Technical feasibility of the Tilting Lock
Structural and morphological analysis of the trench required for the Tilting Lock in a case study at the Haringvliet bridge

Author: P.W.M. Heemskerk
Date: November 30, 2016

Thesis committee: Prof. dr. ir. S.N. Jonkman, TU Delft
Ir. W.F. Molenaar, TU Delft
Ir. B.C. van Prooijen, TU Delft
Ing. C. Poldervaart, Royal HaskoningDHV
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by

P.W.M. Heemskerk

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Student: P.W.M. (Pim) Heemskerk
Menno ter Braaklaan 1, 2624 TA Delft
Student ID: 4097394
E: Pim_heemskerk@hotmail.com
T: +31 (0)6 24 44 73 09

Thesis Committee: Chairman:
Prof. dr. ir. S.N. Jonkman
Delft University of Technology
E: S.N.Jonkman@tudelft.nl
T: +31 (0)15 278 52 78

Daily supervisor:
ir. W.F. Molenaar
Delft University of Technology
E: W.F.Molenaar@tudelft.nl
T: +31 (0)15 278 94 47

ir. B.C. van Prooijen
Delft University of Technology
E: B.C.vanProoijen@tudelft.nl
T: +31 (0)15 278 59 70

ing. C. Poldervaart
Royal HaskoningDHV
E: Carolus.Poldervaart@rhdhv.com
T: +31 (0)10 286 55 36

Institutions: University:
University of Technology Delft
Faculty of Civil Engineering and Geosciences (CEG)
Stevinweg 1, 2628 CN Delft
T: +31 (0)15 278 98 02
W: www.tudelft.nl

Company:
Royal HaskoningDHV
Infrastructure; Rotterdam
George Hintzenweg 85, 3068 AX Rotterdam
W: www.royalhaskoningdhv.com
Preface

This Master’s thesis was written as part of the Master curriculum of Hydraulic Engineering with a specialisation in Hydraulic structures. Hydraulic engineering is a Master track of Civil Engineering at the Delft University of Technology. The research was carried out in collaboration with the Faculty of Civil Engineering and Geosciences and in collaboration with the Dutch engineering company Royal HaskoningDHV (RHDHV).

In my search for a satisfying graduation topic, I have considered many subjects. Due to the press release of RHDHV and the video The Tilting Lock - YouTube, my attention was drawn towards the Tilting Lock. The simplicity and ingenuity of the Tilting Lock are very interesting, but the implementation of the Tilting Lock at an existing location caught the majority of my interests. Eventually, I have chosen to elaborate on the feasibility of the Tilting Lock, although this was not a specified graduation topic in advance.

My gratitude goes out to RHDHV, for supporting me in my ideas to elaborate on the Tilting Lock. I would like to thank ing. C. Poldervaart in particular, for being my supervisor and for giving me the chance to graduate on this topic. In addition, I am grateful that RHDHV provided me with a pleasant working environment and the resources that were needed to complete my Master’s thesis.

Secondly, I want to thank my daily supervisor from the University of Technology Delft; ir. W.F. Molenaar for helping me through all the stages of graduation. Due to his advice and the pleasant meetings, I was able to write this thesis. I would also like to acknowledge my other two committee members from the University of Technology Delft; prof. ir. S.N. Jonkman for chairing my committee and for the critical advice and ir. B.C. van Prooijen for the help related to the hydrodynamic and morphological challenges in my research.

Lastly, a more personal note. I like to express my gratitude to my parents for providing the opportunities to achieve my goals and for their support during my entire study. I would like to thank my friends for their support and motivation, especially Lindsey van der Wijden, as she was always willing to provide me with her educative feedback.

P.W.M. Heemskerk
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Summary

The Tilting Lock is an innovative solution for locations where road traffic crosses marine traffic overhead by a bridge. The occurrence of traffic jams at a bridge during the passage of vessels can be reduced by the construction of a Tilting Lock. By 'submerging' the vessels in the Tilting Lock underneath the bridge, the Tilting Lock increases the available air draft for vessels. Due to the Tilting Lock, vessels can pass a fixed bridge span underneath without hindrance for the road traffic on top of the bridge.

Several studies have been performed on the feasibility of the Tilting Lock. These studies focused among other subjects, on the layout of the several cross sections of the Tilting Lock, the energy consumption of a tilting motion and the costs and benefits of the Tilting Lock. This thesis elaborates on the feasibility regarding the implementation of the Tilting Lock at a case study location; the Haringvliet bridge. Therefore, the main research question of this thesis is:

What is the technical feasibility of the Tilting Lock with regard to the depth that is required for the implementation of the Tilting Lock at the Haringvliet bridge?

The main challenge of the Tilting Lock is the large depth that is required for the implementation of the Tilting Lock. For the case study at the Haringvliet bridge, it was chosen to work with a size of the Tilting Lock (diameter = 56 meter, added air draft = 12 meter), for which the daily opening of the Haringvliet bridge will not be required anymore. From the available alternatives to provide sufficient depth for the Tilting Lock, it was chosen to elaborate on an open trench in combination with a cofferdam foundation to stabilise the Haringvliet bridge piers, as depicted in figure 1. To assess the feasibility of this case study design (the open trench in combination with the cofferdam foundations), the following three critical design aspects have been elaborated on:

- Stability of the subaqueous slopes of the trench for the Tilting Lock.
- Stability of the Haringvliet bridge piers.
- Sedimentation rate in the trench for the Tilting Lock.

![Figure 1: The selected alternative for providing sufficient depth to the Tilting Lock: an open trench in combination with a cofferdam foundation to stabilise the Haringvliet bridge piers.](image-url)
Stability of the subaqueous slopes of the Tilting Lock trench

The stability of the subaqueous slopes of the trench that is required for the Tilting Lock is of great importance for the stability of the Haringvliet bridge piers. Local instabilities of the slopes could lead to failure of the whole pier foundation. Based a qualitative analysis of the sensitivity to liquefaction and breach flow, an optimum inclination for the slopes of the trench was found in 1:5 (11.3°). This slope inclination was found to be stable in a static analysis of the macro stability.

Stability of the Haringvliet bridge piers

For the assessment of the stability of the Haringvliet bridge piers under the excavations that are required for the Tilting Lock trench, two situations were reviewed. Firstly, it was studied to which depth the subsoil of the Haringvliet can be excavated, without endangering the stability of the pile foundation of the Haringvliet bridge piers. It was found that an excavation of 7 meters below the current bed level is allowed. Within the limits of this excavation, a trench of 24 meters deep can be designed. This trench provides sufficient depth for a Tilting Lock with a radius of approximately 17 meter and an added air draft of 5.8 meter, which is smaller than the required size Tilting Lock for the case study. Therefore, it was concluded that the open excavation around the bridge piers is insufficient to be a satisfying design for the case study.

Secondly, it was elaborated on the feasibility of a cofferdam as a stabilising measure for the Haringvliet bridge piers. The cofferdam encloses the soil body between the foundation piles of the Haringvliet bridge piers, which prevents the loss of bearing capacity of the foundation. It was found that the retaining walls for the cofferdam require a length of approximately 50 meters and a section modulus of $\approx 36.500 \, \text{cm}^3/\text{m}$. Based on the required section modulus, the combi wall can be made out of 1 meter thick king pile elements (figure 2). Therefore, it was concluded that the cofferdam foundation is a feasible concept for stabilisation of the Haringvliet bridge piers in the trench for the Tilting Lock.

Sedimentation rate in the trench for the Tilting Lock

The sedimentation in the trench can reduce the available depth and therefore endanger the required UKC for the Tilting Lock. The amount of sedimentation in the trench depends on the amount of deposition and erosion of sediment particles, which both depend on the local flow velocities. Based on an analysis of the energy losses along a streamline through the trench for the Tilting Lock, the local flow velocities through the trench were determined. Depending on the water depth in the trench and the discharge through the Haringvliet estuary, the flow velocities in the trench will be $\approx 70$-$80\%$ of the flow velocities in undisturbed areas of the Haringvliet. It was expected that the sedimentation rate in the trench for the Tilting Lock ($\approx 0.07\, \text{m/} \text{year}$) is ten times larger than in the undisturbed areas of the Haringvliet ($\approx 6.5\, \text{mm/} \text{year}$ [Mulder et al., 2010]). Therefore, it was concluded that maintenance is required on the required depth in the trench for the Tilting Lock.

Conclusions

It was concluded that the technical feasibility of the Tilting Lock at the Haringvliet bridge is not limited by the depth required for the placement of the Tilting Lock and the accompanying measures that are needed for the stability of the Haringvliet bridge. Based on the case study design of the Tilting Lock trench and the cofferdams around the piers of the Haringvliet bridge, it was demonstrated that the implementation of the Tilting Lock in an existing situation will be possible.

The current problems at the Haringvliet bridge, due to the delays in travel time for both the road and marine traffic, are not considered to be sufficient to investigate in potential improvements of the existing situation [Schultz van Haegen, 2016], like the implementation of the Tilting Lock. In addition, the Tilting Lock will not provide a complete conflict free traffic junction, but will only eliminate the daily opening of the Haringvliet bridge. After the implementation of the Tilting Lock, the movable bridge part still has to open to let very large vessels pass. Therefore, it was concluded that the benefits of the Tilting Lock are not yet sufficient to justify the costs of the implementation.
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Chapter 1

Introduction

The Tilting Lock is a solution for existing traffic junctions, where marine traffic intersects with a bridge for road traffic. The available head clearance for the passing vessels at existing bridges is regularly insufficient underneath the fixed spans. Therefore, the opening of a movable bridge part is required to let the vessels pass. The opening of a bridge leads to queueing of the road traffic on top of the bridge. By placing the Tilting Lock underneath an existing bridge (figure 1.1), the available air draft for the vessels will be increased, since the vessels can be submerged to pass the bridge underneath. The Tilting Lock results in a less frequently opening of a bridge and therefore to fewer delays in the travel time for both marine and road traffic.

In The Netherlands, more than a thousand bridges are present, which in most cases connect two banks of a waterway [Rijkswaterstaat, 2007]. The solution of the Tilting Lock is interesting for bridges that accommodate road connections of national importance (highways), but require a frequent opening due to marine traffic with too much air draft to pass the fixed spans of the bridge. The water crossed by a bridge should be sufficiently wide, to prevent that the Tilting Lock would become an obstacle for marine traffic that is small enough to pass the bridge without using the Tilting Lock.

1.1 Problem definition

The main advantage of the Tilting Lock as a locking concept is the energy consumption during the operational phase. The required energy to rotate the Tilting Lock is relatively low (≈ 0.04 kWh [Royal HaskoningDHV, 2014b]), due to the circular shape of the Tilting Lock. Sufficient space is available on top of the Tilting Lock for solar panels to generate energy, so the operational phase of the Tilting Lock can be completely energy neutral [Witteveen and Wolsen, 2015].

The main disadvantage of the Tilting Lock is the draft of the Tilting Lock, which is related to the circular shape of the hull. The required water depth for the Tilting Lock, which is depending on the desired size of the Tilting Lock, is often not available at the selected locations. The adjustments required for implementing the Tilting Lock at a specific location endanger the feasibility of the Tilting Lock.
Previous studies have focused on several aspects of the layout of the Tilting Lock and the economic feasibility of the Tilting Lock [Royal HaskoningDHV, 2014b]. The Tilting Lock was considered to be a valuable and innovative design, which could contribute to the highway system. Several parties, such as Rijkswaterstaat (Department of public works) (RWS), showed interests in the development of the Tilting Lock. Nevertheless, the interests by RWS stocked, because of the absence of economical attractive locations for the Tilting Lock. In addition, there was a lack of an accurate and reliable cost estimation of the integration of the Tilting Lock at a specific location.

1.2 Research objectives and scope

The objective in this master thesis was to perform a feasibility study on the concept of the Tilting Lock. The integration of the Tilting Lock in the surroundings was considered to be the critical aspect for the technical feasibility of the Tilting Lock. Therefore, the main objective in this thesis was to check the feasibility of the Tilting Lock, by elaborating on the integration of the Tilting Lock at a case study location.

The Haringvliet bridge was selected to serve as case study location for this thesis. The Haringvliet bridge is passed on a daily base, by vessels with air drafts up to 25 meter while the available air draft at the fixed spans is only 13 meters at maximum. A Tilting Lock, required to transfer the normative vessels underneath the Haringvliet bridge, has a draft of approximately 34 meter. The available water depth at the Haringvliet bridge is limited to $\approx 10$ meters. Therefore, the feasibility of the Tilting Lock at the Haringvliet bridge will depend on the feasibility of reaching the required depth for the Tilting Lock.

The required depth for the Tilting Lock could be obtained in several ways. From the multiple alternatives available, it was chosen in this thesis to elaborate on an open trench to accommodate the Tilting Lock. Within the design of the trench, it was elaborated on the stability of the subaqueous trench slopes and the stability of the Haringvliet bridge piers.

In addition to the geotechnical challenges of the integration of the Tilting Lock, the preservation of sufficient Under Keel Clearance (UKC) for the Tilting Lock was studied. The objective was to determine whether sedimentation in the trench for the Tilting Lock is likely to endanger the serviceability of the Tilting Lock.

In addition to the technical feasibility, it was intended to refine the economic feasibility of the Tilting Lock by estimating the costs for the integration of the Tilting Lock.

1.2.1 Delimitation of the scope

The scope of this thesis study on the integration of the Tilting Lock at the case study location of the Haringvliet bridge was limited by various assumptions. The most important assumptions for the scope of this thesis, are discussed in the following paragraphs. The more detailed assumptions on the scope of the thesis are discussed in the associated paragraphs.

Material and shape of the Tilting Lock

In previous studies, different materials and shapes for the Tilting Lock were considered [Witteveen and Wolfsen, 2015]. The steel version of the Tilting Lock was considered to be favourable over the concrete version. This was mainly because of the smaller amount of ballast material that is required to locate the point of rotation of the Tilting Lock just below the center.

For non-circular shapes of the hull of the Tilting Lock, problems arose with the location of the center of buoyancy, which drives the Tilting Lock back to the neutral position [Witteveen and Wolfsen, 2015]. Therefore, the Tilting Lock considered in this thesis is made of a circular, steel hull. The energy consumption, the optimal positions of the points of rotation and buoyancy and different kinds of shapes of the Tilting Lock were considered to be out of the scope for this thesis.
Fixating structure and driving mechanism of the Tilting Lock

An external structure is required to keep the Tilting Lock in position underneath a bridge. The required fixating structure was considered to be important for the feasibility of the Tilting Lock, due to the large required dimensions. Furthermore, the driving mechanism for the rotating motion of the Tilting Lock was regarded as important. Because both elements were not expected to be critical for the feasibility of the Tilting Lock, they were considered to be out of the scope of this thesis.

Dimensions of the Tilting Lock

The size of the Tilting Lock can be adjusted to obtain each desired difference in air draft for the vessels. For the case study at the Haringvliet bridge, the dimensions of the Tilting Lock were based on the normative vessel that passes the bridge daily. The assumed dimensions are included in table 1.1.

<table>
<thead>
<tr>
<th>Dimensions Tilting Lock</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Length lock</td>
<td>90 meters</td>
</tr>
<tr>
<td>Width water channels</td>
<td>11.2 meter</td>
</tr>
<tr>
<td>Width lock</td>
<td>56 meters</td>
</tr>
<tr>
<td>Depth water channels</td>
<td>5.6 meter</td>
</tr>
<tr>
<td>Required air draft</td>
<td>13 meters</td>
</tr>
<tr>
<td>Tilting angle</td>
<td>$22^\circ$</td>
</tr>
<tr>
<td>Draft lock</td>
<td>34 meters</td>
</tr>
<tr>
<td>Added head clearance</td>
<td>12 meter</td>
</tr>
</tbody>
</table>

Table 1.1: Dimensions of the Tilting Lock for the case study at the Haringvliet bridge.

1.3 Research questions

1.3.1 Main research question

To fulfill the objective of this thesis, the main research question to be answered in this thesis is:

*What is the technical feasibility of the Tilting Lock with regard to the depth that is required for the implementation of the Tilting Lock at the Haringvliet bridge?*

1.3.2 Supplementary research questions

To be able to answer the main research question, several sub-questions were defined. These sub-questions focus on various aspects of the main research question. The following sub-questions were taken into account:

- What is the Tilting Lock and how does it work? (Chapter 2)
- What are the conditions at the case study location of the Haringvliet bridge? (Chapter 3)
- What are the design alternatives for reaching the depth that is required for the Tilting Lock and what are the critical design aspects of an open trench underneath an existing bridge? (Chapter 4)
  - For what slope inclinations are the subaqueous trench slopes not expected to encounter instabilities? (Chapter 5)
  - Till which depth are excavations in the vicinity of the Haringvliet bridge piers allowed without additional measures to stabilise the bridge piers? (Chapter 6)
  - Is a cofferdam a feasible measure for solving instability of the Haringvliet bridge piers in the case of large excavations? (Chapter 7)
  - Till what extent is sedimentation to be expected in the trench for the Tilting Lock? (Chapter 8)
- What are the costs related to the implementation of the Tilting Lock at the Haringvliet bridge? (Chapter 9.2)
- What is the relation between the size of the Tilting Lock, the tilting angle of the Tilting Lock and the added height by the Tilting Lock to the available head clearance for the passing vessels? (Chapter 9.3)
1.4 Report structure

The main structure of the thesis report is depicted in figure 1.2. The subjects related to the research questions are discussed in the main report. The supplementary and detailed information is included in the several appendices. In general, each chapter has one or more appendices with background information.

![Diagram of report structure]

Figure 1.2: Report structure of this thesis.

For e-copies: The words in the blue font are (hyper)links. By tapping one of these links, the document automatically redirects to the source of the links. Links may refer to: figures, tables, equations, sources, webpages or other chapters and sections within this document. The links of web pages are often curtailed with respect to the complete web-page address.
Chapter 2

The Tilting Lock

The concept of the Tilting Lock is discussed in this chapter. A brief history of the development of the Tilting Lock (2.1) is followed by the purpose of the Tilting Lock (2.2). The layout (2.3) and the principle of operation of the Tilting Lock are discussed (2.4), followed by a brief overview of the expected costs and benefits (2.5).

2.1 History of the Tilting Lock

The Tilting Lock was founded by Carolus Poldervaart, a project manager of Royal HaskoningDHV (RHDHV) and part time skipper. He began to think about solutions to reduce the waiting time for yachts with too much air draft to pass fixed bridge spans and came up with the idea of submerging the yachts to pass the bridge underneath. This eventually resulted in the Tilting Lock. Over several years, the idea was further developed and refined, amongst others by students from different educational levels.

In 2014 the Tilting Lock was introduced to the public, which caused a lot of positive reactions [RTL Z, 2014]. In 2015 the Tilting Lock won the audience award of the Vernufteling, an engineering award for innovative ideas of engineering companies [Koninklijk Instituut Van Ingenieurs (KIVI), 2015]. In the autumn of 2016, the Tilting Lock became a part of an exhibition in the Maritime museum in Rotterdam [Maritiem Museum, 2016]. In 2015 two students of the university of applied sciences Windesheim in Zwolle graduated on their research on the practical feasibility of the Tilting Lock regarding the aspects of stability, required energy for a tilting motion, practical design aspects and life cycle costs [Witteveen and Wolfsen, 2015]. Following on previous graduation projects, the Tilting Lock is now the topic of this graduation thesis at the Technical University of Delft.

2.2 Purpose of the Tilting Lock

The Tilting Lock is a new concept to provide for a conflict free junction between road traffic and (very) high vessels at existing bridge connections over relatively wide waters. In the following paragraphs, the alternatives of the Tilting Lock for solving the classical problem at intersections of road and marine traffic are briefly discussed (2.2.1). In addition, the potential locations for the Tilting Lock and are treated (2.2.2).

2.2.1 Alternative solutions for the Tilting Lock

To provide a connection between two banks, multiple alternatives are available, which are discussed in appendix A.3. The most common alternatives are a ferry service, a bridge or a tunnel. A ferry service is undesirable for economic important connections, as it will not be available at all times and have a large impact on the travel time. Permanent road connections provided by a tunnel or a bridge are more satisfactory. A tunnel does not have any limitations on the head clearance for the marine traffic, but a bridge has. When the limited head clearance becomes a problem, one can
implement a movable span in the bridge. When this movable part is opened, the vessels can pass, but the road traffic has to wait. Other alternatives of solving the limited head clearance problem for the marine traffic, are discussed in appendix A.3.

2.2.2 Potential locations for the Tilting Lock

The potential locations for the Tilting Lock should meet certain requirements, which are listed below:

- Sufficient width of the water, to prevent that the Tilting Lock will act as an obstacle for marine traffic that does not need the Tilting Lock to pass the fixed bridge spans.
- The road on top of the bridge should be of sufficient national importance, to maximise the economic benefits of the realisation of the Tilting Lock.
- Regarding the local conditions, the preliminary water depth was considered to be relevant, because the impact of the draft of the Tilting Lock would be much larger in shallow waters.
- The presence of a movable bridge part was considered to be beneficial, but not critical for the selection of a location. If a movable bridge part was absent, the economic benefit of the Tilting Lock was considered to be absent, as no traffic jams related to the opening of the bridge for passing vessels would occur.

In the Netherlands, four locations can be distinguished which meet the previous mentioned requirements\(^1\), see figure 2.1: the Ketel bridge, the Zeeland bridge, the Moerdijk bridges and the Haringvliet bridge. In appendix A.2 these locations are discussed in more detail. Based on the importance of the road connection (part of the national highway \(A29\)) and the frequent opening of the movable bridge part (at least 6 to 10 times a day in the sailing season), the Haringvliet bridge was selected to serve as case study location for this thesis.

2.3 Layout Tilting Lock

The Tilting Lock system, as depicted in figure 2.2, consists of a floating (steel) structure under a fixed bridge span, that is able to rotate around its longitudinal axis. The floating body kept in position by a system of steel piles, also called the "fixating structure". As the Tilting Lock floats, the connection to the fixating structure includes roller bearings is to allow the Tilting Lock to move the fluctuating water levels.

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\(^1\) Besides these locations in The Netherlands, more potential locations were available worldwide. However, these locations were considered to be beyond the scope of the thesis.
2.3. Layout Tilting Lock

The floating body of the Tilting Lock contains two separate water channels, which are closed off by locking gates. The two internal channels can be subdivided into three sections: one on each side of the overhead bridge and a middle section beneath the bridge. Each section of the internal water channels can be entered by one or two vessels, depending on the sizes of the vessels and the tilting position of the Tilting Lock.

The outer hull of the Tilting Lock differs for the three sections. For the lateral cross sections outside the bridge, a general shape (figure 2.3a) and an adjusted shape with a recess for the cross sections underneath the bridge (figure 2.3b). The main difference between the shapes of the two cross sections of the Tilting Lock is the recess in the hull of the Tilting Lock underneath the bridge. Due to the inwards directed wall, it is prevented that the Tilting Lock will collide with the bridge deck. In addition, these walls prevent the vessels from passing the bridge through the upper internal channel. When the vessels are in the lower internal channel, they can sail to the outer side of the bridge without any limitations. The adjusted cross section of the middle section of the Tilting Lock will have some influences on the design, which were not yet taken into account in the previous research. This includes the reduced buoyancy force of the Tilting Lock and the local disturbance of the internal balance of the Tilting Lock [Witteveen and Wolfsen, 2015].

![Figure 2.3: Characteristic cross sections of the Tilting Lock.](image)

The rotational motion of the Tilting Lock is provided by a driving mechanism of two hydraulic jacks. These hydraulic jacks are connected to the fixating structure and the bulkheads of the Tilting Lock. The point of rotation of the Tilting Lock is located beneath the water surface to increase the effect of the tilting motion. When the Tilting Lock is in the neutral position, the water levels of the internal channels are at the same vertical level as the rotation point of the Tilting Lock, see figure 2.3a. In tilted position, the water level of the upper internal channel of the Tilting Lock will be equal to the outer water surface. The difference between the water level of the lower water channel and the outside water is equal to the added air draft by the Tilting Lock, see figure 2.3b. At the upper ends of the circular hull, straight stretches are added to prevent overtopping of water into the Tilting Lock. Additionally, this stretch will prevent overturning of the Tilting Lock in unforeseen situations.

The three sections of the internal water channels of the Tilting Lock have a length of 30 meters, which makes the total length of the Tilting Lock approximately 90 meters (see table 1.1). The 30 meter length gives space to one large or two smaller sailing vessels. The assumed structural gauge for the vessels includes a width of 6 meters and a height of 25 meters above the water level. The internal channels have a radius of 5.6 meter, which is considered to be sufficient to provide for depth for the vessels. The circular shape provides for an equal depth in each tilted position of the Tilting Lock and is beneficial for the balance of the Tilting Lock [Witteveen and Wolfsen, 2015]. The length of the Tilting Lock and the structural gauge for the vessels can be adjusted to the requirements of a specific location, as the principle of operation of the Tilting Lock will not change.
CHAPTER 2. THE TILTING LOCK

Energy consumption of the tilting motion
An advantage of the Tilting Lock in relation to other shipping locks is the relatively low energy consumption of a locking cycle. In normal shipping locks, large amounts of water have to be pumped into the locking chambers to lift and lower the passing vessels. The energy consumption of the Tilting Lock is important, as it is one of the main targets of the RWS is to consume 20% less energy in 2020 in relation to 2009 [Rijkswaterstaat, 2016b].

In normal weather the energy consumption of a single tilting motion of the Tilting Lock was estimated to be 0.17 kWh [Witteveen and Wolfsen, 2015]. Due to the circular shape of the outer hull, no water is displaced during the tilting of the Tilting Lock, which results in a relatively low consumption of energy. Resistance during the motion is caused by friction between water and the steel hull of the Tilting Lock. This friction is negligible for the relative low rotating velocity (≈ 0.1 m/s) [Witteveen and Wolfsen, 2015]. The estimations on the energy consumption did not include the displacement of water at the adjusted cross sectional shape in the middle section of the Tilting Lock.

To be able to assess the complete energy consumption of the Tilting Lock solution, the complete life cycle of the Tilting Lock should be taken into account. Both the construction and the demolition phase of the Tilting Lock will have a significant contribution to the total energy consumption of the Tilting Lock. As the energy consumption of the Tilting Lock was not expected to be critical for the feasibility of the Tilting Lock, it was considered to be beyond the scope of this thesis.

Ballast required to keep the Tilting Lock in submerged position
To keep the Tilting Lock in its submerged position, ballast material is required [Witteveen and Wolfsen, 2015]. It is important to have sufficient space inside the Tilting Lock to place the ballast. For an optimal distribution of the ballast and to have the rotation point of the Tilting Lock in its desired position, the ballast material is placed in the area between the internal channels of the Tilting Lock. The center of gravity (equal to the rotation point) should be located just below the center of the circular hull of the Tilting Lock [Witteveen and Wolfsen, 2015]. In this way, the energy consumption of the Tilting Lock is slightly increased [Witteveen and Wolfsen, 2015]. In addition, the Tilting Lock will automatically return to its neutral position when one of the driving mechanisms fail.

Extreme water levels
As the Tilting Lock is a floating object, it will move in the vertical direction with the changing water levels of the surroundings. In the neutral position, more head clearance is available between the middle section of the Tilting Lock and the underside of the bridge deck than in tilted position. To prevent a collision in case of extremely high water levels, the Tilting Lock will be put in its neutral position.

Additional ballast can be applied as an extra measure to prevent collision between the Tilting Lock and the bridge. The ballast will lower the position of the Tilting Lock relative to the water surface. During extreme low water levels, some ballast of the Tilting Lock can be removed, to raise the Tilting Lock relative to the water surface level to prevent the Tilting Lock from running aground.

2.4 Principle of operation of the Tilting Lock
The principle of operation of the Tilting Lock could be summarised into three situations. The steps required to transfer a single vessel from one side of a bridge to the other side by the Tilting Lock are, briefly explained in the following points.

1. A vessel only will be able to enter the Tilting Lock when the Tilting Lock is in tilted position. Once the vessel has entered the Tilting Lock in the upper internal channel, the Tilting Lock will tilt from one side to the other. With this tilting motion, the vessel is transferred in the vertical direction, also called "submerged".
2. In this submerged position, the vessel will be able to sail through the lower internal channel of the Tilting Lock to the other side of the bridge. In any other position of the Tilting Lock,
2.5. Economical benefits of the Tilting Lock

The vessel will not be able to pass the bridge, due to recesses in the outer walls of the Tilting Lock underneath the bridge.

3. After the passage of the vessels, the Tilting Lock will tilt back to its initial tilted position. The water level of the internal channel is equal again to the outer water surface and the vessels are able to leave the Tilting Lock.

In figure 2.4 a more detailed flow scheme of the operation principle of the Tilting Lock is included. In appendix A.1, a comic is included to depict working principle of the Tilting Lock. For a visualisation of the working principle of the Tilting Lock is referred to the promotional video on YouTube (The Tilting Lock - YouTube), which could be easily found by using the search queries 'Tilting Lock' or 'Kantelsluis'.

The duration of one locking cycle is estimated at approximately 20 minutes [Witteveen and Wolfsen, 2015], see figure 2.4. As could be seen in this figure, multiple vessels are able to use the Tilting Lock at the same time. With a locking cycle of 20 minutes, a total of three charters or six sailing vessels could be transferred through the lock per water channel. As the Tilting Lock operates in both directions at the same time, a total of 6 to 12 vessels can be handled per hour.

![Sequence of the locking process for channels 1) and 2) [Witteveen and Wolfsen, 2015].](image)

Figure 2.4: Sequence of the locking process for channels 1) and 2) [Witteveen and Wolfsen, 2015].

2.5 Economical benefits of the Tilting Lock

The cost and benefits of the implementation of the Tilting Lock at the case study location of the Haringvliet bridge are investigated by RHDHV in a Maatschappelijke Kosten en Baten Analyse (Social costs and benefits analysis) (MKBA). The considered aspects for the costs and benefits of the Tilting Lock are depicted below, which were estimated for a lifetime of 100 years for the Tilting Lock [Royal HaskoningDHV, 2014a].

<table>
<thead>
<tr>
<th>Costs:</th>
<th>Benefits:</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Development and engineering</td>
<td>4. Reduced delays in travel time</td>
</tr>
<tr>
<td>2. Construction of the Tilting Lock</td>
<td>5. Reduced maintenance costs of bridge</td>
</tr>
<tr>
<td>3. Operation and maintenance</td>
<td>6. Touristic attraction</td>
</tr>
<tr>
<td></td>
<td>7. Less waiting time for the shipping traffic</td>
</tr>
</tbody>
</table>

The initial costs of the realisation of the Tilting Lock were estimated on € 60.000.000 (aspects 1 and 2) [Royal HaskoningDHV, 2014a]. In the second aspect, the design and the construction of
CHAPTER 2. THE TILTING LOCK

The measures required for the implementation of the Tilting Lock in its surroundings were included in the estimation of the costs, as rough estimations at € 1,200,000 + 20% of the total realisation cost [Royal HaskoningDHV, 2014a]. The yearly operation costs (aspect 3) were assumed to be 2% of the initial investments on the Tilting Lock [Royal HaskoningDHV, 2014a]. This resulted in a total cost of all aspects together of € 95,613,000 over the lifetime of the Tilting Lock [Royal HaskoningDHV, 2014a].

The benefits of the Tilting Lock mainly consist of the reduced travel time for the road traffic (aspect 4), that is using the Haringvliet bridge\(^2\). The waiting time of the traffic per opening of the bridge is assumed to be 10 minutes [Royal HaskoningDHV, 2014a]. This results in estimations between € 192,577,000 and € 276,479,000 for the economic benefits over the total lifetime of 100 year of the Tilting Lock [Royal HaskoningDHV, 2014a]. With these estimations, the Tilting Lock will be profitable within 12 to 15 year of operation. aspects 5 is not expected to lead to direct benefits, as the Haringvliet bridge still requires maintenance and operation when the Tilting Lock is present. In addition, the Tilting Lock requires an operator as well as the movable bridge part, which even increase the daily expenses of RWS. aspects 6 and 7 are considered to be extra, unquantified, benefits.

2.6 Key design areas of the Tilting Lock

To determine the key design are of this thesis, three research topics related to the Tilting Lock were drawn. In the following paragraphs, these three design areas are briefly discussed. Eventually, it was chosen to elaborate on the depth that is required for the Tilting Lock.

**Internal structural design of the Tilting Lock**

The first topic focused on the structural strength and the shape of the Tilting Lock. In previous studies, a preliminary design of the internal structure has been designed by RHDHV\(^3\), but this was only for a single lateral cross section of the Tilting Lock. It was expected that the challenges in the structural strength are not critical in the feasibility of the Tilting Lock. This was because the structural layout and the dimensions of the Tilting Lock are very well comparable to large container vessels.

**Fixating structure and driving mechanisms for the Tilting Lock**

The second defined area of research is related to the elaboration of the fixating structure and the rotation mechanisms for the Tilting Lock. The main requirements for the fixating structure will be to resist the external forcing and allow movements of the Tilting Lock within certain limits, for instance, to adapt to the fluctuating water levels.

However, the elaboration of the fixating structures and the driving mechanisms for the Tilting Lock depend on the design of implementation of the Tilting Lock in a local environment. In addition, the research on the driving systems was considered to be mainly outside the body of knowledge of a civil engineering student within the specialisation hydraulic structures.

**Depth required for the Tilting Lock**

Due to the circular shape of the Tilting Lock, a large depth is required for the implementation of the Tilting Lock at a specific location. The required depth depends on the size of the Tilting Lock that needs to be implemented. For the case study at the Haringvliet bridge, a water depth of approximately 38 meters would be required, while the available water depth at the Haringvliet bridge is limited to ≈10 meters. Providing the required depth for the Tilting Lock underneath an existing bridge was expected to be the most critical for the feasibility of the Tilting Lock.

\(^2\)RWS does not acknowledge the delays in travel time of road traffic (aspect 4) in front of a closed bridge as an economical loss. The reasoning is that automobilists know when a bridge will be opened, so they adjust their travel schedule. Therefore, no benefits were taken into account according to the current calculation methods.

\(^3\)Internal report RHDHV

\(^3\)No adequate design is available regarding evacuation of people from the Tilting Lock in the case of emergencies.
Chapter 3

Local conditions at the Haringvliet bridge

As an introduction to the case study location, the local conditions at the Haringvliet bridge are discussed in this chapter. The information on the local conditions was gathered to serve as input during the development of the design for the required depth for the Tilting Lock. First the general aspects of the Haringvliet estuary are discussed in 3.1. Subsequently the local conditions are discussed in 3.2. 3.3 elaborates on the conditions and the design of the Haringvliet bridge.

3.1 The case study location at system level

The case study location of the Haringvliet bridge is located in the South-Western delta of the Netherlands, see figure 3.1. The Haringvliet bridge is the border between the Haringvliet estuary and the Hollandsch Diep. The general aspects of these three areas are discussed in the following paragraphs.

3.1.1 Haringvliet estuary

The Haringvliet is a former estuary between the islands Voorne-Putten, de Hoeksche Waard and Goeree-Overflakkee, and is connected to the Rhine - Meuse delta by the Hollandsch Diep. The estuary was created by a storm surge in the 12th century and originates from the sea.

After the occurrence of a flood in 1953, it was decided in the Delta plan to close off the estuary by the Haringvliet Dam and the accompanying Locks. Since 1970, these structures protect the land areas successfully. Discharge sluices were implemented in the Haringvliet dam over a stretch of approximately 1 kilometer. The Haringvliet dam will be closed during high water at sea to reduce the flood risk [Van Leeuwen et al., 2004]. During times of low discharge from the rivers, the dam will be closed as well, to maintain sufficient water depth for navigation [Rijkswaterstaat, 2011a]. During low waters at sea, the sluices are opened to discharge the water from the upstream rivers on the North Sea.

Before the completion of the Haringvliet Dam, both the Hollandsch Diep and the Haringvliet were subjected to significant tidal influences with water level fluctuations up to 2 meters and more.
By closing of the open connection between the estuary and the North sea, a fixed boundary was created between both areas. The Haringvliet estuary changed from a brackish tidal area to a mainly freshwater basin. Due to the constant inflow of fresh water from the upstream rivers into the Haringvliet, the water in the estuary is fresh. The tide was not absent, as the the Haringvliet was still connected to the North Sea through the rivers Spui, Oude Maas and the Nieuwe Waterweg. However, the tidal influences were reduced to fluctuations of approximately 0.3 meters [Rijkswaterstaat, 2011a]. The closing off of the estuary has increased the safety of the hinterland, but also caused a barrier for the ecosystem. For instance, due to the barrier, the migratory fish could not travel between their different areas of living.

**Future: Kierbesluit**

To re-establish the former tidal character of the Haringvliet estuary, the Kierbesluit was introduced, wherein it was decided to partly open the Haringvliet dam in 2018 [Rijkswaterstaat, 2011a]. Partly opening of the Haringvliet Dam will re-invite the salty water into the Haringvliet. Therefore, it will allow fish migration through the Haringvliet dam. The Kierbesluit will re-introduce small amounts of salty water into the estuary, but the border between the fresh and salty water will not cross the line between Middelharnis and the river Spui [Van Leeuwen et al., 2004]. This was required for agriculture and drinking water, to keep the inlets for fresh water available [Rijkswaterstaat, 2011b].

Although significant beneficial effects for the wildlife, the Kierbesluit will not have big influences on the situation of the Haringvliet regarding the discharges and the water levels [Van Wijngaarden and Ludikhuize, 1997]. The effects of the Kierbesluit on the morphology of the area will be limited [Van Wijngaarden and Ludikhuize, 1997].

### 3.1.2 Hollandsch Diep

The Hollandsch Diep is the connection between the Haringvliet and the rivers Rhine and Meuse, which both mouth in the Hollandsch Diep. Like the Haringvliet, the Hollandsch Diep originates from the sea. In the past, the junction of the Haringvliet and the Hollandsch Diep had a third connection to the Volkerak. This river branch was closed off by the construction of the Volkerak locks and the traffic junction Hellegatsplein between 1957 and 1967 [Rijkswaterstaat, 2016a].

The Hollandsch Diep is a part of the main shipping connection between Antwerp and Rotterdam. This route goes through the Dordtse Kil near the Moerdijk bridges, to the Hollandsch Diep and continues southward through the Volkerak Locks [Rijkswaterstaat, 2009]. This shipping route does not pass the Haringvliet bridge.

Due to pollution of the upstream rivers in the past, parts of the soil layers in the Hollandsch Diep and the Haringvliet are contaminated [De Haan and Verwaart, 1987]. The exact locations of the contaminated soils were not obtained. In the Hollandsch Diep a depot is located, where the dredged polluted soil is stored.

### 3.1.3 Haringvliet bridge

The Haringvliet bridge (figure 3.2) connects the islands Goeree Overflakkee and Voorne-Putten and separates the Hollandsch Diep from the Haringvliet. The Haringvliet bridge is part of the national highway A29, with two driving lanes in both directions. For the local traffic a single driving lane in two directions is present at the eastern side the Haringvliet bridge. At the northern abutment of the Haringvliet bridge, a movable bridge span is included to give passage to very high vessels that are not able to pass the Haringvliet bridge underneath the fixed spans.
3.2. Local conditions Haringvliet estuary

<table>
<thead>
<tr>
<th>Haringvliet Bridge</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Building year</td>
<td>1964</td>
</tr>
<tr>
<td>Total length</td>
<td>1220 meter</td>
</tr>
<tr>
<td>Length span (h.o.h.)</td>
<td>106 meter</td>
</tr>
<tr>
<td>Width between piers</td>
<td>94 meter</td>
</tr>
<tr>
<td>Vertical clearance</td>
<td>14 meter NAP</td>
</tr>
<tr>
<td>Number of spans</td>
<td>12</td>
</tr>
</tbody>
</table>

Figure 3.2: Haringvliet Bridge

Traffic situation

The Haringvliet Bridge is part of the main highway route between Rotterdam and Antwerp. The road traffic intensity at the Haringvliet bridge was identified by RHDHV to use in the analysis of the Maatschappelijke Kosten en Baten Analyse (Social costs and benefits analysis) (MKBA) of the Tilting Lock. The average traffic intensity at the Haringvliet bridge is around 46,000 vehicles during a working day [Royal HaskoningDHV, 2014b]. For the prognosis of the traffic intensities in the near future (2030) an intensity of 75,000 vehicles per working day was predicted, from which 22% of the traffic was expected to consist of trucks [Royal HaskoningDHV, 2014b].

The Haringvliet does not play an important role in the transportation of goods by vessels. The main part of the marine traffic at the Haringvliet bridge consists of recreational vessels, which are dominated by sailing yachts. Vessels with too much air draft need to pass the Haringvliet bridge through the movable bridge part, which is situated close to the northern abutment. The vessels with less air draft are allowed to cross the bridge through each gap.

Information about the intensity and the heights of the marine traffic was not available. One of the charters that passes the Haringvliet Bridge on a daily basis has an air draft up to 25 meters [Witteveen and Wolfsen, 2015]. However, based on experiences can be said that tens of yachts are waiting to pass the bridge at times of nice weather in high season [Witteveen and Wolfsen, 2015]. During summer, the Haringvliet bridge opens 10 to 11 times a day. During the spring and the autumn, the Haringvliet bridge opens 6 to 11 times a day, while in the winter the bridge only opens on requests [Watersportvereniging Numansdorp, 2016].

3.1.4 Problem statement case study location Haringvliet bridge

The Haringvliet bridge is closed off for road traffic each hour during the high season for sailing (April to October), to let the marine traffic pass the bridge. The vessels that are not able to pass the Haringvliet bridge underneath the fixed spans, have to wait until the movable bridge part is opened. When the movable part is opened, the road traffic encounter delay in travel time, which can be related to economical losses (see chapter 2.5).

