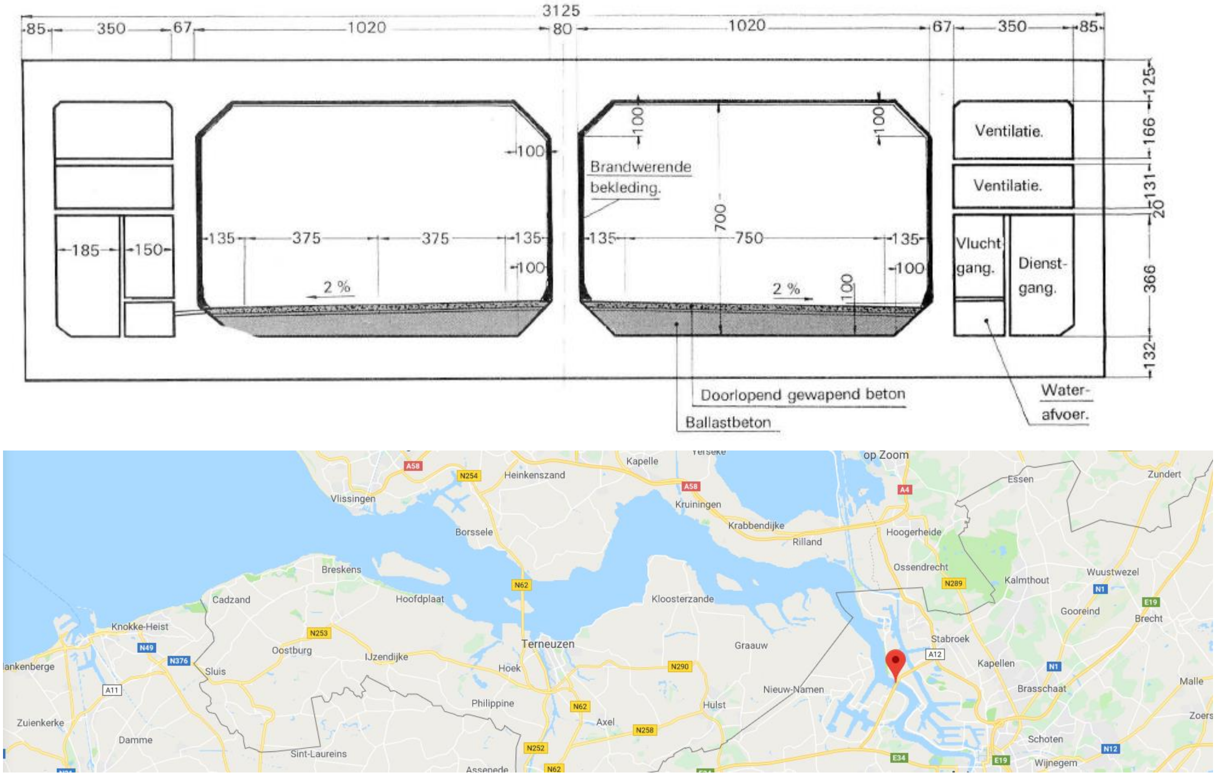


FIGURE 105, LIEFKENSHOEKTUNNEL CROSS SECTION AND LOCATION

Available knowledge

The tunnel is not constructed nor design by any company related to Royal HaskoningDHV or TEC, combined with the fact that this tunnel is not construction in the Netherlands it is not expected to have much information available. It might be more than the Busan-Geoje tunnel since Belgium is still in Europa.

Waterway relevancy

About a year ago the construction of the new Kieldrecht lock has finished. This biggest lock of Europa makes it possible to travel deeper into the port of Antwerp without passing over the Liefkenshoektunnel.

Tunnel relevancy

The Liefkenshoektunnel is the oldest of the four tunnels discussed. Therefore, it is also the least in need of a complete recycling since the new design lifetime is short compared to the other tunnels.

Tunnel dimensions

The number of elements is perfect with 8 elements. This is enough to have some repetition in the process to be optimized but not too much that it becomes an enormous amount of lifting processes. The element length is quite long but not too much.

An extra aspect to consider for the Liefkenshoektunnel is that it is located on the edge of the tidal area of the North Sea, this results in an unknown combination of salt and fresh water. Also, unusual high amount of sediment transport is present. This might result in an overly complicated calculation which is only applicable for a few cases

Construction finished in	1991
Country	Belgium
Location	Antwerp
Name	Liefkenshoektunnel
# elements [-]	8
Type	Segmental
Total length [m]	1136
Element height [m]	9,60
Element width [m]	31,25
Element length [m]	142
Segment length [m]	23,65
Depth ToS [m]	15,65
Foundation	Sand underflow

Wijkertunnel (Camerik, Vermindering Gronddekking op Velsertunnel en Wijkertunnel, 1999)

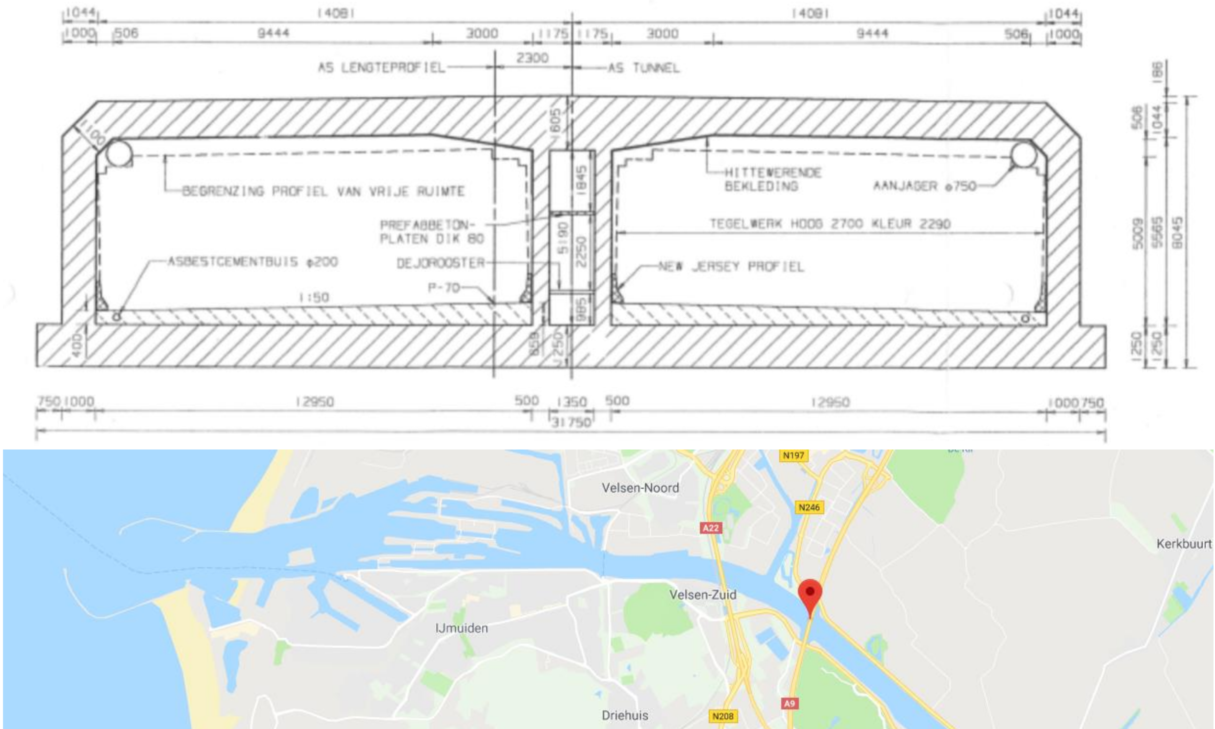


FIGURE 106, WIJKERTUNNEL CROSS SECTION AND LOCATION

Available knowledge
Much information is available for this tunnel. It is constructed not very long ago, and it is in the Netherlands. Also, already some research is done into this tunnel.

Waterway relevancy

The tunnel is in the North Sea canal. With the new Ijmuiden sea-locks close to finishing the Wijkertunnel and the Velsertunnel are the next highest obstacles to the Port of Amsterdam. In the future the big ships will still be travelling through the North Sea Canal since no current plans are available for moving the port to the sea.

Tunnel relevancy

This tunnel is about 25 years old but still in use. Next to the Wijkertunnel is the Velsertunnel. The Velsertunnel is not of the immersed tunnel type and needs replacement, a possible need of extra lanes can be constructed in such a new tunnel. A disadvantage of this tunnel is also a railway tunnel situated next to the Velsertunnel. This should also be replaced to have a complete renewal of the North Sea Canal

Tunnel dimensions

The number of elements, 6, is the lower bound of elements but still fine. Compared to other tunnels is this one relatively low, just reaching above 8 meters. Overall these dimensions are fine.

Construction finished in	1996
Country	The Netherlands
Location	Amsterdam
Name	Wijkertunnel
# elements [-]	6
Type	Segmental
Total length [m]	574
Element height [m]	8,05
Element width [m]	31,75
Element length [m]	92,6
Segment length [m]	23,15
Depth ToS. [m]	18,36
Foundation	Sand underflow

Benelux tunnel (Weeda, 2015)

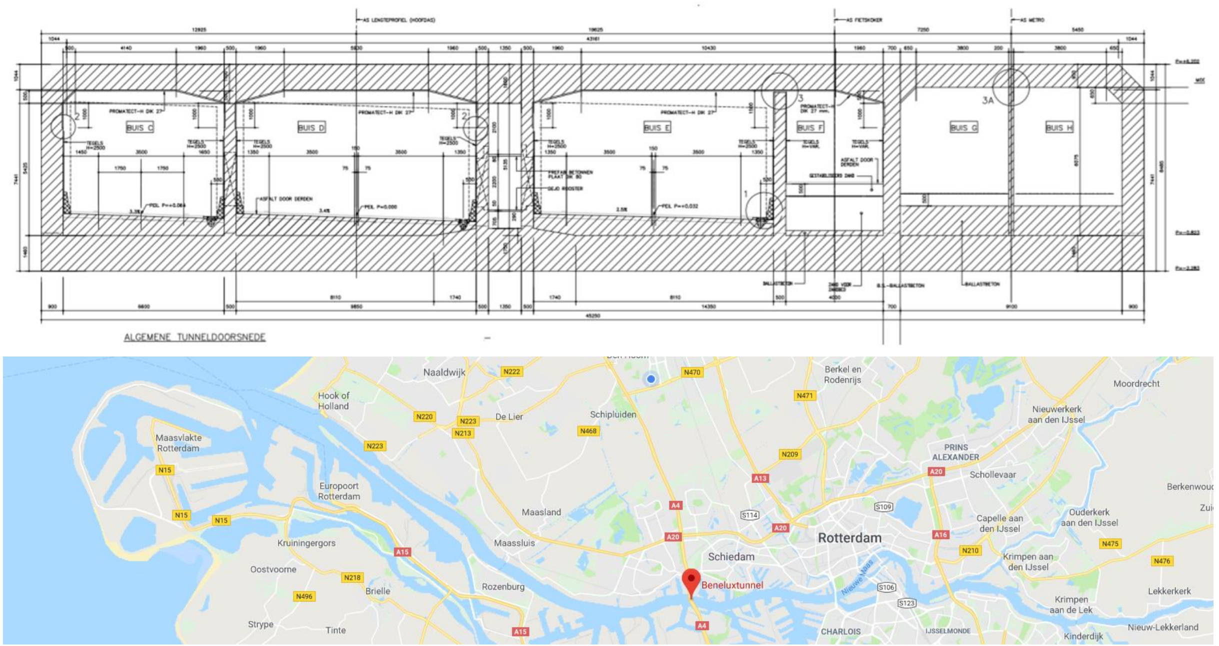


FIGURE 107, BENELUXTUNNEL CROSS SECTION AND LOCATION

Available knowledge

Much information is available for this tunnel. It is constructed not very long ago, and it is in the Netherlands. Also, already some research is done into this tunnel.