Proposed Solution

Multiple alternatives are available to improve the quality of the traffic intersection between road and marine traffic at the Haringvliet bridge (see appendix A.3). In this thesis, the solution of the Tilting Lock is elaborated on to determine whether this can be a satisfying solution to let vessels pass the Haringvliet bridge without interfering with the road traffic. The main requirements for the solution of the traffic problem are:

- Elimination of the daily opening of the movable part of the Haringvliet bridge.
- No hindrance for the road and marine traffic during the construction of the Tilting Lock.

3.2 Local conditions Haringvliet estuary

The characteristic local conditions of the Haringvliet estuary in the vicinity of the Haringvliet bridge are briefly discussed in the following paragraphs. These local conditions are used as input values for the designs made for the required depth for the Tilting Lock.
CHAPTER 3. LOCAL CONDITIONS AT THE HARINGVLIET BRIDGE

3.2.1 Bathymetry
In figure 3.3a the local bathymetry in the vicinity of the Haringvliet Bridge is depicted. Being a former estuary, the Haringvliet and the Hollands Diep shows very fluctuating bathymetry, with several channels and shallow parts. The depth contour profile of the Haringvliet is drawn in figure 3.3b. As can be seen, the maximum water depth underneath the Haringvliet bridge is not more than -11 meters NAP. The surface of the water cross section was estimated on $A = 8350 \text{ m}^2$.

![](image1.png)

Figure 3.3: Water depth in the vicinity of the Haringvliet bridge.

3.2.2 Water discharges and flow velocities
The discharge of water through the Haringvliet estuary depends on the upstream river discharge and the opening or closing of the Haringvliet dam. The Haringvliet dam will be closed during high water at sea to reduce the flood risk [Van Leeuwen et al., 2004]. For low discharge from the rivers, the Haringvliet dam will be closed as well to maintain sufficient water depth for marine traffic [Rijkswaterstaat, 2011a]. In other cases the Haringvliet dam will be opened to flush the Haringvliet estuary. It was expected that the partly opening of the Haringvliet dam (the Kierbesluit) will hardly affect the discharge magnitudes and the local flow velocities [Van Leeuwen et al., 2004].

The discharges at the Haringvliet bridge and the Haringvliet sluices are depicted in figure 3.4 for the neap, the average and the spring tide. As can be seen, the direction of the flow stream will always be in western direction, towards the North Sea. In table 3.1 the distributions over time of the discharges through the Haringvliet are depicted [Havenbedrijf Rotterdam N.V., 2015]. These values do not necessarily coincide with the discharges as depicted in figure 3.4. The flow velocities at a cross section depend on the incoming discharge and the cross section area and are determined according to the general formula $u = Q/A$, for a water level of 0 meter NAP ($A = 8350 \text{ m}^2$)\(^1\).

<table>
<thead>
<tr>
<th></th>
<th>5-percentile</th>
<th>25-percentile</th>
<th>50-percentile</th>
<th>75-percentile</th>
<th>95-percentile</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hollandsch Diep</td>
<td>150 m(^3)/s</td>
<td>330 m(^3)/s</td>
<td>1849 m(^3)/s</td>
<td>2803 m(^3)/s</td>
<td>3612 m(^3)/s</td>
</tr>
<tr>
<td>Haringvliet sluices</td>
<td>0 m(^3)/s</td>
<td>76 m(^3)/s</td>
<td>1737 m(^3)/s</td>
<td>2757 m(^3)/s</td>
<td>3485 m(^3)/s</td>
</tr>
<tr>
<td>Flow velocities at the Haringvliet bridge</td>
<td>0.018 m/s</td>
<td>0.040 m/s</td>
<td>0.221 m/s</td>
<td>0.336 m/s</td>
<td>0.433 m/s</td>
</tr>
</tbody>
</table>

Table 3.1: Discharge distribution over the different river branches for average tide, see figure 3.4 [Havenbedrijf Rotterdam N.V., 2015].

\(^{1}\)The surface of the lateral cross section of the Haringvliet estuary depends on the water level. The flow velocities are calculated for the cross sectional area related to a water level of 0 meter NAP.
3.2. Local conditions Haringvliet estuary

Figure 3.4: Discharge distribution over the different river branches of the South-Western Delta at a 50-percentile discharge condition. The values are the net discharges in m$^3$/s for neap tide / average tide / spring tide [Havenbedrijf Rotterdam N.V., 2015].

3.2.3 Water levels

The water levels in the Haringvliet are controlled by the opening and closing of the Haringvliet dam. Small tidal influences are still present in the Haringvliet estuary due to the connection to the North Sea through the Nieuwe Waterweg, the Oude Maas, the Dordtsche Kil and the Spui. The extreme water levels in the Haringvliet and their probability are given in table 3.2a. The mean water levels and the tidal influences are depicted in table 3.2b, for both the current situation and the future situation after the introduction of the Kierbesluit. It was not expected that the tidal influence will increase after the introduction of the Kierbesluit [Van Leeuwen et al., 2004]. The MHWL and MLWL are expected to change marginally [Van Leeuwen et al., 2004], as can be seen in table 3.2b.

<table>
<thead>
<tr>
<th>Exceeding high water levels</th>
<th>Present</th>
<th>Future</th>
</tr>
</thead>
<tbody>
<tr>
<td>1x per 1000 years</td>
<td>+2.57 m NAP</td>
<td>+2.57 m NAP</td>
</tr>
<tr>
<td>1x per 100 years</td>
<td>+2.36 m NAP</td>
<td>+2.36 m NAP</td>
</tr>
<tr>
<td>1x per 10 years</td>
<td>+2.09 m NAP</td>
<td>+2.09 m NAP</td>
</tr>
<tr>
<td>1x per 2 years</td>
<td>+1.77 m NAP</td>
<td>+1.77 m NAP</td>
</tr>
<tr>
<td>1x per year</td>
<td>+1.63 m NAP</td>
<td>+1.63 m NAP</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Under-run low water levels</th>
<th>Present</th>
<th>Future</th>
</tr>
</thead>
<tbody>
<tr>
<td>1x per 10 years</td>
<td>-0.40 m NAP</td>
<td>-0.40 m NAP</td>
</tr>
<tr>
<td>1x per year</td>
<td>-0.25 m NAP</td>
<td>-0.25 m NAP</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Present</th>
<th>Future</th>
</tr>
</thead>
<tbody>
<tr>
<td>MHWL</td>
<td>0.65 m NAP</td>
</tr>
<tr>
<td>MLWL</td>
<td>0.32 m NAP</td>
</tr>
<tr>
<td>MWL</td>
<td>0.48 m NAP</td>
</tr>
<tr>
<td>Tidal difference</td>
<td>0.33 m NAP</td>
</tr>
</tbody>
</table>

Table 3.2: Water levels at the Haringvliet near Rak Noord (nearby Willemstad).
CHAPTER 3. LOCAL CONDITIONS AT THE HARINGVLIET BRIDGE

3.2.4 Sediment characteristics

The upstream rivers import a lot of sand and mud to the Hollandsch Diep, the Haringvliet and eventually to the North Sea. As could be seen in figure 3.5a, the suspended sediments in the Hollandsch Diep - Haringvliet estuary, consist mainly of non-flocculated silt, with a small partition of flocculated silt.

Due to the higher fall velocity of sand, the sand particles settle relatively fast after entering calm conditions in comparison to the fine suspended sediment particles. Therefore, the majority of the sand particles will settle in the eastern parts of the estuary at the transition of the upstream rivers into the Hollandsch Diep [Mulder et al., 2010]. The fine sediments, like flocculated and non-flocculated silt, have smaller fall velocities and will tend to travel longer distances with the streamlines before they settle. In general, the accumulation in the Haringvliet is relatively small, as almost all suspended sediments are settling in the Hollands Diep. At the Haringvliet bridge, the majority of the suspended sediments already have been settled (see figure 3.5a).

<table>
<thead>
<tr>
<th>Fall velocities</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Non-flocculated silt</td>
<td>0.003 mm/s</td>
</tr>
<tr>
<td>Flocculated silt</td>
<td>0.116 mm/s</td>
</tr>
<tr>
<td>Fine sands</td>
<td>0.289 mm/s</td>
</tr>
</tbody>
</table>

Suspended particle concentration at Haringvliet Bridge

| Minimum | 1 mg/l |
| Maximum | 58 mg/l |
| Median  | 10 mg/l |
| Average | 12 mg/l |
| Top 10% | 20 mg/l |

Concentration suspended sediment

The concentration of the suspended particles in the Haringvliet estuary could be read from figure 3.5a. The total suspended concentration near the Haringvliet bridge is ≈ 13 mg/l, 12 mg/l non-flocculated silt and 1 mg/l flocculated silt. The accompanying fall velocities of the different sediment types are depicted in table 3.5b. In the studies on the morphological influences of the Kierbeshuit on the South-Western Delta, the suspended sediment concentrations of the Haringvliet were assumed between 13.4 mg/l and 19.4 mg/l [Van Leeuwen et al., 2004]. From data analysis of local measurements on the suspended sediment concentration in the Haringvliet, the characteristic concentration values of table 3.5b were obtained. As the yearly sedimentation in the Haringvliet estuary was studied in this thesis, the average value for the suspended sediment concentration (=13 mg/l) was used as normative value for the suspended sediment concentration.

Size sediment particles

The size of the sediment particles that are in suspension or present at the bottom of the Haringvliet estuary are required for the morphological analysis. The particle size distributions of the suspended sediments near the Haringvliet bridge are depicted in table 3.3. With these values, the $D_{50}$ of the suspended particles was determined, which is on average 22.1 µm. The median particle size of the sediment at the bottom of the Hollandsch Diep-Haringvliet estuary was approximated on $D_{50} \approx 110 - 120 \mu$ [Mol, 2003].
3.2. Local conditions Haringvliet estuary

<table>
<thead>
<tr>
<th>Suspended particles</th>
<th>2 µm</th>
<th>10 µm</th>
<th>16 µm</th>
<th>50 µm</th>
<th>63 µm</th>
<th>$D_{50}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>23-1-1992</td>
<td>9.7 %</td>
<td>48.3 %</td>
<td>59.4 %</td>
<td>67.4 %</td>
<td>67.7 %</td>
<td>10.9 µm</td>
</tr>
<tr>
<td>19-3-1992</td>
<td>7.2 %</td>
<td>39.1 %</td>
<td>50.7 %</td>
<td>61.9 %</td>
<td>62.6 %</td>
<td>15.6 µm</td>
</tr>
<tr>
<td>11-5-1992</td>
<td>5.5 %</td>
<td>27.4 %</td>
<td>39.8 %</td>
<td>57.9 %</td>
<td>59.1 %</td>
<td>35.2 µm</td>
</tr>
<tr>
<td>13-7-1992</td>
<td>6.0 %</td>
<td>33.8 %</td>
<td>46.8 %</td>
<td>60.6 %</td>
<td>61.3 %</td>
<td>23.9 µm</td>
</tr>
<tr>
<td>07-9-1992</td>
<td>6.4 %</td>
<td>33.0 %</td>
<td>45.6 %</td>
<td>62.8 %</td>
<td>63.8 %</td>
<td>24.7 µm</td>
</tr>
</tbody>
</table>

Table 3.3: Particle size distribution. Percentages in % of the dry weight of the suspended sediment particles near the Haringvliet bridge [Rijkswaterstaat, 1992]

Initial sedimentation in the vicinity of the Haringvliet bridge

In the Hollandsch Diep, the sedimentation rates are between 50 and 200 mm/year, which gradually mitigates towards the western part of the Haringvliet estuary [Mulder et al., 2010]. The sedimentation in the Haringvliet is on average 2 mm per year in total [Rijkswaterstaat, 2011a]. On the border between the Hollandsch Diep and the Haringvliet estuary, the average sedimentation rates are 1.5 mm sand per year and 5 mm silt per year [Mulder et al., 2010]. This results in a total sedimentation of approximately 6.5 mm per year, which was considered to be the normative reference value for the sedimentation at the case study location.

After the implementation of the Kierbesluit, it was expected that the sedimentation and deposition will stay approximately the same in the Haringvliet in the short term [Van Wijngaarden and Ludikhuize, 1997]. For the long term, a slight increase of the sedimentation was expected, with an exception for the deeper gullies where erosion was to be expected [Van Wijngaarden and Ludikhuize, 1997]. To study the influences of the Kierbesluit on the local morphology, the South-Western delta was subdivided into smaller sections to determine the local sedimentation rates [Van Wijngaarden and Ludikhuize, 1997]. For the areas in the vicinity of the Haringvliet bridge, the sedimentation rates were 8.60 and 6.35 kg/(m²·year) [Van Wijngaarden and Ludikhuize, 1997]. The average of these values (7.5 kg/(m²·year)) was used as the governing value for the initial yearly sedimentation in the vicinity of the Haringvliet bridge.

Contaminated soil layers in the Haringvliet and Hollandsch Diep estuary

Due to the pollution in the upstream rivers in the past, parts of the Haringvliet estuary are contaminated. Nowadays the pollution in the Haringvliet is decreasing, as the river Rhine is significantly cleaner, but the contaminated soil layers are still present. The polluted layers in the Hollands Diep are already covered by clean, newly deposited, soil layers [Van Wijngaarden and Ludikhuize, 1997]. An important boundary condition for the implementation of the Kierbesluit was that no contaminated sediment particles are transported from the South Western Delta to the North Sea, either by permanent or incidental erosion [Van Leeuwen et al., 2004].

3.2.5 Bridge gap selection for case study

For the case study on the feasibility of the Tilting Lock at the Haringvliet bridge, one of the bridge gaps had to be selected. It was chosen to locate the Tilting Lock in the fourth bridge gap of the Haringvliet bridge, between the third and fourth bridge piers. The fourth bridge gap was selected, because of the relatively large initial water depth ($\approx$ -8 meter NAP) and the presence of two identical bridge piers, see appendix C. In addition, this bridge gap is next to one of the two official shipping lanes, which makes the Tilting Lock easily accessible for vessels with dredging an auxiliary access channel.

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2The Tilting Lock will not fit underneath the Haringvliet bridge in the fourth bridge gap with the assumed dimensions of the Tilting Lock for the case study (table 1.1). The available head clearance under the Haringvliet bridge is insufficient. This was not considered as a critical point as the size of the Tilting Lock can be adjusted to fit in this bridge gap, see appendix K.
3.2.6 Soil characteristics

For the initial design of the Haringvliet bridge in 1960, multiple soil tests were performed to map the soil conditions of the area\(^3\). Per pier, two Cone Penetration Test (CPT)’s were made to derive the composition of the subsoil in the Haringvliet estuary [TNO and Geologische Dienst Nederland, 2016], which are included in appendix B. In table 3.4 and figure 3.6 the derived soil conditions were simplified per bridge pier into average soil layers. For each soil layer, characteristic values for the volumetric soil weight (\(\gamma_s\)), the internal soil friction angle (\(\phi'\)) and cohesion (\(c'\)) were assigned according to Dutch standards [Nederlands Normalisatie-instituut, 2012]. As can be seen, both weak and strong soil layers alternate each other till a depth of approximately -32 meters NAP, from where only sand is present.

From the analysis of the different CPT’s in appendix B, it was concluded that the subsoil of bridge pier 3 (CPT 9, figure B.2) contains the worst soil conditions, related to maximum bearing capacity of the bridge pier foundation at a depth of \(\approx -30\) meter NAP. Therefore, CPT 9 was considered to be normative for the case study design. It was expected that better results (i.e. more bearing capacity) will be found when in the other CPT’s are used in calculations.

### Table 3.4: Soil characteristics

<table>
<thead>
<tr>
<th>Soil layer</th>
<th>Cone resistance</th>
<th>Soil type</th>
<th>(\gamma_s)</th>
<th>(\phi')</th>
<th>(c')</th>
</tr>
</thead>
<tbody>
<tr>
<td>Top m NAP</td>
<td>Bottom m NAP</td>
<td>Minimum kg/cm(^2)</td>
<td>Maximum kg/cm(^2)</td>
<td>(kN/m^3)</td>
<td>(^0)</td>
</tr>
<tr>
<td>-4</td>
<td>-8</td>
<td>20</td>
<td>40</td>
<td>sand</td>
<td>19</td>
</tr>
<tr>
<td>-8</td>
<td>-18</td>
<td>10</td>
<td>30</td>
<td>clay + sand</td>
<td>18</td>
</tr>
<tr>
<td>-18</td>
<td>-30</td>
<td>20</td>
<td>120</td>
<td>sand</td>
<td>21</td>
</tr>
<tr>
<td>-30</td>
<td>-32</td>
<td>30</td>
<td>60</td>
<td>clay + sand</td>
<td>20</td>
</tr>
<tr>
<td>-32</td>
<td>-42</td>
<td>80</td>
<td>230</td>
<td>sand</td>
<td>22</td>
</tr>
<tr>
<td>-42</td>
<td></td>
<td>360</td>
<td>400</td>
<td>sand</td>
<td>22</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Soil layer</th>
<th>Cone resistance</th>
<th>Soil type</th>
<th>(\gamma_s)</th>
<th>(\phi')</th>
<th>(c')</th>
</tr>
</thead>
<tbody>
<tr>
<td>Top m NAP</td>
<td>Bottom m NAP</td>
<td>Minimum kg/cm(^2)</td>
<td>Maximum kg/cm(^2)</td>
<td>(kN/m^3)</td>
<td>(^0)</td>
</tr>
<tr>
<td>-3.2</td>
<td>-7</td>
<td>20</td>
<td>50</td>
<td>sand</td>
<td>19</td>
</tr>
<tr>
<td>-7</td>
<td>-15</td>
<td>10</td>
<td>30</td>
<td>clay + sand</td>
<td>20</td>
</tr>
<tr>
<td>-15</td>
<td>-21</td>
<td>20</td>
<td>100</td>
<td>sand + clay</td>
<td>21</td>
</tr>
<tr>
<td>-21</td>
<td>-22</td>
<td>60</td>
<td>160</td>
<td>sand</td>
<td>22</td>
</tr>
<tr>
<td>-22</td>
<td>-25</td>
<td>60</td>
<td>160</td>
<td>sand</td>
<td>22</td>
</tr>
<tr>
<td>-25</td>
<td>-26</td>
<td>350</td>
<td>400</td>
<td>sand + gravel</td>
<td>22</td>
</tr>
<tr>
<td>-26</td>
<td>-27</td>
<td>40</td>
<td>120</td>
<td>sand + clay</td>
<td>21</td>
</tr>
</tbody>
</table>

Since the performed tests in 1960, several things are changed in the subsoil of the Haringvliet bridge. The foundation piles for the Haringvliet bridge are driven, a couple of meters of soil are eroded (from -4 meter to \(-8\) meter NAP) and the testing methods of the soil characteristics have been improved significantly. Therefore, it is recommended for future research to perform new tests to retrieve more recent and more detailed information on the soil conditions at the Haringvliet bridge.

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\(^3\)Since the performed tests in 1960, several things are changed in the subsoil of the Haringvliet bridge. The foundation piles for the Haringvliet bridge are driven, a couple of meters of soil are eroded (from -4 meter to \(-8\) meter NAP) and the testing methods of the soil characteristics have been improved significantly. Therefore, it is recommended for future research to perform new tests to retrieve more recent and more detailed information on the soil conditions at the Haringvliet bridge.
3.2. Local conditions Haringvliet estuary

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Soil Type</th>
<th>Density (g/cm³)</th>
<th>Young's Modulus (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>-27</td>
<td>sand</td>
<td>130</td>
<td>380</td>
</tr>
<tr>
<td>-27.5</td>
<td>clay + sand</td>
<td>50</td>
<td>130</td>
</tr>
<tr>
<td>-29</td>
<td>sand</td>
<td>340</td>
<td>400</td>
</tr>
<tr>
<td>-30</td>
<td>sand</td>
<td>100</td>
<td>260</td>
</tr>
<tr>
<td>-42</td>
<td>sand</td>
<td>360</td>
<td>480</td>
</tr>
</tbody>
</table>

Table 3.4: Soil characteristics of piers 3 and 4, averaged conditions from CPT’s [TNO and Geologische Dienst Nederland, 2016], see appendix B. [10 kg/cm² ≈ 1 MPa]

3.2.7 Hydrogeology

With the large dimensions of the Tilting Lock, it was expected that multiple soil layers will be crossed to implement the Tilting Lock. In the subsoil of the Haringvliet estuary, multiple impermeable clay layers are present, as can be seen in figure 3.7. Between these impermeable layers, several permeable sandy layers are present. The majority of the impermeable soil layers are relatively thin (thickness less than 10 meters) and show multiple interruptions over the longitudinal axis of the estuary, see figure 3.7. Due to the disruptions in the upper impermeable layers, it was assumed that surface and ground waters in the Haringvliet are well connected.

The hydraulic heads for several layers are available at www.dinoloket.nl for multiple locations in the Haringvliet estuary. The hydraulic head decreases at larger depths [TNO and Geologische Dienst Nederland, 2016], which is related to lower water pressures and thus is the inflow of surface water into the soil likely. The differences in the hydraulic heads can be explained by the presence of the polders in the surrounding areas of the Haringvliet. These polders require lower hydraulic heads, to prevent flooding of the polders or bursting up of the soil surface. An extra connection between the permeable soil layers and the surface water of the Haringvliet, will lead to an increase of the inflow of water into the ground and thus to an increased hydraulic head in the permeable soil layers. This can have negative influences on the safety of the surrounding polders.

The influences of the additional connection between the permeable soil layers and the surface water of the Haringvliet were considered to be beyond the scope of this thesis, as already multiple connections are present between the different soil layers of the Haringvliet estuary. The influences of an additional connection were considered to be relatively small.

Figure 3.7: Hydrogeologic conditions of a longitudinal cross section of the subsoil in the Haringvliet [TNO Bouw en Ondergrond, 2008]

Water quality

The surface water of the Haringvliet is fresh [Stuurman and Oude Essink, 2007]. As salty water is more dense, it will tend to stay beneath the less dense fresh water. As can been seen in figure 3.8, is the boundary in the Haringvliet between the brackish ground water and the fresh surface water.
somewhere between -10 to -25 meters NAP. The mixing of fresh and salt water is undesired for the Haringvliet, as it was decided in the Kierbesluit to do not allow any salt water in the eastern parts of the Haringvliet estuary [Van Wijngaarden et al., 2002]. The problems related to the water quality were considered to be beyond the scope.

Figure 3.8: Hydraulic head of the non-fresh ground water levels (≥1000 mg/l). [Geologische Dienst Nederland – TNO, 2013]

3.2.8 Wind and waves
The governing wind conditions in the area of the Haringvliet were assumed to be 32 m/s for a return period of 75 years, based on rules of thumb [Vrijling et al., 2016]. The wave conditions in the Haringvliet estuary were estimated from the wind conditions and the maximum available fetch length in the Haringvliet (≈11 km). The formula by Young and Verhagen [Young and Verhagen, 1996] resulted in a wave height of approximately $H_w = 1.5$ meter at the Haringvliet bridge, see appendix F.2.4.

3.3 Structural conditions Haringvliet bridge

Besides the characteristics of the surroundings of the Haringvliet bridge, the conditions of the bridge itself are of great importance to the design made for the implementation of the Tilting Lock. In the following paragraphs, the conditions of the Haringvliet bridge are discussed. In appendix C a selection of the original design drawings as made for the construction of the Haringvliet bridge are included.

3.3.1 Structural design Haringvliet bridge

The Haringvliet bridge consists of twelve fixed bridge spans, founded on eleven bridge piers and two abutments, see figure 3.10. The bridge deck consists of prefabricated elements, welded together after installation. The bridge consists of two main sections, which have separated bearing schemes. The main span of the bridge stretches from the southern abutment till the basement pier for the movable bridge part section. The northern section of the Haringvliet bridge was founded separately of the main part of the bridge, and stretches from the movable bridge part to the northern abutment.

To bear the steel bridge deck of the Haringvliet bridge, each pier includes two spherical bearings [Civiele technieken De Boer bv, 2009], as depicted in figure 3.10. Each pier at the main span has one unilateral and one general spherical bearing, see figure 3.11. The spherical bearings allow the
bridge deck to deform in the longitudinal direction of the bridge under temperature differences and horizontal traffic loads. The horizontal forces in the lateral direction of the bridge pier were taken by the unilateral spherical bearings.

As dilatation joints are present in the bridge deck at the southern abutment and the pier for the basement of the movable bridge part, it was assumed that the horizontal position of the bridge deck was fixed at the fifth pier, in the middle of the Haringvliet bridge. Therefore, it was assumed that the fifth pier takes the majority of the horizontal forces in the longitudinal direction of the Haringvliet bridge.

Figure 3.10: Simplified top view of the Haringvliet bridge with the locations of the bearings and the degrees of freedom [Maurer Söhne, 2009].

3.3.2 Structural elements bridge piers

The different structural elements of the bridge piers are discussed in the following paragraphs. A bridge pier of the Haringvliet bridge consists globally of the following components depicted in figure 3.12.

- **Sheet pile walls**: The sheet pile walls around the piers were installed to provide for a temporary building pit to build the in-situ bridge pier. The building pit has an oval shape, with outer dimensions of 12 meter by 32.5 meters. After construction of the bridge piers, the sheet pile walls were cut off just above the soil surface and the layer of underwater concrete.

- **Foundation piles**: For the third and fourth pier of the Haringvliet bridge, 154 concrete foundation piles (0.4 x 0.4 meter, pre-tensioned) were installed under inclinations of 1:10.

- **Underwater concrete**: After the installation of the foundation piles, a 3 meters thick layer of underwater concrete was poured around the top ends of the piles, to provide for water tightness of the building pit.

- **In-situ pier**: By the installation of the sheet pile walls and the underwater concrete, a building pit was constructed to provide a dry working place. Inside this building pit, the actual bridge pier and the bridge pier columns were constructed in-situ and consists concrete and masonry. The bottom slab of the pier is casted around the pile heads, which provides a rigid connection between the pier and its foundation.
• **Structural bearings:** On top of the in-situ bridge pier columns, bearings for the bridge deck were installed. In 2010, the original bearings were replaced by spherical bearings [Maurer Söhne, 2009].

• **Prefabricated bridge deck:** The bridge deck of the Haringvliet bridge consists of steel prefabricated elements. Although the prefabricated deck was brought in position per bridge span, no joints are available in the road surface. This indicates that the different bridge deck sections are welded together after the installation. This is endorsed by the fact that only two bearings are present at one bridge pier. For separated bridge decks at least four bearings should be present. This means that all the bridge decks will work as one beam.

• **Bottom protection:** After the construction of the Haringvliet bridge bottom protection was applied to prevent local scour. It is likely that this took place long after the construction of the bridge piers, as the bottom surface of the Haringvliet is currently approximately 4 meters lower than the bottom surface on the original design drawings.

Figure 3.12: Designation of the structural elements of a Haringvliet bridge pier, see appendix C for the original design drawings.
Chapter 4

Design alternatives for the trench of the Tilting Lock

The objective of this thesis was to make a design for the depth required for the implementation of the Tilting Lock, without endangering the stability of the Haringvliet bridge. As can be seen in figure 4.1, the current water depth of the Haringvliet estuary is not sufficient to implement the Tilting Lock. Several alternatives were developed and discussed in 4.1 to obtain this depth. In appendix D the potential alternatives for reaching the required depth are elaborated into more detail. From these alternatives, one was selected to elaborate on. In 4.2 the global design for the selected alternative was discussed in more detail, including the dimensions of the different structural elements. The discussed alternatives are reviewed on several criteria, which are listed below:

- The main requirement on the depth that is required for the Tilting Lock, is sufficient Under Keel Clearance (UKC) to prevent running aground of the Tilting Lock.
- The stability of the bridge piers is in any case vital for the design of the required depth for the Tilting Lock, as otherwise the Haringvliet bridge becomes unserviceable and the initially added value of the Tilting Lock to the traffic junction would be gone.
- As the Tilting Lock will be placed in a morphological active area, it would be beneficial if the sedimentation rate is low under the Tilting Lock. If not, the frequency of maintenance on the depth for the Tilting Lock can be problematic.
- The construction phase of an alternative was taken into account, as the floating body of the Tilting Lock will be prefabricated and installed.

4.1 Alternatives to reach the required depth

In total four conceptual alternatives for the design on the required depth for the Tilting Lock were developed. From a very simple and basic solution (the open trench (4.1.1)) to a very rigid solution which excludes several external influences on the Tilting Lock (the pneumatic caisson (4.1.2)). Between these two extreme alternatives, more moderate alternatives are available. An open trench with measures to stabilise the bridge piers (4.1.3) or an open building pit (4.1.4). These four concept alternatives are briefly discussed in the following paragraphs before one is selected in 4.1.5 to serve as the main alternative for the case study design at the Haringvliet bridge.
4.1.1 Alternative 1: Open trench

The alternative of the open trench is nothing more than excavating a trench till the required depth for the Tilting Lock is reached, as can be seen in figure 4.2.

The open trench has the advantage that it is relatively easy to reach the required depth, as well-known dredging methods and equipment can be used. The main disadvantage of the open trench is the almost inevitable instability of the Haringvliet bridge piers due to the large excavations. As can be seen in figure 4.2, is the soil completely removed over the majority of the pile foundation height. In addition, it was expected that an open trench might attract a lot of sediments, which can lead to a rapid sedimentation of the trench.

Regarding the construction phase of the Tilting Lock, the installation of the Tilting Lock underneath the Haringvliet bridge can be done by a (temporary) extension of the trench, which is depicted on the right side of figure 4.2b. The Tilting Lock can be submerged in this extension of the trench before it is floated into its final position underneath the bridge.

![Figure 4.2: Alternative 1: open trench without bridge pier measures.](a) Lateral cross section of the open trench (b) 3d open trench

4.1.2 Alternative 2: Pneumatic caisson

With a pneumatic caisson, the required depth for the Tilting Lock can be reached without any interference of the foundation of the Haringvliet bridge, see figure 4.3. A pneumatic caisson is a prefabricated, often concrete, caisson that can be immersed into the subsoil, due to the excavation of soil underneath the caisson.

Advantages of the pneumatic caisson are the relatively small area of influence with respect to the open trench alternative and the very low rate of sedimentation inside the caisson. As hardly any water will pass the Tilting Lock underneath, no import of sediment particles will take place. Therefore, the sedimentation rate inside the pneumatic caisson will be low.

Multiple disadvantages of the pneumatic caisson are found. During the construction phase, sufficient air pressure is required in the working chamber underneath the pneumatic caisson. To keep the working chamber free of water intrusion during the subsidence of the caisson. For the required installation depth ($\geq 35$ meter under MWL), the air pressure would exceed the allowable air pressure for manual labour in the working chamber [(Winterkorn & Fang) & (Hof, 2006)].

For the positioning of the Tilting Lock in the pneumatic caisson, a temporary access channel is required. This access channel will have to be dredged, so dredging still required. To sail the Tilting Lock into the pneumatic caisson, a movable wall in the pneumatic caisson is required. Such a removable wall is comparable to the bulkheads used for the immersion of tunnels. In addition, it is not possible to pre-fabricate the whole pneumatic caisson in once, as the required heights of the walls for the non-subsided caisson will not fit underneath the Haringvliet bridge.
4.1. Alternatives to reach the required depth

(a) Lateral cross section

(b) Lateral cross section

Figure 4.3: Alternative 2: Pneumatic caisson.

4.1.3 Alternative 3: Open trench with bridge pier measures

The third alternative is comparable to the open trench alternative, but includes stabilising measures for the Haringvliet bridge piers. As discussed in appendix D.3, many types of measures could be defined. Examples are reinforcing the subsoil of the pier foundations (figure 4.4b), extending the amount of foundation piles (figure 4.4a), limiting the required excavations around the bridge piers (for instance with a sheet pile wall, see figure 4.4c) or a structure to enclose the subsoil of the bridge pier foundation (like a cofferdam, see figure 4.4d).

Due to these measures, the stability of the Haringvliet bridge would be increased. The sedimentation rate and the installation of the Tilting Lock underneath the bridge will be similar to the open trench alternative.

(a) Sketch of added foundation piles

(b) Sketch of a grouted soil body beneath a bridge pier.

(c) Trench with a (local) sheet pile wall to reduce required excavations around the bridge pier.

(d) Temporary building pit with struts to construct a permanent diaphragm wall.

Figure 4.4: Sketches of conceptual alternatives for measures to stabilise a bridge pier

4.1.4 Alternative 4: Open building pit

The building pit is a variation on the pneumatic caisson alternative, but is build in-situ. The encircling of the whole Tilting Lock by in-situ placed soil retaining walls will result in a relatively small area of influence, see figure 4.5. The advantage of the in-situ construction is the potential optimisation of the construction phasing, so the Tilting Lock can be placed in its final position, before the final wall sections of the building pit are installed.

For such large retaining heights, horizontally stabilising elements are inevitable for sheet pile walls. The installation of these elements can be challenging due to the presence of the bridge piers.

Figure 4.5: Alternative 3: Building pit.
and/or the large water depths. The application of struts is limited in the final situation, due to the presence of the Tilting Lock. Placing grout anchors induce large risks for the pile foundation of the Haringvliet bridge piers.

Regarding the sedimentation of the open building pit, the situation will be similar to the pneumatic caisson (alternative 2): as no water will pass the Tilting Lock underneath, no import of sediment particles will take place and therefore will the sedimentation rate be low.

4.1.5 Selection of alternatives for the case study design

To elaborate on the feasibility of the depth that is required for the implementation of the Tilting Lock, two of the proposed alternatives were selected to design into more detail. The open trench (alternative 1) was not expected to provide sufficient depth for the size Tilting Lock in the case study. However, regarding the costs of constructing such an open trench and smaller sized Tilting Locks, it was considered to be vital to know the feasibility of the open trench alternative.

The pneumatic caisson (alternative 2) was considered to be very complicated on the aspects related the construction phase and therefore, expected to be an expensive solution. In addition, placing the Tilting Lock within a pneumatic caisson does not differ much from building a conventional lock. Therefore, the pneumatic caisson alternative was not selected as the case study design.

Both the third and fourth alternatives were considered to be feasible. It was decided to elaborate on the third alternative, because of the limited available time. The third alternative is the expansion of the open trench alternative, which makes that the first and the third alternative are complementary. For the third alternative, it was chosen to elaborate on a cofferdam made of steel elements as the stabilising measure for the foundation of the Haringvliet bridge piers.

4.2 General design of the selected trench alternative

The selected alternative (alternative 3: open trench with bridge pier measures) will be introduced in the following paragraphs. First, the general layout and the dimensions will be discussed (4.2.1). Subsequently, the critical design areas are briefly determined and briefly discussed, as the introduction to the coming chapters in this thesis work.

4.2.1 General dimensions of the trench alternative

In figure 4.6 the lateral cross section of the case study design is depicted, together with the main dimensions. In the flowing paragraphs, the main considerations regarding this chosen design are discussed.
4.2. General design of the selected trench alternative

The required Under Keel Clearance (UKC) for the Tilting Lock

As the Tilting Lock is a floating object in the Haringvliet estuary, sufficient Under Keel Clearance (UKC) is required for the Tilting Lock. Based on assumptions on the required freeboard (1.0 meter), the potential sedimentation in the trench (1.0 meter) and tolerances in the construction methods (0.5 meter) it was concluded that a Under Keel Clearance (UKC) of at least 2.5 meters was required for the design stage of the Tilting Lock.

With the lowest expected water level of -0.4 meter NAP in the Haringvliet estuary (see chapter 3.2.3), the depth that is required for the Tilting Lock at the Haringvliet bridge is at least -36.9 meters NAP. On top of this depth, an additional contribution was taken into account, related to bottom protection to prevent (local) scour around the several detached bodies in the trench (1.0 meter). Examples are the Haringvliet bridge piers and the piles of the structure that is required to fixate the position of the Tilting Lock. Finally was concluded, that the depth that is required for the Tilting Lock is approximate -38 meter NAP.

Cofferdam design

Figure 4.7 gives a close up of the starting design of the cofferdam foundation. A cofferdam is defined as a soil body, enclosed by walls which are connected to each other. As the to-build cofferdam can not be placed directly next to the original sheet pile walls, a distance of 5 meter between the cofferdam wall and the existing sheet pile wall was assumed. This results in a width of the cofferdam of approximately 23.2 meter and a length of 42.5 meter, see figure 4.7.

For the connection between the two retaining walls normally anchors are applied, but not for this case. When anchors will be drilled through the group of foundation piles underneath the Haringvliet bridge piers, it will be inevitable that one or more piles will be damaged and therefore lose their bearing capacity. Failure of one or more foundation piles of the Haringvliet bridge pier was considered to be undesirable. In addition to the potential damaging of the bridge pier foundation, it will be complicated to install anchors properly below the water surface (-8 meter under MWL).

As an alternative for the anchor connection, a system of tension rings was proposed. These rings are completely surrounding the cofferdam and tensioned after they are placed in position. Due to the tension, inwards directed forces will be exerted, providing the cofferdam to act as one structure. For more stability, multiple tension rings can be applied without any large disadvantages for the construction.

Figure 4.7: Global dimensions cofferdam in a lateral cross section (left) and a top view (right).

---

1The UKC was determined for the deepest point of the required trench for the Tilting Lock. However, somewhere along the slope, the UKC for the Tilting Lock will be less, due to the geometry of both the assumed trench design and the circular hull of the Tilting Lock. It was assumed that the small difference in available and required UKC was sufficiently small and will be neglected.
Length and width Tilting Lock trench
Figure 4.8 shows the top view of the open trench for the Tilting Lock. The trench requires a stretch of approximately 170 meters where the full required depth is available. This length includes tolerances (10 meters on each side of the Tilting Lock) and an extension stretch of 60 meters. This extended section of the trench is required for the construction phase. As the Tilting Lock will be prefabricated and sailed in position [Poldervaart, 2013], the trench should have sufficient length to submerge the Tilting Lock outside the Haringvliet bridge. When the Tilting Lock is submerged to the required depth, the Tilting Lock will be sailed to its final position underneath the Haringvliet bridge.

4.2.2 Key design areas trench for the Tilting Lock
For the case study design of the required open trench and the cofferdam foundation, multiple key design areas were determined. For the scope of this thesis, the focus was on four key design areas as depicted in figure 4.9. In the remaining of this thesis, it will be elaborated on these critical aspects for the feasibility of the trench for the Tilting Lock.

Slope stability of the subaqueous trench
For a sufficient safe design, slope instabilities (figure 4.9a) should be prevented, as the stability of the bridge piers are at stake as well. The stability of the slopes mainly depends on the inclination of the slopes, the local conditions and the top loads. In chapter 5 the stability of the subaqueous slopes is discussed. Based on the elaborations in this chapter, it was chosen to perform the design for an inclination of 1:5 (≈ 11.3°) for the subaqueous trench slopes.

Stability of the bridge piers
The stability of the Haringvliet bridge piers (figure 4.9b) is vital for the feasibility of the Tilting Lock. If the Haringvliet bridge becomes unserviceable, the situation at the traffic junction will only be worsened and the added value of the Tilting Lock will be gone.

Regarding the stability of the bridge piers, it was elaborated on two scenarios: the excavation of soil around the bridge pier till an allowable bottom level (chapter 6) and the installation of a cofferdam around the Haringvliet bridge piers to provide sufficient stability to the bridge piers in the trench (chapter 7).

Sedimentation in the trench
The sedimentation rate that is to be expected in the trench (figure 4.9c), is critical in the design for the open trench alternative. When the sedimentation in the trench is large, the serviceability of the Tilting Lock will be less. For large rates of sedimentation, maintenance on the depth of the trench has to be done frequently.

The trench extension can be located on either side of the Haringvliet bridge. Whether this should be on the western or eastern side of the Haringvliet bridge, should be studied in future research.
4.2. General design of the selected trench alternative

With the excavation of the trench and installation of the Tilting Lock, the local hydrodynamical situation will be affected. The amount of sedimentation and erosion at a location is depended on, amongst other aspects, the local flow velocity and the size of the suspended sediment particles. The changes in the hydrodynamics and the morphology in the trench for the Tilting Lock in the Haringvliet estuary are studied into more detail in chapter 8.

4.2.3 Delimitation of the scope of the case study research

In addition to the mentioned key design areas, there are more aspects within the selected case study alternative, that can be elaborated on. However, these additional design areas were considered to be beyond the scope of this feasibility study.

Hydrogeology

The fourth main challenge of designing the trench for the Tilting Lock is related to hydrogeology (figure 4.10). In the present situation multiple connections between the permeable soil layers and the surface water are present already, as can be seen in figure 3.7 in section 3.2.7. When multiple impermeable soil layers are excavated to create the trench for the Tilting Lock, the connections will be intensified even more.

The hydraulic head of the surface water of the Haringvliet was higher than the hydraulic head in the groundwater system of the permeable soil layers [TNO and Geologische Dienst Nederland, 2016]. With the extra connection made by the trench, it will be easier for surface water to flow to the permeable soil layers, which will increase the water pressure in the lower permeable soil layers. This increase in water pressure will raise the hydraulic head in the nearby polders, eventually leading to flooding of the polders. Furthermore, mixing of the surface and groundwater might be unwanted, if differences in water quality are present.

The probability and magnitude of the risks related to the hydrogeology should be assessed in future research, as the hydrogeology problems were considered to be out of the scope for this thesis.

Swell of the subsoil layers

The main aspect that is not taken into account in the case study design on the feasibility of the trench for the Tilting lock, is the potential swell of the subsoil layers. Due to the excavation of the trench, the stress in the subsoil layers will decrease, which will lead to swell. Due to the decrease of effective stresses, small deformations in the subsoil are likely to occur and the bottom level is expected to rise [Vrijling et al., 2016]. Swell of soil layers will lead to less compacted soil layers and which are more susceptible to liquefaction [T. Raaijmakers, 2005]. Besides the effect on the excavations volume, swell of the soil layers will lead to less compacted soil layers. As discussed, loosely packed sand layers are susceptible to liquefaction [T. Raaijmakers, 2005]. Therefore, it was expected that the swell of the soil layers at greater depths will lead to a changed situation for the slope stability of the slopes in the trench for the Tilting Lock. Therefore, it is important to take the effects of swell into account in future research. Although swell was considered to be beyond the scope of this thesis, it might have the following consequences:

- The reduced bearing capacity of the subsoil.
- The reduced slope stability.
- The increased volume of the required excavations.

Fixating structure for the Tilting Lock

In addition to the design of the large depth that is required for the Tilting Lock, more research topics related to the Tilting Lock can be considered. The most critical aspect of the remaining design areas was the design of the structure that is required to fixate the position of the Tilting Lock underneath the Haringvliet bridge.
CHAPTER 4. DESIGN ALTERNATIVES FOR THE TRENCH OF THE TILTING LOCK

The design of the fixating structure will depend on the design of the depth that is required for the Tilting Lock. When the assumed design for the open trench for the Tilting Lock is taken into account, very large piles are required to construct the proposed design of the fixating structure. It was expected that the design of such tubular systems would be feasible, as they are very much comparable to existing structures in the offshore engineering, like offshore windmills or oil rigs.

The fixating structure was considered to be less critical for the feasibility of the Tilting Lock, than the design of the depth that is required to implement the Tilting Lock. Therefore, the feasibility of the fixating structure for the Tilting Lock was considered to be beyond the scope of this thesis.
Chapter 5

Stability of the subaqueous trench slopes

The stability of the subaqueous slopes of the trench that is required for the implementation of the Tilting Lock is of great importance for the stability of the piers of the Haringvliet bridge. Local instabilities of the slopes could lead to the failure of the whole design. In this chapter, the optimal slope inclination and the stability of the subaqueous slopes are studied with respect to the spatial integration of the trench and the Tilting Lock in the local environment of the Haringvliet bridge.

To create insight in the case study problem, figure 5.1 depicts different slope angles that were considered. A too steep slope will lead to a large sensitivity to failure of the subaqueous trench slopes. A too flat inclination will lead to an unnecessarily high volume of excavated soil and the crossing multiple bridge piers.

![Figure 5.1: Different slope inclinations of the subaqueous slopes for the trench of the Tilting Lock, drawn for the case study location at the Haringvliet bridge.](image)

To determine the inclination for which the subaqueous slopes of the trench for the Tilting Lock will be stable, the failure mechanisms of subaqueous slopes were elaborated on in appendix E.1. From this appendix, it was chosen to elaborate on the failure modes liquefaction and breach flow for the present subsoil layers in the Haringvliet estuary (5.1). From the analysis on the liquefaction and the breach flow, an inclination will be found for which the slopes will be stable. This slope inclination will be checked on the macro stability of the subaqueous trench slopes (5.2). The macro stability of the subaqueous slopes was assessed by the software program D-stability (see appendix E.3). In the evaluation (5.3) the performed analysis was discussed and conclusions were drawn.
5.1 Liquefaction and breach flow of the trench slopes

Liquefaction and breach flow have a strong interaction: they could be initiated and intensified by each other [Van den Ham et al., 2012]. The failure of soil layers to liquefaction and breach flow depends on the changes in shear stress in the soil layers and the initial soil conditions, as discussed in appendix E.1. The two failure modes are studied in 5.1.1 and 5.1.2. Subsequently, the influences of these failure modes on the design of the trench slopes are discussed.

5.1.1 Liquefaction

Liquefaction occurs in a fully drained soil layer of loosely packed sand and is initiated by changes in the shear stresses between soil particles, which results in quicksands [Van den Ham et al., 2012]. Liquefaction takes place if at least one sand element is in a metastable stress state [T.P. Stoutjesdijk, M.B. de Groot and Lindenberg, 1998]. This means that the undrained response to any quick change in load, how small it may be, consists of a sudden large increase in pore pressure. Whether a flow slide occurs or not, depends on three parameters: the mean effective stress in the soil, the initial (shear) stresses and the presence of a trigger mechanism.

General conditions for metastability of a subaqueous slope (liquefaction) are listed below [Van den Ham et al., 2012]. A trigger mechanism is always likely to be present, so it is better to increase the safety of the design instead of doing attempts to diminish the trigger mechanism [Van den Ham et al., 2012].

- The sand layer needs to be sensitive to liquefaction (sufficiently loosely packed) over a minimum length, which probably differs between 2 and 5 meter.
- The geometry of the slope needs to be sufficiently unfavourable (steepness & length of the slope are unfavourable).
- Presence of a trigger mechanism; for instance, a small earthquake, the drilling of (sheet) piles, local erosion, sudden water level drops, waves or vibrations in the subsoil by for instance traffic.

Assessment of a slope to liquefaction

As the available data on the subsoil conditions in the Haringvliet estuary was not very detailed and slightly outdated (see chapter 3.2.6), a simple assessment of the probability liquefaction of a subaqueous slope was performed. This simple assessment is justified when at least one of the three following conditions is satisfied [CUR Bouw & Infra, 2008].

1. Liquefaction sensitivity criterion: Liquefaction is negligible if all CPT’s do not show any loosely packed soil layers, thicker than 1 meter, with relative densities $R_e$ below 0.5.
2. Geometric criterion: Liquefaction is negligible if the representative value of the slopes inclination $\alpha_R$ is milder than $1/7 \cdot (H_R/30)^{1/3}$, in which $H_R$ is the representative height of the slope (see figure 5.2).
3. A combination of liquefaction and geometric criteria: The maximum allowable layer thickness of the loosely packed soils is increased to 3 meter and the geometric criterion is expanded to the criteria that $\alpha_R$ is milder than $1/4 \cdot (H_R/30)^{1/3}$ (see figure 5.2).

In appendix E.2 is elaborated on quantitative assessment of the slopes to liquefaction. The relative densities ($R_e$) were estimated according to the relation of Baldi [Van den Ham et al., 2012] and resulted in values of $R_e \approx 0.67$. None of the soil layers had a relative density below 1.5, see appendix E.2.1.

In figure 5.2 the definitions for the slope inclination and the slope height are depicted. For a slope with an height of $H_r = 30$ meters, this resulted in allowable slopes of $\alpha_R = 1:7$ (criteria 2) and $\alpha_R = 1:4$ (criteria 3). In the case of the Haringvliet estuary, no thick impermeable soil layers are present in the top layers of the Haringvliet estuary.

![Figure 5.2: Definitions for $H_R$ and $\alpha_R$](image)
5.1. Liquefaction and breach flow of the trench slopes

(Chapter 3.2.7), that will be crossed during the excavation of the trench for the Tilting Lock. Therefore, it was concluded that the steepest allowable slope inclination regarding the potential liquefaction of the subaqueous slopes was 1:4. With two of the three criteria satisfied for a slope inclination of 1:5, it was concluded that the soil layers beneath pier 3 are not sensitive to liquefaction.

Discussion liquefaction assessment

For a detailed review of the risk of liquefaction, triaxial shear tests are inevitable to determine the sensitivity to liquefaction [Van den Ham et al., 2012]. As these tests are not available for the case study location of the Haringvliet bridge, an advanced evaluation of the liquefaction could not be performed. For more advanced studies on the potential of liquefaction, additional soil research has to be performed. Electronic density measurements, drilled samples of the loosely packed soil layers and a series of triaxial compression tests are at least required plus some custom measurements for this specific case [CUR Bouw & Infra, 2008].

5.1.2 Breach flow

In figure 5.3, the general mechanism of a breach flow is depicted\(^1\). Breach flow slides are initiated by over-steepened slope surfaces, which can exist through (virtual) cohesion due to negative pore pressures [Helbo, 1996]. Due to dilate behaviour of the soil under shear loading, the pore volume might be increased, see figure E.3 from left to right. This will result in negative pore pressures when the inflow of water is restricted. Once water has entered the voids between soil particles, the virtual cohesion will disappear and the particles will rain of the steep surface, resulting in a downhill density flow of a soil-water suspension [Helbo, 1996].

There are roughly two types of breaches, namely controlled and uncontrolled breaching [T. Raaijmakers, 2005]. The controlled breaching is often applied in dredging practice [T. Raaijmakers, 2005]. Uncontrolled breaching is considered to be a failure mode of subaqueous slopes and is, therefore, the relevant type for the design of the trench for the Tilting Lock. The initiation of the slope failure, called the initial breach, advances up-slope with a velocity related to the permeability of the soil and is a gradually process [Mastbergen, 2009]. Initial breaches can be caused, amongst others, by dredging, local erosion, liquefaction or flow slides [Van den Ham et al., 2012]. Breach flow slides also might occur due to the falling or the pouring of soil on top of the slope, which could result in a density flow along the slope. The location of the initial breach has a significant impact on the magnitude of the damage [Van den Ham et al., 2012]. The lower the location of the initial breach, the larger the impact zone as could be seen in figure 5.3 at point (1).

The sediment particles that rain down along the slope, result in a density flow of a soil-water suspension [T. Raaijmakers, 2005]. This is depicted in figure 5.3 by the rightwards directed arrows. If the slope is steep and long enough, the density flow might accelerate and cause even more erosion, because the high velocity and the thickness of the flow may result in Reynolds numbers above the criterion for turbulent flow [T. Raaijmakers, 2005]. Due to erosion, the volume and velocity of the

\(^1\)Figure contains designations in Dutch.
density flow increase till the slope inclination is sufficiently mild to decrease the flow velocity and cause the particles to settle [Van den Ham et al., 2012]. A breach flow intensifies itself, as the raining off of particles will cause the slope to steepen even more [Mastbergen, 2009]. This can lead to the start of new breach flows, as depicted at dots (2), (3) and (4) in figure 5.3

Breach flow slides are only likely to occur in slopes of medium to densely packed fine sand or silt, as these soils have a very low permeability and therefore are able to maintain negative pore pressures for some time [T. Raaijmakers, 2005]. In contrast to liquefaction, the breach will only occur at the surface of a slope and advance gradually and is possible in coarse and densely packed sand which is not liquefiable [Van den Ham et al., 2012].

To be able to evaluate a potential occurrence of the slopes by breach flow mechanism, it is vital to know the soil structure and the grain distribution of the soil layers [Van den Ham et al., 2012]. The hydraulic heads of the soil layers could be of use, as gradients could influence the soil stresses.

**Assessment of breach flow**

To check whether breach flow is a feasible failure mechanism, the three conditions as presented in the CUR 113 will be checked [CUR Bouw & Infra, 2008]. There are two kinds of conditions present, based on the soil parameters and on the slope geometry.

The available data on the subsoil conditions in the Haringvliet estuary was not very detailed and slightly outdated, as discussed in chapter 3.2.6. Therefore, a simple assessment was performed to determine the sensitivity of the subsoil layers of the Haringvliet estuary that will be in contact with the trench of the Tilting Lock. The simple assessment methods available for breach flow failures, where mainly determined for the ‘controlled’ type of breach flow as used in the dredging practice. In this thesis, these simple assessment methods were assumed to be sufficiently satisfying to assess the probability of a breach flow along a slope.

**Soil parameters assessment**

In the assessment methods, a subaqueous slope is considered to be sensitive to breach flow when one of the following conditions are met [CUR Bouw & Infra, 2008].

- The soil contains only sand particles \(D_{50} > 200 \mu m\) and \(D_{15} > 130 \mu m\) or gravel particles \(D_{50} > 500 \mu m\) and \(D_{15} > 250 \mu m\).
- No interfering soil layers (clay or loam) thicker than 0.5 meters.
- No layers sensitive to liquefaction.
- Maximum height of the subaqueous slope of 40 meters.

Based on the qualitative evaluation of the soil parameters, it can be concluded that the slopes are likely to be sensitive for breach flow slides and therefore should be checked in a more advanced assessment.

**Slope geometry assessment**

From the guidelines for the ‘controlled’ breach flow for dredging [Van den Ham et al., 2012], the following statements for a slope design resistant against breach flow failure were drawn. These statements can be used in future designs to reduce the sensitivity of the subaqueous trench slopes to breach flow slides.

- Non-continuous slope, for instance with a berm or protection zones. In these areas, the density flow will be decelerated and the particles in the density flow will be able to settle.
- Prevention of an initial breach
- Slope heights less than 10 meter high were considered to be unfavourable for breach flow slides.
- Slope inclinations should be sufficient mild, so no soil-water suspension density flow can occur. Slopes with inclinations between 1:4 and 1:6 were considered to be sufficient to create a density flow along the slope.

For the design of the slopes, the turning point for breach flow is around slope angles of 1:4 to 1:6...
and lengths above 10 meter [Van den Ham et al., 2012]. It is chosen to consider 1:5 as satisfying for this moment. The same value was taken during the assessment of the scour holes near the Eastern Scheldt barrier, where the local inclination should not exceed 1:5 over stretches of 5 meter or longer [Stoutjesdijk et al., 2012].

Although more than sufficient slope length will be available to develop critical velocities in a density flow at the slope surface, it is possible to include protecting measures to reduce the probability of occurrence.

**Discussion breach flow assessment**

Advanced methods to assess the probability and consequences of breach flow failures are present. For instance the software package HRMbreach by WL—Delft Hydraulics in cooperation with Deltares [Mastbergen, 2009]. For the detailed assessment of the subaqueous slopes on breach flow failure, more data on the subsoil conditions are required, like for instance, the particle size distribution and the porosity of the particles [Mastbergen, 2009].

**Conclusions**

According to the statements related to breach flow failures, it was concluded that the subaqueous slopes of the trench required for the Tilting Lock will be sensitive to breach flow failure under an inclination of 1:5. However, for the remaining of the thesis, it was assumed that an inclination of 1:5 for the subaqueous slopes of the trench for the Tilting Lock will be sufficient. In future research, the sensitivity to breach flow should be studied into more detail.

### 5.2 Macro stability

When the loadings in a slip circle are not in equilibrium, macro stability can occur. As can be seen in figure 5.4, there are in general two types of loadings, the driving forces and the resistance along the slip plane. Driving forces in a sliding circle are for instance the weight of the soil body and the loads on top of the soil surface on the active side. Resistance is in general caused by the shear resistance along the surface of the sliding circle (the slip plane) and the soil weight and additional loads at the passive side of the sliding circle. Additional resistance could be added by structures that penetrate the slip plane of the sliding circle.

Previously performed analyses on sliding circles in subaqueous slopes (without top loads by structures), showed that macro-stability is not the normative failure mode, unless weaker soil layers are present [T. Raaijmakers, 2005]. For the case study location of the Haringvliet bridge, these weaker soil layers are present (see chapter 3.2.6). In addition, significant loads on top of the slope are present due to the bridge piers, which will add additional contributions to the driving and or resisting moments. Therefore, it was expected that the macro stability will be the main failure mode for the subaqueous slopes of the trench that is required for the Tilting Lock.

**Presence of bridge pier**

The piers of the Haringvliet bridge will add a moment to the equilibrium of the sliding circles of the subaqueous trench slopes. Whether the presence of the bridge piers is an active or passive contribution to the momentum equilibrium, depends on the location of the piers in relation to the center of rotation of the sliding circle, see figure 5.5.

To assess the influence of the top loads by the Haringvliet bridge piers, the effects are studied. A subdivision is made between shallow founded structures and structures on pile foundations. A shallow founded structure on the active side of the sliding circle will contribute to the driving
moments of the equilibrium (figure 5.5a), while the shallow founded structure on the passive side of the sliding circle will add resistance (figure 5.5b) [Y.R. Jongerius, 2016]. Pile founded structures can add both driving forces and resistance to the sliding circle equilibrium, when on the active side. Due to the penetration of the slip plane, the piles will add resistance against sliding (figure 5.5c). Although a structure might not be in the normative sliding circle of a slope, it can have influence on the normative slope. Due to the resistance of the pile foundation, the normative sliding circle can be forced to another location as depicted in figure 5.5d.

![Figure 5.5: Influence of a building to the moment equilibrium of a sliding circle [Y.R. Jongerius, 2016].](image)

5.2.1 Sliding circle theory

Many methods for checking the macro stability of slopes are based on dividing the earth mass of the sliding circle into slices with a width \( b \) and a height \( h \), see figure 5.6. After the subdivision of the sliding circle into an arbitrary amount of slices, the forces working on each slide are determined. By considering the moment relative to the center of a sliding circle (with radius \( R \)), the driving and the resisting moments could be determined [Vrijling et al., 2016]. The safety factor of a sliding circle is obtained by dividing the resistance moment by the driving moments, see equation 5.1.

\[
F = \frac{M_r}{M_s + M_w + M_l}
\]

In which:
- \( F \) Factor of safety
- \( M_r \) Resistance moment by shear stress \( kNm/m \)
- \( M_s \) Driving moment by soil weight \( kNm/m \)
- \( M_w \) Driving moment by water pressure \( kNm/m \)
- \( M_l \) Driving moment by loads \( kNm/m \)

In the analysis of the macro stability of slopes is always searched for the normative sliding circle (the plane with the lowest factor of safety \( F \)). Each sliding circle has a radius and center point, which can be different for each sliding circle. The location and size of the normative sliding circle are in general found by multiple iterations, which can be a time consuming task. Software programs, like D-Stability by Deltares, are developed to do the iterative calculations. For values of \( F \) above 1.2 (for temporary works) or 1.3 (for permanent works), the stability check was considered to be satisfied [Vrijling et al., 2016].
5.2. Macro stability

The driving moment on a sliding circle is mainly depending on the mass of the earth (equation 5.2), the water pressure on top of the soil slice (equation 5.3) and the loads on top of the sliding circle (equation 5.4). The moment of resistance (equation 5.5) mainly consists of the shear resistances along the sliding circle. The resistance moment is in depending on the shear stresses \( \tau \) along the slip plane of the circle. The shear strength along the slip plane is depending on the effective stress, the angle of friction and the cohesion of the soil in the considered slice (equation 5.5).

\[
\begin{align*}
\text{Driving moment:} & \quad M_s = R \cdot \sum \gamma_s \cdot b \cdot h \cdot \sin(\alpha) \\
M_w &= R \cdot \sum \gamma_w \cdot b \cdot d \cdot \sin(\alpha) \\
M_l &= R \cdot \sum F \cdot \sin(\alpha)
\end{align*}
\]

\[
\text{Resisting moment:} \quad M_r = R \cdot \sum \frac{b \cdot \tau_{slip}}{\cos(\alpha)} \quad \text{with:} \quad \tau_{slip} = \frac{c + \sigma'_v \cdot \tan(\phi)}{F + \tan(\alpha) \cdot \tan(\phi)}
\]

In which:
- \( b \) Width soil slice
- \( h \) Height soil slice
- \( d \) Average water depth above a soil slice
- \( R \) Radius slip circle
- \( \alpha \) Angle of \( \sigma'_n \) to the vertical axis
- \( \gamma_s \) Volumetric weight of soil \( kN/m^3 \)
- \( c \) Cohesion
- \( \phi \) Angle of internal friction
- \( \sigma'_v \) Effective soil stress \( kN/m^2 \)
- \( \tau_{slip} \) Shear stress along slip plane

5.2.2 Scenarios for macro stability checks

Multiple scenarios were assessed for the checks on the macro stability of the subaqueous slopes of the trench for the Tilting Lock. The scope of this thesis is limited to the following scenarios:

1. Slope stability general slopes, see figure 5.4
2. Slope stability of a single slope including the Haringvliet bridge piers, see figure 5.5

These scenarios are checked for the assumed slope inclination of 1:5, based on the elaborations on the potential liquefaction (5.1.1) and breach flow failure (5.1.2) of the Tilting Lock trench slopes. Other slope angles for were considered to be beyond the scope of this thesis. For the subaqueous slopes without any external loads present (scenario 1), no macro instability was expected to occur, as for these slopes the failure mode of micro instability will be normative [T. Raaijmakers, 2005].

The loads by the piers of the Haringvliet bridge were taken into account in scenarios 2. It was assumed that only the vertical forces on the bridge piers were normative for the slope stability. The horizontal loads on the bridge piers were assumed to be transferred to the subsoil by the foundation piles. At the pile tip level, this results in a linear distributed load. The resulting loads are not different from the situation when only the vertical forces on the foundation of the bridge pier are present. The resulting distributed load by the bridge pier that was taken into account, was 193.5 \( kN/m^2 \) over a width of 14.8 meter (chapter 6.2)\(^3\).

For scenario 2, macro instability of the slopes was expected at the bridge pier halfway the slope, because of the large top load and relatively small amount of soil layers that can provide resistance. The bridge pier on top of the slope was expected to be sufficiently embedded in the subsoil, to prevent large influences on the macro stability.

\(^2\)The water pressures will have an influences on the shear stress along the slip circle, as the effective soil stresses are affected by the water pressure. However, the water pressure does not add a driving moment.

\(^3\)The weight of the soil inside the cofferdam was not taken into account in the defined load condition. Therefore, the load on the soil surface of the trench slope will be higher in reality. However, it was not expected that this would lead to significant differences in the results of the calculations on the macro stability.
5.2.3 D-stability calculations

As the search for the normative sliding circle is a labour intensive task, D-stability was used. For an extended explanation of D-stability calculations and the results, see appendix E.3. The input for the D-stability calculations summarised below. The loads on the slope by the bridge piers were applied as distributed load on the soil surface, see figure 5.7.

- Water level: 0 meter NAP.
- Bottom levels: -8 to -38 meter NAP (Slope height of 30 meters).
- Slope inclination of 1:5.
- Uniform distributed loads: 193.5 kN/m² over 14.8 meters (see chapter 6.2)³.
- Soil level according to the conditions of CPT 9 (see chapter 3.2.6)⁴.
- Forbidden lines at the locations of the bridge piers till a depth of -30 meters NAP.

![Figure 5.7: Input of D-stability for the check on macro stability of the subaqueous trench slopes.](image)

Results macro stability

In appendix E.3.3 the results of the D-stability calculations are discussed in more detail. The D-stability calculations for the first scenario, without any surface loads present, did not result in any normative slip circle as was expected. In scenario 2 no normative sliding circles are found for the upper bridge pier, as can be seen in figure 5.8a. The green areas represent sliding circles with a safety factor over 1.5. Orange areas are safety factors between 1 and 1.5, while red areas represent safety factors lower than 1.0.

The results of scenario 2 showed multiples sliding circle around the lower Haringvliet bridge pier (see figure 5.8a). The normative sliding circle is on the downhill side of the pier on the slope. The normative circle has a safety factor of 1.21 and a radius of 22.99 meter. Therefore, it was concluded that the macro stability of the slopes for the trench, subjected to loads by the Haringvliet bridge piers, is insufficient in open excavations. In a more detailed calculation that was focused on the lower pier of the Haringvliet bridge, the normative circle has a safety factor 1.15 and a radius of 19.63 meter, see figure 5.8b. This safety factor is below the required 1.3 for permanent slopes. The safety against macro stability was considered to be insufficient to provide a safe design of the trench.

For the stability of the bridge piers, a larger cofferdam was required. It was expected that the retaining walls for the cofferdam will have lengths of approximately 50 to 60 meters and therefore

---

³The soil conditions of CPT 9 were used, as this CPT contains the worst soil conditions of the four CPT's available for the case study location at the third and fourth piers of the Haringvliet bridge.
reach to -50 to -60 meter below NAP. These retaining walls will provide for additional resistance against the occurrence of sliding circles. This additional resistance was expected to be more than sufficient to provide for the missing safety in the safety factor. Therefore, it was concluded that the slopes of the trench that is required for the Tilting Lock, are stable under an angle of 1:5 and thus feasible.

Figure 5.8: Results D-stability calculation for the macro stability of the subaqueous slopes of the trench for the Tilting Lock at the cross section where the Haringvliet bridge piers are present.

5.2.4 Verification of the normative sliding circle of D-stability

To verify the software program D-stability, which was initially developed for stability checks on dikes, a supplementary calculation was made to compare the results. The method as discussed in chapter 5.2.1 was used. The normative slip circle was divided into three soil slices, see figure 5.9a. For each soil slice the contributions to the driving and resistance moments was calculated.

As can be seen in equation 5.6, the resulting safety factor is 1.06, which is lower than found in the D-stability calculation. This is probably due to the size of the soil slices, which is in the hand calculation quite large. For less soil slices in D-stability, a lower safety factor is found as well. Therefore was concluded, that the calculation in D-stability gives sufficient reliable results.

\[
F = \frac{M_r}{M_s + M_w + M_l} = \frac{(M_{r,1} + M_{r,2} + M_{r,3}) + M_s}{(M_{s,2} + M_{s,3}) + M_s + M_l} = \frac{(1078 + 2664 + 7807) + 383}{(1319 + 577) + 0 + 9298} = 1.06 \tag{5.6}
\]

Figure 5.9: Results verification of D-stability.

(b) Used parameters: \(\gamma_{s,eff} = 12\), \(R = 19.6\), \(\phi = 30^\circ\), \(c = 0\).
CHAPTER 5. STABILITY OF THE SUBAQUEOUS TRENCH SLOPES

5.3 Conclusions on the slope stability

In the coming paragraphs the discussion (5.3.1), the conclusions (5.3.2) and the recommendations (5.3.3) regarding the slope stability of the trench that is required for the Tilting Lock are discussed.

5.3.1 Discussion on the slope stability calculations

During the elaboration on the slope stability of the subaqueous slopes of the trench for the Tilting Lock, multiple assumptions and simplifications were made to determine the feasibility of the trench slopes under an angle of 1:5 (≈ 11.3°). These most important rough assumptions that will have an effect on the slope stability are discussed in the following paragraphs.

Additional scenarios of macro stability

In the evaluation of the macro stability of the subaqueous slopes for the trench of the Tilting Lock, only a single slope was considered. The actual trench consists of two identical slopes, with both bridge piers present on their surfaces. Therefore two additional scenarios to the macro stability can be considered.

3. Slope stability for double slopes (figure 5.10).
4. Slope stability for double slopes including bridge piers.

The scenarios 3 and 4 were considered too complex to review in a relatively simple macro stability calculation as performed in this thesis or D-stability. When the two slopes are taken into account in a moment equilibrium calculation, sliding circles are likely to intersect and potentially intensify each other. The downhill soil slices in the momentum equilibrium contribute to the resistance momentum of the equilibrium in a sliding circle. In the deepest sections of the trench, the soil slices will be affected by the slip circles from both the left and the right slopes. This might reduce the strength, and thus the resistance, of the soil slice and therefore reduce the resistance of the sliding circle. Till what extent this will have an effect on the normative sliding circles, should be investigated in more detail in future research.

Horizontal loads on piers

The horizontal loadings on the bridge pier were not taken into account in the calculations on the macro stability of the subaqueous slopes of the trench for the Tilting Lock as discussed in 5.2.2. However, the horizontal loads of the bridge piers have to be transferred to the subsoil to make horizontal equilibrium. These horizontal loads will be transferred over the embedded depth of the two retaining walls of the cofferdam. To make horizontal equilibrium, the passive soil pressure should be able to withstand those horizontal loads. This is a different failure mode (horizontal shear sliding), which was considered to be out of the scope of this thesis. However, in more detailed calculations, the influence of the horizontal loads should be taken into account.

Swell of the subsoil

One of the main aspects that was not taken into account in the stability checks of the subaqueous slopes of the trench for the Tilting lock, is the potential swell of the subsoil layers. Due to the excavation of the trench, the stress in the subsoil layers will decrease, which will lead to swell. Due to the decrease of effective stresses, small deformations in the subsoil are likely to occur and the bottom level is expected to rise [Vrijling et al., 2016]. Swell of soil layers will lead to less compacted soil layers and which are more susceptible to liquefaction [T. Raaijmakers, 2005]. It was expected that the swell of the soil layers at greater depths, will lead to a changed situation for the slope stability of the slopes in the trench for the Tilting Lock. Therefore, it is important to take the effects of swell into account in future research.
5.3. Conclusions on the slope stability

Other failure mechanism for subaqueous slopes
In appendix E.1.1, multiple failure mechanisms for subaqueous slopes were discussed to come to the conclusion that the liquefaction, the breach flow and the macro stability failures were the most likely to be the normative conditions. However, the other types of failure should not be neglected in further research.

Three-dimensional effects
The calculations in this chapter are made on simplifications of the lateral cross section of the piers of the Haringvliet bridge. Within these simplifications, the geometry, the loads and the resistances were averaged over the longitudinal cross section of the piers of the Haringvliet bridge and the slopes of the trench for the Tilting Lock. In reality, the situation will be different due to several three-dimensional effects, as can be seen in the example in figure 5.11. The resistance along a sliding circle will be more, because the surface of the sliding plane would be relatively larger that in the two-dimensional situation.

Slope inclination of 1:3
For a slope of 1:3, the pier of the Haringvliet bridge will be much more embedded in the subsoil as depicted in figure 5.1. The macro stability of the slopes in the general trench areas, where no bridge piers are present, will have lower safety factors. It was expected that the stability of the trench slopes regarding macro stability would be not endangered for lower inclinations.

Steeper slope inclinations can also be reached by applying strengthening measures for the slopes. An example of these measures is depicted in figure 5.12, where grout columns penetrate the normative sliding circle.

Measures to reduce the sensitivity to liquefaction
To decrease the sensitivity of the slopes to liquefaction, several options are available. The probability of liquefaction can be reduced by compaction of the soils or construction stone columns, to increase the permeability of the soil layers [Ajanta Sachan, 2013]. The resistance of the soil layers against horizontal movements can be increased by driving additional piles through the liquefiable soil layers to stronger layers, or constructing grouting columns [Ajanta Sachan, 2013]. Compacting the soil layers, for instance by vibrations or deep compaction methods, can be a good option as well to prevent liquefaction [Helbo, 1996].

The foundation piles, driven through a potentially liquefiable soil layer, not only have to resist the vertical loads from the bridge piers, but also the horizontal loads and bending moments due to lateral movements of the liquefied layers. When the foundation piles have insufficient resistance, liquefaction is potentially endangering the stability of the pier foundation of the Haringvliet bridge [Ajanta Sachan, 2013].

Other potential locations for the Tilting Lock
The stability of the subaqueous slopes does very much depend on the local soil conditions, which might differ significantly for other locations than the Haringvliet. The geometry of the trench, the soil conditions, the location of the bridge piers and the foundation of the bridge piers will differ for each location. Therefore, no conclusions on the general slope stability of the trench can be drawn.
5.3.2 Conclusion

From the performed calculations was concluded that no critical aspect related to the stability of the subaqueous slopes was found. The liquefaction and breach flow failure modes were found to be normative for the required slope inclination of the subaqueous slopes, as was shown that the macro stability will not be the normative failure mode.

The subsoil of the Haringvliet is potentially sensitive to liquefaction, which should be studied in more detail in further research. The subaqueous slopes as presented in chapter 4.2 can be designed such, that they are less sensitive to breach flow failure, as discussed in 5.1.2. For more detailed assessments of these two failure modes, more detailed data on the local soil conditions in the Haringvliet estuary are required.

In the case of an open excavation around the piers of the Haringvliet bridge, the moments by the resistance are larger than the driving moments for the normative sliding circle. However, the found safety factor (1.15) was insufficient to completely exclude the failure by macro instability in the case of open excavation for the trench.

The presence of the cofferdam will increase the stability of the overall slope [TAW, 2004]. The retaining walls of the cofferdam will penetrate the normative slip plane by a couple of meters and were considered to add sufficient resistance to the normative sliding circle and the other sliding planes prone to failure, to increase the safety factor to the required levels. Therefore, it was concluded that in the trench slopes sufficient resistance against macro stability failure was present at the case study location of the Haringvliet bridge.

5.3.3 Recommendation

Recommendations for the most critical discussion points related to the stability of the subaqueous slopes are discussed in the following paragraphs.

Improve quality of calculations

For the static stability, a calculation which takes the 3D effects into account should be made, like is possible in the FEM-software PLAXIS. PLAXIS could be used to optimise the design and take the quantification of the effects of liquefaction, breach flow and macro stability into account, together with the 3D-effects. An additional benefit of PLAXIS is that the interaction between the structures (Haringvliet bridge piers and the cofferdam) can be taken into account.

Accurate data on the subsoil conditions in the Haringvliet

For more detailed checks on the sensitivity of the soil layers to liquefaction, breach flow, macro stability and other failure mechanisms, more detailed information is required about the subsoil conditions at the Haringvliet bridge. Therefore, it is recommended to perform additional tests on the soil conditions of the Haringvliet near the Haringvliet bridge to update the available data.

Redesign of the subaqueous slopes

According to the statements related to breach flow failures, it could be concluded that the subaqueous slopes of the trench required for the Tilting Lock will be sensitive to breach flow failure. It is recommended to adjust the design of the subaqueous slopes. Several measures to improve the sensitivity to breach flow are present, see chapter 5.1.2. For instance the inclusion of a berm half way the slopes to decelerate then flow velocities of the density flow (black in figure 5.13).

Under normal morphological conditions, downstream slopes will always flatten and stretch out over a considerable width, while upstream slopes will migrate and can show steepening as well as flattening behaviour [T. Raaijmakers, 2005], (red in figure 5.13). When this variation of the slope geometry would be applied, it would not yield very large reductions of dredging volumes, but the morphological flow profile would be much better without concessions on the slope stability [T. Raaijmakers, 2005]. This would lead to less redistribution of the slopes after the construction of the trench.
5.3. Conclusions on the slope stability

In future research, the design of the slopes for the trench of the Tilting Lock can be optimised to reduce the probability of failure and to reduce the amount of sedimentation in the trench. This optimisation can be done according to the following steps.

- The sensitivity to liquefaction of the trench slopes can be reduced by implementing measures in the case study design. Examples of these measures are:
  - Compaction of soil layers
  - The construction of gravel columns to increase the permeability of the subsoil layers [Ajanta Sachan, 2013].
  - Additional foundation piles to increase the resistance of the soil layers against horizontal movements [Ajanta Sachan, 2013].

- The sensitivity of the subaqueous trench slopes to breach flow failure can be reduced by adding a berm to the slopes (black lines in figure 5.13). Due to the berms, the magnitude of erosion due to the density flow will reduce, as the magnitude of the developed density flow will be smaller.

- The hydrodynamic influences will cause a reshape of the slopes to less steep inclinations of the slopes at the top and bottom (red line in figure 5.13). To prevent initial erosion and sedimentation in the trench this shape can be included in the design. In addition, these flatter toes provide more stability against macro instabilities.
Chapter 6

Stability of the Haringvliet bridge piers under excavation

The stability of the Haringvliet bridge piers will be affected in the case study design, due to the large excavations for the Tilting Lock trench. In figure 6.1 can be seen that the foundation piles will exposed to large excavations. To investigate whether the excavation of the trench is feasible without additional measures to stabilise the bridge piers, it is elaborated on the stability of the bridge pier foundation of the Haringvliet bridge.

The stability of a bridge pier depends the stability of the pile foundation underneath the pier. Therefore, the loads on the piers of the Haringvliet were determined in chapter 6.2 and appendix F. These loads were compared to the original foundation design calculations by RWS (1960). The theory on the bearing capacity of the pile foundation was discussed in F.3 before the capacity of the pile foundation of the Haringvliet bridge piers was determined in 6.3 and appendix F. Conclusions on the feasibility of open excavation around the Haringvliet bridge piers are drawn in chapter 6.4.

![Figure 6.1: Simplification of bridge piers under excavations.](image)

### 6.1 Objective of the stability calculations

The objective of this chapter was to find the level of excavation, for which no additional measures to stabilise the bridge pier were required. This was assessed by a static analysis of the load cases versus the pile resistance of the piers of the Haringvliet bridge. Based on a simple rigidity analysis, it was chosen to assess the stability in only the lateral cross section of the Haringvliet bridge piers. It was considered that the longitudinal cross section of the bridge pier is much stiffer and therefore the pier is less likely to fail in that direction.
CHAPTER 6. STABILITY OF THE HARINGVLIET BRIDGE PIERS UNDER EXCAVATION

The excavation of the soil around the Haringvliet bridge piers, will affect both the bearing capacity of the pile foundation and the magnitude of the loads on the foundation. When the magnitude of the excavation increases, the bearing capacity of the foundation piles will be decreasing. In addition, the loads on the pile foundation will increase, because of the larger momentum by the horizontal loads.

6.1.1 Failure modes bridges

The main reason to review the stability of the piers was to prevent the Haringvliet bridge from becoming unserviceable. It was considered in this thesis that the serviceability of the bridge mainly depends on the stability of the bridge piers. When bridge piers are settling (see figure 6.2), the bridge deck was expected to deform accordingly to the settlements, which would induce a rotation of the bridge deck. The boundary conditions for the rotation of the bridge deck are:

- The bridge deck will not fail structurally if a local rotation due to (uneven) settlements is less than $\beta = 1:100$ (Ultimate Limit State (ULS)).
- The bridge deck will not become unserviceable if the rotations of the deck are less than $\beta = 1:300$ (Serviceability Limit State (SLS))
- The structural bearings of the bridge deck allow for $\delta_{wh} 0.05$ meter of horizontal deformations.

![Figure 6.2: Considered bridge failure modes regarding the open excavation](image)

The allowable rotations of the bridge deck are discussed in appendix F.1 and related to boundary conditions on the settlements of the Haringvliet bridge piers, see table 6.1.

<table>
<thead>
<tr>
<th></th>
<th>SLS</th>
<th>ULS</th>
<th>Depending on:</th>
</tr>
</thead>
<tbody>
<tr>
<td>Horizontal deformation</td>
<td>0.05 meter</td>
<td>0.05 meter</td>
<td>Failing of the bearings</td>
</tr>
<tr>
<td>Vertical deformation</td>
<td>0.18 meter</td>
<td>0.53 meter</td>
<td>Allowable rotation of bridge deck</td>
</tr>
<tr>
<td>Unequal settlements (rotation)</td>
<td>0.015 meter</td>
<td>$\approx 1:800$</td>
<td>Max. hor. deformation bearings</td>
</tr>
</tbody>
</table>

Table 6.1: Boundary conditions for the deformations of a Haringvliet bridge pier under excavation.

Scope limitation

Structural failure of one of the elements in the Haringvliet bridge pier were considered to be out of the scope. It is a realistic failure mode that the foundation piles will fail under a load combination, for instance on the shear capacity by horizontal loads or compression force due to a significant increase load. In this thesis was assumed that both the bridge pier and the bridge deck are rigid elements, which were not able to deform.

Till a depth of -17 meters NAP, the foundation piles of the Haringvliet bridge are completely encircled by sheet piles. Therefore, excavation of soil does not necessarily lead to a situation where the foundation piles are not embedded in soil anymore. As the current state of the connection between the sheet pile walls and the underwater concrete of the bridge piers is unknown, it was
6.2. Overview of the loads on the bridge piers

considered that the sheet pile walls are not sufficiently strong to withstand the differences in soil surface. Therefore, the pile foundation of the Haringvliet bridge piers is assessed as being fully exposed to the soil excavations.

6.1.2 Calculation approach

To determine the excavation levels that are allowed around the Haringvliet bridge piers, a static analysis was conducted to assess the induced pile loads in relation to the bearing capacity of the pile foundation. The stability of the bridge pier were assessed according to equation 6.1. The design values of the bearing capacity of the pile foundation \( R_d \) have to be larger than the design values of the loads on the individual foundation piles \( F_d \).

The design values for the induced loads and the bearing capacity are determined according to equations 6.2 and 6.3. The magnitude of the safety factors \( \gamma \) depend on the specific situation and the material. These safety factors are elaborated on in appendix F.1.2.

\[
\begin{align*}
F_k & \leq R_k & F_d & \leq R_d \\
F_{\text{rep}} &= \psi \cdot F_k & F_d &= \gamma \cdot F_{\text{rep}} \\
R_k &= \frac{R_{\text{cal}}}{\xi} & R_d &= \frac{R_k}{\gamma}
\end{align*}
\]

In which:

- \( F_k \) Characteristic load (SLS) kN
- \( F_d \) Design load (ULS) kN
- \( F_{\text{rep}} \) Representative load kN
- \( R_k \) Characteristic resistance (SLS) kN
- \( R_d \) Design resistance (ULS) kN
- \( R_{\text{cal}} \) Calculated resistance kN
- \( \gamma \) Safety factor (see appendix F.1.2)
- \( \psi \) Load factor (assumed 1.0)
- \( \xi \) Reduction factor for unfavourable permanent load

To assess the stability of the foundation of the Haringvliet bridge piers, multiple scenarios are reviewed. First of all, a difference is made between the SLS and ULS. In addition, several levels of the soil surface are reviewed. The calculations will be performed for six surface levels:

- Initial soil surface level of -4 meter NAP
- Current bottom level of -8 meter NAP
- Potential excavation levels: -10 meter NAP, -15 meter NAP, -20 meter NAP, -25 meter NAP

6.2 Overview of the loads on the bridge piers

To assess the stability of the Haringvliet bridge in the several designs for the required depth for the Tilting Lock, the loads on the bridge piers are required. Appendix F.2 elaborates extensively on the loads on the Haringvliet bridge piers. The loads on the bridge piers of the Haringvliet bridge are determined as characteristic loads \( (F_c; \text{unfactored}) \) and design loads \( (F_d; \text{factored}) \). In the following paragraphs, the loads on the Haringvliet bridge pier will be briefly discussed. Subsequently are these loads translated to loadings on the individual foundation piles.

6.2.1 Loads on the Haringvliet bridge piers

The overview of the load in lateral direction of the Haringvliet bridge piers, are included in figure 6.3 for both the original design loads and the revised loads according to the modern calculations methods as described in the Dutch standards (NEN 9997). Since the design of the Haringvliet bridge in 1960, several load conditions have been changed. In addition, the rules to assess the different loading cases have been changed as well. According to the modern calculation methods, the loadings on the bridge piers have been increased, which is mainly because of safety factors on the loads. In the original design calculations, no safety factors were applied on the loads, only on the bearing capacity of the pile foundation.
CHAPTER 6. STABILITY OF THE HARINGVLIET BRIDGE PIERS UNDER EXCAVATION

As can be seen in figure 6.3, not all the loads on the Haringvliet bridge piers were taken into account in this chapter. The neglected loads were:

- The loading by ice (Load I) was considered to be beyond the scope of this case study, as the value for ice loading in the initial design calculations of the Haringvliet bridge comparable to the force by ship collisions.
- Due to uneven settlement of a bridge pier, the pier will have a small rotation. Due to the small rotation, the vertical loads will become eccentric and induce an additional moment on the pile foundation. This contribution was sufficient small in relation to the other loadings to be neglected, see appendix F.2.8.