Waterway relevancy

The Maas and Port of Rotterdam are developing very fast and moving slowly to the North Sea. With the construction of the 2nd Maasvlakte the tunnels deep inland become less problematic since the biggest vessels will moor close to the sea. This makes dredging and the 2nd Benelux tunnel less relevant.

Tunnel relevancy

This tunnel is constructed about 20 years ago. It is located next to the 1st Benelux tunnel. If extra lanes are needed the first Benelux tunnel can be replaced but the second tunnel might be recycled and immersed again.

Tunnel dimensions

The number of elements, 6, is the lower bound of elements but still fine. Compared to other tunnels is this one relatively wide, about 50% more than the Wijkertunnel and Liefkenshoektunnel, this might result in being less relatable to other tunnels.

Construction finished in	2003
Country	The Netherlands
Location	Rotterdam
Name	2nd Benelux Tunnel
# elements [-]	6
Type	Segmental
Total length [m]	900
Element height [m]	8,50
Element width [m]	45
Element length [m]	140
Segment length [m]	20
Depth ToS [m]	18
Foundation	Gravel

Busan-Geoje tunnel **(Matwan, 2019)**

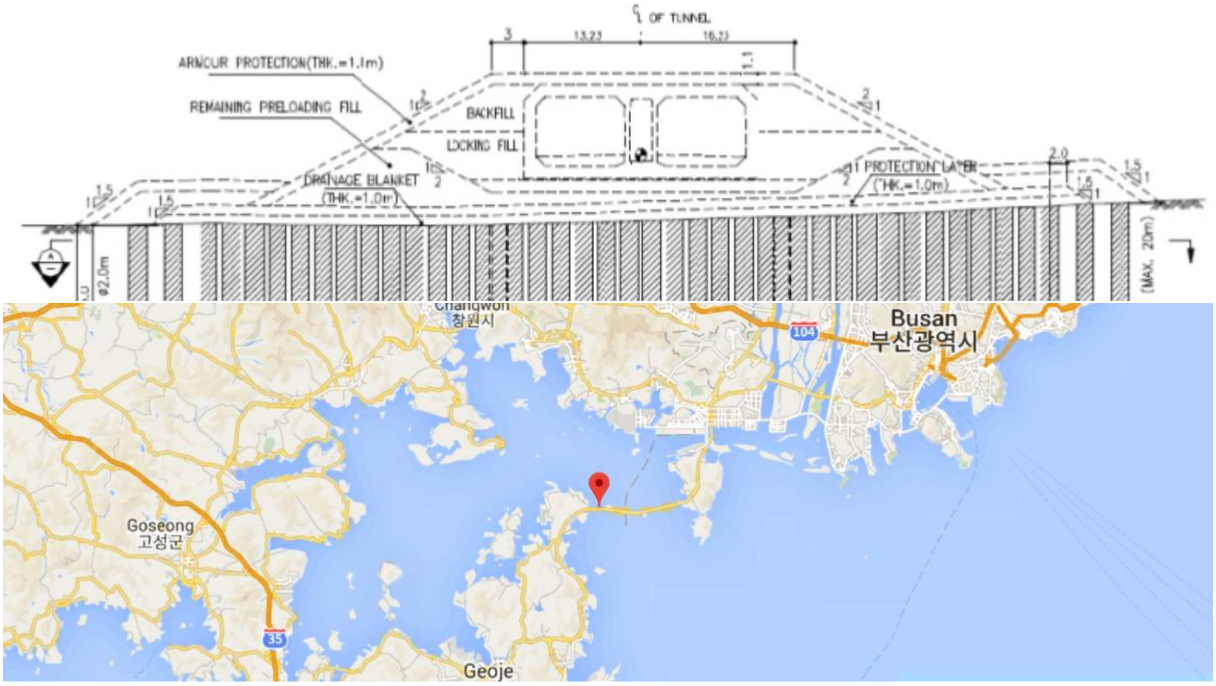


FIGURE 108, BUSAN-GEOJE TUNNEL CROSS SECTION AND LOCATION

Available knowledge

The tunnel is not constructed nor design by any company related to Royal HaskoningDHV or TEC, combined with the fact that this tunnel is not construction in the Netherlands it is not expected to have much information available. It might be even less information than the Liefkenshoektunnel since this one is constructed in South-Korea

Waterway relevancy

aa

Tunnel relevancy

The tunnel construction is only finished about a decade ago. It is probably necessary to construct extra lanes. It would be a waste to discard this entire tunnel if dredging is needed.

Tunnel dimensions

The amount of tunnel elements for this tunnel is too high. Lifting this high number of elements makes it a very extensive process. The dimensions of the single elements are fine.

Construction finished in	2010
Country	South Korea
Location	Busan-Geoje
Name	Busan–Geoje tunnel
# elements [-]	18
Type	Segmental
Total length [m]	3240
Element height [m]	10,00
Element width [m]	26,50
Element length [m]	180
Segment length [m]	22,5
Depth ToS [m]	50,00
Foundation	Sand compacting piles

Case choice

	Available knowledge	Waterway relevancy	Tunnel relevancy	Tunnel dimensions	Score
Liefkenshoektunnel	7	6	6	7	6,77
Wijkertunnel	8	9	6	7	7,31
Benelux tunnels	8	6	7	7	7,23
Busan-Geoje tunnel	6	7	8	6	6,38

TABLE 34, MCA RESULTING SCORES

F. Vertical balance calculation

In this appendix a calculation is made regarding the vertical balance in each of the construction stages. The criteria for this balance calculation are the uplift factor of safety, available lifting forces and required freeboard. The following table shows the calculation of the upward or downward forces based on the given densities.

SITUATION SKETCH

For this calculation many situations need to be considered (all nine vertical load cases). For all of these cases a sketch is given in appendix B. Below the situation sketch of load case V-III is given.

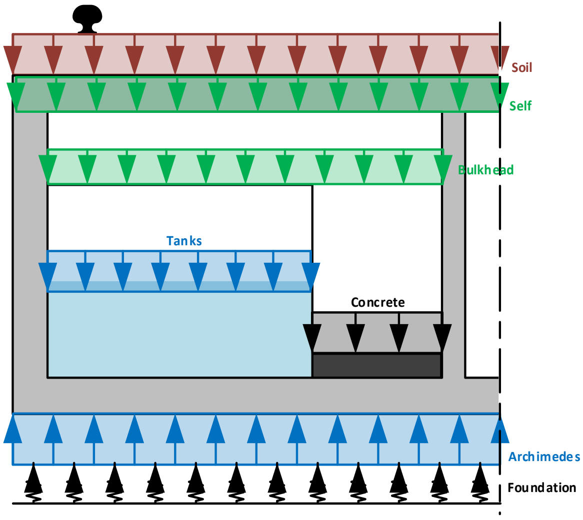


FIGURE 109, VERTICAL BALANCE CALCULATION; SITUATION SKETCH LOAD CASE V-III

METHOD

For this calculation not all the material densities are known exactly, so a upper and lower boundary is given if needed. The dimensions of the tunnel elements are known, combining these volumes with the densities results in all the required loads.

This results in a minimum and maximum value for the upwards and the downward forces for each load case. With these values the most unfavorable combinations are used to calculated the factor of safety or the required freeboard using:

$$FoS = \frac{downwards_{min}}{upwards_{max}}$$
$$Lifting\ force_{full\ tanks} = downward - upwards$$
$$Lifting\ force_{empty\ tanks} = downwards - upwards - weight\ tanks$$
$$freeboard = (buoyancy - downwards)/(force/\ meter\ freeboard)$$

The criteria are given in chapter 2, and are summarized in Table 35

Factor of Safety; Operational	1.05
Factor of Safety; Construction	1.024
Applied lifting force	4000 [kN]
Required freeboard	0.15 [m]

TABLE 35, VERTICAL BALANCE CALCULATION; METHOD CRITERIA

INPUT PARAMETERS

Parameters	Value	Unit	Source
$\rho_{water;min}$	10.00	kN/m^3	(TEC, 1993)
$\rho_{water;max}$	10.25	kN/m^3	''
$\rho_{concrete;min}$	24.214	kN/m^3	''
$\rho_{concrete;max}$	24.914	kN/m^3	''
$\rho_{ballast\ concrete}$	23.00	kN/m^3	''
$l_{element}$	95.6	m	''
$w_{element;floor}$	31.75	m	''
$w_{element;roof}$	30.25	m	''
$h_{element}$	8.045	m	''
$A_{face;total}$	244.07	m^2	''
$A_{face;ballast\ concrete}$	13.71	m^2	''
$A_{face;soil}$	43.11	m^2	''
$A_{face;element\ concrete}$	91.43	m^2	''
$A_{face;tubes}$	152.64	m^2	''
Parameters	**Value**	**Unit**	**Source**
$t_{bulkhead}$	0.3	m	Design choice
$\boldsymbol{bulkhead\ indent}$	0.7	m	Design choice
$w_{ballast\ tanks}$	10	m	Design choice
$l_{ballast\ tanks}$	16	m	Design choice
$n_{ballast\ tanks}$	4		Design choice

TABLE 36, VERTICAL BALANCE CALCULATION; INPUT PARAMETERS

RESULTS

The resulting forces during each load case are shown in Table 37.Table 37, Vertical balance calculation; Resulting loads

Load case	Construction step	downwards min	downwards max	upwards min	upwards max
V-I	A.1 – A.4	278885	289125	229914	235662
V-II	A.5	281135	291374	229914	235662
V-III	A.6	303151	313941	229914	235662
V-IV	A.7 – A.13	278431	289221	229914	235662
V-V	A.14 – C.8	241339	248008	229914	235662
V-VI	C.9 – C.10	241339	248008	229914	235662
V-VII	C.10 – C.11	241339	248008	229914	235662
V-VIII	C.12	241339	248008	0	0
V-IX	C.13 →				

TABLE 37, VERTICAL BALANCE CALCULATION; RESULTING LOADS

This is directly translated into the important design aspects (FoS or lifting force and freeboard range).