6.2.2 Load combinations

To determine the loads on the individual piles of the piers of the Haringvliet bridge, several loading scenarios were reviewed. These scenarios are elaborated into more detail in appendix F.1.3. For the Serviceability Limit State (SLS), it was assumed that the bridge piers were only subjected by the characteristic values of routine loadings, like the self-weight of the piers and the traffic loads. In the Ultimate Limit State (ULS) the design loads for extreme situations were taken into account, which meant for this case study a ship could collide with a bridge pier.

Besides the SLS and ULS, two scenarios regarding the water levels in the Haringvliet estuary are reviewed. High water levels will induce larger upward directed water pressures underneath the underwater concrete layer. Low water levels will result in more downward directed loads. The maximum water level +2.6 meters NAP was used and -0.4 meter NAP was used for the minimum water level in the Haringvliet estuary, see table 3.2. The resulting load combinations are depicted in table 6.2:

<table>
<thead>
<tr>
<th>Limit state</th>
<th>Loads</th>
<th>Water level</th>
</tr>
</thead>
<tbody>
<tr>
<td>SLS</td>
<td>ABCDIE</td>
<td>-0.4 m NAP</td>
</tr>
<tr>
<td>ULS</td>
<td>ABCDIE</td>
<td>-0.4 m NAP</td>
</tr>
<tr>
<td>ULS</td>
<td>ABCDIEK</td>
<td>-0.4 m NAP</td>
</tr>
<tr>
<td>ULS</td>
<td>ABIE</td>
<td>+2.6 m NAP</td>
</tr>
<tr>
<td>ULS</td>
<td>ABIK</td>
<td>+2.6 m NAP</td>
</tr>
</tbody>
</table>

Table 6.2: Load combinations for the assessment of the stability of the Haringvliet bridge piers.

---

2In the calculations of the self weight, the sheet pile walls are neglected, as the connection between the sheet pile walls and the bridge piers are unknown. Therefore, it is unknown if the connection between the sheet pile walls and the bridge piers are sufficient to exchange forces. This is justified, as sheet pile walls are considered to provide for their own stability.
6.3. Bearing capacity of the Haringvliet bridge foundation

6.2.3 Individual foundation pile loads

The individual pile loads of the foundation of the Haringvliet bridge piers were determined by using a simplified design of the lateral cross section of the bridge piers. The inclined foundation piles were simplified to sixteen pile rows, in eight points of the lateral cross section of the bridge pier. The resulting thirty-two vectors were in equilibrium with the horizontal and vertical forces on the bridge piers. For a more detailed explanation, see appendix F.2.9. The maximum individual pile forces in the lateral cross section of the Haringvliet bridge piers are depicted in table 6.3 for discussed load combinations and several levels of excavation around the bridge piers.

<table>
<thead>
<tr>
<th>Soil level m NAP</th>
<th>Compression in foundation</th>
<th>Tension</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>SLS: ABCDEI</td>
<td>ULS: ABCDEI</td>
</tr>
<tr>
<td>-8</td>
<td>589 338</td>
<td>762 473</td>
</tr>
<tr>
<td>-10</td>
<td>594 334</td>
<td>768 467</td>
</tr>
<tr>
<td>-15</td>
<td>634 294</td>
<td>821 414</td>
</tr>
<tr>
<td>-20</td>
<td>663 264</td>
<td>862 373</td>
</tr>
<tr>
<td>-25</td>
<td>683 245</td>
<td>889 346</td>
</tr>
</tbody>
</table>

Table 6.3: (Extreme) individual pile loads ($F_d$) for the different load combinations. The governing water levels are at maximum +2.6 meter NAP and at minimum -0.4 meter NAP.

The calculated individual pile loads do contain a small error, due to the simplification of the pile lay-out. As the majority of the piles are located closer to the centre of the pier in reality, the piles will bear less stresses as calculated in table 6.3. This is favourable when the individual foundation piles are examined, but unfavourable for the calculation of the extreme stresses in the outer piles. However, it was considered that this discrepancy is covered sufficiently by the usage of safety factors in the calculations.

Distributed loads

The loads on the Haringvliet bridge piers, transferred through the foundation piles to the subsoil, will eventually be distributed over a certain area. This distributed loads were calculated by equation 6.4 at the level of the tips of the foundation piles. In table 6.4 the resulting distributed loads are included. Summation of the vertical loads on the Haringvliet bridge piers result in an equally distributed load of 193.5 kN/m$^2$ at the pile tip level.

$$\sigma = \frac{\sum V}{w \cdot l} + \frac{\sum M}{1/6 \cdot l^2 \cdot w^2} \tag{6.4}$$

In which:

- $\sum V$ total of the acting vertical forces \(kN\)
- $w$ width of the assessed area \(14.8 m\)
- $l$ length of the assessed area \(33.2 m\)
- $\sum M$ total of the acting moments \(kNm/m\)

6.3 Bearing capacity of the Haringvliet bridge foundation

To check whether sufficient bearing capacity is present in the subsoil of the Haringvliet bridge piers, the bearing capacity of a single foundation pile is determined. An assessment of the pile group was not required, as a check on the bearing capacity of a single pile is sufficient for rigid structures.\(^5\)\[Nederlands Normalisatie-instituut, 2012.\]

\(^3\)Although the piles are in an inclined position, this will not be taken into account in this calculation, as the inclination is rather small (1:10). The most severe influence for the piles is on the shaft friction length, which is smaller in case the inclination is neglected.

\(^4\)The weight of the soil layers between the foundation piles of the bridge piers were excluded from this calculation.

\(^5\)As the foundation of the Haringvliet bridge piers consist of a 3 meter thick concrete layer, in which all the pile heads were embedded during the casting of the concrete, the bridge pier was considered to be rigid.
CHAPTER 6. STABILITY OF THE HARINGVLIELT BRIDGE PIERS UNDER EXCAVATION

<table>
<thead>
<tr>
<th></th>
<th>A</th>
<th>B</th>
<th>C</th>
<th>D</th>
<th>E</th>
<th>I</th>
<th>K</th>
</tr>
</thead>
<tbody>
<tr>
<td>ULS +2.6 m NAP</td>
<td>67.6</td>
<td>22.1</td>
<td>21.3</td>
<td>2.2</td>
<td>44.0</td>
<td>31.1</td>
<td>98.7</td>
</tr>
<tr>
<td>ULS -0.4 m NAP</td>
<td>124.9</td>
<td>29.4</td>
<td>35.5</td>
<td>3.7</td>
<td>14.1</td>
<td>20.2</td>
<td>47.5</td>
</tr>
<tr>
<td>SLS -0.4 m NAP</td>
<td>94.7</td>
<td>24.5</td>
<td>23.7</td>
<td>2.4</td>
<td>9.4</td>
<td>20.2</td>
<td>47.5</td>
</tr>
</tbody>
</table>

Table 6.4: Distributed (extreme) individual pile loads for the different load contributions on the piers of the Haringvliet bridge.

6.3.1 Pile bearing capacity theory

In appendix F.3 it is elaborated on the theory of the bearing capacity of foundation piles. As can be seen in figure 6.4, the bearing capacity of a foundation pile under compression consists of three main components:

- Tip bearing capacity ($R_{b;cal}$)
- Positive shaft friction ($R_{s;cal}$)
- Negative shaft friction

The main part of the resistance against settlements of a foundation pile is in general obtained by the tip resistance, which can be calculated with the method of Koppejan\(^6\) (appendix F.3.1). The shaft friction (appendix F.3.2) works over the complete length of the foundation piles and can be either positive or negative with regard to the bearing capacity of the piles\(^7\).

When soil is excavated around foundation piles, the amount of shaft friction will decrease. In addition, the subsoil layers will be relieved, which leads to less bearing capacity. This loss of resistance can be related to a reduction in cone resistance [Nederlands Normalisatie-instituut, 2012]. The reduction of the cone resistance will affect both the pile tip bearing capacity and the shaft friction along a foundation pile (appendix F.3.3).

6.3.2 Results bearing capacity calculations

In appendix F.4 is elaborated on the bearing capacity of the pile foundation of the Haringvliet bridge. The calculations on the bearing capacity were performed for the CPT with the worst conditions regarding the bearing capacities of foundation piles as discussed in section 3.2.6.

The bearing capacity of a single foundation pile was determined for different levels of soil excavation, as discussed in 6.1.2. The contributions of the tip bearing capacity ($R_{b;cal}$) and the positive pile shaft friction ($R_{s;cal}$) were combined to calculate the total bearing capacity ($R_{c;cal}$). Subsequently, the characteristic and design values for the bearing capacity were calculated according to equations 6.2 and 6.3. In table 6.5a the results of the calculations on the bearing capacities are included\(^8\).

Verification of the found bearing capacity of a single pile

In the original design calculations by RWS, the bearing capacity for a single pile under the bridge pier foundation of the Haringvliet bridge was determined at 56 ton (= 550 kN) [Rijkwaterstaat - Directie bruggen, 1961]. In the past, it was common to use safety factors with values up to 2 to 3. With a safety factor of 3, the initial characteristic value as used in the original design calculations would be 1648 kN, which is very much comparable to the found values in table 6.5a.

---

\(^6\)The announced reduction of the factor related to the cone resistance in the Koppejan calculation [Nederlands Normalisatie-instituut, 2015] was not taken into account.

\(^7\)It was considered that the inclination of the foundation piles does not have significant influences on the bearing capacity of the pile foundation.

\(^8\)In the calculations, the effects of the negative shaft friction were neglected, as it was found that the ULS loading scenarios are normative in the conducted static analysis.
6.3. Bearing capacity of the Haringvliet bridge foundation

6.3.3 Bearing capacity pile foundation versus compression load cases

With both the individual pile loads and the bearing capacity of the individual foundation piles determined, the allowable excavation depth can be determined. It was checked in table 6.5b at which excavation depth, the design load at an individual pile \( F_d \) is larger than the design resistance of the soil \( R_{cd} \). The normative values for SLS and ULS are underlined in table 6.5. From the table can be concluded that the bearing capacity of the piles underneath the bridge pier is sufficient in SLS, to allow excavation till a depth of -20 meter NAP. However, the ULS was normative, as the allowable excavation is approximately -15 meter NAP.

6.3.4 Checks based on rules of thumb

To check the assumed allowable excavation to -15 meters NAP, several small check based on rules of thumb are performed to check the stability of the piers of the Haringvliet bridge. To following checks are performed:

- Rotational stability
- Vertical bearing capacity (shallow foundation)
- Settlements of the Haringvliet bridge pier

**Rotational stability**

For rotational stability of shallow foundations, it was required that the line of action of the resulting force is situated inside the core of the structure, see equation 6.5 [Vrijling et al., 2016]. The most unfavourable scenario regarding the rotational stability, would be the one with large horizontal forces and small vertical force (scenarios ULS ABIK, see table 6.3). The line of action of the resulting forces in the discussed load scenario was found to be within the moment center of the subsoil for larger excavations depths than -16 meter NAP, see appendix F.5.3.

\[
\varepsilon_R = \frac{\sum M}{\sum V} \leq \frac{1}{6} w \quad (6.5)
\]

In which:
- \( \varepsilon_R \) Distance from the centre to the point of the resulting force [m]
- \( \sum V \) Sum of the acting vertical forces [kN]
- \( \sum M \) Sum of the acting moments (Horizontal force \( \cdot \) arm) [kN\( \cdot \)m]
- \( w \) Width of the shallow foundation, depends on the excavation depth [m]
CHAPTER 6. STABILITY OF THE HARINGVLIET BRIDGE PIERS UNDER EXCAVATION

Vertical bearing capacity
As it was possible that the complete bridge pier and its foundation will be a part of the sliding plane, local vertical bearing capacity of the subsoil underneath the pile foundation was determined in appendix F.5.2. To evaluate the stability the foundation of the bridge pier was simplified to a shallow foundation. By checking the vertical bearing capacity with the method of Prandtl and Brinch Hansen [Vrijling et al., 2016], it was concluded that the subsoil have sufficiently vertical bearing capacity.

Settlement of a single foundation pile
In appendix F.5.1 the settlements of a single pile under the maximum loads were determined. The settlement of a single pile under maximum loading will be approximately 0.14 meter. It can be concluded that the boundary conditions in ULS (0.53 meter) on the overall settlements are not exceeded, as the expected settlements are only 0.14 meter. For SLS the boundary condition was 0.18 meter, which is quite close to the expected settlements in ULS. As the maximum loads in SLS are lower than in ULS and the settlements depend on the load, it is expected that this boundary condition will be satisfied as well.

It should be remarked, that the calculation of the settlement is not a representative value for the actual settlements that are to be expected. Multiple aspects that have a significant influence on the settlement of a structure were not taken into account:

- The pile foundation has already settled a couple of millimetres under the permanent loadings.
- The settlements of the bridge piers due to swell of the soil layers under excavation and the settlements of the pile foundations due to the destressing of the soil were not taken into account.
- The uneven deformations due to the differences in the individual pile loads were not taken into account.
- The dynamic loading types, like the collision with a ship, will result in an elastic deformation of the subsoil. Eventually the structure will be pushed back to its initial position.

6.4 Conclusion bearing capacity bridge piers
6.4.1 Discussion
In the following paragraphs, the discussion points of the calculations on the bearing capacity of the pier foundation of the Haringvliet bridge are discussed.

Horizontal loads
In case of a temporary high (dynamic) load, like the force by a ship collision, it is more appropriate to assess the situation by linear elastic or plastic analysis. In this method the deformations of the materials are taken into account. The elastic and plastic deformations of an element are used to absorb the large horizontal forces.

Bearing capacity
The used calculation is conservative, as the most unfavourable cone reduction value is used for all the cone resistances in the calculation. Especially for the larger excavations, this has significant influence. The bearing capacity of the pile group is not taken into account. However, the pile group has influences on the negative shaft friction, which was considered to be out of the scope. The maximum loads on one pile row are used to check the resistance, but in reality the loads will tend to redistribute over the pile group, when the loaded piles will settle. Therefore more bearing capacity of the whole pier will be present than assumed in this assessment.

Structural resistance against loads
In this thesis was assumed that the structural elements of the Haringvliet bridge piers are in good conditions, sufficient to retain the several load cases. However, in further research it should be investigated whether the capacity of the structural elements are indeed sufficient. For instance the
6.4. Conclusion bearing capacity bridge piers

Resistance of the foundation piles against horizontal deformations can be a critical issue. Important in this assessment will be the current conditions of the Haringvliet bridge piers. Small damages to the concrete structure might result in less resistance.

6.4.2 Conclusions

Based on the static analysis performed in this chapter, it was concluded that excavation around the piers of the Haringvliet bridge is allowed till a depth of -15 meter NAP, which is 7 meters below the initial bottom level of the Haringvliet. For larger excavations, the subsoil will not have sufficient bearing capacity to withstand the pile loads and the Haringvliet bridge piers will settle. The magnitude of these settlements are likely to be much more than the set boundary conditions for settlements.

With the allowable excavation depth of 7 meters around the Haringvliet bridge piers, a trench can be designed with a maximum depth of approximately 24 meters below NAP. As depicted in figure 6.5 a depth of approximately 22 meters (-30 meter NAP) was required around the piers of the Haringvliet bridge to construct the trench required in the case study design. Therefore, it was concluded that the alternative of an open trench is not sufficient to provide sufficient depth to the Tilting Lock at the Haringvliet bridge.

![Figure 6.5: Trench for the Tilting Lock related to the allowable excavations around the Haringvliet bridge pier.](image)

6.4.3 Recommendation

As the excavation of the soil, without applying any protection to the piers of the Haringvliet bridge, was found to be an unsatisfying solution for the size of the Tilting Lock, stabilising measures are required for the piers. In chapter 4.2 was already assumed that such a structure was required. For this case study, the feasibility of a cofferdam around the piers of the Haringvliet bridge was studied in chapter 7.

Calculation methods

With regards to the calculation methods of the bearing capacity of the Haringvliet bridge piers it is recommended to perform calculations on the dynamic behaviour of the bridge piers. The linear plastic and elastic deformations of the bridge piers and the subsoil should be taken into account.

Different sized Tilting Locks

Although it was concluded that the open excavation around the Haringvliet bridge piers was not satisfying for this case study, it might be for other locations or smaller sized Tilting Locks. In figure 6.5 the allowable excavation depth is drawn. As can be seen, the maximum trench depth would be around 24.3 meters. This allows for a Tilting Lock with a draft of approximately 20 meters, to keep sufficient UKC underneath the Tilting Lock.
CHAPTER 6. STABILITY OF THE HARINGVLIET BRIDGE PIERS UNDER EXCAVATION
Chapter 7

Cofferdam foundation for stability of the Haringvliet bridge piers

In chapter 6 was found that the allowable excavation around the piers of the Haringvliet bridge will not be sufficient to reach the depth that is required for the trench that is required for the implementation of the Tilting Lock in the case study at the Haringvliet bridge. Therefore, it was concluded that measures are required to stabilise the Haringvliet bridge piers.

Multiple alternatives for the stabilising measures are available, but for this feasibility study it was chosen to elaborate on a cofferdam made of steel combi wall elements, see chapter 4.2. In appendix G.1 two recent building projects are used as reference projects. From both the reference projects it was concluded that a design of a cofferdam as stabilising element of the bridge piers is feasible.

- The shallow founded bridge pier of the railroad bridge in the Waal near Lent, which was enclosed by a diaphragm cofferdam to allow the dredging of an ancillary channel.
- The construction of the North-South line in Amsterdam, where large excavations near pile founded buildings took place.

An overview of the selected design layout for the cofferdam foundation for the Haringvliet bridge piers is given in figure 7.1. In section 7.1 the global design of the cofferdam foundation is discussed, together with the potential failure modes and reference projects. To check the feasibility of the cofferdam as a stabilising measure for the Haringvliet bridge piers, the first steps in the determination of the required combi wall elements are conducted in section 7.2. In section 7.3 the performed elaborations are discussed and conclusions are drawn.

![Figure 7.1: Impression of the alternative to reach the depth that is required for the Tilting Lock: the open trench in combination with the cofferdams to stabilise the Haringvliet bridge piers.](image)
CHAPTER 7. COFFERDAM FOUNDATION FOR STABILITY OF THE HARINGVLIET BRIDGE PIERS

7.1 Global design of a cofferdam

A cofferdam is defined as a soil body, enclosed by walls, which is able to transfer horizontal and vertical forces to the subsoil [TAW, 2004]. To increase the rigidity of the cofferdam, the retaining walls of the cofferdam are connected to each other by anchors at one or multiple levels [CUR, 2005]. The main function of a cofferdam is ensuring the stability of the enclosed soil body, but a cofferdam can also be used as a water retaining structure.

As depicted in figure 7.2, a structure can be called a cofferdam if the ratio between the retaining height and the width of the cofferdam (B/H) is between 0.7 and 1.5 [CUR, 2005]. Inside a cofferdam, the passive and active soil pressure zones overlap as depicted in figure 7.2b. When the requirement is not met, the two retaining walls are not each other's area of influence 7.2c. In general a cofferdam exists of the following structural elements [TAW, 2004]:

- **Front retaining wall**: The front wall is subjected to the main elevation differences in soil surface levels. The main loads on the front wall are the active soil pressure, the horizontal stabilising force and potential top loads, depending on the situation.

- **Back retaining wall**: Provides the stability of the front wall. In case the front wall is prone to deform, the back wall will add additional resistance through the horizontal connection between the two walls. The resistance of the back wall is provided by the rigidity of the wall and the passive soil pressure from the enclosed soil body.

- **Horizontal connection (anchor)**: A horizontal connection between both retaining walls makes sure that both retaining walls work together. In general, the horizontal connection is made by anchors.

- **Enclosed soil body**: Provides for the main component of the shape retention of the cofferdam, as it consists of well compacted soil between the two retaining walls.

![Figure 7.2: Definitions of a cofferdam [CUR, 2005].](image)

7.1.1 Case study layout of the cofferdam foundation

For the case study, the layout of the cofferdam foundation of the Haringvliet bridge piers is designed. In figure 7.3 the lateral cross section and the top view of the cofferdam foundation are depicted. The considerations regarding the design of the cofferdam foundation for the Haringvliet bridge piers are discussed in the following paragraphs. The following aspects are treated:

- Width and length of the cofferdam in top view.
- Retaining height of the cofferdam walls.
- Length of the retaining walls.
- Material of the retaining walls.
- Type and location of horizontal stabilising elements.
7.1. Global design of a cofferdam

Figure 7.3: Global dimensions cofferdam in a lateral cross section (left) and a top view (right).

Width and length of the cofferdam in the top view
The width of the cofferdam was determined by the construction phase of the cofferdam foundation. The retainer walls of the cofferdam cannot be installed right next to the sheet pile walls of the Haringvliet bridge piers, due to the inclination of the foundation piles and the required distance to safely install the combi wall elements (≈2 meter [TAW, 2004]).

The rule of thumb for the dimensions of a cofferdam is that for a larger distance between the two retainer walls of the cofferdam, less rigid and less long retainer walls are required [TAW, 2004]. For larger distances, more soil is enclosed, which increases the rigidity of the cofferdam significantly. For the feasibility study the required distance between the retainer walls and the existing bridge pier was considered to be 5 meters1. This resulted in the dimensions as depicted in figure 7.3.

Retaining height cofferdam walls
To determine the required strength and length of the retainer walls of the cofferdam, the difference in height of the soil surface is required. The elevation difference in soil surface that have to be retained by the walls of the cofferdam depend on the required excavation levels for the trench that is required for the Tilting Lock. Over the lateral cross section of the Haringvliet bridge piers, the retainer walls of the cofferdam have to retain the largest elevation difference in soil surface levels at the downhill side of the bridge pier. Therefore, the retainer wall at the downhill side of the cofferdam was considered to be critical for the feasibility of the cofferdam as a foundation for the Haringvliet bridge piers2. The accompanying retaining heights for the cofferdam walls are depicted in figure 7.3.

Material of the retaining walls
In general, the retainer walls of a cofferdam are made out of steel sheet pile or combi wall elements. The main alternative for the steel walls are diaphragm walls, which are often applied in the vicinity of existing buildings [TAW, 2004] (see chapter G.1). In both reference projects for the cofferdam foundation (appendix G.1), diaphragm walls are used to obtain stabilisation of the building foundations in the vicinity of deep excavations.

1 The required distance for the retainer walls to the existing sheet pile wailes of the Haringvliet bridge pier, was considered to be 5 meters. This estimation was based on the following components:
   - The distance from the foundation pile tips to the outer edge of the sheet walls (1.3 meter)
   - The pile inclination width (2.4 m)
   - The additional width for the transfer of loads to the bottom (4 · Deq = 1.6 meter)
   - Construction tolerances (1 meter)
   - A free board space (1 meter)

2 It should be noted that the maximum required excavation depth next to the cofferdam is about equal to the foundation layer of the pile foundation of the Haringvliet bridge piers.
CHAPTER 7. COFFERDAM FOUNDATION FOR STABILITY OF THE HARINGVLIET BRIDGE PIERS

In this feasibility study was chosen to focus on a cofferdam made out of steel walls. The main reason for this choice was related to the construction phase. The underwater construction of a diaphragm wall requires, in contradiction to steel retaining walls, either a new construction method or a temporary building pit. Both measures were considered to be out of the scope of this thesis.

Type and location of horizontal stabilising elements

The collaboration between the front and back retaining walls of the cofferdam requires a horizontal connecting element. In general, this connection is made by conventional anchors, applied and prestressed at different heights of the retaining walls.

These conventional anchors were considered to be unsatisfactory in the case of the cofferdam foundation for the Haringvliet bridge piers. To install these anchors, a cable has to be driven through the enclosed soil body of the cofferdam, where the foundation piles of the bridge piers are located. The risk of hitting and damaging one or more foundation piles during the drilling of the anchors, and thus losing bearing capacity, was considered to be too high. In addition to the potential damage to bridge pier foundation, it is complicated to install conventional anchors well below the water surface. Therefore, it was concluded that an alternative for the conventional anchors was required.

The tension rings

As the alternative for the anchor connection, system of pretensioned cables around the cofferdam was proposed, also called 'tension rings'. As can be seen in figure 7.4, will these tension rings provide an inwards directed horizontal force to the retaining walls, due to pre-stressing of the cables. The cables can be applied relatively easily around the cofferdam, as they can be assembled above the water level before they are submerged to the surface of the bottom, where they can be installed. Due to the presence of the bottom, the installation height of the tension rings can be easily facilitated. During the excavation of the trench, multiple 'tension rings' can be applied at different heights.

The magnitude of the inward directed force of the tension rings depends on the local curvature of the tension rings. It is required that the shape of the cofferdam is adjusted as such, that the inward directed force will be equal over the whole circumference. For the scope of this thesis it was assumed that the tension rings are sufficient strong to provide for the required horizontal load.

Shape of the cofferdam foundation

The strength of the retaining walls depend on the global shape of the cofferdam. The contribution of the oval shape of the cofferdam will increase the rigidity of the cofferdam foundation with regards to a square version. In addition, the combination of a squared cofferdam and the tension rings as horizontal stabilising elements will induce a very complicate distribution of the internal stresses in the cofferdam walls. The influences of the shape of the cofferdam were considered to be beyond the scope of this thesis.

Length of the retaining walls

The length of embedded section of the retaining walls of the cofferdam can be estimated by the rule of thumb 0.5 to 1.0 times the retaining height [TAW, 2004]. This would result in a length of 33 to 44 meters. However, the load conditions for the cofferdam in the case study differ significantly.

---

3 A single conventional anchor only provides for a horizontal force in one direction. In case of the cofferdam foundation for the Haringvliet bridge piers, a three dimensional situation is present. This indicates that multiple anchors at a single height level are required to stabilise the retaining walls in all directions.

4 Other alternative to fixed the horizontal position of the retaining walls can be a concrete girder on top of the retaining walls. By pouring a reinforced concrete girder or plate, the top of the walls can be connected and thus rotatable fixed. However, the elaboration of this alternative was considered to be beyond the scope of this thesis.
from a normal cofferdam and might influence the situation in both positive and negative direction. The water pressure on both sides of the cofferdam are equal (positive effect) and large top loads are present due to presence of the Haringvliet bridge pier (negative effect). Therefore, it was concluded that a special assessment of the situation is required to determine the length of the retaining walls.

7.1.2 Failure modes cofferdams

To assess the feasibility of the cofferdam as a foundation for the Haringvliet bridge piers, the potential failure modes of cofferdams are studied. In figure 7.5 an overview is given of the main failure modes of a cofferdam [TAW, 2004]. The failure modes are elucidated briefly in the list below. Subsequently, the probability of occurrence in the is qualitatively treated in the list.

- **Failure mode A:** Internal stability: When the maximum shear stress capacity of the enclosed soil is exceeded, the cofferdam will lose its shape retention and deform into a parallelogram (figure 7.5a). For the cofferdam foundation of the bridge piers, it was expected that this failure mode is not normative, as the enclosed soil body is reinforced by the pile foundation of the bridge piers.

- **Failure mode B:** Horizontal shear deformation of the cofferdam: the passive resistance in front and inside the cofferdam is too low to withstand the driving forces. The cofferdam will displace horizontally (figure 7.5b). For the cofferdam foundation, it is not expected that the cofferdam will displace in the horizontal direction, as the height difference between the subsoil in front of both retaining walls, is only 4 meters as can be seen figure 7.3.

- **Failure mode C:** Tilting of the front wall: the passive resistance in front of the cofferdam is too low to withstand the driving forces, the front wall will rotate around the anchor point (figure 7.5c). This is a realistic failure mode of the cofferdam foundation for the bridge piers. However, for this case study was considered that this failure mode can be prevent by applying tension rings at multiple levels, which will reduce the probability of a rotated front retaining wall.

- **Failure mode D:** Tilting of the cofferdam: Due to disequilibrium of the driving forces and the resistance around a point inside the cofferdam (figure 7.5d). For the cofferdam foundation, it is not expected that the cofferdam will rotate, as the height difference between the subsoil in front of both retaining walls, is only 4 meters as can be seen figure 7.3.

- **Failure mode E:** Tilting of the cofferdam: Due to disequilibrium of momentum around a point underneath the cofferdam (figure 7.5e). For the cofferdam foundation the same reasoning holds as for failure mode D.

- **Failure mode F:** Deformation of the retaining walls (figure 7.5f). The structural failure of the retaining walls is considered to be realistic. The Haringvliet bridge piers will induce relative high top loads that need to be transferred to the subsoil.

- **Failure mode G:** Anchor breach: due to too large forces in the anchor (figure 7.5g). For the cofferdam foundation for the Haringvliet bridge and the tension rings to provide the horizontal stability, this was considered to be an important failure mode to assess. However, the strength of the tension rings was considered to be beyond the scope of this thesis.

- **Failure mode H:** Macro instability: Is identical to the slope failure as discussed chapter 5 (figure 7.5h).

From the qualitative assessment of the failure modes of the cofferdam, it was concluded that the focus of the remaining of this chapter is on the front retaining wall of the cofferdam to assess the feasibility of the cofferdam foundation.

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5The required length of the walls will introduce problems to the construction of the cofferdam. The maximum available height between the bridge deck and the soil surface was $\approx 20$ meter, from which $\approx 12$ meters above the water surface of the Haringvliet estuary.
CHAPTER 7. COFFERDAM FOUNDATION FOR STABILITY OF THE HARINGVLIET BRIDGE PIERS

(a) Internal stability  
(b) Horizontal shear  
(c) Tilting of the frontwall  
(d) Tilting of the cofferdam  
(e) Rotation of the cofferdam  
(f) Exceeding section modulus front wall  
(g) Anchor breach  
(h) Macro stability failure of the slip plane

Figure 7.5: Failure modes of a cofferdam [CUR, 2005].
7.2. Structural dimensioning of the cofferdam foundation

7.1.3 General design approach of a cofferdam

For the dimensioning of the design of a cofferdam, multiple calculation methods are available in the Dutch standards [TAW, 2004]:

- Classical calculation method (deterministic, no Finite Element Method (FEM)).
- Deterministic calculation (semi-probabilistic with help of FEM).
- Probabilistic calculation (full FEM).

For the feasibility study in this thesis, it was considered that a classical calculation would be sufficient to assess the feasibility of the cofferdam as a foundation for the Haringvliet bridge pier. The steps in the classical calculation method are defined as follows [CUR, 2005]:

1. Determine whether the active and passive soil wedges of the two retaining walls cross each other. If not, the retaining walls can be assessed individually.
2. Determine the length of the front wall like a normal sheet pile wall. From this calculation, the anchor force, the momentum line and the shear force line for the front wall can be obtained.
3. With the active soil pressure and the anchor force known, the maximum occurring passive soil pressure in the enclosed soil body of the cofferdam can be determined, by for instance the method by Homberg.
4. The dimensions of the back wall of the cofferdam can be determined with the passive soil pressure and the anchor force.
5. The internal stability of the cofferdam has to be check by the method by Terzaghi.
6. The horizontal stabilising element (for instance the conventional anchor, but here the tension rings) have to be dimensioned.
7. Final step is to assess the total stability of the cofferdam, for instance the horizontal equilibrium and the macro stability.

7.1.4 Limitation of the scope of the feasibility study for the cofferdam

Based on the analysis of the failure modes of a cofferdam (7.1.2), the conclusion on the reference projects for the cofferdam (G.1) and the analysis of the global approach for the design calculation sequence for a cofferdam (7.1.3), it was chosen to elaborate on the dimensions of required walls for the cofferdam (steps 1 and 2) to prevent the structural failure of the retaining walls of the cofferdam foundation (failure mode F; figure 7.5f).

It was assumed, that both the front and back wall of the cofferdam foundation for the Haringvliet bridge require identical dimensions, although the back wall will have to retain less elevation differences than the front wall of the cofferdam. Therefore, it was considered that the calculations for the pressure by the passive soil wedge were not required to dimension both the front and the back wall of the cofferdam for the Haringvliet bridge piers (steps 3 and 4). The checks on the internal stability and the total stability of the cofferdam were considered to be beyond the scope of thesis (steps 5, 6 and 7).

7.2 Structural dimensioning of the cofferdam foundation

In the previous paragraphs, the design for the cofferdam foundation is discussed and the feasibility of the cofferdam as a foundation was qualitatively studied by means of reference projects in appendix G.1. In the following paragraphs, it is elaborated on the feasibility of the cofferdam into more detail by means of performing dimensioning calculations of the front retaining wall.

7.2.1 Retaining wall schematisation by Blum

As the first step in the classical approach is fulfilled \((23.2/21.7 = 1.07 < 1.5)\), the second step is to determine the length of the front retaining wall. The required dimensions for the retaining walls of the cofferdam are determined by the calculation method by Blum. In the method by Blum, three parameters are considered to be unknown and will be determined in the calculation, see figure 7.6:

- The embedded depth \(d\).
- The anchor force \(T\).
- The substitute force \(R\).
These parameters can be found by considering the deflections at anchor point \( E \) to be equal to zero [Vrijling et al., 2016]. It is assumed that the retaining walls are fixed at their bottom side. Therefore, the found length of the sheet pile walls should be increased by \( \approx 20\% \) of theoretical embedded length \( d \) [Vrijling et al., 2016]. The calculation method by Blum includes the following steps:

1. Compute the theoretical embedded depth (\( d \)) using the condition that the horizontal deflection at the anchor point (\( E \)) must be zero. The found theoretical embedded depth should be multiplied by 1.2 for the total embedded length.
2. Compute the equilibrium of moments around the toe of the sheet pile (point \( D \)) in order to determine the anchor force (\( T \)).
3. Compute the substitute force (\( R \)), from the horizontal equilibrium (\( \sum H = 0 \)).
4. Compute the horizontal stress diagram, the vertical stress diagram and the moment diagram.
5. Compute the required section modulus for the retaining walls (\( W_{eff,y} \)) and choose a suitable sheet pile or combi wall element.

### 7.2.2 Load conditions soil retaining walls

To be able to determine the required dimensions for the cofferdam wall element with the method by Blum, the load conditions for the front retaining walls of the cofferdam foundation are required. In figure 7.7 the overview of the assumed load conditions for the assumed case study design are depicted. The following paragraphs elaborate on the different parameters of figure 7.7.
7.2. Structural dimensioning of the cofferdam foundation

Anchor force / tension rings $(T)$

As discussed before in the description of the layout of the cofferdam foundation, it will be relatively easy to install multiple tension rings at different heights. However, for the simplification of the calculation in this thesis, a single tension ring is assumed at the top of the retaining walls ($a = 0$).

Active and passive soil pressures $(F_{\gamma:a} \text{ & } F_{\gamma:p})$

The horizontal loads by the soil wedges on the active and passive sides of the retaining wall depend on the effective weight of the soil layers $\gamma$, the soil pressure coefficients for active and passive soils $K$ and the height of the soil layers as can be seen in equations 7.1 and 7.2.

\[
F_{\gamma:a,h} = \frac{1}{2} \cdot (\gamma_s - \gamma_w) \cdot (h + d) \cdot K_{\gamma:a} \tag{7.1}
\]

\[
F_{\gamma:p,h} = \frac{1}{2} \cdot (\gamma_s - \gamma_w) \cdot (d)^2 \cdot K_{\gamma:p} \tag{7.2}
\]

In which:
- $K_{\gamma:a,h}$ Active soil pressure coefficient 0.33 [-]
- $K_{\gamma:p,h}$ Passive soil pressure coefficient 2.11 [-]
- $\gamma_s$ Volumetric weight of the soil 21 $kN/m^3$
- $\gamma_w$ Volumetric weight of water 10 $kN/m^3$
- $h$ Retaining height cofferdam walls 22 $m$
- $d$ Theoretical embedded depth cofferdam walls [-] $m$

The active and passive soil pressure coefficients differ for the loading case (soil load or top load) and are determined in appendix G.2.1. As the soil surface at the passive side of the retaining is under and negative inclination ($\beta = -11.3^\circ$), the value of the passive soil pressure coefficient $K_p$ was lower than the standard value ($K_p = 3$). The composition of the soil layers in the Haringvliet was simplified to a homogeneous sandy soil layer. It was assumed that cohesion is absent, the internal friction angle is $\phi = 30^\circ$ and the volumetric weight of the soil is $\gamma_s = 21 \ kN/m^3$. These values of the soil parameters are most in line with the general composition of the subsoil in the Haringvliet estuary (see chapter 3.2.6).

Top loads by the Haringvliet bridge piers $(q_{pier})$

The distributed loads on top of the cofferdam consists of two components, the loads by the water column on top of the cofferdam and the loads by the bridge piers. The loads by the water can be neglected, as there is no difference in water level between both sides of the cofferdam. The loads by the superstructure of the Haringvliet bridge piers ($q_{pier} \approx 193.5 \ kN/m^2$, see chapter 6.2) can not be neglected.

The distributed top loads by the Haringvliet bridge pier were taken into account at a height $b$ above the pile tip level. Because of the pile foundation of the Haringvliet bridge piers, the loads will be transferred to the subsoil by positive pile shaft friction ($\approx 2/3$ of the bearing capacity) and tip bearing capacity ($\approx 1/3$ of the bearing capacity), see chapter 6. It was assumed that for $b = 5.5$ meter ($\approx 1/4$ of the length of the foundation piles) the working height of the simplified bridge pier loads is sufficient representative. The distributed vertical loads were translated to a horizontal load on the retaining walls by the active soil pressure factor $K_{sur:a}$ (equation 7.3). The active soil pressure factor is discussed in appendix G.2.1.

\[
\Delta F_{h,\text{sur}} = K_{sur:a} \cdot q_{pier} \cdot (b + d - h) \tag{7.3}
\]

In which:
- $K_{sur:a}$ Active for surface load 0.33
- $b$ Assumed height of the distributed loads 5.5 $m$

\[\text{The horizontal loadings of the pier were assumed to be negligible for the scope of this thesis. The normative horizontal load (Load K, ship collision) was expected to be prevented by applying additional measures around the Haringvliet bridge piers, to reduce the probability of occurrence. In addition, it was considered that the majority of the horizontal forces would "dissipate" in the elastic deformations of the structural pier elements and the deformations in the subsoil.}\]
Silo effect
In relation to normal sheet pile walls, additional horizontal loads on the retaining walls of a cofferdam can be expected due to the so-called silo mechanism [CUR, 2005]. In the Dutch design rules, this mechanism is often neglected, while German methods assume a 25% increase of the active pressure [CUR, 2005]. For this thesis, the Dutch approach is used and therefore the silo effect has been neglected.

7.2.3 Calculation method retaining wall by Blum
For the assumed dimensions, the ’requirement’ to be an cofferdam was met \((23.2/21.7 = 1.07 < 1.5)\). In appendix G.2 the five steps of the calculation method by Blum are elucidated, which are briefly described in the following paragraphs.

Step 1. The theoretical embedded depth
The different load simplifications of figure 7.7 are related to deformations of the anchor point \((E)\), see figure 7.8 and equation 7.4. In appendix G.2.2 it is elaborated on the required length for the retaining walls. The found formula’s were put in an excel-file, together with the known values for the parameters. By iteration, the embedded depth \(d\) and thus the total length of the sheet-pile wall \(L_T\) were determined by iteration. This resulted in the following:
- Theoretical embedded depth cofferdam walls \((d) = 25.1\) meter.
- Total length retaining wall \((L_{tot}) = 52.1\) meter.

\[
\begin{align*}
\sum_{i=1}^{4} u_i &= 0 \\
&= u_1 + u_2 + u_3 + u_4 = 0 \\
&\text{In which:} \\
&\begin{aligned}
u_1 &\text{ Deflection due to anchor force } T \\
u_2 &\text{ Deflection due to horizontal active soil pressure} \\
u_3 &\text{ Deflection due to horizontal passive soil pressure} \\
u_4 &\text{ Deflection due to distributed load by the Haringvliet bridge piers}
\end{aligned}
\end{align*}
\]

\[\sum M_D = 0 \]

\[
T \cdot (h + d - a) = 1/6 \cdot K_{\gamma,a} \cdot \gamma_s \cdot (h + d)^3 - 1/6 \cdot K_{\gamma,p} \cdot \gamma_s \cdot d^3 + 1/2 \cdot q_{pier} \cdot K_{\gamma,sur:a} \cdot (d + b)^2
\]
Step 3. Horizontal equilibrium
The substitute force $R$ can be determined by assessing the horizontal equilibrium, see equation 7.6. This horizontal force is equivalent to the horizontal soil pressure at the lower part of the right-hand side of the retaining wall in figure 7.6. The result of the calculation of the substitute force was $R = 1965 \text{kN}$.

\[
\sum H = T + 1/2 \cdot K_{\gamma,p} \cdot \gamma_x \cdot d^2 - 1/2 \cdot K_{\gamma,a} \cdot \gamma_s \cdot (d + b)^2 = q_{\text{pier}} \cdot K_{\text{sur,p}} \cdot (d + b) = R \quad (7.6)
\]

Step 4. Horizontal, vertical and moment stress diagrams
The fourth step in the Blum calculation is to determine the different stress diagrams. The horizontal stress diagram was found by analysing the previous found horizontal forces, see figure 7.9a. The shear force diagram was found by calculating the area under the resulting horizontal stresses diagram, see figure 7.9b. The shear stress diagram will start at the top of the retaining wall with the horizontal pressure by the pier [Vrijling et al., 2016]. At point $D$, the shear force should equal zero, which is the case in the performed calculations.

The moment diagram is found by calculating the area under the shear stress diagram, see figure 7.9c. The moment diagram starts with 0 kNm at the soil surface and should reach its maximum at the point $D$, where the shear force is zero [Vrijling et al., 2016]. In the case of the retaining walls for the cofferdam of the Haringvliet bridge piers, the shear stress is zero at two locations ($\approx -21$ meter and $\approx -37$ meter). This can be explained, by the large values for the passive soil pressure that are required to make equilibrium in the horizontal direction. In appendix G.2.4 more detailed figures are included.

Figure 7.9: Stresses in the retaining wall of the cofferdam.
Step 5. Determine section modulus

The required section modulus for the retaining walls can be determined according to equation 7.7. The maximum positive moment in the retaining wall was near 6754 $kN/m$. The accompanying required section modules for a steel profile would be around 28,740 $cm^3/m$. The largest negative moment found was -8574 $kNm/m$, which would require a section modulus of 36.500 $cm^3/m$.

$$W = \frac{M_{\max}}{f_{y,dt}} = \frac{M_{\max}}{235} \quad (7.7)$$

A combi wall, for instance made out of king piles (see figure 7.10), has a maximum section modulus of approximately 45.530 $cm^3/m$ [ArcelorMittal, 2016]. The required section modulus is 0.8 times as large as the largest available section modulus for steel combi wall elements. Therefore, it was concluded that a cofferdam made out of steel combi wall elements is feasible.

7.2.4 Optimisations of the calculation simplifications

The found length of the retaining wall of 52 is extremely long. The performed calculation on the required length and strength of the retaining walls has multiple conservative assumptions that lead to an unnecessary long required length. In the coming paragraphs, multiple options for a more refined calculation are discussed:

- Simplification of the top load $q_{pier}$.
- Size of the active soil wedge
- Soil characteristics
- Amount of tension rings

A. Loads simplifications

The top load by the bridge piers can be treated in more detail by assessing the situation by taken the top loads into account over a restricted area. In case a two-sided restricted top load is assumed, the equations as discussed in the previous paragraphs change. In figure 7.11a the load simplification as presented in the Dutch standards (CUR 166) are included. For simplicity, the horizontal distributed load was assumed to work at a single point load $F_{eh,q}$ at a depth $b$.

In appendix G.2.3 the adjusted top load simplification is elaborated on in more detail. The length of the retaining walls decreases, due to the refined top load definitions. Two scenarios are reviewed, which differ in the width ($s$ in figure 7.11a). In the first scenario, the top loads by the Haringvliet bridge piers are assumed over the complete width ($s = 14.8$ meter). In the second scenario was assumed that the top loads are distributed over both the front and the back retaining walls of the cofferdam. Therefore, the width of the top loads was divided by two ($s = 7.4$ meter).

B. Size active soil wedge

When the size of the active soil wedge is larger that the width of the cofferdam, the amount of active pressure on the retaining wall of the cofferdam should be reduced (see figure 7.11b). The width of the active soil pressure wedge is approximately 27.2 meters for a retaining wall length of 52 meters. This indicates that the actual horizontal soil pressure on the retaining walls of the cofferdam was overestimated in the previous calculations.

C. Reinforced soil body

Due to the presence of the pile foundation in the enclosed soil body of the cofferdam, it would be more appropriate to assess the soil as being reinforced. The presence of the foundation piles in the soil influences the vertical and horizontal deformations [Korff, 2009]. This would lead to smaller values for the active soil pressure coefficient $K_{\gamma,a,h}$. To assess the influence of a lower $K_{\gamma,a,h}$, the calculations are repeated for a $K_a = 0.22$. The results are depicted in table 7.1.
7.3. Conclusions and discussion of the cofferdam foundation

In the following paragraphs the results of the feasibility study on the cofferdam as foundation for the Haringvliet bridge piers are discussed. Subsequently, the conclusions are drawn and recommendations for future research are given.

### Results sensitivity analysis

To create insight in the sensitivity of the required length of the retaining walls to the discussed parameters in the previous paragraphs, calculations are performed. The results are depicted in table 7.1 and are based on the discussed values. The same point of action for the top loads as in the previous calculations \( b = 5.5 \) meter was used.

From table 7.1 it was concluded that the length that is required for the retaining walls of the Tilting Lock can be optimised to have significant reductions in material use. In addition, multiple other aspects can reduce the required length of the retaining walls even more, which are discussed above and in chapter 7.3.1.

<table>
<thead>
<tr>
<th>Initial</th>
<th>Adjusted top load</th>
<th>Reinforced soil ( K_{\gamma,a,h} = 0.22 )</th>
<th>Combined</th>
</tr>
</thead>
<tbody>
<tr>
<td>Embedded depth ( d )</td>
<td>25.1 m</td>
<td>20.4 m</td>
<td>19.3 m</td>
</tr>
<tr>
<td>Total length walls ( L_T )</td>
<td>52.1 m</td>
<td>46.5 m</td>
<td>45.2 m</td>
</tr>
<tr>
<td>Reduced length ( \Delta L_T )</td>
<td>-</td>
<td>5.7 m</td>
<td>6.9 m</td>
</tr>
</tbody>
</table>

Table 7.1: Results optimisation of the calculations on the required length of the retaining walls.

### Amount of tension rings

In the current calculations, only one anchor point was taken into account, located at the top of the retaining wall. It was found, that for applying an anchor that the top of the retaining wall is the most favourable situation (least moment in the retaining wall). When the position of this single anchor force was lowered, it was found that the required length of the retaining wall increased as well. For each 1.0 meter that the location of the anchor was lowered, the embedded depth increased approximately with a 1.15 to 1.3 meter. This indicates that the length of the wall was increased, together with the span of the wall between point \( R \) and the anchor point. This increases the prevailing moment in the retaining wall. It was expected, that for more applied anchors or tension rings, the length that is required for the retaining walls will decrease.

---

*(a) Horizontal soil pressure by two side restricted top load [CUR, 2005].
(b) Area of influence of the active soil wedge, using straight slip planes (Coulomb) [Vrijling et al., 2016].

Figure 7.11: Top load simplification.*
CHAPTER 7. COFFERDAM FOUNDATION FOR STABILITY OF THE HARINGVLIET BRIDGE PIERS

7.3.1 Discussion
Based on the performed calculations the feasibility of the cofferdam as foundation for the Haringvliet bridge piers is discussed in the following paragraphs.

Optimisation of the cofferdam dimensions
For the assumed dimensions, the ‘requirement’ to be an cofferdam was met \( \frac{23.2}{21.7} \approx 1.07 < 1.5 \). Therefore, it was concluded that the width of the cofferdam might be expanded even more in further design, as for wider cofferdams the required length of the walls will be less [TAW, 2004].

Disadvantages of the increasing the width of the cofferdam, are the increased retaining height due to the inclined bottom level of the trench for the Tilting Lock and the increased circumference of the cofferdam. A larger circumference relates to the usage of more material and thus higher costs. Therefore, was concluded that the design of the cofferdam can be optimised regarding the dimensions and the costs. However, optimising or changing the dimensions of the cofferdam was considered to be beyond the scope of this thesis, as was sought for the feasibility of the cofferdam.

Tension rings
The horizontal stabilising elements of the tension ring are not elaborated on. The advantages and disadvantages as found during the elaboration on the cofferdam foundation for the Haringvliet bridge piers are listed below.

- Easy construction methods.
- Known construction method (pre-tensioning of steel cables).
- No interference with the existing pile foundation of the Haringvliet bridge.
- Unknown strength.
- Unknown force distribution between tension ring and cofferdam.
- Unknown internal force distribution in the walls of the cofferdam foundation.

In this thesis, it was assumed that the tension rings can be designed as such, that the required anchor force \( \approx 700 \text{kN} \) can be taken. However, when multiple tension rings are applied around the cofferdam, the required induced force by a single tension ring will decrease. It is recommended to study the feasibility of the tension rings into more detail in future research.

Influences on the Haringvliet bridge piers
Deformations of the retaining walls of the cofferdam are inevitable, as the passive horizontal pressures are not activated when no deformations are present. Due to the deformations, it is unavoidable that destressing of the enclosed soil of cofferdam walls will occur. This destressing will induce settlements of the pile foundation of the Haringvliet bridge, due to the loss of positive pile shaft friction. Settlements near deep excavations and adjacent buildings are never two-dimensional. Three dimensional effects can cause either an increase or a reduction of the settlements depending on local conditions [Korff, 2009]. In addition to the settlements due to stressing, multiple other causes of settlements are present in the vicinity of deep excavations [Korff, 2012]:

- Settlements of the foundation layer.
- Settlements due to the reduction of pile tip bearing capacity as a result of reduced stress levels.
- Re-mobilisation of shaft friction, either positive or negative skin friction.
- Redistribution of pile load from the building.
- Horizontal pile deformation.

Horizontal bridge pier loads
The horizontal loads on the bridge piers were not taken into account, as discussed in chapter 7.2.2. It was expected that the majority of the horizontal forces would "dissipate" in the elastic and plastic deformations of the structural elements of the bridge pier and deformations in the subsoil, which was enclosed by the cofferdam.
7.3. Conclusions and discussion of the cofferdam foundation

Soil characteristics
The horizontal pressures by the active and passive soil wedges were determined for homogeneous sand ($\gamma_s = 21 \, kN/m^3$, $\phi = 30^\circ$, $c = 0$). The subsoil of the Haringvliet estuary contains other soil layers as well, which were not taken into account.

Construction phasing
As the available head clearance underneath the Haringvliet bridge is limited, problems can arise for the construction phasing. The construction phasing is discussed in chapter 9.

7.3.2 Conclusion on the feasibility of the cofferdam as a foundation
It is concluded that a cofferdam made out of steel combi wall elements will be feasible, based on the basic calculations on the required section modulus (36.500 $cm^3/m$) for the retaining walls of the cofferdam foundation for the Haringvliet bridge. The largest combi wall elements available on the market have a section modulus (45.530 $cm^3/m$ [ArcelorMittal, 2016]) that is sufficiently higher than the required section modulus for the retaining walls of the cofferdam (36.500 $cm^3/m$). Therefore, it was concluded that even larger elevation differences can be obtained. Larger allowable excavations are related to a larger trench for the Tilting Lock, which indicates that even larger sized Tilting Locks are feasible for the case study at the Haringvliet bridge.

In the first, conservative, approach for the calculations on the required length for the retaining walls of the cofferdam was found that lengths up to 52 meters are required. However, in the optimisation of the calculation simplifications was concluded that much more favourable lengths ($\approx 40$ meter) can be obtained with more detailed calculations.

7.3.3 Recommendations
The recommendations for future research on the cofferdam as a foundation for the Haringvliet bridge piers are discussed in the following paragraphs.

Detailed calculations
For further research, it was recommended to study the feasibility the cofferdam accordingly to the conventional calculation methods as described in chapter 7.1.3. For more detailed calculations, the use of Finite Element Method (FEM)-software was recommended, certainly because of the presence of the bridge piers and the expected three dimensional effects. In these FEM models, it possible to take the influences and the deformations of the Haringvliet bridge piers into account as well.

Tension rings
The current design of the tension rings was based on a basic, qualitatively study. For further research it is recommended to elaborated on the tension rings. Especially the force transfer between the tensioned cables and the cofferdam will be interesting to be studied into more detail.

In the current calculations on the retaining walls for the cofferdam, only one anchor point was taken into account, located at the top of the retaining wall. In further design calculations, multiple anchor forces along the retaining wall should be considered. These multiple anchor forces were expected to further reduce the required section modulus for the cofferdam, without (significantly) increasing the length of the required retaining wall.

Optimisation regarding costs
In this chapter, the feasibility of the preliminary design for the cofferdam as a foundation for the Haringvliet bridge piers was assessed. In further research, it is recommended to optimise the designs made. Besides the more detailed calculations and the checks on the untreated failure modes, the design of the cofferdam can be optimised regarding the costs. It is expected that more favourable designs can be found regarding the costs.
CHAPTER 7. COFFERDAM FOUNDATION FOR STABILITY OF THE HARINGVLIET BRIDGE PIERS
Chapter 8

Hydrodynamic and morphological influences of the Tilting Lock trench

By constructing the required trench and the Tilting Lock underneath the Haringvliet bridge, the morphological situation in the Haringvliet estuary will be affected. The feasibility of the Tilting Lock depends on the available Under Keel Clearance (UKC), as the Tilting Lock will not be operational when it runs aground. This chapter elaborates on the rate of sedimentation in the trench for the Tilting Lock. In this way, the required frequency and magnitude of maintenance on the depth of the trench to provide sufficient UKC for the Tilting Lock can be assessed.

A high frequency of required dredging to keep sufficient UKC under the Tilting Lock is considered to be negative for the feasibility of the Tilting Lock. When a large amount of sedimentation in short time is expected, the design of the trench should be adjusted to allow easy access for maintenance on the depth of the trench. This might be both complicated and expensive for the design of the Tilting Lock as well for the operational phase of the Tilting Lock.

The amount of sedimentation in an area depends, amongst others, on the amount of discharge and the local flow velocities in the considered area. For this thesis, the reviewed area was limited to the area where the largest risks for the feasibility of the Tilting Lock were expected: underneath the Tilting Lock. The local flow velocities depend on the geometry of the considered area and the discharge through the area. In section 8.1 estimations on the local flow velocities and the discharge through the area underneath the Tilting Lock are made.

In section 8.2, several scenarios were drawn to estimate the rate of sedimentation in the trench for the Tilting Lock. The defined scenarios for sedimentation were assessed using the results of section 8.1. Conclusions on the sedimentation in the trench are discussed in section 8.3. In appendices H and I more detailed information on specific topics of the fluid mechanics and the morphology of the trench for the Tilting Lock are included.

8.1 Hydrodynamic situation in the Tilting Lock trench

To estimate the local flow velocities in the vicinity of the Tilting Lock, the hydrodynamic situation in the trench was studied. The Tilting Lock will be a detached body in a flow regime. It was expected that the streamlines around the Tilting Lock will tend to bend both down- and sideways to go around the Tilting Lock (see figure 8.1b). The discharge directed at the Tilting Lock will pass the Tilting Lock either alongside or underneath. The amount of the discharge that will pass the Tilting Lock underneath, through the red area of figure 8.1a, can be expressed as a percentage
of the initial incoming discharge. When the amount of discharge is known, the local flow velocities can be determined from the assumed geometry of the lateral cross section of the trench. This ratio was required for the estimations on the deposition and erosion in the trench for the Tilting Lock, which are described in section 8.2.

(a) The reviewed area (red) in the hydrodynamic analysis.  
(b) Very rough sketch of the expected streamlines around the Tilting Lock

Figure 8.1: Potential flow distribution.

The dimensions of the area reviewed for sedimentation were limited to the width of the Tilting Lock and the length of the trench for the Tilting Lock, as depicted in the red area in figure 8.1a. The geometries of the Tilting Lock and the trench are rather complicated for a preliminary calculation. Therefore, the geometry of the Tilting Lock and the trench in the lateral cross section, are simplified to average values for the water depth and the blocking height by the Tilting Lock. In figure 8.2 the simplification on the geometry of the Tilting Lock and the trench are depicted. In appendix H.1.2 the simplifications of the cross sections are discussed in more detail.

8.1.1 Energy dissipation along a streamline

Each water particles in a flow will follow a certain path, a so-called streamline. Along these streamlines, the water particles will encounter several types of resistance, which will lead to the dissipation of energy. Resistance along a streamline can be caused by wall friction or turbulent flow. In general, the water particles will tend to follow the streamlines with the least resistance.

In this thesis, it was assumed that a streamline underneath the Tilting Lock will encounter a similar amount of energy dissipation over the reviewed length as a streamline in the undisturbed areas of the Haringvliet estuaries. The energy dissipation along a streamline is very much related to the local flow velocity and water depth of an area, as could be seen in the formulas used later on in this thesis. As the geometry of the area underneath the Tilting Lock was assumed to be constant (only the water depths will fluctuate with the changing water levels), the amount of resistance along a streamline depends on the local flow velocities in the area. The objective of the section was to find the discharge, for which the energy dissipation along the streamline through the trench will be identical to the energy dissipation along an undisturbed streamline in the Haringvliet estuary.
8.1. Hydrodynamic situation in the Tilting Lock trench

Contributions to the energy dissipation

Over the assumed length of the reviewed stretch of the trench for the Tilting Lock (500 meter), multiple contributions of potential energy dissipation along the streamlines could be defined. A situation sketch of the longitudinal cross section of the trench is included in figure 8.3 to depict the different energy dissipation contributions.

The in- and outflow losses along the streamlines at the upstream and downstream edges of the trench were expected to be very limited. After a short period of time, the slopes of the trench would be reshaped into a more smooth shape under the hydrodynamic conditions [T. Raaijmakers, 2005]. Without the sharp transitions at the edges of the trench, the detachment of the streamlines will not take place anymore and, therefore, energy dissipation will be absent. The losses due to the detached bodies in the streamlines (the fixating structure and the bridge piers) were left out of the scope because they are not in the reviewed area (bridge piers) or relatively small (piles fixating structure).

For the scope of this thesis, it was assumed that the energy dissipation along a streamline through the trench only depends on the wall friction and the outflow losses at the downstream side of the Tilting Lock, see equation 8.1. The wall friction contribution was assumed to depend on the friction between the streamlines, bottom of the trench and the hull of the Tilting Lock. The inflow losses due to the wake at the upstream side of the Tilting Lock were neglected. The outflow losses at the downstream side of the Tilting Lock were due to the detachment of the flow at the edge of the Tilting Lock.

The separate parts of equation 8.1 are explained in detail in the following paragraphs. In appendix H.1.1 the other types of energy dissipation along the streamlines through the trench for the Tilting Lock are discussed.

\[
\Delta H_{w,trench} = \Delta H_{w,fric} + \Delta H_{w,of} \quad \& \quad \Delta H_{w,trench} = \Delta H_{w,bar}
\]  

(8.1)

In which:

- \(\Delta H_{w,trench}\): Energy dissipation of streamline through Tilting Lock trench \([\text{m}]\)
- \(\Delta H_{w,bar}\): Energy dissipation of a streamline in the undisturbed Haringvliet \([\text{m}]\)
- \(\Delta H_{w,fric}\): Energy loss by wall friction in the Tilting Lock trench \([\text{m}]\)
- \(\Delta H_{w,of}\): Energy loss by outflow at downstream edge of the Tilting Lock \([\text{m}]\)
CHAPTER 8. HYDRODYNAMIC AND MORPHOLOGICAL INFLUENCES OF THE TILTING LOCK TRENCH

Energy dissipation in undisturbed areas Haringvliet estuary $\Delta H_{w;har}$
The energy dissipation in the undisturbed situation of the Haringvliet estuary is depending on the length of the reviewed area and the friction gradient, see equation 8.2. In the uniform flow assumption, the gradients in bed, water surface and friction level are identical. Therefore, it was assumed that the friction gradient $i_{w;har}$ is equal to the water surface gradient. With the Chézy equation (8.3) the water level inclination could be determined. The Chézy-coefficient $C$ is in general between 50 and 500 m$^{1/2}$/s. A more precise estimation could be made by applying the equation 8.4.

As the depth of the Haringvliet was negligible with respect to its width ($8 < 1200$ meter), the hydraulic radius $R_{Har}$ was assumed to be equal to the water depth ($8$ meter). The equivalent Nikuradse numbers $k$ were estimated for alluvial bed material between 0.046 and 0.14 (see table at equation 8.6). The lowest values of the $k$-value were used, as these deliver higher initial energy dissipation. With the assumed average maximum flow velocity under the Haringvliet bridge of $\approx 0.45$ m/s (see chapter 3.2.2) the water level inclination was assumed on $9.7 \cdot 10^{-6}$.

$$\Delta H_{w;har} = i_{w;har} \cdot L$$  \hspace{1cm} (8.2)

$$u = C \cdot \sqrt{R_{Har} \cdot i_{w;har}} \rightarrow i_{w;har} = \frac{(u/C)^2}{R_{Har}}$$  \hspace{1cm} (8.3)

$$C = 5.75 \cdot \sqrt{g \cdot \log(12 \cdot R_{Har}/k)}$$  \hspace{1cm} (8.4)

In which:
- $i_{w;har}$ Friction gradient 9.7 $\cdot 10^{-6}$
- $u$ Flow velocity 0.45 m/s
- $C$ Chézy value 51 m$^{1/2}$/s
- $R_{Har}$ Hydraulic radius of the Haringvliet estuary 8 m
- $k$ Nikuradse value for alluvial material 0.046 - 0.14
- $L$ Length of reviewed area 500 m

Wall friction $\Delta H_{w;fric}$
The total energy loss due to wall friction along a streamline was calculated according to equation 8.5 [Battjes, 2002]. Two boundaries that can cause wall friction were defined: the steel hull of the Tilting Lock and the alluvial bottom of the Haringvliet. For both contributions the water level friction gradient $i_{w;fric;i}$ were determined with equation 8.7. As could be seen, the friction coefficient $c_{f;i}$ depends on the hydraulic radius $R_{i}$ of the cross section and the equivalent $k_{i}$-values of Nikuradse.

$$\Delta H_{w;fric} = \sum_{i} i_{w;fric;i} \cdot L_{i}$$ \hspace{1cm} (8.5)

$$\frac{1}{\sqrt{c_{f;i}}} = 5.75 \cdot \log \left( \frac{12 \cdot R_{i}}{k_{i}} \right)$$  \hspace{1cm} (8.6)

$$i_{w;fric;i} = \frac{c_{f;i} \cdot U^2}{g \cdot R_{i}}$$  \hspace{1cm} (8.7)

In which:
- $i_{w;fric}$ Water level inclination due to wall friction [-]
- $L_{i}$ Length over which friction i works 90 [m]
- $c_{f;i}$ Friction coefficient for material i [-]
- $R_{i}$ Hydraulic radius material i ($A/P$) [m]
- $A$ Area [m$^2$]
- $P_{i}$ Wet perimeter material i [m]
- $k_{steel}$ Nikuradse k-value for steel 0.0005 - 0.002 [-]
- $k_{bottom}$ Nikuradse k-value for alluvial material 0.046 - 0.14 [-]

For cross sections with hydraulically rough flow, an estimation for the friction coefficient $c_{f;i}$ was made in equation 8.6 [Battjes, 2002]. A flow through natural conduct with alluvial material is almost always hydraulically rough [Battjes, 2002]. The values of the roughness $k$ of both steel
hull of the Tilting Lock and alluvial material of the bottom of the trench were approximated by
the estimations by Nikuradse [Battjes, 2002]. The lower values of the $k$-values were used, as these
results in an overestimation of the friction coefficient $c_f$ and thus an overestimation of the energy
dissipation along the streamline through the Tilting Lock trench.

The hydraulic radius $R$ depends on the wet perimeter $P_i$ of the considered wall (either the hull
of the Tilting Lock or the bottom of the trench), and the surface of the water in the lateral cross
section of the Tilting Lock. The surface of the flow through the trench underneath the Tilting
Lock depends the water level at the surface, as the Tilting Lock will be able to move with the
tidal motions. As multiple water surface levels were taken into account (see section 8.1.2), multiple
values for the hydraulic radius were used in the calculations.

**Outflow losses $\Delta H_{w,of}$**

For the determination of the outflow losses at the downstream edge of the Tilting Lock, balance
equations were used over section 2, 3 and 4 of figure 8.4. In the simplification, three cross sections
are defined: upstream of the Tilting Lock, underneath the Tilting Lock and downstream the Tilting
Lock. The water depths in the cross sections of the simplification coincide with the water depths
in the vicinity of the Tilting Lock as assumed in appendix H.1.2.

![Figure 8.4: Longitudinal cross section of the trench that is required for the Tilting Lock.](image)

In each cross section at least two of the three equilibria hold, see equations 8.8, 8.9 and 8.10.
Between cross sections 2 and 3, the energy head levels and the volume of the discharge are the same,
but the momentum flux can not be calculated. Between cross sections 3 and 4, the momentum
fluxes and the volume are the same at the cross sections, but the energy head level will differ.
The energy dissipation along the streamline through the trench due to the outflow losses was
determined by the differences in energy head level between the cross sections up- and downstream
of the Tilting Lock.

$$\Delta H_{w,of} = H_2 - H_4$$ (8.8)

Energy head level:

$$H_i = z_i + d_i + \frac{u_i^2}{2 \cdot g}$$ (8.9)

Momentum flux:

$$F_i = \frac{1}{2} \cdot \rho \cdot g \cdot d_i^2 + \alpha \cdot \rho \cdot u_i^2 \cdot (a_i)$$ (8.10)

Discharge:

$$q_i = u_i \cdot \mu \cdot a_i$$ (8.11)

In which:

- $H$: Energy head level $m$
- $z$: Distance between reference level and bed level $m$
- $d$: Local water depth $m$
- $u$: Local depth averaged flow velocity $m/s$
- $F$: Momentum flux $N/m$
- $q$: Discharge $m^2/s$
- $h$: Water level surface at deepest point of trench $m$ NAP
- $a$: Open area beneath valve $m$
- $\mu$: Factor for inflow losses -

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CHAPTER 8. HYDRODYNAMIC AND MORPHOLOGICAL INFLUENCES OF THE TILTING LOCK TRENCH

It should be noted that the energy losses along the streamlines between cross sections 2 and 4 due to the wall friction were neglected in the calculation of the outflow losses. This means that the energy head level in front of and underneath the Tilting Lock will be the same. In reality, energy dissipation along the streamline due to wall friction will take place.

8.1.2 Main approach of the hydrodynamic calculations

Based on the discussed theory, different steps were taken to determine the ratio between the incoming discharge and the discharge through the trench for the Tilting Lock:

1. Determine the energy dissipation along a streamline in the undisturbed Haringvliet ($\Delta H_{w,\text{har}}$).
2. Determine the energy dissipation along a streamline through the trench for the Tilting Lock ($\Delta H_{w,trench}$), based on wall friction and outflow losses.
3. Reduce the discharge through the trench till the initial energy dissipation and the dissipation in the new situation are equal.
4. Determine the flow velocities through the trench for the Tilting Lock from the geometry of the lateral cross section and the discharge through the trench.

Scenarios hydrodynamic calculations

The ratio for the discharge through the trench for the Tilting Lock was determined for multiple scenarios. As discussed before, the energy dissipation along a streamline will depend on the local flow velocity and the local water depths and local water depths depend on the water surface levels. In total five characteristic water levels for the Haringvliet estuary were taken into account, see table 8.1.

The local flow velocities depend on the discharge. For each water level of table 8.1, ten different values for the discharge through the Haringvliet estuary were used. These values for the discharges varied between 150 m$^3$/s and 3800 m$^3$/s, which are the 5 and 95-percentile of the discharges through the Haringvliet Estuary at the Haringvliet bridge (see section 3.2.2).

<table>
<thead>
<tr>
<th>Exceeding high water levels</th>
<th>Rak Noord</th>
</tr>
</thead>
<tbody>
<tr>
<td>1x per 1000 years</td>
<td>+2.57 m NAP</td>
</tr>
<tr>
<td>1x per year</td>
<td>+1.63 m NAP</td>
</tr>
<tr>
<td>Average water level</td>
<td>+0.46 m NAP</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Under-run low water levels</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>1x per 10 years</td>
<td>- 0.40 m NAP</td>
</tr>
<tr>
<td>1x per year</td>
<td>- 0.25 m NAP</td>
</tr>
</tbody>
</table>

Table 8.1: Used water levels in the hydrodynamic review, see table 3.2.

8.1.3 Discharge versus energy dissipation

In appendix H.2.1 a first estimation of the energy dissipations was made to verify the results of the calculations made in Matlab. In equation 8.12 the outcomes of the calculation are depicted, which show that the caused friction along the streamlines underneath the Tilting Lock will be about twice the energy dissipation in the undisturbed situation of the Haringvliet estuary. As energy dissipation along a streamline depends on the geometry (assumed constant) and the local flow velocities. Therefore, it was concluded that the flow rate through the section underneath the Tilting Lock was overestimated.

$$\Delta H_{w,trench} = \Delta H_{w,fric} + \Delta H_{of} = 9.6 \cdot 10^{-4} + 1.7 \cdot 10^{-2} = 1.8 \cdot 10^{-2} \text{ m}$$  \hspace{1cm} (8.12)

$$\Delta H_{w,trench} \gg \Delta H_{w,har} = 9 \cdot 10^{-3} \text{ m}$$

The energy losses by the wall friction were a factor 10 smaller than the energy dissipation by the outflow losses. This indicated that the outflow losses were more important for the determination of the energy dissipation over the stretch, but not sufficient to exclude the wall friction factor from the calculations.
8.1. Hydrodynamic situation in the Tilting Lock trench

Matlab calculation

As was concluded in the hand calculation that the discharge through the section underneath the Tilting Lock was assumed too high, the discharge was lowered in the subsequent calculations. The discharge through the trench could be determined by iterating the discharge and energy dissipation through the trench until the energy dissipation was equal to the energy dissipation along streamlines in undisturbed areas. As the iteration takes a lot of calculation cycles, the calculations were conducted in Matlab. The detailed description of the Matlab file is included in appendix H.2.2.

In table 8.2 the outcomes of the calculations in Matlab on the energy dissipation and discharges are depicted. In appendix H.2.2 the complete results are included. It was found, that the difference in discharges between the trench \( q_{\text{trench}} \) and the undisturbed areas of the Haringvliet \( q_{\text{har}} \) are related to each other by a relatively stable ratio between the discharge through the section underneath the Tilting Lock and the discharge through a comparable, undisturbed section of the Haringvliet estuary. This ratio was constant for different magnitudes of the discharge through the Haringvliet, but was different for changing water levels in the Haringvliet estuary.

From the Matlab calculations it was concluded, that a larger amount of discharge was expected to flow through the section underneath the Tilting Lock for higher water levels. In table 8.2 the different discharge ratios versus the water levels in the Haringvliet are included. This made sense, as for higher water levels the UKC of the Tilting Lock was larger. With more UKC a larger cross section for the streamlines underneath the Tilting Lock is available, which reduces the influence of wall friction on the energy dissipation along the streamlines.

The ratios between the flow velocities in the undisturbed areas of the Haringvliet \( u_{\text{har}} \) and the flow velocity underneath the Tilting Lock \( u_{\text{trench}} \) are less broadly distributed than the ratios for the discharge. This is visualised in figures 8.5a and 8.5b. This is because the local flow velocities depend on the size of the cross sectional areas. For higher water levels, the cross sectional areas underneath the Tilting Lock are larger and therefore reduce the effect of the higher discharge through the trench.

<table>
<thead>
<tr>
<th>Water level [m NAP]</th>
<th>Water depth [meter]</th>
<th>UKC [meter]</th>
<th>Ratio ( q_{\text{trench}}/q_{\text{har}} )</th>
<th>Ratio ( u_{\text{trench}}/u_{\text{har}} )</th>
<th>min. ( u_{\text{trench}} )</th>
<th>max. ( u_{\text{trench}} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>-0.40</td>
<td>36.5</td>
<td>2.5</td>
<td>50.6 %</td>
<td>70.3 %</td>
<td>0.014</td>
<td>0.317</td>
</tr>
<tr>
<td>-0.20</td>
<td>36.7</td>
<td>2.7</td>
<td>52.9 %</td>
<td>71.1 %</td>
<td>0.014</td>
<td>0.320</td>
</tr>
<tr>
<td>0.46</td>
<td>37.4</td>
<td>3.4</td>
<td>60.7 %</td>
<td>73.4 %</td>
<td>0.014</td>
<td>0.330</td>
</tr>
<tr>
<td>1.63</td>
<td>38.5</td>
<td>4.5</td>
<td>75.7 %</td>
<td>77.8 %</td>
<td>0.015</td>
<td>0.350</td>
</tr>
<tr>
<td>2.60</td>
<td>39.5</td>
<td>5.5</td>
<td>89.3 %</td>
<td>81.6 %</td>
<td>0.016</td>
<td>0.367</td>
</tr>
</tbody>
</table>

Table 8.2: For an initial depth of \( h_0 = 8 \) meter.

![Figure 8.5a](image1.png)  ![Figure 8.5b](image2.png)

(a) Ratios of the discharges (1:1.1 to 1:2).
(b) Ratios of the flow velocities (1:1.2 to 1:1.4).

Figure 8.5: Relations between the undisturbed Haringvliet and the trench for different water levels in the Haringvliet.
CHAPTER 8. HYDRODYNAMIC AND MORPHOLOGICAL INFLUENCES OF THE TILTING LOCK TRENCH

8.1.4 Conclusions on the hydrodynamic analyses

The calculated flow rate and flow velocities with the Matlab file were sufficiently satisfying to estimate the morphological changes caused by the implementation of the Tilting Lock and its trench in the Haringvliet estuary. The ratios to find the discharges and flow velocities through the trench for the Tilting Lock are depicted in table 8.2 and should be applied on the discharges and flow velocities in the undisturbed areas of the Haringvliet.

It can be concluded that the estimation of the flow velocities in this thesis was not complete, but sufficient to make a satisfying conservative assessment of the sedimentation in the trench for the Tilting Lock. The discrepancy due to the overestimation of the flow velocities has limited influences on the reliability of the performed estimation.

Discussion on the hydrodynamic analyses

In general, the discharge ratios, that were found and elaborated on in the previous paragraphs, will overestimate the amount of flow volume through the trench for the Tilting Lock. In the calculations made, not all contributions to the energy dissipation along the streamlines were taken into account. However, this slightly overestimation of the discharge ratios was considered to be satisfying for a conservative approach in the sediment deposition estimation of section 8.2.7. An overestimated discharge would introduce more sediment particles to the area underneath the Tilting Lock than in the real situation. For the estimation of erosion in the vicinity of the Tilting Lock, the found velocity values were less satisfying, as overestimated discharges lead to overestimated flow velocities. Higher flow velocities are related to higher rates of erosion.

8.2 Sedimentation scenarios

The transport of sediment through the area of the Tilting Lock could be described by using a standard balance, as depicted in equation 8.13 [Bosboom and Stive, 2013] and figure 8.6. The amount of sedimentation in a certain area is dependent on the amount of incoming and outgoing water discharge with a suspended sediment concentration and the deposition and erosion of sediment particles. The net deposition in the area of the Tilting Lock was related to differences in the bed level elevation by equation 8.14.

\[
S_{\text{in}} - D + E = S_{\text{out}}
\]  

(8.13)

\[
\frac{\delta z_{\text{bed}}}{\delta t} = \frac{D - E}{\rho_s}
\]  

(8.14)

In which:

- \( S_{\text{in}} \) Total sediment transport rate: \( \text{kg/(m}^2 \cdot \text{s)} \)
- \( D \) Deposited sediment particles: \( \text{kg/(m}^2 \cdot \text{s)} \)
- \( E \) Eroded sediment particles: \( \text{kg/(m}^2 \cdot \text{s)} \)
- \( \delta z_{\text{bed}}/\delta t \) Change in bed level elevation: \( \text{m/s} \)
- \( \rho_s \) Density of the sediment particles: 1000 \( \text{kg/m}^3 \)

In the following paragraphs is elaborated on different scenarios to estimate the sedimentation rate in the Tilting Lock trench. These scenarios were based on the assumption that the non-flocculated silt particles are not able to settle within the reviewed area of the trench. Therefore, the sedimentation rate estimation was based on the flocculated silt particles:

- Suspended sediment concentration of \( c = 1.5 \text{ mg/l.} \)
- Fall velocity \( w_s = 0.116 \text{ mm/s.} \)

8.2.1 Approach scenarios

To determine the potential sedimentation in the trench for the Tilting Lock, two approaches were applied and compared. In total five scenarios were reviewed distributed over two approaches, see
8.2. Sedimentation scenarios

The first approach (figure 8.7) was based on the amount of incoming suspended sediment concentration and the factors that limit the deposition of those suspended particles. The second approach (figure 8.8) was based on the expected net deposition in the Haringvliet estuary near the Haringvliet bridge without the Tilting Lock in place. It was expected that by applying two different approaches, a reliable estimation for the sedimentation in the trench for the Tilting Lock could be made. In table 8.5 the resulting sedimentation rates of each scenario is depicted.

In all scenarios for the sedimentation was assumed that the transport of sediment particles only takes place by suspended load transport. The bed load transport was neglected in this thesis, as the maximum flow velocities in the Haringvliet estuary (max. 0.21 m/s, see appendix 3.2.2) were not expected to exceed the critical flow velocity for erosion as determined in the assessment of the influences of the Kierbesluit on the morphology in the South-Western delta of the Netherlands (0.4 m/s [Van Wijngaarden and Ludikhuize, 1997]). In appendix I.1.1 the different types of sediment transport are discussed in more detail. The influences of waves on the morphology in the trench for the Tilting Lock were neglected as well, because the local water depths were 8 meters or more and the wave action in the Haringvliet was limited.

The streamlines through the trench for the Tilting Lock were expected to directly follow the bottom of the trench. For the first three scenarios, it was assumed that suspended sediment particles were present over a height 8 meters in the trench of the Tilting lock. This entails that a sediment particle at the top of each water column in the trench has to cross 8 meters of water in the vertical direction to settle at the bottom. In reality, the height of the water columns differs over the longitudinal and lateral cross sections of the trench.

Several parameters were determined before the potential sedimentation in the trench for the Tilting Lock was estimated. All of these parameters are briefly elucidated in the coming paragraphs, but the detailed elaborations on most of these parameters are discussed in appendix I.1.

**Figure 8.7:** Scenarios 1, 2 and 3 to determine the sedimentation in the trench of the Tilting Lock, based on the incoming and outgoing sediment transport of a certain area.

**Figure 8.8:** Scenarios 4 and 5 to determine the sedimentation in the trench of the Tilting Lock, related to the sedimentation rate in the undisturbed areas of the Haringvliet estuary.
CHAPTER 8. HYDRODYNAMIC AND MORPHOLOGICAL INFLUENCES OF THE TILTING LOCK TRENCH

8.2.2 Scenario 1: Full deposition

In scenario 1, the amount of sedimentation underneath the Tilting Lock was determined by assessing the amount of incoming suspended sediments into the trench. It was assumed that all the incoming sediment particles by suspended load transport, would be deposited in the trench for the Tilting Lock. In addition, the erosion of the newly deposited particles was not taken into account.

The amount of sediment that will be transported into the trench depends on the concentration of suspended particles and the flow velocity as can be seen in equation 8.15. The formula for the rate of deposition in the trench is depicted in equation 8.16 [Bosboom and Stive, 2013]. As discussed in section 3.2.4, two types of sediment are in suspension in the vicinity of the Haringvliet bridge: non-flocculated silt \( c \approx 12 \text{ mg/l} \) and flocculated silt \( c = 1.5 \text{ mg/l} \). As could be seen, the deposition of suspended particles depends on the fall velocity of the particle and the concentration of suspended particles.

\[
S = c \cdot u \tag{8.15}
\]

\[
D = w_s \cdot c \quad \text{or} \quad D = \frac{Q_{in} \cdot c}{A_{trench}} \tag{8.16}
\]

In which:
- \( S \) Sediment transport rate \( \text{kg/(m}^2 \cdot \text{s}) \)
- \( c \) Sediment concentration \( \text{mg/l or kg/m}^3 \)
- \( u \) Local flow velocity \( \text{m/s} \)
- \( D \) Deposition rate \( \text{kg/(m}^2 \cdot \text{s}) \)
- \( w_s \) Fall velocity \( \text{m/s} \)
- \( c \) Concentration \( \text{kg/m}^3 \)
- \( Q_{in} \) Incoming discharge \( \text{m}^3/\text{s} \)
- \( A_{trench} \) Surface trench \( 28,000 \text{ m}^2 \)

The amount of discharge over the width of the trench for the Tilting Lock \( (q_{har}) \) was determined by assuming that the discharge through the Haringvliet estuary was equally distributed over the cross section of the Haringvliet near the Haringvliet bridge. According to the assumed water depth of 8 meters in the undisturbed area in the Haringvliet estuary and the width of 56 meters of the Tilting Lock, 5.4\% of the total discharge through the Haringvliet will go through the area \( (8 \cdot 56/8350 = 5.4\%) \). In table 8.2, the ratios between the discharge through the Haringvliet and the discharge through the trench of the Tilting Lock are included.

8.2.3 Scenario 2: Limited deposition

In scenario 2 the found value of deposition in scenario 1 was reduced by taking into account the time that sediment particles need to settle in the trench. Due to the very fine sediments in suspension, it was expected that the major part of the suspended particles would not be able to settle within the length of the trench. These suspended particles would travel along the streamlines without settling in the trench.

Settling length

A suspended particle in a water column will cover a horizontal distance before it reaches the bottom. The amount of particles that could settle within a predefined length depends on the settling length of the suspended particles. This settling length depends on the fall velocity of the suspended particle, the height of the particle in the water column and the local flow velocity, see equation 8.17 and figure 8.9.

\[
L_{set} = u \cdot \frac{h}{w_s} \tag{8.17}
\]

Figure 8.9: Definitions related to the settling length of a suspended sediment particle.
8.2. Sedimentation scenarios

**Settling length**

In which:

- $L_{set}$: Settling length \[m\]
- $h$: Water depth \[m\]
- $u$: Flow velocity \[m/s\]

**Fall velocity**

The fall velocity of a sediment particle depends, amongst others, on the size and the shape of the particle. As addressed in section 3.2.4, the majority of the suspended sediment at the Haringvliet bridge in the current situation consist of non-flocculated silt and a small part of flocculated silt [Van Wijngaarden and Ludikhuize, 1997]. The fall velocities of the different sediment types are depicted in table 3.5b.