Load case	Factor of Safety	Minimum case	Maximum case
V-I	1.183		
V-II	1.193		
V-III	1.286		
V-IV	1.181		
V-V	1.024		
V-VI		-16338 to 5678	-4472 to 18094
V-VII		-16338 to 5678	-4472 to 18094
V-VIII		-0.610 to 0.151	-0.196 to 0.565

TABLE 38, VERTICAL BALANCE CALCULATION; RESULTING FOS OR LIFTING AND FREEBOARD RANGES

G. Bulkhead calculations

In this appendix the calculations for the bulkheads are discussed. The bulkhead is only present during construction so load factors of 1 are applied. The material properties are the same as defined in section **3.2.7**. Different bulkheads are calculation as mentioned in section **4.1.2**

SITUATION SKETCH

InFigure 110 the situation sketch of the bulkhead is given.

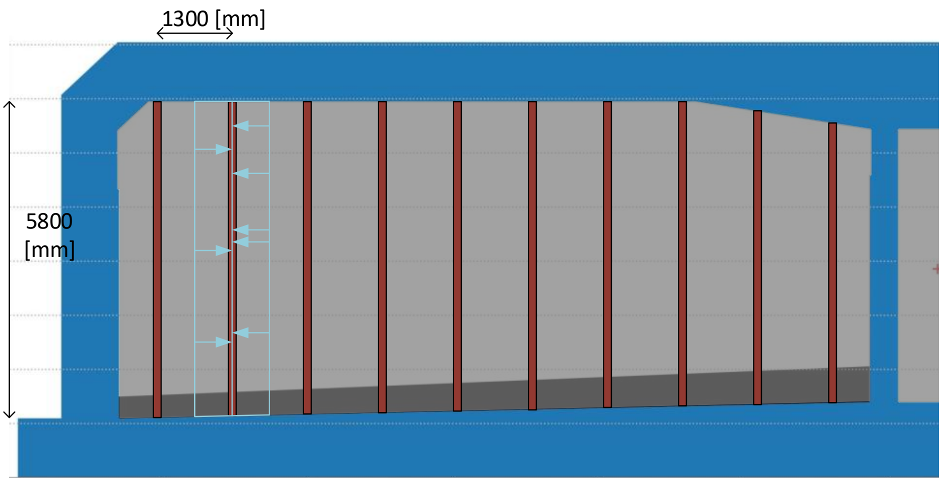


FIGURE 110, BULKHEAD CALCULATION; SITUATION SKETCH

METHOD

The bulkheads are subjected to a water load. Per element face the load is increasing over depth. The This load is first carried by the concrete in the horizontal directions, then transported to the steel HEB-profile-beams in vertical direction. Both the steel and concrete are assumed to have a simple support without rotational rigidity.

For the steel calculation no buckling is assumed since buckling support is present due to the concrete. The steel using is class S235. The resulting check is shown below, the M_{Rd} is known for each beam type. The $M_{Ed,extra} = M_{Ed} * 110\%$ due to the risk measure.

$$U.C. = \frac{M_{Ed;extra}}{M_{Rd}}$$

For the concrete the calculations are made according to EC2 9.2. A minimal reinforcement ratio for both the longitudinal and shear reinforcement is assumed.

$$A_{s,min,long} = 0.26 * \frac{f_{ctm}}{f_{yk}} * b_t * d, but > 0.0013 * b_t * d$$

$$A_{s,min,shear} = 0.08 * \sqrt{f_{ck}}/f_{yk}$$

For the moment EC2 6.1 is used. A bi-linear concrete and a yielding steel is used. The concrete cover is based on the reinforcement diameter and the environment in which the bulkheads will be used.

$$z = d - \beta * x_u = d - \beta * \frac{Nc}{\alpha * h_{tube} * f_{cd}}$$

$$U.C. = \frac{M_{Ed;extra}}{M_{Rd}} = \frac{1.1 * \frac{1}{8} * q * l^2}{A_s * z * f_{yk}}$$

For the shear EC2 6.2 is used. The crack angle is set at $\theta = 21.8°$ is used since the maximum pressure in the compressive strut is orders of magnitude bigger than the applied shear force. All the values are shown below.

$$U.C. = \frac{V_{Ed;extra}}{V_{Rds;min}} = \frac{1.1 * 0.5 * q * l}{\frac{A_{sw}}{s} * z * f_{yd} * \cot(\theta)}$$

The total usage of materials can be calculated knowing the dimensions of the bulkheads and the dimensions of the beams and concrete. For each of the joints 2 bulkheads are needed (both sides), for each tunnel 2 traffic tubes are present.

INPUT PARAMETERS

Parameters	Value	Unit	Source
ρ_{water}	10.25	kN/m^3	(TEC, 1993)
$Depth_{joint:1\&7}$	17.123	m	"
$Depth_{joint:2\&6}$	27.507	m	"
$Depth_{joint:3\&5}$	25.676	m	"
$Depth_{joint:4}$	24.914	m	"
h_{tube}	5.8	m	"
w_{tube}	12.95	m	"
$distance\ beams$	1.30	m	Design choice
Concrete cover	24	mm	Code
α	0.75	-	Code
β	0.39	-	Code
$Reinforcing\ steel\ class$	B500	-	(TEC, 1993)
$Steel\ class$	S235	-	"
$Concrete\ class$	C30/37	-	"

The parameters for the steel profiles are standard values given in steel profile tables and are:

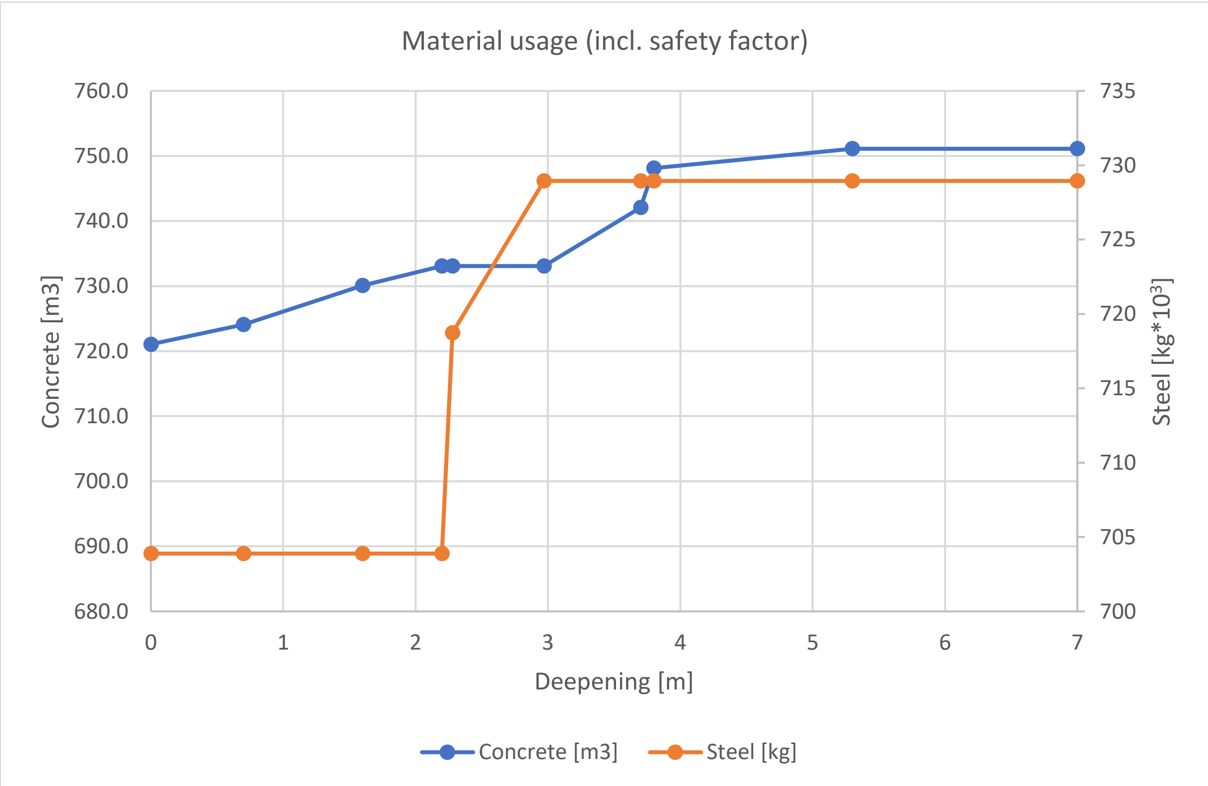
Profile type:	W [m3]	Mrd [kNm]	G8 [kg/m]
HEB400	0.002884	677.7	158
HEB450	0.003551	834.5	174
HEB500	0.004287	1007.4	191
HEB550	0.004971	1168.2	203
HEB600	0.005701	1339.7	216
HEB650	0.006480	1522.8	229
HEB700	0.007340	1724.9	245
HEB800	0.008977	2109.6	267

RESULTS

The results are unity checks for each of the element faces. This depends on the actual amount of deepening. Keeping the unity checks below 1 the following minimum dimensions are required for a given amount of deepening

	Joint 1&7	Joint 2&6	Joint 3&5	Joint 4
Beam type	HEB500	HEB600	HEB650	HEB700
Maximum deepening [m]	All	7+	2.28	2.97
Beam type			HEB700	HEB800
Maximum deepening [m]			7+	7+
Concrete thickness	300	340	370	380
Maximum deepening [m]	All	3.7	1.6	0.7
Concrete thickness		350	380	390
Maximum deepening [m]		7+	3.8	2.2
Concrete thickness			390	400
Maximum deepening [m]			7+	3.7
Concrete thickness				410
Maximum deepening [m]				5.3

Based on this data the total material usage can be calculated. Based on this the following graph can be plotted:

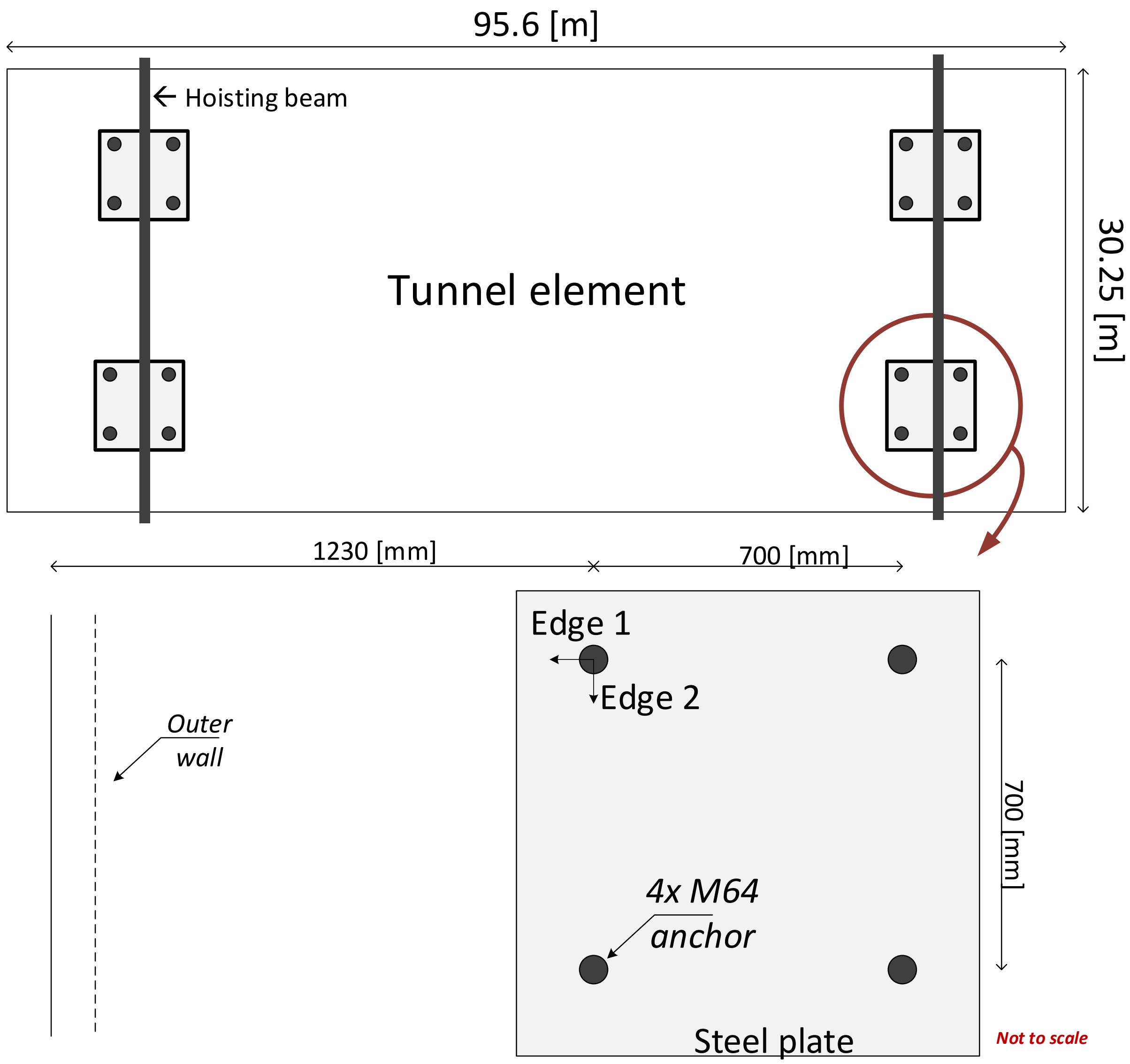


H. Hoisting wires & anchors calculation

In this appendix the calculation of the connection of the hoisting wires to the anchors is calculated.

SITUATION SKETCH

Below a situation sketch is given (according to the final design).



METHOD

Three aspects need considering regarding the hoisting wires: the anchors, the hoisting beam and the cross-sectional effect.

The anchors are calculated according to NEN-EN 1992-4. The lifting applies pure tension to the wires however water flow induces a small shear force, it is assumed at 10% of the tension force. Regarding tension 6 failure modes are present, regarding shear 4 modes are present; these are shown in Figure 111 and Figure 112. Tension failure *d) 'Combined pull-out and concrete failure'* and shear failure *b) 'Steel failure with lever arm'* are not discussed since the first is not required for post-installed fasteners and the lay-out is without a lever arm.

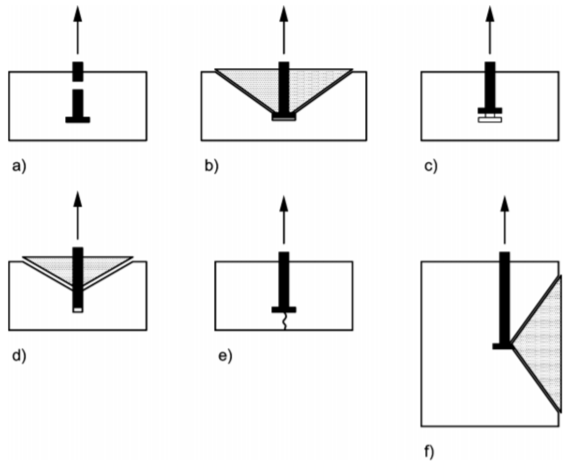


FIGURE 111, TENSION FAILURE MODES

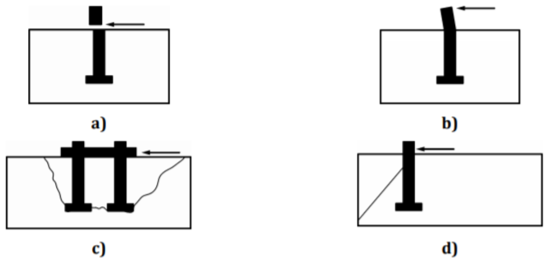


FIGURE 112, SHEAR FAILURE MODES

Due to multiple anchor being present the loads are:

$$N_{Ed,group} = 1500\ [kN]$$
$$N_{Ed,single} = 375\ [kN]$$

$$V_{Ed,group} = 150\ [kN]$$
$$V_{Ed,single} = 37.5\ [kN]$$

Tension mode A; Steel failure of fastener

$$N_{Rk,s} = A_{s,min} * f_{uk}$$
$$N_{Rd,s} = N_{Rk,s}/\gamma_{Ms,tension}$$
$$U.C. = N_{Ed,single}/N_{Rd,s}$$

Tension mode B; Concrete cone failure

$$N_{Rk,c}^{0} = k_1 * \sqrt{f_{ck}} * h_{eff}^{1.5}$$
$$N_{Rk,c} = N_{Rk,c}^{0} * \frac{A_{c,N}}{A_{c,N}^{0}} * \psi_{s,N} * \psi_{re,N} * \psi_{ec,N} * \psi_{M,N}$$
$$N_{Rd,c} = N_{Rk,c}/\gamma_{Mp}$$
$$U.C. = N_{Ed,group}/N_{Rd,c}$$

Tension mode C; Pull-out of fastener

$$N_{Rk,p} = k_2 * A_h * f_{ck}$$
$$A_h = \frac{\pi}{4}(d_h^2 - d_a^2)$$
$$N_{Rd,p} = N_{Rk,p}/\gamma_{Mp}$$
$$U.C. = N_{Ed,single}/N_{Rd,p}$$

Tension mode E; Concrete splitting failure

No reinforcement needed if:

$$c \geq 1.2 * c_{crsp}$$
$$h \geq h_{min}$$

Tension mode F; Concrete blow-out failure

Not required if:

$$c \geq 0.5 * h_{eff}$$

Shear mode B; Shear load without lever arm

$$V_{Rk,s} = A_{s,min} * f_{uk} * k_{50} * f_{reduction}$$
$$V_{Rd,s} = V_{Rk,s}/\gamma_{Ms,shear}$$
$$U.C. = V_{Ed,single}/V_{Rd,s}$$

Shear mode C; Concrete pry-out failure

$$V_{Rk,cp} = k_8 * N_{Rk,c}$$
$$V_{Rd,cp} = \frac{V_{Rk,cp}}{\gamma_{Mc}}$$
$$U.C. = V_{Ed,total}/V_{Rd,cp}$$

Shear mode D; Concrete blow-out failure

$$V^0_{Rk,c} = k_9 * d_{nom}^{\alpha} * l_f^{\beta} * \sqrt{f_{ck}} * c_1^{1.5}$$
$$\alpha = 0.1\left(\frac{l_f}{c_1}\right)^{0.5}$$
$$\beta = 0.1\left(\frac{d_{nom}}{c_1}\right)^{0.2}$$
$$V_{Rk,c} = V^0_{Rk,c} * \frac{A_{c,V}}{A^0_{c,V}} * \psi_{s,V} * \psi_{h,V} * \psi_{ec,V} * \psi_{\alpha,V} * \psi_{re,V}$$
$$V_{Rd,c} = \frac{V_{Rk,c}}{\gamma_{Mc}}$$
$$U.C. = V_{Ed,total}/V_{Rd,c}$$

Combined Steel failure of fastener

$$U.C. = \left(\frac{N_{ed,group}}{N_{Rd,s}}\right)^2 + \left(\frac{V_{ed,group}}{V_{Rd,s}}\right)^2$$

Combined other failure modes than steel

$$U.C. = \left(\frac{N_{ed,group}}{N_{Rd,s}}\right)^2 + \left(\frac{V_{ed,group}}{V_{Rd,s}}\right)^2$$

Or

$$U.C. = \frac{\frac{N_{ed,group}}{N_{Rd,s}} + \frac{V_{ed,group}}{V_{Rd,s}}}{1.2}$$

INPUT PARAMETERS

Parameters	**Value**	**Unit**	**Source**
Anchors per plate	4		Design choice
Distance edge1	1230	mm	ʺ
Distance edge2	19000	mm	ʺ
Opposite edge1	25000	mm	ʺ
Opposite edge2	19000	mm	ʺ
Distance rows	700	mm	ʺ
Distance columns	700	mm	ʺ
Diameter anchor	64	mm	Design choice

γ_{mc}	1.50	-	Code
$\gamma_{Ms,re}$	1.15	-	"
$\gamma_{Ms,tension}$	1.87	-	"
$\gamma_{Ms,shear}$	1.56	-	"
k_1	7.5	-	"
k_2	8.9	-	"
k_{50}	50	-	"
k_3	2	-	"

RESULTS

The proposed lay-out consist of 4x M64 anchors. The parameters for this anchor type are defined by the European Product Specification. The distance between the anchors is set at 700 $[mm]$. The others are each discussed below. The exact formulas can also be found in NEN-EN 1992-4 and the section mentioned. Below the unity checks are given regarding each failure mode.

Tension		**Shear**		**Combination**	
Failure	*U,C,*	*Failure*	*U.C.*	*Failure*	*U.C.*
a) Steel	0.371	a) Steel with lever arm	0.062	Steel	0.144
b) Concrete cone	0.970	c) Concrete pry-out	0.049	Other than steel	0.971
c) Pull-out	0.437	d) Concrete edge	0.062		
e) concrete splitting	n.a.				
f) Concrete blow-out	n.a.				

I. Soil adhesion calculation

In this appendix the soil adhesion variants are calculation in more detail. Below four different situations are calculated with each different input variables. This calculation is discussed in section 2.5.3 and 4.3.1.

SITUATION SKETCH

In Figure 113 a situation sketch is given of the element under tension, but ‘sticking’ to the soil foundation.