**Suspended sediment concentration**

Both the flow velocity and the sediment concentration differ over the height of a water column as depicted in figure 8.10. In this thesis was assumed that the flow velocities were equally distributed over the height of the water column. The concentration of suspended particles was assumed to be distributed over the height of a water column by the Rouse distribution, as depicted in equations 8.18 and 8.19. In appendix I.1.2 is elaborated on the distribution of the sediment concentration over the height of a water column and the chosen values for the different parameters. The distribution of the sediment concentration over the water column was used to estimate the percentage of the concentration of the suspended particles in the water column that will settle in the trench, see figure 8.10.

\[
C(z) = C_a \left( \frac{h - z}{z} \cdot \frac{a}{a - z} \right)^{z_*} \tag{8.18}
\]

\[
z_* = \frac{w_s}{\kappa \cdot u_\star} \tag{8.19}
\]

In which:

- $C(z)$: Concentration at depth $z$ \[kg/m^3\]
- $C_a$: Concentration at reference depth $a$ \[0.0015$ (1.5) \[kg/m^3 (mg/l)\]
- $h$: Water depth \[8 \text{ m}\]
- $z_*$: Calculated depth \[m\]
- $a$: Reference depth \[6.5 \text{ m}\]
- $\kappa$: Rouse number \[0.8 \text{ [-]}\]
- $\kappa$: Von Karman constant \[0.4 \text{ [-]}\]
- $u_\star$: Shear stress velocity \[m/s\]

**Limited deposition of particles related to the settling length**

The required settling length was taken into account in the sediment balance by computing a ratio between the settling length and the length of the trench for the Tilting Lock. The ratio for the settling length is elaborated on in appendix I.1.3. The results of the elaboration on the settling lengths for the different suspended sediment types are depicted in table 8.3. It was assumed that the deposition of the non-flocculated silt particles within the length of the trench could be neglected. This is because the settling lengths already exceeded 690 meters for the non-flocculated silt by the lowest flow velocities found in the Haringvliet estuary, see table 8.3. Therefore only the contribution of flocculated silt particles to the deposition in the trench remained.
### Sedimentation and Morphological Influences of the Tilting Lock Trench

<table>
<thead>
<tr>
<th>Sediment type</th>
<th>Fall velocity $w_s$</th>
<th>Suspended concentration</th>
<th>$L_{set}$ $h = 0.1$ m</th>
<th>Settle height $L_{set} = 500$ m</th>
<th>% depo</th>
</tr>
</thead>
<tbody>
<tr>
<td>Non-flocculated silt</td>
<td>0.003 mm/s</td>
<td>12 mg/l</td>
<td>691 m</td>
<td>0.1 m</td>
<td>11.3%</td>
</tr>
<tr>
<td>Flocculated silt</td>
<td>0.116 mm/s</td>
<td>1.5 mg/l</td>
<td>17 m</td>
<td>4.1 m</td>
<td>72.0%</td>
</tr>
<tr>
<td>Fine sands</td>
<td>0.289 mm/s</td>
<td>0 mg/l</td>
<td>7 m</td>
<td>10.3 m</td>
<td>100%</td>
</tr>
</tbody>
</table>

Table 8.3: Fall velocities per sediment type and their concentration, see section 3.2.4 and appendix I.1.3 [Van Wijngaarden and Ludikhuize, 1997].

#### 8.2.4 Scenario 3: Erosion

Scenario 3 included the potential erosion of sediment particles in the estimations of scenarios 1 and 2. The magnitude of the erosion was calculated according to equation 8.20 [Van Wijngaarden and Ludikhuize, 1997]. In this formula, the erosion flux depends on the ratio between the local flow velocity in the trench and the critical flow velocity for the erosion of the sediment particles.

$$ E = M \cdot \left[ \left( \frac{u}{u_{cr,E}} \right)^2 - 1 \right] \quad \text{for} \quad u > u_{cr,E} $$

$$ E = 0 \quad \text{for} \quad u \leq u_{cr,E} $$

In which:
- $E$: erosion flux \( \text{kg} / (\text{m}^2 \cdot \text{s}) \)
- $u$: flow velocity \( \text{m} / \text{s} \)
- $u_{cr,E}$: Critical flow velocity for erosion \( \text{m} / \text{s} \)
- $M$: Erosion rate parameter \( \text{kg} / (\text{m}^2 \cdot \text{day}) \)

The critical flow velocity for erosion was determined according to the stability parameter by Shields ($\theta$). When the Shields parameter of a streamline at a certain location exceeds the critical Shields parameter ($\theta_{cr}$), bottom particles will be set in motion at that location. For the case study on the trench of the Tilting Lock, the critical erosion velocity was determined for both particles in suspension (0.24 m/s) and the particles at the bottom of the Haringvliet estuary (0.25 m/s), see appendix I.2.3.

The erosion parameter $M$ was assumed to be 0.5 \( \text{kg} / (\text{m}^2 \cdot \text{day}) \) in the studies on the effects of the Kierbesluit on the morphology of the South-Western delta [Van Wijngaarden and Ludikhuize, 1997]. As the same area was reviewed in this thesis, it was assumed that this value for the erosion parameter $M$ was sufficiently reliable. In further research, the value of the parameter should be refined according to the specific local conditions at the Haringvliet bridge.

The amount of erosion to be expected in the trench for the Tilting Lock is included in table 8.4 and discussed into more detail in scenarios 4 and 5. The amount of erosion in the trench for scenario 3 is the same as the amount of erosion as calculated for scenario 5.

#### 8.2.5 Scenario 4: Sedimentation in undisturbed situation

The sedimentation rate in the undisturbed area of the Haringvliet estuary was used as starting point for scenario 4. This sedimentation rate was estimated to be 7.5 \( \text{kg} / (\text{m}^2 \cdot \text{day}) \), as discussed in section 3.2.4, and was considered as the net deposition that will take place. The gross deposition can be calculated, by estimating the erosion of sediment particles in the undisturbed situation, see equation 8.21. The amount of erosion can be calculated with the assumed flow velocities in the Haringvliet estuary (section 3.2.2) and equation 8.20.

---

1 Besides being interesting for the sedimentation in the trench for the Tilting Lock, erosion of the bottom might cause problems as well. As discussed in chapter 5, erosion might be the cause of instability of the subaqueous slopes of the trench. In this thesis, the problems due to erosion were considered to be out of scope, as it was expected that the potential problems could be solved by bottom protection.
8.2. Sedimentation scenarios

\[ D_{\text{net}} = D_{\text{gross}} - E \rightarrow D_{\text{gross}} = D_{\text{net}} + E \]  

(8.21)

In table 8.4 the probability of the discharges through the Haringvliet (section 3.2.2) was related to the accompanying flow velocities in both the undisturbed area \( (u_{\text{har}}) \) and the trench for the Tilting Lock \( (u_{\text{trench}}) \). In the “Erosion rate” column, the erosion rate per second for these specific flow velocities is given. In the subsequent column, the amounts of erosion in the undisturbed Haringvliet and the trench were translated into a yearly value of the probability of occurrence of the discharge in time.

<table>
<thead>
<tr>
<th>Discharge</th>
<th>Flow velocity</th>
<th>Erosion rate</th>
<th>incl time %</th>
<th>∆E</th>
</tr>
</thead>
<tbody>
<tr>
<td>( Q_{\text{har}} ) m(^3)/s</td>
<td>( u_{\text{har}} ) m/s</td>
<td>( u_{\text{trench}} ) m/s</td>
<td>Time %</td>
<td>( u_{\text{har}} ) kg/m(^2)/s</td>
</tr>
<tr>
<td>95-perc. 3612</td>
<td>0.433</td>
<td>0.350</td>
<td>5%</td>
<td>13.0 ( \times ) 10(^{-6})</td>
</tr>
<tr>
<td>75-perc. 2803</td>
<td>0.336</td>
<td>0.270</td>
<td>20%</td>
<td>5.5 ( \times ) 10(^{-6})</td>
</tr>
<tr>
<td>50-perc. 1896</td>
<td>0.227</td>
<td>0.176</td>
<td>8%</td>
<td>5.5 ( \times ) 10(^{-6})</td>
</tr>
<tr>
<td>Total Erosion</td>
<td></td>
<td></td>
<td></td>
<td>93.9</td>
</tr>
</tbody>
</table>

Table 8.4: Erosion rates for \( u_{\text{cr,E}} = 0.249 \) m/s.

### 8.2.6 Scenario 5: Sedimentation in trench

In scenario 5, the found total yearly deposition in the undisturbed areas of the Haringvliet area of scenario 4 was reduced by the amount of expected erosion inside the trench for the Tilting Lock. The amount of erosion in the trench for the Tilting Lock depends on the local flow velocities in the trench, as calculated in table 8.2. The most right column \( (∆E) \) of table 8.4 depicts the differences in erosion between the undisturbed situation and the situation in the trench for the Tilting Lock. From this column was concluded that the difference in erosion rates between the undisturbed Haringvliet and the trench are relatively large. Therefore, more deposition of suspended particles in the trench for the Tilting Lock was expected in comparison with the undisturbed situation in the Haringvliet estuary.

### 8.2.7 Results scenarios

The sedimentation rates for the five situations were estimated and summarised in table 8.5. From the table, it was concluded that the two approaches delivered comparable results. The situations with and without erosion had similar estimations of the sedimentation in the trench for the Tilting Lock. For the situations without erosion in the trench, the maximum value for the sedimentation was approximately 102-108 kg/(m\(^2\)·year). For the situations where the erosion was taken into account, the expected sedimentation in the trench was around 78-84 kg/(m\(^2\)·year). These estimations for the sedimentation underneath the Tilting Lock are around 10 to 15 times as large as the sedimentation in the undisturbed Haringvliet in the vicinity of the Haringvliet bridge, which is \( \approx 7.5 \) kg/(m\(^2\)·year) as discussed in section 3.2.4.

<table>
<thead>
<tr>
<th>( Q_{\text{har}} ) m(^3)/s</th>
<th>( D_1 )</th>
<th>( D_2 )</th>
<th>( D_3 )</th>
<th>( D_4 )</th>
<th>( D_5 )</th>
</tr>
</thead>
<tbody>
<tr>
<td>N-F silt</td>
<td>F silt</td>
<td>N-F silt</td>
<td>F silt</td>
<td>N-F silt</td>
<td>F silt</td>
</tr>
<tr>
<td>MHWL</td>
<td>88.6</td>
<td>1198</td>
<td>150</td>
<td>135</td>
<td>108</td>
</tr>
<tr>
<td>MWL</td>
<td>60.2</td>
<td>814</td>
<td>102</td>
<td>92</td>
<td>73</td>
</tr>
<tr>
<td>MLWL</td>
<td>52.5</td>
<td>709</td>
<td>89</td>
<td>80</td>
<td>64</td>
</tr>
</tbody>
</table>

Table 8.5: Sedimentation based on the incoming particles in kg/(m\(^2\)·year)
CHAPTER 8. HYDRODYNAMIC AND MORPHOLOGICAL INFLUENCES OF THE TILTING LOCK TRENCH

Density of the deposited material
The amount of sedimentation in the trench should be related to changes in the bed level elevation, which can be done according to equation 8.14. Different densities of the deposited material will lead to very different dimensions of sedimentation. It was considered that the density of the deposited material would differ over time. The longer the particles are at the bottom, the more densification of the soil will occur. As table 8.6 depicts, the scatter of bed level changes is relatively large for different densities of the soil.

The densification of the newly deposited soil was considered to be beyond the scope of this thesis. A density value of 1200 kg/m$^3$ was assumed for the deposited soil particles, to calculate the amount of sedimentation in meters per year. When this value for the density of the deposited particles is applied on the sedimentation rate in the undisturbed situation in the Haringvliet estuary (7.5 kg/(m$^2$ · year)), this resulted in an estimated sedimentation of 6.3 mm/y. This bed level change is very much comparable to the estimation of the initial sedimentation in the Haringvliet of 6.5 mm/y, see section 3.2.4. In further research, the densification of the soil should be investigated into more detail.

<table>
<thead>
<tr>
<th>Density (kg/m$^3$)</th>
<th>Sedimentation Rate (mm/y)</th>
</tr>
</thead>
<tbody>
<tr>
<td>7.5 kg/(m$^2$ · y)</td>
<td>7.5 mm/y</td>
</tr>
<tr>
<td>10 kg/(m$^2$ · y)</td>
<td>10 mm/y</td>
</tr>
<tr>
<td>20 kg/(m$^2$ · y)</td>
<td>20 mm/y</td>
</tr>
<tr>
<td>50 kg/(m$^2$ · y)</td>
<td>50 mm/y</td>
</tr>
<tr>
<td>80 kg/(m$^2$ · y)</td>
<td>80 mm/y</td>
</tr>
<tr>
<td>100 kg/(m$^2$ · y)</td>
<td>100 mm/y</td>
</tr>
<tr>
<td>250 kg/(m$^2$ · y)</td>
<td>250 mm/y</td>
</tr>
<tr>
<td>400 kg/(m$^2$ · y)</td>
<td>400 mm/y</td>
</tr>
<tr>
<td>600 kg/(m$^2$ · y)</td>
<td>600 mm/y</td>
</tr>
<tr>
<td>800 kg/(m$^2$ · y)</td>
<td>800 mm/y</td>
</tr>
<tr>
<td>1000 kg/(m$^2$ · y)</td>
<td>1000 mm/y</td>
</tr>
</tbody>
</table>

Table 8.6: Sedimentation for different densities of the deposited sediments

The characteristic density of the material to settle is assumed to be approximately 1000 kg/m$^3$, which is based on an assumed porosity of 0.35-0.4. The characteristic density of deposited silt is in general lower than 1200 kg/m$^3$ [Sonke, 1996]. As the Haringvliet bridge is in between both, it is assumed that 6.5 mm is a reasonable estimation. with 7.5 kg/m$^2$ and 0.0065 m/j, the resulting density would be 1150 kg/m$^3$. However, the chosen density is very sensitive in relation to the amount of sedimentation in mm/y.

8.3 Conclusion hydrodynamics and morphology
During the performed calculations on the hydrodynamic and morphological situations at the Haringvliet bridge and the influences of the implementation of the Tilting Lock, multiple assumptions were made. The main aspects of discussion are treated in the following paragraphs. Firstly the size of the considered area of the trench in the calculations is discussed (8.3.1). Subsequently, the hydrodynamical (8.3.2) and morphological aspects (8.3.3) are discussed. In 8.3.4 the conclusion on the potential sedimentation in the trench for the Tilting Lock are drawn.

8.3.1 Discussion size considered area of the trench
The size of the area considered in the morphological calculations of this chapter was rather large and generalised. By the averaging of the dimensions over the width of Tilting Lock, the details of the geometry of the trench and the Tilting Lock were neglected.

Lateral cross sections of the trench
When smaller lateral cross sections will be reviewed, more specific results were found. The deeper parts of the trench for the Tilting Lock were expected to have lower flow velocities, compared to the less deep sections of the trench. This is because the streamlines through the deeper areas of the trench will encounter more energy dissipation along the streamlines, due to the larger wake at the downstream side of the Tilting Lock. A larger wake is related to more turbulent water flow and thus to more energy dissipation. In addition, the UKC of the Tilting Lock is in general smaller in the deeper sections of the trench, which is related to more contribution by the wall friction to the energy dissipations along the streamlines.
Lower flow velocities are related to a higher percentage of suspended sediment concentration that will be able to settle down, but also to a smaller amount of incoming suspended sediment. In addition, the probability of erosion of the newly deposited sediment particles is lower for lower flow velocities. Therefore, it was expected that the rate of sedimentation in the deeper parts of the Tilting Lock trench would be higher than in the current estimations made in this thesis. In contradiction, the sedimentation in the less profound section of the trench was expected to be less with respect to the made estimations.

**Longitudinal cross sections of the trench**

When longer stretches of the streamlines through the trench for the Tilting Lock are taken into account, the effect of the Tilting Lock on the energy dissipation along the streamline will be less. The energy dissipations by the Tilting Lock will be spread over a larger area, which results in a lower relative influence of the Tilting Lock.

Over the length of a streamline, the heights of the water column differ as well as the flow velocities along the streamlines. In the performed calculations was assumed that the height of the water column was equal to the initial water depth in the Haringvliet (8 meters) and the flow velocities as found in section 8.1 for the section underneath the Tilting Lock were representative for the whole trench. In reality, both the water depth and the flow velocities will differ along the streamlines through the trench for the Tilting Lock.

**Size cross sectional area Haringvliet**

The flow velocities in the undisturbed Haringvliet estuary were determined by dividing the discharges by a constant cross sectional area of the Haringvliet at the Haringvliet bridge. In reality, the surface of the cross sectional area changes with the fluctuating water levels in the Haringvliet estuary. The sensitivity of the calculated flow velocities is rather large, as shown in table 8.7.

<table>
<thead>
<tr>
<th>Water level NAP m</th>
<th>Cross sectional area $m^2$</th>
<th>Flow velocities $m/s$</th>
</tr>
</thead>
<tbody>
<tr>
<td>-0.4</td>
<td>7870</td>
<td>0.24</td>
</tr>
<tr>
<td>0.0</td>
<td>8350</td>
<td>0.23</td>
</tr>
<tr>
<td>0.6</td>
<td>9070</td>
<td>0.21</td>
</tr>
<tr>
<td>1.6</td>
<td>10.270</td>
<td>0.18</td>
</tr>
<tr>
<td>2.6</td>
<td>11.470</td>
<td>0.17</td>
</tr>
</tbody>
</table>

Table 8.7: Sensitivity of the flow velocities in the Haringvliet area to changes in water surface levels for a constant discharge of 1893 $m^3/s$.

For the expected discharge through the section underneath the Tilting Lock, it was assumed that the discharge through the Haringvliet estuary was equally distributed over the cross sectional area of the Haringvliet. However, the deeper sections of the estuary will attract more discharge than the shallow areas.

**8.3.2 Discussion hydrodynamics**

The determination of the local flow velocities in the vicinity of the Tilting Lock has some flaws which could be improved in further research. In this section, the points of discussion with regard to the changes in the hydrodynamic situation due to the implementation of the Tilting Lock are briefly discussed.

**Neglected friction factors in energy dissipation approach**

In the calculation of the energy dissipation along the streamlines through the trench of the Tilting Lock, multiple friction factors were neglected in this thesis. When those friction factors are included in the calculations, the found discharge ratios would decrease. The exclusion of these friction factors caused that the discharges and the flow velocity through the trench for the Tilting Lock were estimated higher than they would be in reality.
CHAPTER 8. HYDRODYNAMIC AND MORPHOLOGICAL INFLUENCES OF THE TILTING LOCK TRENCH

Local measurements, numerical modelling or CFD
The quality of all the made estimations could be improved by changing the method of calculation. In practice, it is tried to estimate the velocities on local measurements, which are not available for this thesis. A more modern approach is the application of Computational Fluid Dynamics (CFD)-software. Besides the common usage in coastal and river engineering, CFD’s are a common tool in the design of marine equipment [Battjes, 2002]. To improve the quality of the fluid mechanical calculations, it is recommended to make a numerical model or to use a CFD software package.

8.3.3 Discussion morphology
In the used approach to estimate the potential sedimentation of the trench for the Tilting Lock multiple assumptions were made which should be reconsidered in further research.

Bed load transport
Bed load transport of the bottom particles in the Haringvliet was expected to be negligible, as the main part of the flow velocities in the Haringvliet estuary were far below the critical flow velocity for erosion ($u_{crl,E}$) as determined for the assessment of the influences of the Kierbeshut on the morphological situation in the South-Western delta of the Netherlands. However, in appendix I.2.3 was proved that the critical flow velocity for motion of sediment particles for the situation in the Haringvliet estuary was significantly lower (0.24 m/s). Therefore, it could be concluded that the bed load transport should have been taken into account, at least in scenarios 1 to 3. However, in scenarios 4 and 5 the mode of sediment transport was of no importance. Therefore it was concluded that the estimations in scenarios 1 to 3 were overestimated, due to the relatively conservative approach.

Critical flow velocity for erosion
The critical flow velocity for erosion was determined in this thesis, based on the stability parameter by Shields. This parameter describes whether a sediment particle will be set in motion or not. Therefore this parameter is not fully representative for the critical flow velocity for erosion, as the particles are not necessarily brought in suspension for the found values for the critical flow velocities by the Shields parameter.

In the current design, the critical flow velocities were determined for the $D_{50}$’s of the suspended particles and the particles on the bottom of the Haringvliet estuary. However, the subsoil of the Haringvliet consists of much more types of sediment particles. Some of the (weaker) soil layers present at different depths will be exposed to the mainstream of the Haringvliet, by the excavation of the trench for the Tilting Lock, see figure 8.11. Especially the weaker layers around -7 to -18 m NAP and -21 to -22 m NAP could be sensitive to erosion, see figure 8.11. The probability of erosion and the impact of the consequences (mainly on the slope stability), should be investigated in further research.

Figure 8.11: Lateral cross section of the Tilting Lock and the required trench, with the multiple soil layers of the Haringvliet designated.
8.3. Conclusion hydrodynamics and morphology

Tilting motion Tilting Lock
Due to the tilting motion of the Tilting Lock, it was expected that some turbulent flow can be created in the Tilting Lock trench. This turbulent flow will have an influence in the local hydrodynamic situation in the areas beneath the Tilting Lock. Therefore, it was expected that the motions of the Tilting Lock will have influence on the sedimentation of particles in the area.

8.3.4 Conclusion on the potential sedimentation of the trench

In conclusion on the sedimentation of the trench for the Tilting Lock, it could be said that the expected sedimentation in the trench for the Tilting Lock will be around $80 \text{ kg/(m}^2 \text{ year)}$. For an assumed density for the deposited soil of $\rho_s = 1200 \text{ kg/m}^3$, the rise of the bed level in the trench would be around 0.07 meter per year. This sedimentation is significantly more than the sedimentation in the current conditions in the Haringvliet estuary near the Haringvliet bridge. With an assumed tolerance in Under Keel Clearance (UKC) for the Tilting Lock related to the potential sedimentation in the trench of 1 meter, it would take up to 14 years before maintenance on the depth of the trench is required.

Flushing of the area beneath the Tilting Lock was expected to occur, but to which extent this will be effective was unclear. The efficiency of the flushing to fight the sedimentation beneath the Tilting Lock could not be determined to a satisfying level, mainly because of the overestimation of the flow velocities in the trench. In addition, will the stability of the particles and thus the rate of erosion, depend on the density of the newly deposited particles. The lower the density of the deposited particles, the higher the probability of being brought in suspension again. Further research will be required to be able to quantify the flushing character of the trench for the Tilting Lock.

Maintenance on the depth of the trench
Although the amount of sedimentation in the trench will be limited at the case study location of the Haringvliet, maintenance on the depth of the trench will be required. No conclusion could be drawn on the exact frequency of maintenance that will be needed, as the range of sedimentation in the trench depends on various factors, which were estimated only very roughly in this thesis. For the operational phase of the Tilting Lock it is recommended that the available UKC of the Tilting Lock will be monitored frequently, to be able to exactly determine when maintenance is required.

The method to perform maintenance on the depth of the trench for the Tilting Lock will be a challenge. The deepest sections of the trench will be hard to reach with the common types of dredging equipment. It is, therefore, likely that special dredging equipment should be developed to perform the maintenance of the trench. For instance, this dredging method could be based on water injection dredging. Another option could be to completely remove the Tilting Lock from its position during times of maintenance, but this will add significant challenges to the design of the fixating structures for the Tilting Lock.

Other potential locations for the Tilting Lock
At other potential locations, where there are other types of sediment particles, a complete different situation for the implementation of the Tilting Lock will occur. At other locations, larger and heavier particles could be in suspension, leading to more deposition and less erosion of the sediment particles in the trench for the Tilting Lock. Depending on the exact location and circumstances, the sedimentation could potentially be a large factor regarding the design and operation costs of the Tilting Lock.

At other locations, with different conditions, the amount of sedimentation in the trench could be different, but it was expected that the magnitude of the discharge ratios would be similar. This because the ratio between the discharges in the undisturbed situation without the Tilting Lock and the discharges through the trench underneath the Tilting Lock, are depended on the geometry of the trench and the Tilting Lock. The influences of the discharge on the discharge ratio are very limited.
8.3.5 Recommendations

Multiple parameters were estimated very roughly in this thesis, because the lack of reliable data on the flow velocities and the characteristics of the suspended particles. With more recent data, better estimations could be made. More research is required to improve the estimations on the changes in morphology related to the implementation of the Tilting Lock and the trench. With more reliable estimations of the changes in morphology, a more adequate planning and cost estimation of the maintenance on the depth of the trench can be made.

A disadvantage of the used approach in this thesis is the very generalised geometry of the Tilting Lock and the trench. In addition, not all friction aspects along a streamline through the Tilting Lock trench were taken into account. More detailed calculations can increase the amount of calculations required, but not necessarily improve the quality of the results. This because the sedimentation is depended on other parameters that will have more influence on the sedimentation rate in the trench for the Tilting Lock, than the degree of detailed hydrodynamic calculations. Examples are the volumetric density ($\rho_s$) of the newly deposited sediment particles or the critical flow velocity for erosion ($u_{cr}$).

For more in-depth elaborations on the sedimentation in the trench for the Tilting Lock, it is recommended to focus on the following aspects:

- Determine the probability of erosion of the weaker layers in the subsoil of the Haringvliet. The erosion of these weaker soil layers might lead to instabilities of the subaqueous trench slopes.
- In the performed research it is proven that erosion of the newly deposited particles will occur. However, the magnitude of this erosion will depend on the rate of compaction of the deposited sediments. When the magnitude of the flushing character of the trench is sufficient, less maintenance on the depth of the trench is required.
- The method to perform maintenance on the depth of the trench for the Tilting Lock has to be designed.
Chapter 9

Evaluation of the case study design

The developed design for the trench of the Tilting Lock is discussed in this chapter. The construction phasing of the Tilting Lock trench and the installation of the cofferdam are treated in section 9.1. A cost estimation of the conducted trench design is made in 9.2. Chapter 9 ends with a discussion on the design that is made in the case study (10.1).

9.1 Global construction phasing for the Tilting Lock trench

To realise the trench that is required for the Tilting Lock, a construction phasing is inevitable. The general construction phasing of the proposed design of the trench can be described in different phases. The most important steps are discussed into more detail in the following paragraphs. The steps of the construction phase are listed in the enumeration below:

1. Removal of the initial bottom protection around the Haringvliet bridge piers.
2. Installation of the retaining walls of the cofferdam.
3. Start soil excavations for the trench and install the tension rings.
4. Install foundations for the fixating structures for the Tilting Lock.
5. Apply bottom protection in the trench.
6. Install superstructure of one of the fixating structures for the Tilting Lock.
7. Sail the Tilting Lock into the trench and submerge the Tilting Lock till the required depth by installing ballast in the Tilting Lock.
8. Move the submerged Tilting Lock underneath the Haringvliet bridge and connect to the installed fixation structure.
9. Install the second fixating structure and connect to the Tilting Lock.
10. Finish the installation of the Tilting Lock.

Removal of initial bottom protection (step 1.)
As discussed in section 3.3.2, bottom protection is present around the Haringvliet bridge piers (see figure C.2). Before the installation of the cofferdams, this bottom protection needs to be removed. Otherwise, the installation of the combi wall elements will be significant harder.

Construction of the retaining walls (step 2.)
The construction of the retaining walls for the cofferdam foundation require special attention. Due to the presence of the foundation of the Haringvliet bridge piers and the limited height available underneath the bridge deck, the installation can be complicated. In the following paragraphs multiple aspects related to the installation of the retaining walls are discussed. The general phasing of the installation of the retaining walls of the cofferdam is depicted in figure 9.1 and described in the enumeration below:
CHAPTER 9. EVALUATION OF THE CASE STUDY DESIGN

2.1 The first wall elements should be driven into the subsoil till the top of the wall is just above the water level (1. and 2. in figure 9.1a).

2.2 Next step is to place an additional combi wall element on top of the installed combi wall element and weld them together (3. in figure 9.1b).

2.3 After the weld has been checked and approved, the combi wall element can be pressed further into the subsoil (3. in figure 9.1b).

2.4 Subsequently the next combi wall element can be placed and welded onto the previous wall elements (5. in figure 9.1c).

2.5 Subsequently, the welded combi wall elements can be pressured further into the subsoil (6. in figure 9.1c).

2.6 This phasing is repeated till the full combi wall is installed (7. and 8. figure 9.1d).

2.7 The last wall element is installed at the required depth by the help of an extension piece (9. in figure 9.1e).

The combi wall elements have to be pressed into the subsoil of the Haringvliet estuary. Other methods of installing combi wall elements are available (like installation by vibrations), but undesirable in the vicinity of buildings and structures [CUR, 2005]. Advantage of installation by pressure is the low height required for installation. Disadvantage of the pressing of combi walls, is the higher costs and the longer time required to install the elements. In addition, pressuring the wall elements into the subsoil is only possible from above the water surface.

The maximal available height between the subsoil of the Haringvliet and the deck of the Haringvliet bridge is ≈ 20 meter. Above the water level, the maximum head clearance is ≈ 12 meter. Because of the restricted working height underneath the Haringvliet bridge, the length of the wall elements is limited.

The installation of the last element of the retaining walls for the cofferdam requires specialised construction equipment, as the equipment to install combi wall elements can not operate beneath the water surface. Therefore, the last element requires an extension piece to the installation equipment.

**Dredging methods (step 3.)**

The dredging of the soil in the vicinity of the Haringvliet bridge piers require special attention. The excavation around the bridge piers should take place with mechanical excavations methods to prevent damage to the cofferdam and the piers of the Haringvliet bridge. The usage of, for instance, cutter head dredgers have a high risk for the stability of the subaqueous trench slopes and the nearby bridge piers. The excavation of the major part of the trench does not necessarily require specialised dredging material and can be done separately of the areas in the vicinity of the Haringvliet bridge piers.

As discussed in chapter 3.2.6, contaminated soil is present in the Haringvliet estuary. The excava-
9.1. Global construction phasing for the Tilting Lock trench

Installation of the tension rings (step 3.)
The tension rings that are required for the horizontal stabilisation of the cofferdam walls can be installed relatively easy, following these steps:

3.1 When the required level for the tension rings are reached with the excavation, the excavation in that area should be paused. During the pause, the cables for the tension rings can be wrapped around the cofferdam and assembled at the subsoil surface.

3.2 The next step will be to stress the cables till the required stress level. Consequently, due to the inward directed resulting forces, the tension rings will be fixed in position.

3.3 After the installation of the tension rings, the excavation of the trench can continue and steps 1. and 2. are repeated.

Figure 9.2: Sequence of excavation of the Tilting Lock trench and the installation of the tension rings around the cofferdam walls.

Bottom protection (step 5.)
Around detached bodies in flow streams local scour is common, as can be seen in figure 9.3. Bottom protection is often applied to prevent significant scour in the vicinity of detached bodies. As it was expected that local erosion might take place around the bridge piers and the piles of the fixating structure for the Tilting Lock, bottom protection is taken into account in the construction phasing and cost estimation. The bottom protection was taken into account for \(1/4\) of the surface of the trench that is required for the Tilting Lock.

Figure 9.3: Scour around a cylinder (Breusers / Randkivi, 1991) cited in [Schiereck et al., 2000].
Floating element of the Tilting Lock (step 7.)
The construction phasing of the Tilting Lock itself is not treated, as the Tilting Lock is a floating steel object that can be fabricated at a shipyard and transported to its final position. When arrived at the final location, sufficient ballast has to be applied to submerge the Tilting Lock to the required depth. Therefore, the construction phasing of the floating body of the Tilting Lock is considered to be independent of the construction of the trench and thus beyond the scope of this construction phasing.

9.2 Costs of the implementation of the Tilting Lock
The costs of the design of the trench for the Tilting Lock at the case study location at the Haringvliet bridge were determined according to rules of thumb in appendix J. In this appendix several assumptions about the cost estimations are discussed. An overview of the cost estimations of the implementation of the Tilting Lock is depicted in table 9.1.

<table>
<thead>
<tr>
<th>Cost item</th>
<th>Estimated costs</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dredging works (volume 1,480,000 m³)</td>
<td>€ 10,545,000</td>
</tr>
<tr>
<td>Bottom protection (surface 26,500 m²)</td>
<td>€ 795,000</td>
</tr>
<tr>
<td>Cofferdams (2 pieces)</td>
<td>€ 9,080,000</td>
</tr>
<tr>
<td>Direct costs</td>
<td>€ 20,500,000</td>
</tr>
<tr>
<td>Design uncertainties (15%)</td>
<td>€ 3,100,000</td>
</tr>
<tr>
<td>Contractors cut (35%)</td>
<td>€ 8,300,000</td>
</tr>
<tr>
<td>Total costs Tilting Lock trench</td>
<td>€ 31,900,000</td>
</tr>
</tbody>
</table>

Table 9.1: Overview cost estimation of the trench that is required for the Tilting Lock, see figure J.1.

The construction costs of the Tilting Lock itself were considered to be out of the scope within this thesis, as no additional elaborations on structural elements like the floating body and the fixating structures of the Tilting Lock were made in this thesis. In previous research were the costs of these components of the Tilting Lock system estimated on 60 million euros, see chapter 2.5. The cost estimation of the Tilting Lock (€60 million) was based on a Tilting Lock of 190 meter long and a diameter of 48 meter [Royal HaskoningDHV, 2014a]. It was assumed that for a Tilting Lock of 90 meter long and a diameter of 56 meters, the costs would not be significantly different.

The total cost estimations that were made exclude the costs that are required for the operational phase of the Tilting Lock. During the operational phase, costs on maintenance and operation are to be expected. In the MKBA, these costs were taken into account in a percentage of 2% of the initial building costs related to the Tilting Lock [Royal HaskoningDHV, 2014b].

9.2.1 Economic feasibility of the Tilting Lock
To review the feasibility of the Tilting Lock in relation to the costs and the benefits, the effects of the made design in the case study are discussed in the following paragraphs. Subsequently, these influences are related to the other potential solutions to solve the problem with the interfering traffic flows at the Haringvliet bridge, as discussed in appendix A.3.

The added construction costs due to the trench and the required cofferdam are much more than estimated in previous research. In the previous estimations, the costs for dredging were estimated on €2 million [Royal HaskoningDHV, 2014b]. These additional costs make that the Tilting Lock is not the cheap solution that it was thought to be in relation to other alternatives, as will be discussed in the next paragraphs (9.2.2). To assess the economic feasibility of the Tilting Lock at the Haringvliet bridge, it was recommended to investigate to following aspects into more detail in future research:
9.2. Costs of the implementation of the Tilting Lock

- The intensity and the height distribution of the vessels passing the Haringvliet bridge. This data can be used to find the optimum size for the Tilting Lock in relation to the dimensions and intensities of the vessels and to quantify the benefits of the Tilting Lock in comparison to other alternatives.
- To increase the monetary benefits of the Tilting Lock, it was recommended to quantify the delays in travel time in the current situation with regular opening of the Haringvliet bridge. These delays should be included in official calculation models to determine the exact benefits of the Tilting Lock.

**Smaller sized Tilting Locks**

A possibility to increase the technical and economical feasibility of the Tilting Lock can be a smaller sized Tilting Lock. Based on the case study at the Haringvliet bridge, it can be concluded that the technical feasibility of the trench for the Tilting Lock will improve for smaller sized Tilting Locks, as the cofferdam foundation for the bridge piers might be unnecessary and less excavations of subsoil are required in case of a smaller Tilting Lock.

An example of such a smaller lock with significant benefits for the technical feasibility of the trench that is required for the Tilting Lock, has a radius of approximately 17 meter and an added air draft of 5.8 meter, see appendix K. The absence of the cofferdam foundations for the Haringvliet bridge piers would reduce the costs of the Tilting Lock. In chapter 6 it was found that an excavation of 7 meters below the current bed level is allowed around the Haringvliet bridge piers. With the assumed slope inclination of 1:5, this results in a trench of 24 meters deep, which would be sufficient for this size Tilting Lock.

However, the economic benefits would decrease as well for smaller sized Tilting Locks. This is because the added height to the available head clearance for the vessels will be smaller for smaller sized Tilting Locks than for larger sized Tilting Locks. Therefore, an optimum should be found between the size of the Tilting Lock, the accompanying trench that is required and the economic benefits of the added height to the available head clearance for vessels. This should be studied in future research.

**9.2.2 Competitiveness of the Tilting Lock concept**

In this thesis, it was discussed that the Tilting Lock will be the most competitive at locations where a relatively large water is crossed by a road traffic bridge. In addition, the traffic should experience significant delays due to the regular opening of the bridge. The competitiveness of the Tilting Lock regarding other potential solutions is discussed in the following paragraphs. The advantages and disadvantages of the other potential solutions are briefly discussed in the following enumeration, together with their suitable locations and the estimated costs:

- **Jacking up bridge**
  - *Suitable for:* Locations where small differences in available and required air draft for vessels (≈ 1 meter) are required at existing bridges.
  - *Costs:* Relatively cheap (≈ €1 million euros).
  - *Advantage:* Less conflicts between marine and road traffic.
  - *Disadvantage:* Only a small added height to the air draft for vessels and the movable bridge part still needs to open to give passage to large vessels.

- **Tunnel**
  - *Suitable for:* Situations where no limitations on air draft vessels are required, but small limitation on head clearance for road traffic are allowed.
  - *Costs:* Costly (≈ €600 million or more)
  - *Advantage:* No conflicts between road and marine traffic.
  - *Disadvantage:* Very costly measure for a relative small problem with regards to the economical losses.
CHAPTER 9. EVALUATION OF THE CASE STUDY DESIGN

- **New bridge**
  - Suitable for: Locations where small limitations on the available air draft for vessels are allowed, without delays for both marine and road traffic.
  - Costs: Moderate (≈ €200 million or more).
  - Advantage: Hardly any conflicts between marine and road traffic, but movable bridge part still need to open for very large vessels.
  - Disadvantage: Large impact on the surroundings and more complicated construction phasing without interference of the traffic flows.

- **Tilting Lock**
  - Suitable for: Locations where small limitations on the available air draft for vessels are allowed.
  - Costs: Moderate (≈ €100 million).
  - Advantage: Hardly any conflicts between marine and road traffic, movable bridge part still needs to open for very large vessels. Additional advantage is that the construction of the Tilting Lock can be done without hindrance of the traffic flows.
  - Disadvantage: Marine traffic encounters delays in travel time due to locking cycle.

Recap competitiveness Tilting Lock
With the elaboration on the trench that is required for the implementation of the Tilting Lock, a recap on the competitiveness of the Tilting Lock can be made. The Tilting Lock does not provide a completely conflict free traffic intersection between road and marine traffic. There will always be vessels that do not fit in the internal channels of the Tilting Lock and therefore require the opening of the movable bridge part to pass the Haringvliet bridge. However, the amount of required openings of the movable bridge part will be lowered to an almost negligible amount in relation to the current situation.

As the Tilting Lock cannot completely prevent the need to open the movable bridge part from the traffic junction at the Haringvliet bridge, it can be concluded that the Tilting Lock is not the ultimate solution to problems regarding limited air draft for vessels.

9.3 Scaling of the Tilting Lock
In the conducted case study, a design was made for the trench that is required to implement the Tilting Lock at the Haringvliet bridge. During the elaboration on the case study, it was found that other sized Tilting Lock can be favourable. In addition, it is interesting to know the relation between the different dimension parameters of the Tilting Lock. With these relations, the optimal size of the Tilting Lock can be found for any potential location for the Tilting Lock

9.3.1 Research approach
To elaborate on the relations between the different parameters of the Tilting Lock, the general lateral cross sections of the Tilting Lock and the lateral cross sections of the Tilting Lock underneath the bridge are reviewed. In appendix K the dimensions of the Tilting Lock in these cross sections are determined by geometric relations.

Assumptions
In the conducted research, several assumptions have been made on certain parameters. These assumptions are characteristic for the Tilting Lock that is required for the Haringvliet bridge and are in line with the assumptions made on the case study design of the Tilting Lock (see table 1.1). For other locations, different assumptions might be more appropriate.

- **Tilting angle** (θ) of the Tilting Lock is 22° to one side. The angle between the two tilted positions of the Tilting Lock is in total for 44°.
- **Radius of the Tilting lock** is 28 meters (diameter = 56 meter).
- **Required Under Keel Clearance (UKC)** is 4 meters for the Tilting Lock.
- **Length of the Tilting Lock** is 90 meters.
- **Internal channels** have a radius of 5.6 meter (diameter = 11.2 meter).
9.3. Scaling of the Tilting Lock

- Structural gauge for the width of the vessels is 6 meter.
- The inclined walls of the Tilting Lock are under the same angle as the tilting angle \( \theta \).
- Additional height of the outer hull to prevent wave overtopping is 1.5 meter.

Designation of the characteristic parameters

In figure 9.4 the characteristic dimensional parameters of the Tilting Lock are depicted. The geometric relations to determine the values of these parameters can be found in appendix K.

- \( R \) is the radius of the Tilting Lock.
- \( \delta h \) is the submerged depth of the center of the Tilting Lock.
- \( 2 \cdot \delta h \) is the added air draft of the Tilting Lock to the clearance height of the vessels.
- \( \theta \) is the tilting angle of the Tilting Lock.
- \( K \) is the air draft of the Tilting Lock in the general lateral cross sections of the Tilting Lock.
- \( T \) is the air draft of the Tilting Lock in the lateral cross sections underneath the bridge.
- \( U \) is the total draft of the Tilting Lock, measured from the water line towards the lowest point.

9.3.2 Analyses of the dimensional parameters

In the following paragraphs, several analyses are done on the influences of the characteristic parameters of the Tilting Lock. First, the relations of the tilting angle \( \theta \) and the radius of the Tilting Lock \( R \) and the different vertical parameters are studied. Secondly, the found relations are translated to multiple scaling scenarios. Lastly, it is elaborated on the influences of the varied parameters on the trench that is required for the implementation of the Tilting Lock.

Scaling relations

To investigate the relations between the different dimensional parameters of the Tilting Lock, the influence of varying the two characteristic parameters \( R \) and \( \theta \) of the Tilting Lock are investigated.

In figure 9.5a the different characteristic parameters are depicted in a graph for several tilting angles \( \theta \) of the Tilting Lock, for a constant Tilting Lock radius of \( R = 28 \text{ meter} \). As can be seen, is an optimum found for the added air draft of the Tilting Lock \( 2 \cdot \delta h \) at a tilting angle of \( 25^\circ \).

In figure 9.5b the different characteristic parameters are depicted in a graph for a constant tilting angle \( \theta = 22^\circ \) and several radii of the Tilting Lock \( R \). As can be seen, are the values of the dimensions parameters linear increasing for larger radii. It is found that the relations of the parameters to each other are as follows:

- The relation between obtained added air draft \( 2 \cdot \delta h \) versus the radius of the Tilting Lock \( R \) is 1:2.2.
- The relation between the height of the Tilting Lock in the sections outside the bridge \( K \) and the radius of the Tilting Lock \( R \) is 1:1.6.
- The relation between the required head clearance for the Tilting Lock underneath a bridge \( T \) and the radius of the Tilting Lock \( R \) is 1:6.1.
- The relation between the radius of the Tilting Lock \( R \) and the required depth for the Tilting Lock \( U \) is 1:1.2.

According to figure K.4 it is to be expected that the height of the Tilting Lock section underneath
the bridge decreases \((T)\) for larger tilting angles. As the height of the Tilting Lock underneath the bridge is not assessed during the tilting motion. What the realistic relation between the available height that is, should be checked in future research.

(a) Parametric relations for a varied tilting angle \(\theta\) and a constant radius \(R = 28\) meter.
(b) Parametric relations for a constant tilting angle \(\theta = 22^\circ\) and a varied radius \(R\).

Figure 9.5: Scaling relations for varied tilting angles of the Tilting Lock \((\theta)\) and varied radii of the Tilting Lock \((R)\).

Scaling scenarios
In the analysis on the scaling of the Tilting Lock, a couple of characteristic scenarios are reviewed. An example is the size Tilting Lock that can fit in a trench, that does not require the cofferdam foundation to stabilise the Haringvliet bridge piers. The assessed scaling scenarios are listed below. The accompanying dimensions of the Tilting Locks for the situations are included in table 9.2.

1. The smallest feasible Tilting Lock and at the same time, the largest Tilting Lock for which no excavation is required near the Haringvliet bridge piers.
2. The largest Tilting Lock, for which no additional measures to the first bridge pier are required.
3. The largest Tilting Lock, for which no excavations are required around the second bridge piers.
4. The largest Tilting Lock, for which no additional measures to the second bridge pier are required.
5. The largest Tilting Lock that fits underneath the Haringvliet bridge (air draft of +13 meter).

<table>
<thead>
<tr>
<th>Scenarios</th>
<th>(R)</th>
<th>(U + UKC)</th>
<th>(2 \cdot \delta h)</th>
<th>(T)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.</td>
<td>11</td>
<td>16.5</td>
<td>3.1</td>
<td>8.6</td>
</tr>
<tr>
<td>2.</td>
<td>17</td>
<td>23.9</td>
<td>5.8</td>
<td>9.6</td>
</tr>
<tr>
<td>3.</td>
<td>28</td>
<td>37.4</td>
<td>5.4</td>
<td>11.4</td>
</tr>
<tr>
<td>4.</td>
<td>34</td>
<td>44.8</td>
<td>13.5</td>
<td>12.3</td>
</tr>
<tr>
<td>5.</td>
<td>38</td>
<td>49.7</td>
<td>15.4</td>
<td>13.0</td>
</tr>
</tbody>
</table>

Table 9.2: Dimensions of the characteristic Tilting Lock sizes for a constant tilting angle of \(\theta = 22^\circ\).

The smallest feasible Tilting Lock under the assumed dimensions for the structural gauge of the passing vessels (6 meter width), has a radius of 11 meters and 3 meters of added air draft to the head clearance of the vessels. This Tilting Lock requires an available air draft underneath a bridge of at least 8.6 meters. The required water depth is 12.5 meters (without UKC).

Influence on the size of the Tilting Lock trench
The different sizes of the Tilting Lock can be related to the size of the trench that is required to implement the Tilting Lock underneath a bridge. In the following paragraphs is elaborated on the relation between the radius of the Tilting Lock \((R)\) and the excavations around the Haringvliet bridge piers. Also the relation between the radius of the Tilting Lock \((R)\) and the volume of the excavation that is required to construct the Tilting Lock trench.
In figure 9.6a are different radii of the Tilting Lock \((R)\) related to the excavations depths around the Haringvliet bridge piers (the orange line for the first pier, the light blue for the second pier). The green line depicts the maximum depth of the trench that is required for the Tilting Lock. In the figure is also depicted, for which radii of the Tilting Lock a cofferdam foundation is required to stabilise the Haringvliet bridge piers (the red line for the first pier and the darker blue line for the second pier). For example, a cofferdam is required around the first bridge pier, when the radius of the Tilting Lock is larger than 17 meters. This coincides with the second scenario of table 9.2.

In figure 9.6b the excavation volume of the trench for a Tilting Lock with a length of 90 meters is related to the different ratios of the Tilting Lock. As can be seen, is the relation between the radius and the excavation volume exponential.

Figure 9.6: Scaling relations for varied radii of the Tilting Lock and the size of the trench required for the implementation of the Tilting Lock.

9.3.3 Conclusions scaling of the Tilting Lock

In the following paragraphs the found results will be discussed and subsequently conclusions will drawn on the scaling of the Tilting Lock.

Discussion on the performed research

The reviewed lay-out of the Tilting Lock is based on the described Tilting Lock in chapter 2. In future research other lay-outs of the lateral cross section of the Tilting Lock can be considered, which will give different results. The conducted research only focused on the geometry of the inner channels, the outer hull and the partition walls between the inner channels and the outside water and the internal channels. Other aspects, like the space for the required ballast in the Tilting Lock or for the locking gates, were assessed. In future research it is recommended to take these aspects into account, as they are vital for the balance of the Tilting Lock during the tilting motions.

For smaller structural gauges for the passing vessels, it was expected that the added air draft \((2\cdot\delta h)\) would be smaller. In addition, it was expected that for different dimensions of the structural gauge and the internal water channels, the design of the lateral cross section of the Tilting Lock can be optimised.

For smaller sized Tilting Locks, the technical feasibility of the trench that is required for the implementation of the Tilting Lock will improve. However, the economic benefits of the Tilting Lock would decrease, as the added air draft to the available head clearance of the vessels will be less. Less added air draft implies that less vessel can use the Tilting Lock to pass the Haringvliet bridge. Therefore, more vessels will require the opening of the movable bridge part of the Haringvliet bridge in case of a smaller sized Tilting Lock.
Conclusions on the parametric relations of the Tilting Lock

In the performed analyses it was found that the tilting angle $\theta$ of the Tilting Lock and the radius $R$ are the two most important parameters related to the size of the Tilting Lock and the amount of added air draft for the vessels. The following conclusions were drawn from the conducted analyses:

- The relation between the radius of the Tilting Lock and the added head clearance for the vessels is linear.
- The relation between the required depth of the trench for the Tilting Lock and the radius of the Tilting Lock is linear.
- The optimum tilting angle $\theta$ of the Tilting Lock (with a radius $R$ of 28 meters) regarding the added air draft for the vessels is $\approx 25^\circ$.
- The relation between the excavation volume of the trench that is required for the Tilting Lock and the radius of the Tilting Lock is exponential.
- The relation between obtained added air draft versus the radius of the Tilting Lock is $\rightarrow 2 \cdot \delta h : R = 1:2.2$.
- The relation between the height of the Tilting Lock in the sections outside the bridge and the radius of the Tilting Lock is $\rightarrow K : R = 1:1.6$.
- The relation between the required head clearance underneath a bridge and the radius of the Tilting Lock is $\rightarrow T : R = 1:6.1$.
- The relation between the radius of the Tilting Lock and the required depth for the Tilting Lock is $\rightarrow R : U = 1:1.2$.

With the conducted research, the used size of the Tilting Lock in the case study can be evaluated. Regarding the scaling of the Tilting Lock, the following conclusions were drawn for the case study at the Haringvliet bridge:

- To obtain the desired added air draft for the vessels of 12 meters ($2 \cdot \delta h$) for the case study, a larger sized Tilting Lock ($R \approx 31$ meters) is required than the applied size of the Tilting Lock in the case study.
- The maximum available head clearance underneath the Haringvliet bridge ($T \approx 13$ meters), allows for larger sized Tilting Locks (radius up to 38 meters) than applied in the case study.
- The implementation of a larger Tilting Lock than in the case study, will lead to a significant larger required depth for the Tilting Lock trench $U$, see table 9.2.
- The open trench (alternative 1, chapter 4) can provide for sufficient depth for a Tilting Lock with a radius $R$ of 17 meters ($2 \cdot \delta h = 5.8$ meters). Therefore, no measures will be required to stabilise the Haringvliet bridge piers for this size Tilting Lock.
Chapter 10

Discussion and conclusions

The main objective of this thesis was to investigate the feasibility of the Tilting Lock with respect to the depth required for the placement of the Tilting Lock. To fulfil this objective, a case study was conducted on the implementation of the Tilting Lock at the Haringvliet bridge. The results of the performed case study are discussed in 10.1 and recommendations for future research are drawn. The main research question and the supplementary question are answered in the conclusions in 10.2.

10.1 Discussion and recommendations

The case study on the technical feasibility of the Tilting Lock that was performed in this thesis increases the insights in the requirements for the implementation of the Tilting Lock at a specific location; the Haringvliet bridge.

In the following paragraphs, the results that are found this thesis are discussed and recommendations are given for future research on the specific topics. First, the case study design of the trench that is required to provide sufficient UKC for the Tilting Lock at the Haringvliet bridge is discussed in 10.1.1. In addition, implications for future research related to the case study design are given in this section. Secondly, the feasibility of the other potential alternatives for providing sufficient depth for the Tilting Lock (as discussed in chapter 4) are discussed in 10.1.2. Lastly, the influences of the case study results on the global design of the Tilting Lock are discussed in 10.1.3.

10.1.1 Discussion case study design at Haringvliet bridge

To evaluate the case study design of the trench that is required for the Tilting Lock, the aspects studied in this thesis are briefly discussed in the following paragraphs. The found results depend very much on the local conditions and are therefore characteristic for the case study at the Haringvliet bridge. These local conditions might differ significantly for other locations. The required size of Tilting Lock, the location of the bridge piers, the soil conditions, the type of bridge pier foundation, the hydrodynamic situation and thus the geometry of the trench will differ for each location. Therefore, it was concluded that the found results for the Haringvliet bridge case study are not necessarily suitable for other locations.

Stability of subaqueous trench slopes

In chapter 5 was concluded that the subaqueous trench slopes are expected to be technically feasible for inclinations of 1:5 (≈ 11.3°). From basic checks, it was concluded that the trench slopes are potentially sensitive to liquefaction. The macro stability was assessed into more detail. Due to the presence of the cofferdam walls, it was not expected that macro stability failures will be normative. Therefore, it was concluded that the subaqueous trench slopes are not expected to fail at the case study location of the Haringvliet bridge.
CHAPTER 10. DISCUSSION AND CONCLUSIONS

The resulting design of the trench that is required for the Tilting Lock contains less steep slopes than was expected at the start of the elaboration (slope inclination of 1:5 instead of 1:3). Due to the milder slope, the amount of excavation that is required for the construction of the trench was doubled (excavation volume increase from 720,000 m$^3$ to 1,480,000 m$^3$). The milder slopes also resulted in larger excavations around the Haringvliet bridge piers. However, the feasibility of the case study design was not really affected by this larger excavations, as the Haringvliet bridge piers were still found to be stable after the inclusion of a cofferdam foundation for the bridge piers.

In future research, the design of the trench slopes can be made more hydrodynamic friendly, by including milder top and bottom sections of the slopes. It is also recommended to investigate the feasibility of steeper, reinforced slopes in future research. Reinforcing the slopes will decrease the sensitivity of the subsoil to liquefaction, will reduce the excavation volume of the subsoil and will reduce the magnitude of the excavation around the Haringvliet bridge piers.

Stability bridge piers under excavations
In chapter 6 it was found that the Haringvliet bridge piers will remain stable for excavations up to 7 meters below the current soil surface level (-15 meter NAP). In this thesis, it was demonstrated that excavations around the bridge piers will be feasible. However, a cofferdam is required in the case study design to be able to implement the Tilting Lock underneath the Haringvliet bridge without endangering the stability of the bridge piers.

Cofferdam foundation for stability of the Haringvliet bridge piers
In chapter 7 the feasibility of the cofferdam as a foundation for the Haringvliet bridge piers was studied. In the current elaborations, no critical blank spot was found regarding the feasibility of the cofferdam as a foundation for the bridge piers. Based on the static analysis of the required dimensions for the retaining walls of the cofferdam, it was concluded that the cofferdam as a foundation for the piers of the Haringvliet bridge seems to be feasible.

In the elaboration on the cofferdam was focused on the main structural elements of the cofferdam wall: the retaining walls. This is the second of seven steps in the design guidelines for cofferdams by the Dutch standards [CUR, 2005]. It is recommended to perform the subsequent steps in the design guidelines. In addition, it is recommended for future research to elaborate on the deformations of the structural elements of the cofferdam, like the retaining walls and the Haringvliet bridge piers. In addition, the feasibility of the tension rings as a horizontal stabilising measure for the cofferdam walls should be studied into more detail.

Recommendation geo-technical aspects
The design of the trench for the Tilting Lock can be optimised significantly in relation to the geotechnical aspects of the slope stability, the bearing capacity of the pier foundation and the cofferdam. For future research, it is recommended to use Finite Element Method (FEM) software to analyse the interaction between the different structural elements and the subsoil of the Haringvliet estuary and to determine the deformations of the several structural elements.

Sedimentation in the Tilting Lock trench
In chapter 8 the potential sedimentation in the trench for the Tilting Lock was studied. It was found that the sedimentation in the trench for the Tilting Lock will be approximately ten times as large as in the Haringvliet estuary in the current situation ($\approx 65$ mm/year versus $\approx 6.5$ mm/year).

Although a basic approach was used in this thesis based on the generalised geometry of the Tilting Lock and the trench, it is demonstrated that the amount of sedimentation in the trench will be a problem. Therefore, it is recommended to investigate the methods to perform maintenance on the depth of the trench. The deepest sections of the trench will be hard to reach with the common types of dredging equipment, as the Tilting Lock will be in the way.

To prevent sedimentation in the trench for the Tilting Lock, adjustments to the case study design might be considered in future research. The most interesting solution could be filling the trench for
the Tilting Lock with the gel that is developed for the gel sluice [Hydraulic Engineering, 2014]. As vessels are able to sail through the gel, the Tilting Lock will still be able to rotate when surrounded by the gel. As this gel cannot be penetrated by water, the import of (suspended) sediment particles will be prevented.

Regarding the elaboration on the sedimentation rate in the trench that is required for the Tilting Lock, the following recommendations are drawn:

- Extend the estimations on the discharges and local flow velocities in the trench of the Tilting Lock by using Computational Fluid Dynamics (CFD) software.
- Elaborate on the method to perform the maintenance on the depth of the trench for the Tilting Lock.
- During the lifetime of the Tilting Lock, it is recommended monitor the available UKC of the Tilting Lock frequently. In this way, it will be possible to exactly determine when maintenance is required on the depth of the trench for the Tilting Lock.

**Effects of the Tilting Lock on the water storage system of the South-Western delta**

Due to the implementation of the Tilting Lock at the Haringvliet, the water storage system in the South-Western delta of The Netherlands will be influenced. The water storage capacity of the Haringvliet is an important factor in the safety against flooding of large areas in the Netherlands.

The water storage capacity of the Haringvliet estuary is not influenced by the implementation of the Tilting Lock, as the Tilting Lock is floating. The implementation of the Tilting Lock in the system would increase the water level for a very short period of time, but this rise of the water level will be levelled as long as the connection with the North Sea stays present. Therefore, the effects on the water storage system were considered to be beyond the scope, but in future research, this aspect should be considered before the final decision on the realisation of the Tilting Lock can be made.

The Tilting Lock and the trench can have an influence on the discharge through the Haringvliet. The Tilting Lock will add resistance to the streamlines through the Haringvliet as can be seen in chapter 8.1. The extent to which this will affect the (extreme) water levels has to be studied in future research.

**Missing aspects in the case study research**

In addition to the subjects that were studied in this thesis, more research topics within the case study design can be investigated in future research. The most important aspects that should be studied in future research are discussed in 4.2.3 and summarised below:

- The fixating structure that is required to keep the Tilting Lock in place. Due to the relatively large depth of the trench for the Tilting Lock (≈ 38 meters) the design of the fixating structure was expected to be challenging.
- Driving mechanism to perform the tilting motions of the Tilting Lock.
- Influences of the trench on the hydrogeology in the Haringvliet estuary, see section 3.2.7.
- Effects due to swell of the subsoil of the trench that is required for the Tilting Lock, see .

### 10.1.2 Discussion concept alternatives for the trench

From the alternatives discussed in chapter 4.2, the open trench (alternative 1) and the open pier with measures for the bridge piers (alternative 3) were studied in this thesis as alternatives for providing sufficient depth for the Tilting Lock. With the results of the performed case study, feasibility of the other two alternatives, the pneumatic caisson (alternative 2) and the open building pit (alternative 4), can be reviewed.
The pneumatic caisson (alternative 2) can be considered to be unnecessarily expensive and complicated. When such a large concrete structure is desired, one should reconsider the choice for a Tilting Lock, since the construction of the pneumatic caisson is very much comparable to a conventional locking complex. However, when a conventional locking complex is build to add the same amount of air draft to the head clearance of the vessels, the required construction depth would be significantly less. Therefore, it was concluded that the pneumatic caisson as a solution to provide sufficient depth for the Tilting Lock is unfeasible.

Regarding the alternative with stabilising measures for the Haringvliet bridge piers (alternative 3), it was found that the cofferdam can be a feasible design for stabilisation of the bridge piers. However, this does not necessarily mean that the cofferdam is the best solution to stabilise the Haringvliet bridge piers under large excavations. Other alternatives, like expanding the amount of foundation piles underneath the foundation of the Haringvliet bridge piers, can be satisfying stabilising measures to secure the stability of the Haringvliet bridge. Therefore, different alternatives for the stabilising measures should be considered in future research.

The open building pit (alternative 4), can be considered as an interesting alternative to elaborate on in future research. The results of the performed research on the retaining walls of the cofferdam foundation (chapter 7) indicate that the large retaining height that is required, does not have to be a major problem.

Eventually, it can be concluded that the chosen concept of open trench including the cofferdam walls still seems to be the most favourable alternative regarding the design alternatives for the case study at the Haringvliet bridge. Therefore, it is recommended to continue with the elaboration of this design.

10.1.3 Discussion of global design of the Tilting Lock

Scaling of the Tilting Lock

In chapter 9.3, the scaling of the Tilting Lock regarding the radius of the Tilting Lock and the tilting angle of the Tilting Lock are elaborated on. With the found relations, the Tilting Lock can be scaled towards the required size, the available water depth, the available head clearance underneath a bridge or the desired added air draft by the Tilting Lock.

The disadvantage of the research is the limitation of variables taken into account. For the operation of the Tilting Lock, the internal mass balance is very important. Within the performed study, this balance was not taken into account. Therefore, it is recommended to investigate the relations between the different dimensional parameters of the Tilting Lock into more detail.

Energy consumption of the Tilting Lock

One of the initial benefits of the Tilting Lock was the relatively low energy consumption of a locking cycle [Witteveen and Wolfsen, 2015]. However, the case study design as described in this thesis is expected to have a significant contribution to the total energy consumption of the Tilting Lock, due to the large required excavations for the Tilting Lock trench.

To study the energy consumption of the Tilting Lock in future research, it is recommended to take the complete life cycle of the Tilting Lock into account. Both the construction and the demolition phase of the Tilting Lock will have a significant contribution to the total energy consumption of the Tilting Lock. Regarding the energy consumption of the Tilting Lock, it was recommended to investigate the following aspects in future research:

- The energy consumption of the trench that is required to provide sufficient depth for the Tilting Lock.
- The optimal shape of the Tilting Lock regarding the energy consumption of a tilting motion and the depth that is required for the Tilting Lock.
- The total energy consumption of the life cycle of the Tilting Lock in relation to the energy consumption of a conventional shipping lock.
10.2 Conclusions

Shape of the Tilting Lock
As can be seen in figure 10.1, other shapes of the Tilting Lock are already studied in previous research [Witteveen and Wolfsen, 2015]. It was found that less circular hull shapes will require less depth. However, at the same time, the energy consumption to rotate the Tilting Lock will increase. The overall energy balance could be favourable in case of a less circular shape of the hull, as the excavation of the trench and the installation of the cofferdam require a lot of energy as well. For these reasons, the effect of different shapes of the Tilting Lock should be studied into more detail in future research.

Figure 10.1: Sketch of a different hull shape versus the common circular shape [Witteveen and Wolfsen, 2015].

Economic feasibility of the Tilting Lock at a case study location
To assess the economic feasibility of the Tilting Lock at the Haringvliet bridge, a cost estimation of the reviewed case study design was made. The benefits of the implementation of the Tilting Lock are not reviewed in this thesis, it is recommended to further investigate several aspects into more detail:

- The intensity and the height distribution of the vessels passing the Haringvliet bridge. This data can be used to find the optimum size for the Tilting Lock in relation to the dimensions and intensities of the vessels and to quantify the benefits of the Tilting Lock in comparison to other alternatives.

- Assess the added value of the Tilting Lock in relation to the potential alternatives for solving the problems of the interfering traffic flows at the Haringvliet bridge, like jacking up of the existing bridge or realising a new bridge connection.

- To increase the monetary benefits of the Tilting Lock, it was recommended to quantify the delays in travel time in the current situation with regular opening of the Haringvliet bridge. These delays should be included in official calculation models to determine the exact benefits of the Tilting Lock.

10.2 Conclusions
The objective of this thesis is to perform a feasibility study on the concept of the Tilting Lock. To elaborate on this objective, the main research question and several supplementary research questions were drawn in chapter 1.3. In the following paragraphs, these questions are answered.

10.2.1 Main research question
To fulfil the thesis objective the main research question to be answered is:

*What is the technical feasibility of the Tilting Lock with regard to the depth that is required for the implementation of the Tilting Lock at the Haringvliet bridge?*

From the performed case study in this thesis, it is concluded that the technical feasibility of the Tilting Lock at the Haringvliet bridge is not limited by the depth required for the placement of the Tilting Lock and the accompanying measures for the bridge. Based on the case study design of the Tilting Lock trench and the cofferdams foundation for the Haringvliet bridge piers, it is demonstrated that the implementation of the Tilting Lock in an existing situation will be possible.
Stability of subaqueous trench slopes

The analysis of the stability of the subaqueous slopes of the trench focuses on answering the question: "For what slope inclinations are the subaqueous slopes not expected to encounter instabilities?". This resulted in the following conclusions:

- Liquefaction of a soil layer in the subaqueous slopes can be a normative failure mode, due to the presence of permeable soil layers in the subsoil of the Haringvliet estuary.
- Breach flow failure of the subaqueous slopes of the trench for the Tilting Lock can be normative, due to the presence of multiple partly impermeable soil layers in the subsoil of the Haringvliet estuary.
- Based on rules of thumb for the failure modes on liquefaction and breach flow, it is concluded that the subaqueous slopes of the trench are not expected to encounter instabilities for a slope inclination of 1:5 (11.3°).
- The macro stability of the subaqueous slopes of the trench for the Tilting Lock subjected to loads by the Haringvliet bridge piers is not the normative failure mode for the trench slopes with an inclination of 1:5.
- No critical aspect for the feasibility of the Tilting Lock trench was found regarding the stability of the subaqueous slopes for the trench of the Tilting Lock.

Stability Haringvliet bridge piers under excavation

The study on the bearing capacity of the foundation of the Haringvliet bridge piers is performed to answer the following question: "Till which depth are excavations allowed in the vicinity of the piers of the Haringvliet bridge without additional measures to stabilise the bridge piers?". This resulted in the following conclusions:

- Excavation around the piers of the Haringvliet bridge without additional measures is allowed to a depth of -15 meter NAP, which is 7 meters below the initial bottom level of the Haringvliet.
- The allowable excavation depth of 7 meters is not sufficient to create a trench for the Tilting Lock with the required depth (38 meters) in the case study.
- The trench that can be constructed by open excavation without additional measures to stabilise the Haringvliet bridge piers, provides sufficient depth (24 meters) to a smaller sized Tilting Locks (diameter ≤ 17 meter, added air draft ≈ 5.8 meter).

Cofferdam foundation for stability of the Haringvliet bridge piers

As is concluded for the case study that the trench required for the Tilting Lock cannot be constructed without measures, additional measures are studied to stabilise the Haringvliet bridge piers. A cofferdam foundation for the Haringvliet bridge piers is studied to give answer to the following question: "Is a cofferdam a feasible measure for solving instability of the Haringvliet bridge piers in case of large excavations?". The elaboration resulted in the following conclusions:

- Based the basic calculations of the required section modulus for the retaining walls of the cofferdam, it is concluded that the cofferdam required to stabilise the Haringvliet bridge piers under the excavations is technically feasible.
- Combi wall elements which meet the requirements on the minimum required section modulus for the retaining walls (≈ 32.500 cm³/m) of the cofferdam are available.
- Pre-tensioned cables (‘tension rings’) around the cofferdam are the most favourable alternative to provide the required horizontal stabilisation of the retaining walls of the cofferdam, based on a qualitative elaboration.
- An open trench with a cofferdam foundation for the Haringvliet bridge piers made of combi walls is a technically feasible design for providing sufficient depth to larger sized Tilting Locks (diameter ≥ 17 meters, draft ≥ 20 meters).
10.2. Conclusions

Sedimentation in trench

To assess the sedimentation rate in the trench that is required for the Tilting Lock, a basic approach is used to answer the following question: "Till what extend is sedimentation in the trench for the Tilting Lock to be expected?" The following conclusions are drawn regarding the sedimentation in the Tilting Lock trench:

- The sedimentation rate in the trench for the Tilting Lock (≈ 65 mm/year) will in the order of ten times the sedimentation rate in the undisturbed areas of the Haringvliet (6.5 mm/year).
- The rise of the bed level in the trench for the Tilting Lock is expected to be 0.07 meter per year, based on the estimated sedimentation rate of 80 kg/(m$^2$.year) and an assumed density for the deposited soil of $\rho_s = 1200$ kg/m$^3$.
- It is concluded that maintenance on the depth of the trench for the Tilting Lock is required, although the amount of sedimentation in the trench will be limited for the case study location of the Haringvliet bridge in relation to other potential locations.
- The effects of the introduction of the Kierbesluit on the flow velocities, the import of sediment particles and the morphology were expected to be small in the Haringvliet estuary [Van Leeuwen et al., 2004]. Therefore, it is concluded that the effects of the Kierbesluit on the sedimentation rate in the Tilting Lock trench are small as well.

To study the morphological situation in the trench for the Tilting Lock, it is elaborated on the influences on the hydrodynamic situation. From the basic approach of the situation, the following conclusions were obtained:

- The ratio between the discharge through the trench for the Tilting Lock and the discharge through an identical, undisturbed section of the Haringvliet estuary is approximated between 0.5 and 0.9.
- The ratio between the local flow velocity in the trench for the Tilting Lock and flow velocity in an identical, undisturbed section of the Haringvliet estuary is approximated between 0.7 and 0.8.
- Both ratios depend on the amount of discharge through the Haringvliet estuary and the water level in the Haringvliet estuary.

Costs of implementation

The costs related to the construction of the case study design are estimated to answer the following question: "What are the costs related to the implementation of the Tilting Lock at the Haringvliet bridge?". This resulted in the following conclusions:

- The estimated construction costs for trench that is required to implement the Tilting Lock are approximated on €31.9 million.
- The total costs of implementing the Tilting Lock are estimated on €92 million.

Size Tilting Lock

To investigate the changes of the Tilting Lock at other locations, a study has been performed on the relation of the size of the Tilting Lock to answer the following question: "What is the relation between the size of the Tilting Lock, the tilting angle of the Tilting Lock and the added height by the Tilting Lock to the available head clearance for the passing vessels?" The performed analysis was based on either a constant radius of the Tilting Lock ($R = 28$ meter) or on a constant tilting angle ($\theta = 22^\circ$). For other values for these constants, other results will be obtained.

From the analysis the following conclusions were drawn:

Figure 10.2: Parameter designation of the Tilting Lock.
• The tilting angle of the Tilting Lock and the radius are the two most important parameters related to the size of the Tilting Lock and the amount of added air draft for the vessels.

• The optimum tilting angle $\theta$ of the Tilting Lock (with a radius of 28 meters) regarding the added air draft for the vessels is between $\approx 25^\circ$.

• The relation between obtained added air draft versus the radius of the Tilting Lock is $2 \cdot \delta h : R = 1:2.2$.

• The relation between the height of the Tilting Lock in the sections outside the bridge and the radius of the Tilting Lock is $K : R = 1:1.6$.

• The relation between the required head clearance underneath a bridge and the radius of the Tilting Lock is $T : R = 1:6.1$.

• The relation between the radius of the Tilting Lock and the required depth for the Tilting Lock is $R : U = 1:1.2$.

With the performed analysis on the size of the Tilting Lock, the following conclusions were drawn for the Tilting Lock at the case study at the Haringvliet bridge:

• To obtain the desired added air draft for the vessels of 12 meters ($2 \cdot \delta h$) for the case study, a larger sized Tilting Lock ($R \approx 31$ meters) is required than the applied size of the Tilting Lock in the case study.

• The maximum available head clearance underneath the Haringvliet bridge ($T \approx 13$ meters), allows for larger sized Tilting Locks (radius up to 38 meters) than applied in the case study.

• The implementation of a larger Tilting Lock than in the case study will lead to a significant larger required depth for the Tilting Lock trench $U$.

• The open trench (alternative 1, chapter 4) can provide for sufficient depth for a Tilting Lock with a radius $R$ of 17 meters ($2 \cdot \delta h = 5.8$ meters). Therefore, no measures will be required to stabilise the Haringvliet bridge piers for this size Tilting Lock.

Conclusions on the feasibility of the Tilting Lock at the Haringvliet bridge

In thesis is demonstrated that the implementation of the Tilting Lock at the Haringvliet bridge is technically feasible. However, the current problems at the Haringvliet bridge, due to the delays in travel time for both the road and marine traffic, are not considered to be sufficient to investigate potential improvements of the existing situation [Schultz van Haegen, 2016]. In addition, the Tilting Lock will not provide a complete conflict free traffic junction but will eliminate the daily opening of the Haringvliet bridge. After the implementation of the Tilting Lock, the movable bridge part still has to open to let very large vessels pass. Therefore, it was concluded that the benefits of the Tilting Lock are not yet sufficient to justify the costs of the implementation.
Bibliography


[Rijkswaterstaat, 2016b] Rijkswaterstaat (2016b). Waterstand t.o.v. NAP.


Abbreviations

CFD
Computational Fluid Dynamics

CPT
Cone Penetration Test

FEM
Finite Element Method

MCA
Multi-Criteria Analysis

MER
Milieu Effecten Rapportage (Environmental Effects Report)

MHWL
Mean High Water Level

MKBA
Maatschappelijke Kosten en Baten Analyse (Social costs and benefits analysis)

MLWL
Mean Low Water Level

MWL
Mean Water Level

NAP
Normaal Amsterdams Peil

RHDHV
Royal HaskoningDHV

RWS
Rijkswaterstaat (Department of public works)

SLS
Serviceability Limit State

UKC
Under Keel Clearance

ULS
Ultimate Limit State
Abbreviations
Appendix A

The Tilting Lock

A.1 Principle of operation of the Tilting Lock

Each picture of figure A.1 shows one phase of the locking sequence. To understand the order of appearance in channel B, one should read horizontal from upper left (1) to lower right (12). For the order of channel A, one should read horizontal from lower right (12) to upper left (1). The description below the sketches explains what is happening in each channel. Each time step in figure A.1 contains the following sketches: front view to see the status of the gates (upper left), lateral cross section to see the status of the presence of a vessel in the channels A and B (upper right) and a top view to see the status of the location of the vessels with reference to the channels (central).
APPENDIX A. THE TILTING LOCK

Channel A

12. Yacht is moored in the third section of the internal channels, while a second yacht is waiting in front of the Tilting Lock.

11. Yacht is leaving the Tilting Lock, while a second yacht is entering the Tilting Lock.

10. Yacht is moored in the first section of the internal channels.

9. Yacht is moored in the first section of the internal channels.

8. Yacht is sailing through the second section of the internal channels.

7. Yacht is moored in the third section of the internal channels.

6. Yacht is moored in the third section of the internal channels, while a second yacht is waiting in front of the Tilting Lock.

5. Yacht is leaving the Tilting Lock, while a second yacht is entering the Tilting Lock.

4. Yacht is moored in the first section of the internal channels.

3. Yacht is moored in the first section of the internal channels.

2. Yacht is sailing through the second section of the internal channels.

1. Yacht is moored in the third section of the internal channels.

Channel B

1. Yacht is waiting in front of the Tilting Lock.

2. Yacht is entering the Tilting Lock.

3. Yacht is moored in the first section of the internal channels.

4. Yacht is moored in the first section of the internal channels.

5. Yacht is sailing through the second section of the internal channels.

6. Yacht is moored in the third section of the internal channels.

7. Yacht is moored in the third section of the internal channels.

8. Yacht is leaving the Tilting Lock.

9. No yachts

10. No yachts

11. No yachts

12. No yachts
A.1. Principle of operation of the Tilting Lock

Figure A.1: Locking principle of the Tilting Lock.
APPENDIX A. THE TILTING LOCK

A.1.1 Fixating structure
To keep the Tilting Lock in position a fixating structure will be necessary. Without this fixating structure, the Tilting Lock would float away with the currents or the winds.

In the current design of the Tilting Lock, a tubular system is designed to keep the structure in position. It is assumed that the piles will be driven into the subsoil, comparable to a monopile of an offshore wind will. The piles will consist of multiple sections, which are removable from each other when required. By lifting the top part, the fixing structure could be removed. The first estimation of the diameter of the piles was 1 meter.

Requirements
The requirements for the fixing structure can be related to the Tilting Lock. These requirements are listed below.

- Horizontal movements of the Tilting Lock are allowed within \( \approx 0.5 \) – 1.0 meter tolerance.
- Vertical movements of the Tilting Lock should be allowed to follow the water level differences.
- The Tilting Lock will be fixated by two structures, one on both short sides of the lock.
- The Tilting Lock is infinity stiff in the longitudinal direction.
- Connection point between Tilting Lock and fixing structure will be in or nearby the pivot point of the Tilting Lock (-6 meter NAP).
- The driving mechanism is located at \( \approx +2 \) m NAP.

A.1.2 Loads
The forcing that the fixating structure of the Tilting Lock will need to resist can be divided into two categories: the forces that will directly work on the fixating structure and the forces that will act on the Tilting Lock, but have to be resisted by the fixating structure. It was expected that the forcing transferred from the Tilting Lock to the fixating structures will be more significant than the loads that are directly working on the fixing structure. As the main components of the loads will work either on the Tilting Lock or on the fixating structure (wind, waves and currents), the direct forcing on the fixating structure will be neglected in the first designs.

- Wind Force
- Current force
- Wave force
- Scour
- Ice pressure
- Temperature differences
- Ship collision

A.2 Potential locations for the Tilting Lock
To be able to perform a reliable feasibility study, a case study location will be introduced. Four locations in The Netherlands were discussed in this appendix, before one was selected as the case study location for this thesis.

A.2.1 Potential locations in the Netherlands
Four locations in the Netherlands are discussed in more detail in the following paragraphs, see figure A.2. In table A.1 and figure A.3 the characteristics of the four bridges are depicted. To be suitable for the Tilting Lock, a location has to meet some requirements, as discussed in chapter 2.2.2:

- Sufficient width of the water, to prevent that the Tilting Lock will act as an obstacle for marine traffic that does not need the Tilting Lock to pass the fixed bridge spans.
- The road on top of the bridge should be of sufficient national importance, to maximise the economic benefits of the realisation of the Tilting Lock.
- Regarding the local conditions, the preliminary water depth was considered to be relevant, because the impact of the draft of the Tilting Lock would be much larger in shallow waters.
- The presence of a movable bridge part was considered to be beneficial, but not critical for
the selection of a location. If a movable bridge part was absent, the economic benefit of the Tilting Lock was considered to be absent, as no traffic jams related to the opening of the bridge for passing yachts would occur.

![Map of possible locations for a Tilting Lock in the Netherlands](image)

**Figure A.2: Possible locations in the Netherlands for a Tilting Lock.**

**Ketel bridge**
The Ketel bridge was part of the national highway A6 between Lelystad and Urk since 1970. With the two driving lanes in both directions, the bridge has an average traffic intensity of 45,600 vehicles per day. On the eastern side, there was also a connection for the local traffic [Wegenwiki, 2015a]. In 2013, the movable parts in the southern parts of the Ketel bridge were renovated [Rijkswaterstaat, 2015]. Beneath the Ketel bridge a navigation channel is dredged, with a depth of 5.5 meters below MWL [Navionics Inc., 2015].

**Haringvliet bridge**
The Haringvliet bridge separates the estuaries of the Haringvliet and the Hollands Diep and has been built in 1964. This bridge was a part of the national highway A29 between Numansdorp and Willemstad and contains two driving lanes in each direction. Next to the highway there was a small lane for local traffic on top of the bridge. The bridge has 8 spans of approximately 106 meters, but the total length of the bridge was 1,220 meters. At the northern part of the Haringvliet bridge, a movable bridge span was located to let high yachts pass. On a daily base in 2010, 48,000 vehicles passed the Haringvliet bridge [Wegenwiki, 2016a]. The maximum water depth beneath the Haringvliet bridge was around 11 meters [Navionics Inc., 2015].

**Zeeland bridge**
The Zeeland bridge was built in 1965 with a span of 5.022 meter. As part of the secondary road N256 between Goes and Zierikzee, the bridge crosses the Eastern Scheldt with a total of 52 spans. In both directions, one driving lane was present with on the side a small bicycle lane. On the northern end of the bridge, a movable part was included to give passage to vessels with a large height. On a daily base, 12,200 vehicles crossed the bridge in 2014 [Wegenwiki, 2015d]. The governing maximum water depths are between 15 and 25 meters [Navionics Inc., 2015].

**Moerdijk bridges**
The Moerdijk bridges cross the Hollandsch Diep and provide for the connections between the provinces Zuid-Holland and Brabant. The bridge for road traffic was part of the national highway A16. Within 600 meters of the road traffic bridge, two railway bridges are located. In the current situation, these bridges are the most important connection between the two banks of the Hollands Diep. The bridges did not have a movable part, so no traffic jams were caused due to opening the bridges for passing vessels. On a daily base, 140,000 vehicles passed the Moerdijk bridge in 2011 [Wegenwiki, 2015b].
APPENDIX A. THE TILTING LOCK

<table>
<thead>
<tr>
<th>Ketel bridge</th>
<th>Zeeland bridge</th>
</tr>
</thead>
<tbody>
<tr>
<td>Total length: 800 meters</td>
<td>Total length: 5022 meters</td>
</tr>
<tr>
<td>Length span: 80 meters</td>
<td>Length span: 95 meters</td>
</tr>
<tr>
<td>Vertical clearance: 13 meters</td>
<td>Vertical clearance: 15 meters</td>
</tr>
<tr>
<td>Maximum water depth: 5.5 meters</td>
<td>Maximum water depth: 15 to 25 meters</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Haringvliet bridge</th>
<th>Moerdijk bridges</th>
</tr>
</thead>
<tbody>
<tr>
<td>Total length: 1220 meters</td>
<td>Total length: 1040 meters</td>
</tr>
<tr>
<td>Length span: 94 meters</td>
<td>Length span: 100 meters</td>
</tr>
<tr>
<td>Vertical clearance: 13 meters</td>
<td>Vertical clearance: 10.9 meters</td>
</tr>
<tr>
<td>Maximum water depth: 11 meters</td>
<td>Maximum water depth: 9 meters</td>
</tr>
</tbody>
</table>

Table A.1: Main characteristics bridges

Figure A.3: Potential locations of the Tilting Lock in the Netherlands

A.2.2 Selection of the case study location

In table A.2 the locations are reviewed on the requirements for the potential case study locations. From this table it was concluded that the Haringvliet bridge will be the most promising location for the Tilting Lock, based on the following observations:

- The available water depth at the Ketel bridge was considered to be too limited. In combination with the importance of the bridge in the traffic system, the Ketel bridge was not considered to be the best case study location.
- The Haringvliet bridge has a road connection of significant importance where road traffic experiences delays in travel time on a regular base. In addition, the first impression of the local conditions was the best in relation to the other potential locations for the Tilting Lock.
- The economic importance of the Zeeland bridge was considered too low, due to the secondary road that passes the bridge.
- The Moerdijk bridges are the most important bridges considered in this case study selection, but have a big disadvantage because of the 500 meter stretch between the car traffic and the railway bridges. In addition, both the Moerdijk bridges do not have any movable bridge.
A.3 Alternatives for the Tilting Lock

The main function of the Tilting Lock will be to create a conflict free traffic junction between road traffic and yachts with very high air drafts, which do not fit beneath the available bridge. For this classic traffic problem, several alternative solutions are already available and often applied. In the following paragraphs, these alternatives are discussed.

The alternative solutions are subdivided into three types, discussed from the perspective of the road traffic that is passing the marine traffic. In either situation, a maximum head clearance is present for one of the two traffic flows.

- Crossing the waterway overhead (bridge).
- Crossing the waterway underneath (tunnel/aqueduct).
- A detour around the location of the congestion.
- Innovative solutions, like the Tilting Lock, can improve the situation at locations where one of the above solutions is already present.

### A.3.1 Bridge

A bridge is a fixed or (partly) movable connection between two banks that are separated by for instance rivers, lakes, canyons or traffic lanes. In this thesis is focused on bridges over larger waters, like estuaries, rivers or lakes.