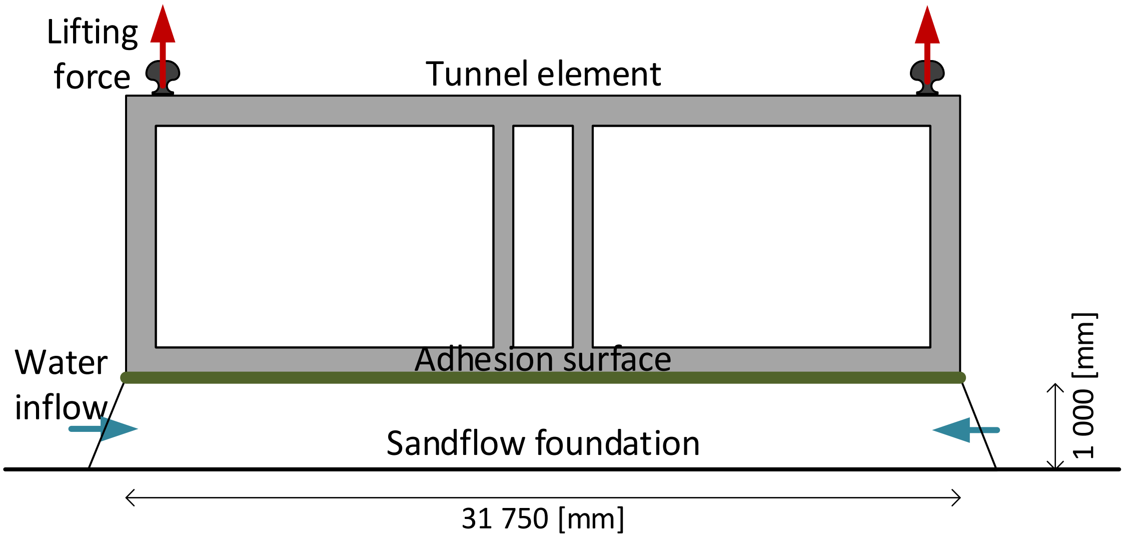


FIGURE 113, SOIL ADHESION SITUATION SKETCH; CROSS SECTIONAL VIEW

METHOD

As mentioned a Darcy flow is assumed and used. Other options are available but discarded already in section 4.3.1.

$$\sigma_{adhesive} = \frac{F_{breaking}}{L_{element} * W_{element}}$$

$$\Delta h = \frac{\sigma_{adhesive}}{\rho_w}$$

$$l_{flow,average} = \frac{W_{element}}{n_{sides}} * 0.5$$

$$q_{per\ meter} = \frac{t_{foundation} * k * \Delta h}{l_{flow,average}} * n_{sides}$$

$$V_{needed;per\ meter} = t_{breakout} * w_{element}$$

INPUT PARAMETERS

Parameters	Value	Unit	Source
$l_{element}$	95.6	m	(TEC, 1993)
$w_{element}$	31.75	m	(TEC, 1993) & Design choice
$F_{breaking}$	4000	kN	Design choice
$t_{foundation}$	1	m	(TEC, 1993)
ρ_{water}	10.25	kN/m^3	(TEC, 1993)
$t_{breakout}$	5	mm	(TEC, 2017) & (Mei, Yeung, & Liu, 1995)
$Permeability\ k$	0.05	m/day	(Baber & Luniss, 2013)

RESULTS
With these parameters the expected breakout time can be calculated. In Xx the result is shown for all the variants:

Variant	Change in design	Breakout time [hrs]
Standard	n.a.	2025
Improved breakout thickness	0.05 → 0.02	941
Improved soil permeability	0.05 → 2	23.5
Increased tension	4000 → 6400	14.7
Reduced width	31.75 →15.75	1.79

J. Wijkertunnel rotational capacity calculation

In this appendix the new alignment is calculated. First the rotation required with respect to the lowering is calculated. Secondly the GINA seal needed for this rotation is designed.

Rotation required

For the rotation the three options are discussed. For each of these a line-model is constructed as discussed in section 2.6.1 as similar to appendix C. The tunnel consists of 6 elements at which a maximum rotation can occur. Different to the previous mentioned calculated the GINA expansion is kept variable. The result is a dependency of the maximum deepening on the GINA expansion for each of the variants. The result of this is shown in Figure 114.

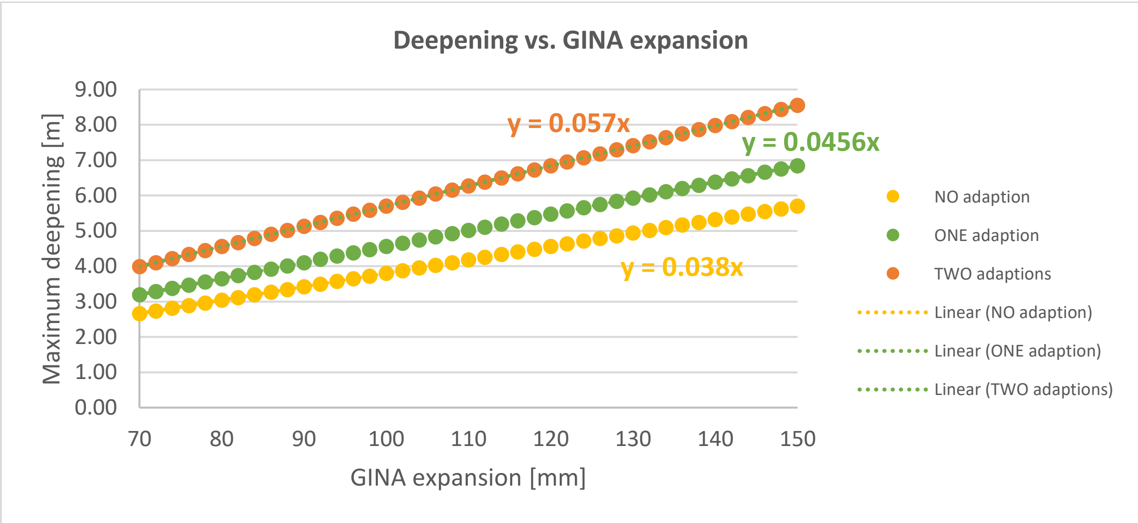


FIGURE 114, DEEPENING VS. GINA EXPANSION

GINA seal design

Here the GINA seal is calculated, based on the loads during immersion and lifetime.

SITUATIONAL SKETCH

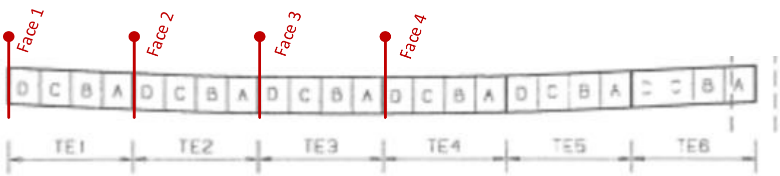


FIGURE 115, WIJKERTUNNEL ELEMENT FACES

METHOD

The calculation is divided in three parts: a general compression of the GINA seal, secondly extra compression and relaxation, thirdly the compression required for the rotation. (this is calculated for each of the element heads/faces as shown in Figure 115). The depth of the GINA seal differs per element so each element is considered.

$$l_{seal} = 2 * (width + height)$$
$$Face\ force = A_{tunnel} * \rho_{water} * d_{average}$$
$$\Delta T_{compression} = \alpha * L_{tunnel} * (T_{max} - T_0)$$

INPUT PARAMETERS

Parameters	Value	Unit	Source
$\boldsymbol{l_{element}}$	95.6	m	(TEC, 1993)
$\boldsymbol{h_{seal}}$	7.995	m	"
$\boldsymbol{w_{seal}}$	29.75	m	"
$\boldsymbol{Expansion\ coeff.}$	0.000001		"
$\boldsymbol{T_{max}}$	16.5	˚C	"
$\boldsymbol{T_{min}}$	-3.5	˚C	"
$\boldsymbol{\rho_{water}}$	10.25	kN/m^3	"

RESULTS

Results 1st section

GINA location	Old force [kN/m]	Deepened force [kN/m]
Face 1	390	390
Face 2	532	627
Face 3	673	859
Face 4	734	954

Results 2nd & 3rd section

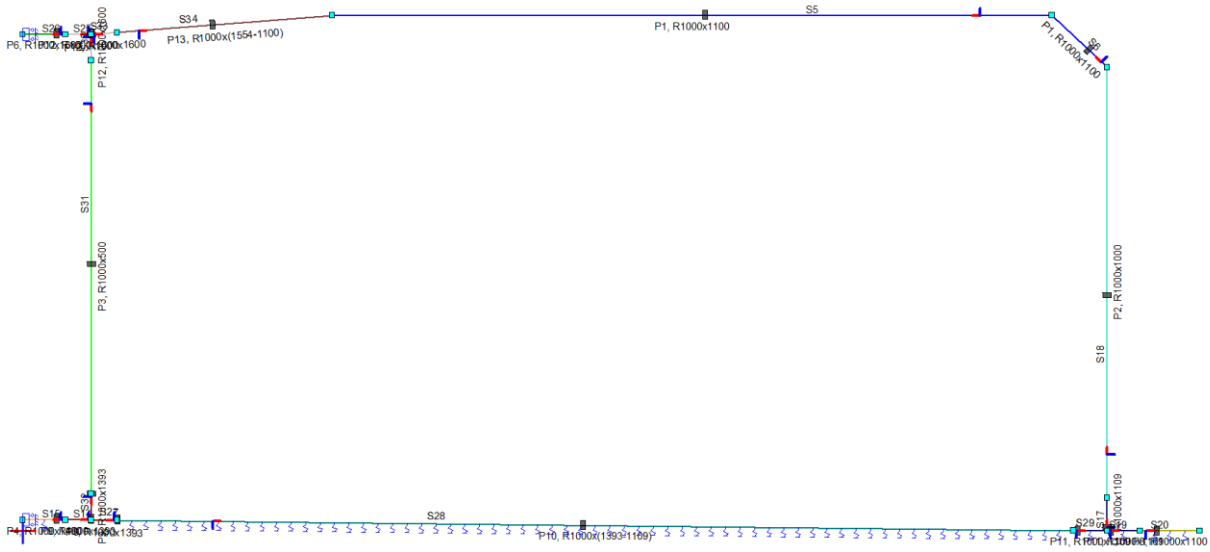
	Compression	Relaxation	
Construction uncertainty	6	-6	mm
Misplacement	2	-8	mm
Settlements	1.7	-1.7	mm
Closure joint	0	-2.5	mm
Temperature	6.2	-12.9	mm
Rotational extension	0	63.96	mm

K. Final position model

In this appendix a cross-sectional model is constructed and the higher internal forces due to increase in water pressure are calculated. This is based on an increasing lowering of the element and therefore an increasing variable water load. The model is constructed in MatrixFrame, then all the different loads are applied. Finally, the internal forces are shown regarding the water level increase and soil cover thickness reduction.

Cross-section in MatrixFrame

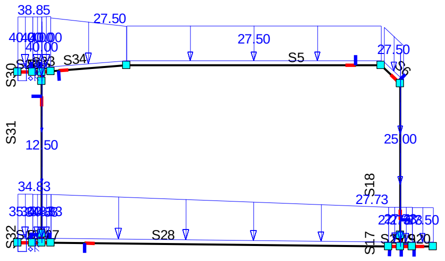
The model in MatrixFrame is as follows:



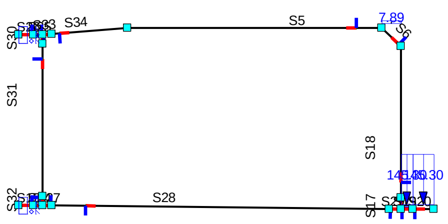
Loads

The following loads are applied (according to section 3.3.1):

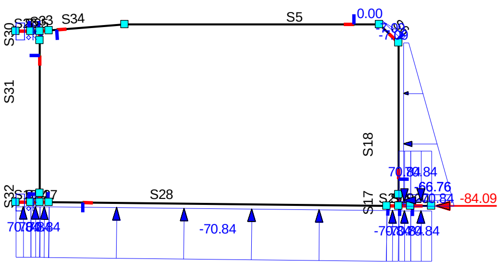
Self-weight



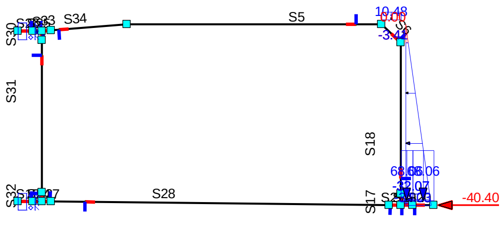
Soil settlement adhesion load



Permanent water load

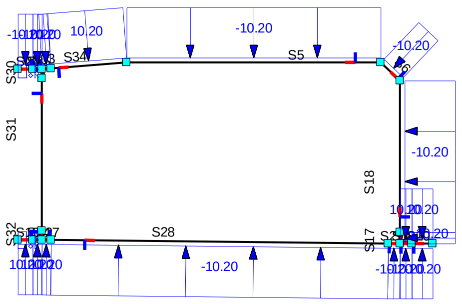


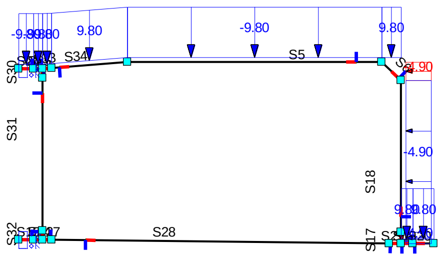
Permanent soil load



Variable water load

Variable soil load





Internal forces

The resulting internal forces are:

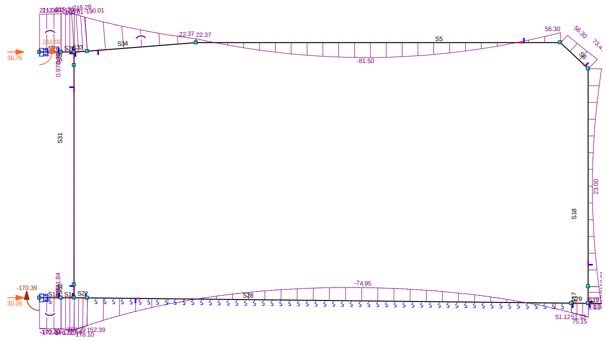


FIGURE 116, MOMENT CHANGE; WATER INCREASE (PER METER)

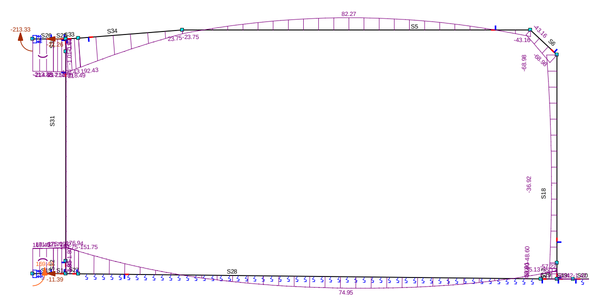


FIGURE 117, MOMENT CHANGE; COVER DECREASE (PER METER)

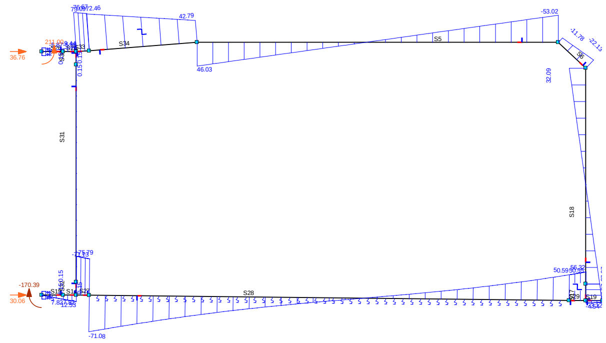


FIGURE 118, SHEAR CHANGE; WATER INCREASE (PER METER)

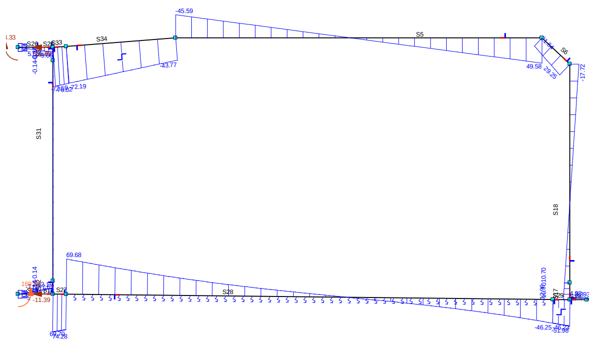


FIGURE 119, SHEAR CHANGE; COVER DECEASE (PER METER)

L. Prestressing model set-up

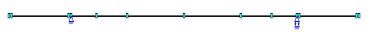
In this appendix the prestressing model details are discussed. The MatrixFrame model input is shown together with all the load combinations and resulting internal forces.

MatrixFrame model

The following parameters are used to construct the model. This is followed by all the different loads applied on the model.

Parameter	Value	Unit	Formula	Description
$\mathbf{A_{concrete}}$	91.43	m^2	(Burggraaf, Overbeek, & Vervuurt, 2007)	Cross-section total area
$\mathbf{I_{xx}}$	915.81	m^4	″	2nd moment inertia xx
$\boldsymbol{E_{concrete}}$	32 000 000	kN/m^2	(TEC, 1993)	Concrete Youngs modulus
$\boldsymbol{l_e}$	95 600	mm	(TEC, 1993)	Element length
$\boldsymbol{l_s}$	23 900	mm	(TEC, 1993)	Element length
$\boldsymbol{w_e}$	31 750	mm	″	Element width

TABLE 39, PRESTRESSING MODEL SET-UP TUNNEL PARAMETERS



Load combination A

This combination describes the element just after it is released from the riverbed. It is still immersed deep. The variance in density in concrete results in big differences in downward loads. Two situations are discussed (case A.1 and A.2), a heavy element and light water and vice versa. The load of the self-weight acts over the entire element.

$$q_{self\_min} = \rho_{concrete,min} * A_{concrete} = 2213.9\ [kN/m]$$
$$q_{self\_max} = \rho_{concrete,max} * A_{concrete} = 2277.9\ [kN/m]$$

The weight of the bulkheads is based on the same density of concrete as the structural concrete. However, the change of the density does not have a significant impact on this amount on concrete, the average density is used. The thickness of these bulkheads is assumed to be 0.3 meter and it is assumed to be a point load acting 0.7 meter from the end of the element.

$$F_{bulkhead} = t_{bulkhead} * \rho_{concrete,ave} * A_{bulkhead} = 1124.8[kN]$$

The buoyancy force is equal to the weight of the displaced water. Like the self-weight two situations are discussed. This loads act from the starting point of the bulkhead to the end of the bulkhead (also assumed to start acting after 0.7 meter).

$$q_{buoyancy,min} = \rho_{water,min} * A_{element} = 2440.1\ [kN/m]$$
$$q_{buoyancy,max} = \rho_{water,max} * A_{element} = 2501.1\ [kN/m]$$

The forces in the lifting wires are 1000 $[kN]$ per wire. For each support two wires are located

$$F_{support} = n * F_{wire} = 2000\ [kN]$$

The weight of the ballast tanks is variable and results in a vertical balance. The tanks are assumed distribute the load evenly over the entire tunnel width. The density of the ballast water is equal to the density used in the buoyancy calculation. The exact dimensions of the ballast tanks are variable but are split in two locations, the acting length is similar for the minimum and maximum tanks. Note that as discussed in section 4.1.1 some of the ballast concrete stays in place, this is located beneath the ballast tanks. The value calculated below is a summation of the remaining concrete and the ballast tanks.

$$F_{ballast\ tanks,min} = \sum F_{v\,heavy\ element,light\ water} = 13\ 841\ [kN]$$
$$F_{ballast\ tanks,max} = \sum F_{v\,light\ element,heavy\ water} = 25\ 705\ [kN]$$

The maximum tanks are assumed to be 10-meter-wide in both tubes. And a length of 16 meters per tank

$$q_{ballast\ tanks,min} = \frac{F_{ballast\ tanks,min}}{32} = 432.5\ [kN/m]$$
$$q_{ballast\ tanks,max} = \frac{F_{ballast\ tanks,max}}{32} = 803.3\ [kN/m]$$

On the face of the element a normal force acts due to the water pressure, for this the average water density is used. The sum of this normal force does not act at the centre of gravity of the element. This results in the following extra forces (cross-section assumed to be square shape). The eccentricity of the 'triangle shape part' is equal to $\frac{1}{6}h = 1.341\ [m]$.

$$N_{face} = \frac{\rho_{water} * (h_{top} + h_{bottom})}{2} * A_{face} = 271.2 * A_{face} = 66\ 192\ [kN]$$
$$M_{face} = \frac{\rho_{water} * h_{element}}{2} * A_{face} * e = 13\ 330\ [kNm]$$

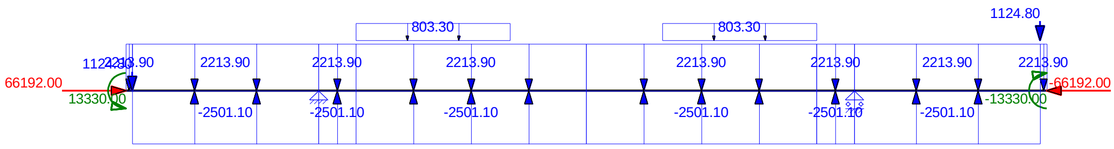


FIGURE 120, PRESTRESSING MODEL SET-UP LOAD COMBINATION A

Load combination B

This combination is like the previous, but the element is not yet released from the riverbed. A total hoisting force of $F_{hoisting} = 12\ 000\ [kN]$ is present. Compared to the previous an extra load is applied, this force acts over the entire tunnel element:

$$q_{adhesion} = \frac{F_{hoisting} - F_{support}}{l_{element}} = 83.68\ [kN/m]$$

The extra hoisting locations for breaking the adhesion are represented by two point loads, located 10 meters from the middle.

$$F_{hoisting,extra} = 2 * 1500 = 3000\ [kN]$$

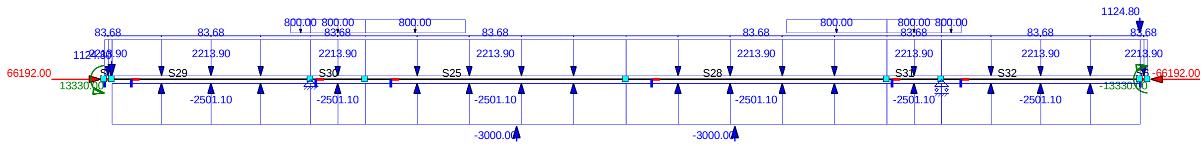


FIGURE 121, PRESTRESSING MODEL SET-UP LOAD COMBINATION B

Load combination C

This load combination describes the tunnel just before breaking out of the water. All the loads are the same as in combination A except the normal force at the face. This is reduced to:

$$N_{face} = \frac{\rho_{water} * h_{element}}{2} * A_{face} = 9\,940\ [kN]$$

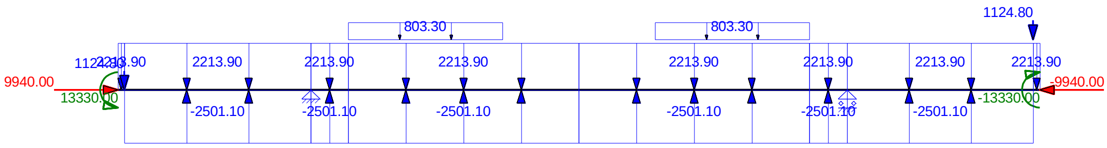


FIGURE 122, PRESTRESSING MODEL SET-UP LOAD COMBINATION C

Load combination D

The element is now balanced in the water. The buoyancy force and the lifting force are removed from Matrix frame. This is replaced by an elastic foundation; the compression of the spring represents the depth of the element. The spring constant can be calculated by multiplying the element width by the water density (giving 2 situations for the minimum and maximum)

In this calculation the width at the top of the element should be used instead of the maximum width (due to the ears at the bottom). To compensate for these ears a small buoyancy force is introduced of:

$$q_{ears,min} = (0.75 * 1.1 * \rho_{water,min}) * 2 = 16.5\ [kN/m]$$
$$q_{ears,min} = (0.75 * 1.1 * \rho_{water,max}) * 2 = 16.9\ [kN/m]$$
$$K_{min} = 30.25 * \rho_{water,min} = 302.5\ [kN/m^3/m]$$
$$K_{max} = 30.25 * \rho_{water,max} = 310.1\ [kN/m^3/m]$$

The forces in the ballast tanks are set such that the freeboard is in the range $0.195 - 0.205\ [m]$, which means a spring compression of $7.84 - 7.85\ [m]$

The ballast loads for heavy element situation are

$$q_{ballast} = 158\ [kN/m]$$

The ballast loads for the light element situation are

$$q_{ballast} = 526\ [kN/m]$$

For the forces on the face it is assumed the element is emerged 0.2 meter out of the water.

On the face of the element a normal force acts due to the water pressure, for this the average water density is used. The sum of this normal force does not act at the centre of gravity of the element. This results in the following extra forces (cross-section assumed to be square shape). The eccentricity of this force equals $\frac{1}{2} h_{element} - \frac{1}{3} h_{immersed} = 1.408\ [m]$

$$N_{face} = \frac{\rho_{water} * h_{immersed}}{2} * (A_{face} - 0.2 * W_{e,top}) = 271.2 * A_{face} = 9\,404\ [kN]$$
$$M_{face} = N_{face} * e = 13\,236\ [kNm]$$

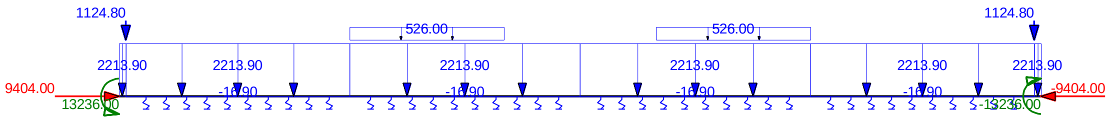


FIGURE 123, PRESTRESSING MODEL SET-UP LOAD COMBINATION D

<u>Load combination E</u>

For this load combination nothing differs from the load combination D except for an incidental wave load. This load is essentially a water elevation in a sinusoidal shape. The Wijkertunnel is in the Noorzeekanaal and is not expected to be transported over sea. This canal is a zone 3 waterway, having a maximum significant wave height of 0.6 meters. (Europees Parlemenent & Raad, 2006).

For ease of use this sinusoidal shape is transformed in a trapezium shape with the same are (see Figure 124, Sinusoidal to Trapezium).

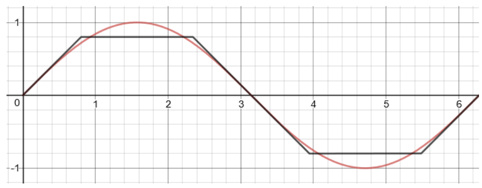


FIGURE 124, SINUSOIDAL TO TRAPEZIUM

The worst governing load is a wave with the wavelength double the element length. In this case the wave can be described as $A\sin(BX) = 0.6\sin\left(\frac{2\pi}{191.2}x\right)$. The trapezium has a kink at $x\!: \frac{0.95261}{B} \approx 29\ [m]$ and $y\!: 0.95261 * A = 0.572\ [m]$

The width on which this force acts is $30.25\ [m]$.

$$q_{wave,min} = h_{wave} * \rho_{water,min} * w_{e,top} = 173.0\ [kN/m]$$
$$q_{wave,max} = h_{wave} * \rho_{water,max} * w_{e,top} = 177.3\ [kN/m]$$

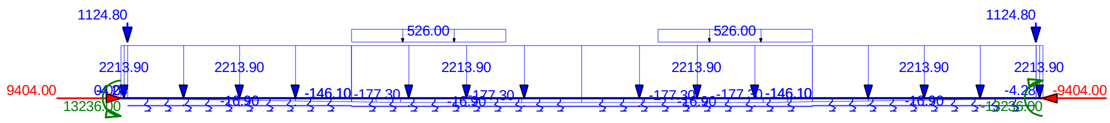


FIGURE 125, PRESTRESSING MODEL SET-UP LOAD COMBINATION E

<u>Load combination F</u>

This load combination is the same as load combination E but instead of a water elevation the wave results in a water level decrease

All the loads modelled in Matrix Frame are shown in appendix L. For ease of use the ballast tanks are placed in a different load group with a load of $q = 1000\ [kN/m]$ using load combinations the preferred ballast tank weight is achieved.

Load case	Ballast factor	Load case	Ballast factor
A.1	0.8033	A.2	0.4325
B.1	0.8033	B.2	0.4325
C.1	0.8033	C.2	0.4325
D.1	0.526	D.2	0.158
E.1	0.526	E.2	0.158
F.1	0.526	F.2	0.158

TABLE 40, LONGITUDINAL MODEL; BALLAST FACTORS

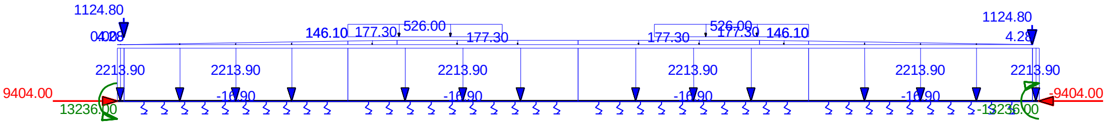


FIGURE 126, PRESTRESSING MODEL SET-UP LOAD COMBINATION F

Resulting envelope A.1 – C.2

The following internal forces occur according to load case A.1 to C.2 for an unknown position of the ballast tanks:

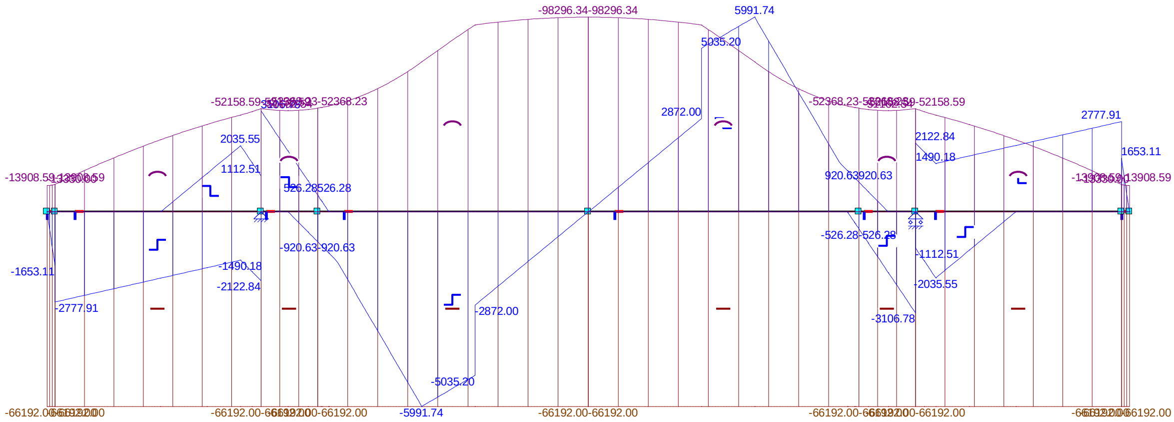


FIGURE 127, PRESTRESSING MODEL RESULTING ENVELOPE CASE A.1-C.2

Resulting envelope D.1 + E.1 + F.1

The following internal forces occur according to load case D.1, E.1 and F.1 for an unknown position of the ballast tanks:

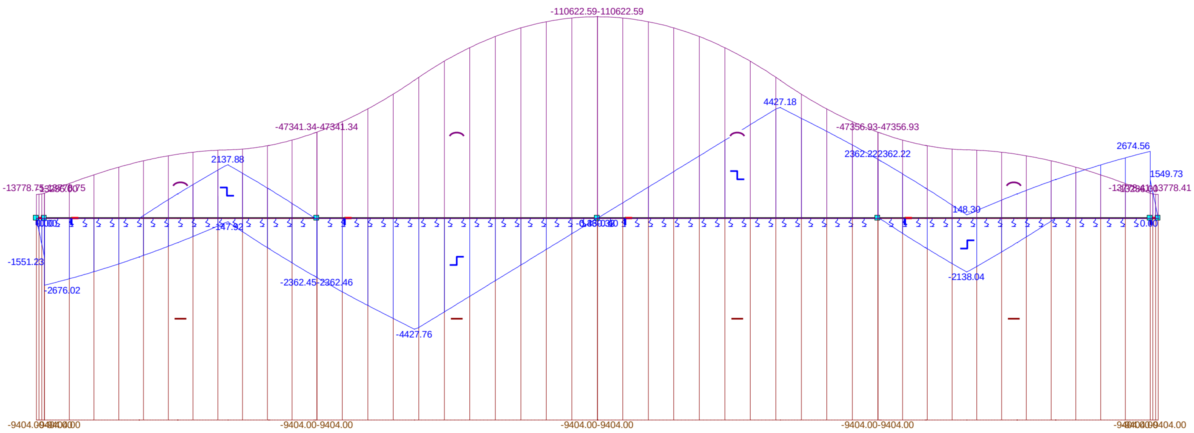


FIGURE 128, PRESTRESSING MODEL RESULTING ENVELOPE CASE D.1+E.1+F.1

Resulting envelope D.1 + E.1 + F.1

The following internal forces occur according to load case D.2, E.2 and F.2 for an unknown position of the ballast tanks:

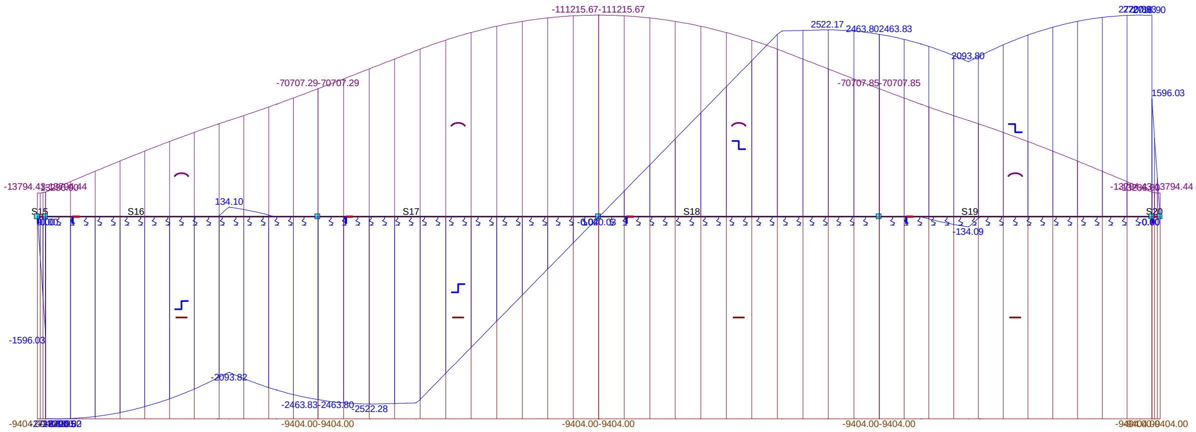


FIGURE 129, PRESTRESSING MODEL RESULTING ENVELOPE CASE D.2+E.2+F.2

Model variant 1

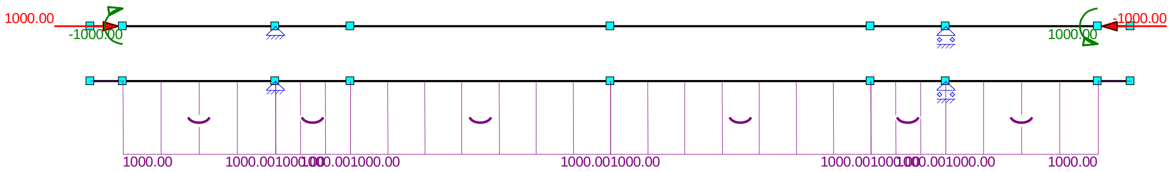


FIGURE 130, PRESTRESSING MODEL VARIANT 1 INPUT

Model variant 2

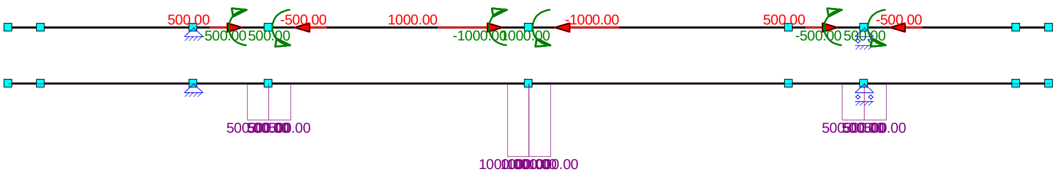


FIGURE 131, PRESTRESSING MODEL VARIANT 2 INPUT

Model variant 3

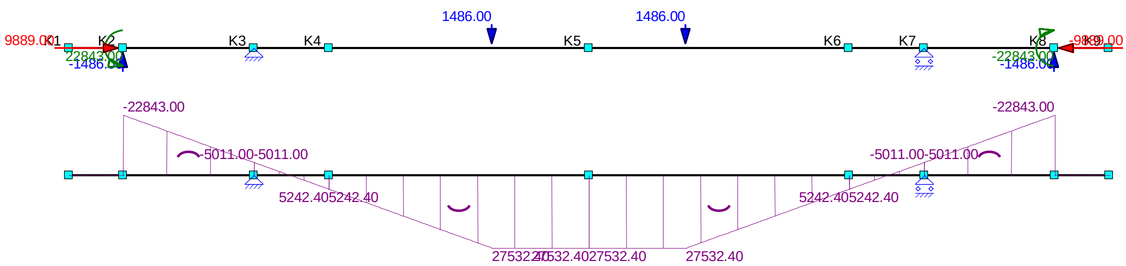


FIGURE 132, PRESTRESSING MODEL VARIANT 3 INPUT

Model variant 4

-

M. Prestressing calculation

In this appendix the calculation for selection the optimal prestressing lay-out and the verification is done

SITUATION SKETCH

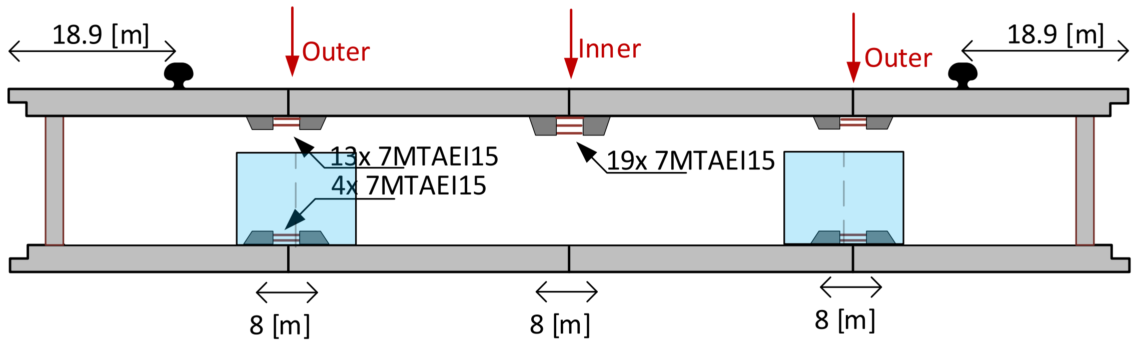


FIGURE 133, POST-TENSIONED PRESTRESSING LAY-OUT; LONGITUDINAL VIEW

METHOD

Different locations of the ballast tanks are used to create internal forces based on the load combinations. These are used as input in all the variants. The other part of the input is amount of prestressing wires used (and therefore the maximum amount of tension in a single wire). This maximum value is supplied by the prestressing wires producer. The relaxation as discussed in 2.3.5 is subtracted. The output of these calculations is the number of wires needed and a total length of the wires, based on the optimal location of the ballast tanks.

Variant 1; Connecting outer segments

The prestressing force is introduced. A number of prestressing wires is given for both the floor and the roof. The eccentricity of these wires is discussed above. These force from these wires are translated in a force P1 and P2. The force P2 is acting at the centre of the cross-section. P1 is an eccentric force, acting at the given eccentricity.

$$M_p = P_1 * e_{P1}$$

Then the resulting internal forces are combined with the prestressing input.

$$M_{total} = M_{model} + M_p$$
$$N_{total} = N_{model} + N_p$$

Using the cross-sectional parameters the resulting stresses in the outer fibres are calculated using the following equation:

$$\sigma_{top} = \frac{M_t}{W_t} + \frac{N}{A} \text{ and } \sigma_{bot} = \frac{M_b}{W_b} + \frac{N}{A}$$

This value can be checked according to the design requirements

Variant 2; Connecting each joint

The calculation for variant 2 is essentially the same as for variant 1. The only difference being that de calculation is made for each individual segment joint. These inner or outer joints also differ in amount of prestressing wires.

Variant 3; Bend tendons

In this variant a single layer of tendons is used, however this layer is connected to both the roof and the floor. These angled tendons lower the effective normal force and introduces point loads at the locations of the kinks.

$$\alpha = \tan^{-1}\left(\frac{height}{kinked\ length}\right)$$
$$N_p = P * \sin(\alpha)$$
$$V_p = P * \cos(\alpha)$$
$$V_{total} = V_{model} + V_p$$

The maximum allowed shear force is set at $V_{Rd} = 9\ [MN]$.

INPUT PARAMETERS

Parameter	Value	Unit	Source
$M_{roof;model}$	Variable	kN	MatrixFrame
$M_{floor;model}$	Variable	kN	MatrixFrame
N_{model}	Variable	kN	MatrixFrame
V_{model}	Variable	kN	MatrixFrame
z_{roof}	3.085	m	(TEC, 1993)
z_{floor}	-2.760	m	ʺ
$A_{concrete}$	91.43	m^2	ʺ
I_{xx}	915.81	m^4	ʺ
e_{roof}	2.785	m	Design choice
e_{floor}	-2.310	m	Design choice
$F_{p;max}$	Variable	kN	Design choice

FIGURE 134, PRESTRESSING CALCULATION; INPUT PARAMETERS

RESULTS

The complete result is an enormous excel sheet for with values for each range of variables. Below only the results used above is shown; being variant 2 with 7MTAEI15 wires.

MatrixFrame internal forces	Load case A.1 - C.2		Load case D.1 – F.2	
	Outer	Inner	Outer	Inner
Moment roof [kNm]	-56 368	-98 296	-69 799	-109 197
Moment floor [kNm]	-6 067	-35 559	-4 486	-34 920
Normal force [kN]	-9 940	-9 940	-9 404	-9 404
Shear force [kN]	2 083	737	3101	1079

FIGURE 135, PRESTRESSING CALCULATION; MATRIXFRAME INTERNAL FORCES

The resulting unity checks are shown below

Unity check	Load case A.1 - C.2		Load case D.1 – F.2	
	Outer	Inner	Outer	Inner
Roof stresses	0.766	0.749	0.999	0.925
Floor stresses	0.931	0.928	0.984	0.962
Shear forces	0.231	0.082	0.345	0.120

FIGURE 136, PRESTRESSING CALCULATION; UNITY CHECKS SEGMENTS