Each bridge has an available air draft for traffic that passes the bridge underneath. The available air draft for the traffic underneath can be increased, when the bridge contains a movable part. However, the passage of traffic over the bridge will be restricted during the times of opening. At locations where a bridge is already present and the conflict between the road traffic and the marine traffic increases in severity, four options are available:

- Construct a new bridge
- Increase the available air draft underneath the bridge by jacking up the bridge deck
- Implement an innovative solution (see appendix A.3.4).
- Remain in the existing situation

#### New bridge

By construction a new bridge with more air draft, larger ships will be able to cross the bridge without interrupting the traffic flow on top of the bridge. This solution requires a significant investment of money and would influence both road and marine traffic flows. Based on the construction costs of the reference bridges in table A.3, the costs of a new bridge connection are estimated on €200 million or more. The advantage of this solution is the opportunity to update both traffic lanes to new requirements, like an increased capacity of the bridge for the road traffic by adding more driving lanes. The disadvantage of a bridge is that the air draft of the fixed spans of a bridge will be limited.

#### Jacking up bridge

Jacking up is a method to increase the air draft underneath existing bridges. The bridge deck level is raised by hydraulic jacks and strutted by temporary struts. These temporary struts are later on replaced by permanent ones. Reference projects in the Netherlands on the jacking up of

<table>
<thead>
<tr>
<th>Width water</th>
<th>Road class</th>
<th>Max. depth water</th>
<th>Movable bridge part</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ketel bridge</td>
<td>800 meter</td>
<td>0</td>
<td>5.5 meter</td>
</tr>
<tr>
<td>Haringvliet bridge</td>
<td>1220 meter</td>
<td>+</td>
<td>11 meter</td>
</tr>
<tr>
<td>Zeeland bridge</td>
<td>5022 meter</td>
<td>-</td>
<td>15 to 25 meter</td>
</tr>
<tr>
<td>Moerdijk bridges</td>
<td>1040 meter</td>
<td>++</td>
<td>9 meter</td>
</tr>
</tbody>
</table>

Table A.2: Overview potential case study locations for the Tilting Lock
APPENDIX A. THE TILTING LOCK

<table>
<thead>
<tr>
<th>Length</th>
<th>Tacitus bridge</th>
<th>Pont du Normandie</th>
</tr>
</thead>
<tbody>
<tr>
<td>Available air draft</td>
<td>1055 meter</td>
<td>2143 meter</td>
</tr>
<tr>
<td>Construction costs</td>
<td>€268 million</td>
<td>€233 million</td>
</tr>
</tbody>
</table>

Table A.3: Reference projects regarding the construction costs of a new bridge over the Haringvliet [Wegenwiki, 2016b], [Wegenwiki, 2015c], [Le Havre Chamber of Commerce and Industry, 2008].

bridges showed that the increase in height is generally around 1 meter, see table A.4. The general consensus is that jacking up a bridge is done for relative short bridges, with a maximum length of 1 kilometer for the Tacitus bridge near Ewijk.

<table>
<thead>
<tr>
<th>Increased height</th>
<th>Galecopper bridge Utrecht¹</th>
<th>Botlek bridge Rotterdam²</th>
<th>Zuid-Willemsvaart bridge Veghel³</th>
<th>Tacitus bridge Ewijk⁴</th>
</tr>
</thead>
<tbody>
<tr>
<td>Construction year</td>
<td>0.7 to 0.9 meter</td>
<td>1 meter</td>
<td>1 meter</td>
<td>1 meter</td>
</tr>
<tr>
<td>Length bridge</td>
<td>327 meter</td>
<td>1971</td>
<td>2014</td>
<td>2015</td>
</tr>
<tr>
<td></td>
<td></td>
<td>505 meter</td>
<td>170 meter</td>
<td>1000 meter</td>
</tr>
<tr>
<td>Costs</td>
<td>4 Million Gulden</td>
<td>€ 3.3 Million</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table A.4: Figures jacking up of bridges.

A.3.2 Tunnel

Another alternative for the Tilting Lock is a tunnel for the road traffic. By constructing a tunnel, the road traffic would pass the marine traffic underneath, redeeming the limitation on the air draft for the marine traffic, but introducing a height limitation to the road traffic. In general, the road traffic has a smaller range in air draft, which makes it more convenient to apply a restricted height for passage for the road traffic. As the tunnel would be in the subsoil, the construction phase would not interfere with the existing bridge and the visual pollution would be limited. The costs of a tunnel connection would be relatively high, as can be concluded from the reference projects in table A.5.

When the connection between the A29 sections north and south of the Haringvliet is made by a tunnel, the Haringvliet bridge would no longer be a part of the highway traffic system. This would solve the economic problems of traffic jams on the national highway. In the situation with a new tunnel for the highway, the Haringvliet bridge could still be used for the local traffic.

<table>
<thead>
<tr>
<th>Western Scheldt tunnel⁵</th>
<th>Sluiskil tunnel⁶</th>
<th>Roertunnel⁷</th>
<th>Pannderdensch Canal tunnel⁸</th>
</tr>
</thead>
<tbody>
<tr>
<td>Length tunnel</td>
<td>6.600 meters</td>
<td>1145 meters</td>
<td>2450 meter</td>
</tr>
<tr>
<td>Depth</td>
<td>-60 meters NAP</td>
<td>Bored</td>
<td>-12 meters Immersion</td>
</tr>
<tr>
<td>Construction method</td>
<td>Bored</td>
<td>Bored</td>
<td>Bored</td>
</tr>
<tr>
<td>Costs</td>
<td>€663 Million</td>
<td>€160 Million</td>
<td>€125 Million</td>
</tr>
</tbody>
</table>

Table A.5: Figures Western Scheldt & Sluiskil & Roer & Pannderdensch Canal Tunnel

¹www.youtube.com & www.rijkswaterstaat.nl
²www.digibron.nl & www.digibron.nl
³www.rasenberginfra.nl & www.struktoninfraotechnieken.nl
⁴www.struktoninfraotechnieken.nl
⁵www.westerscheldetunnel.nl
⁶www.sluiskiltunnel.nl & www.westerscheldetunnel.nl/nl/sluiskiltunnel.html
⁷www.nl.wikipedia.org & www.wegenwiki.nl/Roertunnel
⁸www.nl.wikipedia.org
A.3.3 Detour

A detour is an option for traffic junctions where the passage of the road traffic will be restricted for longer periods and a second passage of the waterway is available in the vicinity of the existing bridge. In figure A.4 the detour at locks of Bruinisse is depicted. There are two roads over both gates of the shipping lock, which allow the road traffic to pass the lock independently of the locking operation.

A detour is satisfying for waterways with a relatively small width, as a secondary road connection can be constructed relatively easy and at low costs. For wider waterways, the lengths (and therefore the costs) of a second connection would be significantly higher.

A.3.4 Innovative alternatives

Besides the discussed traditional solutions to the traffic intersection problems, more innovative designs like the Tilting Lock are present. In general, these innovative solutions require an existing bridge connection for the road traffic, as the solutions do not necessarily provide for a road connection. Examples of such innovative ideas are depicted in figure A.5.

- Floating lock (figure A.5a).
- Falkirk Wheel (figure A.5b) [British Waterways, ].
- Dalmuir drop lock [Eadon Consulting, 2015].
- Circle bridge (figure A.5c).

![Figure A.5: Innovative alternatives for the Tilting Lock](Image)
A.3.5 Keep existing situation

If the existing bridge is kept in the original state, the quality of the junction will remain the same. No benefits will be gained, but investments are absent as well.

Vessels that have too much air draft and do not want to wait in front of a bridge, have several opportunities to lower their air draft. For instance taking in weight to increase their draft. Striking their masts or attach a mass to the top of the mast to let the ships pass the bridge in inclined position are other options. However, not all vessels are able to perform these actions, as not all masts will be able to be lowered or the required additional air draft is too much to be compensated by sailing under an angle.

A.3.6 Recapitulation alternatives

At locations where the required additional air draft is limited, the jacking-up of a bridge was considered to be the best solution. In relation to the Tilting Lock, this alternative has the advantage that the vessels would not have to wait before they can pass the bridge and therefore does not encounter delays in travel time. In addition, the costs of jacking up a bridge will be lower than the costs related to the Tilting Lock.

Both the alternatives of a new bridge and a tunnel applies for situations where the current situation is not able to meet the requirements for the near future, for instance related to traffic capacity or durability of the existing bridge. Constructing a new road traffic connection will be a costly operation.

A detour could be a satisfying solution for small stretches of water. As the Tilting Lock will be mostly interesting at locations where long spans are required, the detour was not considered to be a very competitive alternative for the Tilting Lock.

The Tilting Lock could be a satisfying solution for situations where large additional air drafts for the marine traffic are required. In addition, it is beneficial for the economic feasibility of the Tilting Lock when the existing bridge is in relatively good conditions and meets the requirements related to traffic capacity for the near future. In other cases, it was considered that it would be more satisfying to update the road connection with a new bridge or a tunnel connection, despite the bigger required investments.
Appendix B

Soil characteristics Haringvliet estuary

In the following figures the original Cone Penetration Test (CPT)'s for the construction of the Haringvliet bridge in 1960 are depicted. In figure B.1 the location of the CPT's in relation to the Haringvliet bridge are depicted.

The CPT’s are used to determine the bearing capacity of the Haringvliet bridge piers in chapter 6. It was chosen to only work with CPT 9, as this CPT was assessed as the least favourable regarding the bearing capacity around the foundation pile tip level (≈ −30 meters NAP).

Figure B.1: Location of the Cone Penetration Test (CPT) for the original design of the Haringvliet bridge pier foundations [Rijkswaterstaat - Directie bruggen, 1961].
Figure B.2: Subsoil conditions at the third pier of the Haringvliet bridge pier (CPT 9) [TNO and Geologische Dienst Nederland, 2016].
Figure B.3: Subsoil conditions at the third pier of the Haringvliet bridge pier (CPT 10) [TNO and Geologische Dienst Nederland, 2016].
Figure B.4: Subsoil conditions at the fourth pier of the Haringvliet bridge pier (CPT 11) [TNO and Geologische Dienst Nederland, 2016].
Figure B.5: Subsoil conditions at the fourth pier of the Haringvliet bridge pier (CPT 12) [TNO and Geologische Dienst Nederland, 2016].
Appendix C

Original design drawings of the Haringvliet bridge

In the following pages, the original design drawings of the Haringvliet bridge are included to underline the descriptives of chapter 3.3. The following subjects are depicted:

- Overview of the Haringvliet bridge (figures C.1 till C.2).
- The selected bridge gap (figure C.3).
- The Haringvliet bridge piers (figures C.4 till C.8).
- The foundation of the Haringvliet bridge piers (figures C.9 till C.11).
- The construction phasing of the Haringvliet bridge (figure C.12).

The global steps of the construction of the Haringvliet bridge were (figure C.12):

1. Installing sheet pile walls for building pier
2. Create a cofferdam
3. Installation the foundation piles
4. Pour underwater concrete
5. Construct the pier in the dry-building pit
6. Place the prefabricated deck
7. After the completion of the pier, the building pit is filled up with water and the sheet pile walls are cut off just above the underwater concrete.
Figure C.1: Areal overview of the Haringvliet bridge [Rijkswaterstaat - Directie bruggen, 1961].
(a) Side view of the Haringvliet bridge.

(b) Topview of the Haringvliet bridge, the hatched areas mark where bottom protection is present.

Figure C.2: Overviews of the Haringvliet bridge [Rijkswaterstaat - Directie bruggen, 1961].
APPENDIX C. ORIGINAL DESIGN DRAWINGS OF THE HARINGVLIET BRIDGE

Figure C.3: Overview of the selected bridge gap for the Tilting Lock [Rijkswaterstaat - Directie bruggen, 1961].

Figure C.4: General lateral cross section at the bridge piers [Rijkswaterstaat - Directie bruggen, 1961].
Figure C.5: Side view of the third Haringvliet bridge pier [Rijkswaterstaat - Directie bruggen, 1961].
Figure C.6: Front view of the third Haringvliet bridge pier [Rijkswaterstaat - Directie bruggen, 1961].
Figure C.7: Top view of the third Haringvliet bridge pier [Rijkswaterstaat - Directie bruggen, 1961].
Figure C.8: Front view of the temporary sheet pile walls required for the construction of the Haringvliet bridge [Rijkswaterstaat - Directie bruggen, 1961].
Figure C.9: Simplification of the front view of the pile foundation of the fourth Haringvliet bridge pier [Rijkswaterstaat - Directie bruggen, 1961].
APPENDIX C. ORIGINAL DESIGN DRAWINGS OF THE HARINGVLIEFT BRIDGE

Figure C.10: Top view of the pile foundation of the fourth Haringvliet bridge pier [Rijkswaterstaat - Directie bruggen, 1961].
Figure C.11: Specifications of the foundation piles [Rijkswaterstaat - Directie bruggen, 1961].
APPENDIX C. ORIGINAL DESIGN DRAWINGS OF THE HARINGVLIET BRIDGE

Figure C.12: Construction phasing bridge piers [Rijkswaterstaat - Directie bruggen, 1961].
Appendix D

Alternatives for providing the required depth

The following alternatives were considered to reach the depth that is required for the Tilting Lock. These very general alternatives are developed into more detail in this appendix. The objective of this chapter is to be able to evaluate all the alternatives and select one of the alternatives to elaborated on in more detail.

1. Open trench (D.1)
2. Pneumatic caisson (D.2)
3. Open trench with measures for the bridge piers (D.3)
4. Open building pit (D.4)

D.1 Open trench

The first alternative presented is a dredged trench. The main advantage of this trench is that the construction would be relatively easy and cheap, as the required depth could be reached by well-known dredging methods and equipment. The disadvantage of the open trench is the almost inevitable instability of the piers of the Haringvliet bridge due to the excavation around the foundation of the Haringvliet bridge piers. The first sketches of the open trench alternative are depicted in figure D.1. In appendix D.5.1 larger sketches are included, including a top- and a side view of the first alternative.

![Figure D.1: Sketches alternative 1: open trench.](image)

Another disadvantage of the trench is the potential sediment trap that the trench can be. As the surface of the lateral cross section of the Haringvliet increases, the flow velocities will decrease. This might lead to accretion in the trench and endanger the operation ability of the Tilting Lock.
APPENDIX D. ALTERNATIVES FOR PROVIDING THE REQUIRED DEPTH

on the long term.

As multiple subsoil layers have to be crossed reach to the required depth, the failure of some soil layers might be expected. As discussed in section B, several impermeable layers are present. Also was concluded/assumed that no sealed soil layers are available in the Haringvliet estuary and in combination with the small water surface inclination $i_w$, no groundwater flow or groundwater pressure will be present under the impermeable soil layers.

Main issues would be the reduced bearing capacity for the pile foundation and the increase of moments in the piles by the horizontal loadings. For the potential bridge pier instability, several measures are already developed. These measures are included in alternative 3, see appendix D.3.

Slope of the subaqueous trench

For the preliminary design of the trench a natural slope with an inclination of 1:3 is assumed (both sketches in figure D.1). However, this slope is rather steep for subaqueous conditions according to [Helbo, 1996]. In figure D.2 multiple slopes are depicted. From this figure, it is clear that a too flat slope would be undesired, as multiple bridge piers are crossed underneath. This crossing of bridge piers will be one of the main difficulties in the design of the trench.

If the stability of the slope is reviewed into more detail, different inclinations and layout possibilities can be considered, together with the possibility to decide whether slope strengthening measures are required or not. The stability of the slopes will also have an iterative impact on the bridge stability. The inclination of the slopes will have a significant effect on the morphological aspects, as the flow velocities depend on the cross sectional area used by the flow stream.

D.2 Pneumatic Caisson

The prefabricated method to reach the required depth is the application of a pneumatic caisson. With the immersion of pneumatic caisson into the subsoil, the vertical stability would be provided by the prefabricated walls. The main selling point of the pneumatic caissons is the use of space, which will be limited when compared to the open trench. The width of the required pneumatic caisson will be similar to the diameter of the Tilting Lock and the interference with the bridge is likely to be minimal or absent. In figure D.3 a preliminary sketch is included. In appendix D.5.1, larger figures are available, including the top- and side view of the third alternative.
Pneumatic caissons are used in multiple projects. For instance, the abutments of the sea lock in Ijmuiden will be constructed with the use of pneumatic caissons. Also, a part of the noord-zuidlijn in Amsterdam is executed with these kinds of caissons in the vicinity of buildings founded on (wooden) piles, to create the starting shaft for the Tunnel Boring Machine.

Existing pneumatic caissons are generally limited to dimensions of 60 x 60 meter\(^1\). For the considered size of the Tilting Lock (length \(\approx 30\) meter), this means that two caissons are required, which should be connected after immersion.

With the implementation of a prefabricated caisson, the risk of sedimentation of the caisson could be prevented by adding a sealing system, that closes off space beneath the floating Tilting Lock. The sealing should be able to adapt to the vertical movements of the lock with the fluctuating water levels. The sealing should not be completely water tight, as space beneath the Tilting Lock should be flushed now and then to refresh the water and get rid of the sediment particles that had the chance to enter the enclosed area. As long water does not enter the area beneath the Tilting Lock, sediment will not be able to enter the area underneath as well.

**Construction methods**

A segmented construction of the pneumatic caisson is required, as the required height of the caisson walls is too high in relation to the available head clearance underneath the Haringvliet bridge. First, the substructure will be prefabricated, sailed in position and lowered onto the bottom of the Haringvliet. Next, the side walls will be extended to a satisfying height before the whole pneumatic caisson could be immersed to the required depth.

Another negative aspect of the pneumatic caisson method is the high air pressure that is required to keep the working chamber free of water intrusion. When personal has to work in pressurised working chambers, strict legislation applies to the execution. In the caisson law is stated that the maximum allowable depth to do manual work in the working chamber is 35 meters below MWL (Winterkorn & Fang) & (Hof, 2006). At larger depths, one should work with automatised jets and suction pipes. This automatised equipment is not yet available but under research.

**Access trench**

A potential disadvantage of the pneumatic caisson is the accessibility for the Tilting Lock into the caisson. Some kind of access trench will be required to let the Tilting Lock sail into the pneumatic caisson. The access trench is temporary and therefore allowed to be accreted.

To sail the Tilting Lock into the pneumatic caisson, a movable wall in the pneumatic caisson is required. Such a removable wall is comparable to the bulkheads used for immersing tunnel elements. Similar bulkheads were already considered in other feasibility studies [Van Corven, 2015].

### D.3 Open trench with bridge pier measures

When the stability of the Haringvliet bridge piers are assessed and found not sufficient to allow for the required excavations related to an open trench, measures to increase the stability of the Haringvliet bridge piers might be satisfying.

#### D.3.1 Bridge pier measures

In the following paragraphs, several options to increase the stability of the Haringvliet bridge piers are listed.

**No measures (open excavation)**

The most simple variation would be in the case that the bridge piers does not need any additional measures. In this variation, the bridge piers stay stable during the execution and final phase, while

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\(^1\)According to J. Bogaards, Geotechnical consultant at Royal HaskoningDHV (RHDHV).
APPENDIX D. ALTERNATIVES FOR PROVIDING THE REQUIRED DEPTH

(a) Open excavation  (b) Strengthen pier foundation

(c) Strengthened slopes  (d) Single wall

Figure D.4: Overview alternative 3: bridge pier measures

the subsoil layers beneath the piers are being dredged. An impression of this variation is already depicted before in figure D.4a.

**Expansion of amount of foundation piles**

When the bridge pier is considered to be unstable, the foundation of the bridge pier could be extended. With longer and more foundation piles added to the existing foundation, the stability could be increased. To connect these piles to the existing foundation the underwater concrete floor should be extended, see figure D.4b. It is likely that additional (temporary) measurements are required to execute this alternative at a satisfying level. It should be questioned if the connection between the existing structure and the additional parts have sufficient strength to transfer the loads.

**Strengthen slopes**

To increase the stability of the bridge piers, it can be chosen to steepen the slope inclinations, so the foundation piles of the Haringvliet bridge are not necessarily excavated, as depicted in figure D.4c.

**Protection of bridge piers by walls**

Several wall types are available to increase the stability of the bridge pier foundation, an example is included in figure D.4d. For the layout of such wall, a huge amount of variations could be thought of.

**Fender system**

As a significant quantity of foundation piles is available beneath the bridge piers, it might be feasible to have a stable bridge during non-exceptional situations. This means that the bridge will not settle under normal conditions like wind, wave or current conditions, but collapse due to drifting ice or ship collisions. To provide for a safe design, a fender system can be designed that...
D.3. Open trench with bridge pier measures

prevents the occurrence of such collisions.

(Grout) injections
The subsoil beneath the concrete floor and between the foundation piles could be strengthened by for instance the injection of grout. Firstly, this will prevent the soil from washing away with the currents. Secondly, the structure will act as one solid piece, which will be better able to resist the increased moments due to the excavation. This grouted structure could be simplified to shallow foundation which is embedded in the slopes of the trench, see figure D.5. In this way, the character of the foundation is changed and should be approached differently.

Conclusion on measures for the Haringvliet bridge pier
Although multiple, feasible solutions are available to increase the stability of the Haringvliet bridge piers under the required excavations to construct the trench that is required for the Tilting Lock, it was chosen to only select one to continue the design. The retaining sheet pile walls were considered to be the most interesting to investigate in the remaining of this thesis.

D.3.2 Construction materials vertical walls
In the following paragraphs, multiple types of vertical walls that can be used to construct retaining walls around the Haringvliet bridge pier are listed. For this thesis, it was chosen to focus on the feasibility of sheet pile and combi wall elements for the retaining walls.

- The most common method of making a building pit is to drive sheet piles into the soil to withstand the external forces. However, the retaining height of sheet piles without vertical stabilisers is not that much. This retaining height could be increased by applying combi-walls instead of sheet piles. Normally retaining sheet piles could resist soil till a height of 6-7 meters without any anchoring on the top parts. Struts or anchors will be required for larger retaining heights. The execution of sheet pile walls is hard below the water surface, but not impossible.

- An often applied method for retaining walls in large excavations is the diaphragm wall. These concrete walls can reach to great depths and withstand quite some force. However, it is not possible to construct diaphragm walls underwater. With a temporary pivot dike or a building put to reach the surface of the bottom, the diaphragm walls could be installed.

- Another in-situ method of constructing is the pouring of underwater concrete. This is often applied to make concrete slabs as floors for building pits. If it is possible to submerge and place accurately the form-work, this method could also be used to pout vertical walls to retain the soil. However, the execution would be very complicated. Therefore, it is likely that it will be easier to apply prefab elements.

- Sandwich wall (Amsterdam central station within the noord-zuidlijn)
- Pile wall (driven piles / in situ piles)

D.3.3 Shapes retaining walls
Multiple shapes can be thought off regarding the structure constructed with the retaining sheet pile walls. From a single sheet pile wall, to a U-shaped alternative or a complete encircled Tilting Lock or bridge pier. This is tried to show in figure D.6.

The assessed alternative will be the square shape (figure D.6c), as this one is considered to be the
most promising regarding the feasibility of the cofferdam. This is also called a cofferdam. When it is proven that this alternative is sufficiently safe, other shapes could be considered.

![Sheet pile wall shape alternatives](image)

(a) Single sheet pile wall.  
(b) U-shaped sheet pile wall.  
(c) Square like encircled bridge piers.  
(d) Oval shaped encircled bridge piers.

Figure D.6: Sheet pile wall shape alternatives.

D.4 Open building pit

The fourth alternative to reach the required depth is by excavation in an open building pit. This open building pit could be made with several methods, which are briefly described below. The walls of the building pit need to resist the forcing by both water and soil level over relatively large height differences. The building pit alternative has the most potential alternatives, as many materials and layouts of building pits could be considered. In addition, the construction method, building phasing, permanent / temporary building and so on could differ as well. It is tried to make a comprehensible overview. The options will be listed and shortly discussed.

**Construction methods**

Limitations of applying sheet pile walls would be the large retaining height, which in several layouts will have a magnitude of 10 meters. As the sheet piles need to be placed below the water surface level, the applying of anchors is almost infeasible and should be prevented. The application of struts is not feasible as the Tilting Lock will need to be between both sheet pile walls. Additionally, the sheet pile walls have to be placed at water depths of 8 to 11 meters and at locations with a limited head clearance due to the presence of the bridge.

**Construction materials horizontal stabilisation of vertical walls**

The buildings pits can be stabilised by several methods. During the construction phase, the vertical strutting is no problem in general. During the final phase, no struts or similar should be in the way of the tilting lock. Therefore, the types of horizontal stabilisation methods for the final phase are limited.

- Struts
D.5. Evaluation of the MCA

In table D.1 the results of the Multi-Criteria Analysis (MCA) for the conceptual alternatives are listed. A more elaborated explanation of the scores can be found below.

Alternative 1: Trench alternative
The completeness of the solution of the trench is low for the case study location at the Haringvliet bridge and the desired size of the Tilting Lock. Multiple additional structures or measures are likely to be required to come to a satisfying design. The utilisation of space is low, as the area occupied by the trench is relatively large when compared to the other conceptual alternatives.
APPENDIX D. ALTERNATIVES FOR PROVIDING THE REQUIRED DEPTH

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<tr>
<td>Completeness of solution</td>
<td>–</td>
<td>+</td>
<td>++</td>
<td>+</td>
</tr>
<tr>
<td>Simplicity</td>
<td>++</td>
<td>–</td>
<td>0</td>
<td>-</td>
</tr>
<tr>
<td>Utilisation of space</td>
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<td>-</td>
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Table D.1: MCA design alternatives.

The trench will eventually fill up or create a new equilibrium state of morphology in the Haringvliet estuary. The demolition of the open trench will not require any action, as the trench will eventually fill up. Therefore, the life cycle analysis of the open trench is rated positively. The other way around, the maintenance was expected to be high, as the depth of the trench has to be maintained during the lifetime of the Tilting Lock.

The feasibility of the execution and the simplicity of the solution largely depend on the required additional measures, just like the costs. For only the trench, without measures, all these aspects would score very positive with respect to the other alternatives.

**Alternative 2: Pneumatic caisson**

The solution of the pneumatic caisson is quite complete, as it entails the stability of the bridge, the requirement of maintenance on sedimentation and creates easy opportunities for the fixating structure. The utilisation of space in this conceptual alternative is rather good, as the piers of the Haringvliet bridge are not affected and maintenance will hardly be required. However, the simplicity of this solution is rather low due to the complicated technology of excavation under high air pressure and the required measures for getting the Tilting Lock in place. After the lifetime of the Tilting Lock, it will be hard to remove the caisson, which results in a low score on the life cycle analysis. The costs for a pneumatic caisson are estimated to be significantly higher than for the other alternatives.

**Alternative 3: Open trench with bridge pier measures**

The open trench alternative scores very well on the completeness of the solution, as this is the extended version of alternative 1: the open trench. The lacking point in the first alternative (providing sufficient stability for the Haringvliet bridge piers), is corrected by the bridge pier measures. This decreases the simplicity of the solution and the feasibility of the execution.

The utilisation of space and the required maintenance of this alternatives are equal to these aspects in alternative 1. The life cycle of the bridge pier measures includes the removal of the installed structures, which also lead to additional costs. This resulted in lower scores on both aspects.

**Alternative 4: Open building pit**

The building pit alternative is comparable to the second alternative of a pneumatic caisson. Because the retaining walls of the open building pit can be constructed by more straightforward construction methods, the simplicity and feasibility of the execution are higher than for the second alternative.

The completeness of the solution, the utilisation of space and the maintenance are expected to be the same for the second and fourth alternative. For the life cycle analysis, the fourth alternative has a better grade, as it was expected that the sheet pile wall can be relatively easily removed compared to a pneumatic caisson. The costs of the building pit were expected to be lower than the pneumatic caisson, but higher than the bridge pier measures of alternative 3.
Conclusion MCA
From table D.1 was concluded that there is no clear best alternative. As the trench seemed to be the most logical alternative, this alternative was selected to elaborate into more detail. As it is likely that the piers require stabilising measures, it was decided to study the feasibility of a cofferdam as an extra foundation for the Haringvliet bridge piers.

Drawings alternatives open trench and pneumatic caisson

D.5.1 Alternative 1: Open trench

![Diagram of alternative 1: open trench](image1)

Figure D.9: Lateral cross section of alternative 1: open trench.

![Diagram of alternative 1: open trench](image2)

Figure D.10: Longitudinal cross section of alternative 1: open trench.
APPENDIX D. ALTERNATIVES FOR PROVIDING THE REQUIRED DEPTH

Figure D.11: Top view of alternative 1: open trench.

D.5.2 Alternative 3: Pneumatic caisson
D.5. Evaluation of the MCA

Figure D.12: Lateral cross section of alternative 3: pneumatic caisson.

Figure D.13: Longitudinal cross section of alternative 3: pneumatic caisson.
Figure D.14: Top view of alternative 3: pneumatic caisson.
Appendix E

Stability of the subaqueous trench slopes

In figure E.1 the slope excavations under different angles are depicted. For slopes with angels between 1:3 and 1:5 only one pier per side will be crossed underneath by the embankments. The required excavation depths at the piers for the different slope inclinations are included in table E.1.

![Figure E.1: Different slope inclinations of the subaqueous slopes for the trench of the Tilting Lock, drawn for the case study location at the Haringvliet bridge.](image)

<table>
<thead>
<tr>
<th>Slope</th>
<th>Excavation depths</th>
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<tbody>
<tr>
<td></td>
<td>First pier</td>
</tr>
<tr>
<td>1:3</td>
<td>14.45</td>
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<tr>
<td>1:4</td>
<td>18.34</td>
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<td>1:5</td>
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<td>1:7</td>
<td>23.34</td>
</tr>
<tr>
<td>1:8</td>
<td>24.17</td>
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</table>

Table E.1: Dimensions of excavations on the side of the bridge piers, excavation levels with respect to the current soil surface at -8 meter NAP
E.1 Subaqueous slope stability

Due to the presence of the Haringvliet bridge piers halfway the trench slopes, it was considered important to investigate the slope stability for the feasibility of the case study for the trench of the Tilting Lock at the Haringvliet bridge. Problems with the stability of the slopes were expected for the locations where the bridge piers are present.

Recently the stability of subaqueous slopes has been studied frequently, because of multiple failures or near failures of river banks and flood defences in The Netherlands. The interaction between flood defence failure and subaqueous flow slides was studied, but the effects of flow slides on dunes and structures have not been investigated yet [Van der Krogt, 2015]. The influences of structures on the stability of slopes in flood defences were studied [Y.R. Jongerius, 2016], although not for subaqueous conditions.

E.1.1 Failure modes of subaqueous slopes

In general, slopes fail due to the occurrence of sliding circles, which could be initiated by several causes as depicted in figure E.2. The presence of a structure was expected to primarily affect the macro stability of the slopes [Y.R. Jongerius, 2016]. In general, all flow slides need a trigger. In the case of the Haringvliet bridge, this trigger might be vibrations from the road traffic through the bridge pier, a ship collision with the Tilting Lock or the bridge pier or potential dangerous stages during the execution phase.

The failure modes of the subaqueous slopes are briefly discussed in the following paragraphs. The most critical failure modes are elaborated on in more detail in remaining of this chapter.

![Figure E.2: Failure modes subaqueous slopes (partly taken from Jonkman and Schweckendiek, 2015)](image)

**Macro instability**

The main failure mode of slopes is an imbalance in the equilibrium of moments, which leads to a slip circle in the slope. The imbalance in the equilibrium of driving and resisting moments is called macro instability. If the forces along a slip circle are not in equilibrium, a sliding circle will occur. The radius and the position of the center point of the slip circle are depending on the local conditions of the assessed slope. Multiple sliding circles should be checked to find the normative slip circle in a slope. These iterative calculations are performed often with software packages like D-stability.

Due to the presence of the bridge pier on the slope, additional driving or resisting moments are added to the equilibrium in comparison to the situation for slopes without structures.

**Liquefaction**

Liquefaction can be described as a soil layer of loosely packed sand that instantly liquefies, which results in quicksands [Van den Ham et al., 2012]. Loosely packed sand layers are likely to be present in areas where young marine sands have been deposited relatively fast and along (former) river beds where erosion and sedimentation of sand are continuously taking place [Helbo, 1996], like is the case in the Haringvliet estuary.

With the excavation of soil, the stress conditions in the soil layers can be changed in both horizontal and vertical direction. Due to the increase or decrease of shear stresses, small movements of the soil particles can occur as depicted in figure E.3. Whether the changes in shear stress cause loosening...
E.1. Subaqueous slope stability

or compacting of the soil layers, depends on the initial packing of the soil particles.

The changes in the density of the soil layers will cause an increase or decrease of the void ratios between the particles leading to over- or under pressure in the voids. A decrease of void ratio will lead to an increase of pore pressures, which might lead to static liquefaction. Liquefaction will occur when the original void ratio of the soil layer is above the dry critical void ratio [Helbo, 1996]. This implies that the effective stresses between the soil particles are decreased, due to the increased pore pressure. If the effective stresses are lower than and the shear stress, failure will occur. The complete liquefaction of a soil layer (when effective stress is completely absent) will hardly occur [Van den Ham et al., 2012].

If a soil layer is compacted due to the redistribution of the particles, liquefaction of the soil layer might occur as the increase in water pressures will decrease the effective stress between the soil particles. Liquefaction of a soil layer can cause a sudden sliding plane in a whole slope in a relative short period of time [T. Raaijmakers, 2005]. As the shear stresses in a soil layers increases for steeper and longer slopes, the probability of liquefaction of these slopes is larger than for milder and shorter slope [Helbo, 1996].

![Figure E.3: Schematising of (reversible) behaviour of densely and loosely packed grains subjected to shear stress](T. Raaijmakers, 2005)

**Breach flow**

When the changes in shear stresses cause the particles to become more loosely packed, as depicted in figure E.2, the void ratio between the particles will increase. The increase of the void ratio leads to a temporary vacuum between the particles, when the inflow of water is restricted. During the vacuum, the slope can be stable under steeper inclinations than for situations without the vacuum. After the voids are finally filled with water, the maximum slope inclination will be at its natural inclination.

When a slope is steepened during the presence of vacuum in the particle voids, for instance due to dredging or local scour around structures, a breach flow slide might be initiated. Due to the loss of the vacuum in the void ratios, particles start to rain down the slope, causing a density flume along the slope. This active failure mode propagates gradually, as the density flume might cause more erosion, and could eventually lead to a complete failure of a subaqueous slope. A breach flow slide is an uncontrolled variant of the known “breach process” in the dredging engineering, which is used by suction dredgers [Van den Ham et al., 2012].

**Piping**

Due to the in- or outflow of groundwater into the subaqueous slope, local instability of soil particles might be caused. The outflow of soil particles could be sufficient to trigger slope instability. Although the in- or outflow of groundwater is less expected for submerged slopes than for normal slopes above the water level, piping could still be a reason for instability.

For the case study location of the Haringvliet, it was found that the hydraulic head in the groundwater was lower in the subsequent soil layers with respect to the surface waters as discussed in
chapter 3.2.7. Therefore, it was not expected that piping would be a critical failure mechanism for the subaqueous slopes for the trench of the Tilting Lock.

**Erosion**

Like the in- or outflow of groundwater could lead to the outflow of soil particles from the bottom, the currents of the surface waters could lead to erosion of the bottom and thus the trigger for slope instability. The erosion or sedimentation of soil particles at the bottom influences the slope stability through changes in the geometric profile of the bottom slope [Van den Ham et al., 2012]. The influences of the morphological conditions in the trench of the Tilting Lock are discussed in chapter 8. For the stability of the subaqueous slopes, it was expected that the failure by erosion can be prevented by the application of bottom protection. Therefore, the failure mode by erosion was considered to be beyond the scope of this thesis.

**Swell of the bottom surface**

The subaqueous slopes for the trench of the Tilting Lock are created by excavation of the initial bottom of the Haringvliet estuary. The excavation of the trench is likely to induce swell of the subsequent soil layers. The stress redistribution due to the swell could lead to failure of the subaqueous slopes. However, swell of the deeper soil layers underneath the trench for the Tilting Lock was considered to be beyond the scope of this thesis.

**Conclusion failure modes**

From the analysis of the failure modes for subaqueous slope, it was concluded that the slope for the trench of the Tilting Lock should be checked on the sensitivity to liquefaction and the breach flow. When slope inclinations are found for which the sensitivities are sufficient satisfying, the macro stability of subaqueous slopes should be studied into more detail.

**E.2 Liquefaction**

**E.2.1 Relative density \( R_e \)**

To check whether liquefaction is a feasible failure mechanism, the three conditions as presented in the easy validation by the CUR 113 will be checked [CUR Bouw & Infra, 2008], see chapter 5.1.1. For the first condition, the relative density was required.

With the correlation depicted in equation E.1, a fair estimation of the relative density of the sand layers could be made [Van den Ham et al., 2012]. This relation between the cone resistance \( q_c \) and the relative density of sand \( R_e \) also includes some empirical constants, which differ per relation. The coefficients of (Baldi et al. 1982) are used [CUR Bouw & Infra, 2008].

The lower part of the slope gives the largest contribution to liquefaction till a depth of 1/2 the slope height beneath the toe. For effective vertical stresses between 20 kPa and 200 kPa, one can say that sand with \( R_e > 0.67 \) is not sensitive for liquefaction, while for \( R_e < 0.33 \) the sand layer will be highly sensitive for liquefaction [CUR Bouw & Infra, 2008].

In table E.2 the results for the situation under the third pier of the Haringvliet bridge (CPT 9) are depicted. As all the found relative densities are above 0.67, it was concluded that the soil layers were not sensitive to liquefaction in the first of three conditions.

\[
R_e = A \cdot \ln \left[ \frac{q_c}{B \cdot (\sigma^\prime_{v0})^C} \right] \quad (E.1)
\]
E.3 Macro stability in D-stability

In which:

\( R_e \) Relative density of sand
\( q_c \) Average cone resistance per meter in the CPT \( kN/m^2 \)
\( \sigma'_{vo} \) Effective vertical stress \( kN/m^2 \)
\( A \) Empirical constant of Baldi 0.4
\( B \) Empirical constant of Baldi 0.14
\( C \) Empirical constant of Baldi 0.6
\( \gamma_w \) Standardised volumetric weight of the soil layer \( kN/m^3 \)
\( \sigma_{v,soil} \) Weight of the soil layer \( kN/m^2 \)
\( p_{top} \) Water pressure on top of the soil layer \( kN/m^2 \)
\( p_{soil} \) Water pressure inside the soil layer \( kN/m^2 \)

Pier 3; CPT 9

<table>
<thead>
<tr>
<th>Soil layer</th>
<th>Top m NAP</th>
<th>Bottom m NAP</th>
<th>Height m</th>
<th>Average ( q_c ) ( kN/m^2 )</th>
<th>( \gamma_w )</th>
<th>( \sigma_{v,soil} ) ( kN/m^2 )</th>
<th>( p_{top} ) ( kN/m^2 )</th>
<th>( p_{soil} ) ( kN/m^2 )</th>
<th>( \sigma'_{vo} ) ( kN/m^2 )</th>
<th>( R_e )</th>
</tr>
</thead>
<tbody>
<tr>
<td>3.8</td>
<td>8</td>
<td>4.2</td>
<td>400.0</td>
<td>19</td>
<td>79.8</td>
<td>38</td>
<td>80</td>
<td>37.8</td>
<td>2.3</td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>8.5</td>
<td>0.5</td>
<td>320.0</td>
<td>20</td>
<td>10</td>
<td>38</td>
<td>85</td>
<td>42.8</td>
<td>2.2</td>
<td></td>
</tr>
<tr>
<td>8.5</td>
<td>19</td>
<td>10.5</td>
<td>200.0</td>
<td>18</td>
<td>189</td>
<td>38</td>
<td>190</td>
<td>126.8</td>
<td>1.7</td>
<td></td>
</tr>
<tr>
<td>19</td>
<td>26</td>
<td>7</td>
<td>880.0</td>
<td>20</td>
<td>140</td>
<td>38</td>
<td>260</td>
<td>196.8</td>
<td>2.2</td>
<td></td>
</tr>
<tr>
<td>26</td>
<td>26.5</td>
<td>0.5</td>
<td>600.0</td>
<td>20</td>
<td>10</td>
<td>38</td>
<td>265</td>
<td>201.8</td>
<td>2.1</td>
<td></td>
</tr>
<tr>
<td>26.5</td>
<td>31</td>
<td>4.5</td>
<td>1300.0</td>
<td>20</td>
<td>90</td>
<td>38</td>
<td>310</td>
<td>246.8</td>
<td>2.3</td>
<td></td>
</tr>
<tr>
<td>31</td>
<td>31.5</td>
<td>0.5</td>
<td>640.0</td>
<td>18</td>
<td>9</td>
<td>38</td>
<td>315</td>
<td>250.8</td>
<td>2.0</td>
<td></td>
</tr>
<tr>
<td>31.5</td>
<td>42</td>
<td>10.5</td>
<td>1600.0</td>
<td>20</td>
<td>210</td>
<td>38</td>
<td>420</td>
<td>355.8</td>
<td>2.3</td>
<td></td>
</tr>
</tbody>
</table>

Table E.2: Soil characteristics of pier 3 \([10 \, kg/cm^2 = 1 \, MPa]\)

Note: The situation for the areas where the bridge pier will act as a load on top of the slope surface is not taken into account yet. This load will increase the effective stresses in certain layers significantly. However, as the piles of the bridge pier transfer their pile tip force at a depth of -24 meter NAP, it is considered that these forces will only work in layers deeper than this. With only one normative layer beneath -24 meter NAP in CPT 9, this layer will be checked. As could be seen below, this still results in a safe situation.

\[
\sigma'_e = 315 + 1160 = 1475 kN/m^2 \quad \quad R_e = 0.4 \cdot \ln \left[ \frac{640}{0.14 \cdot 1475^{0.6}} \right] = 1.6 \quad (E.2)
\]

Note: The soil parameters are used as presented in the original CPT’s. However, due to the time these parameters have changed due to the erosion of several layers, see the section about bearing capacity. These changed parameters should be applied here as well. The reduction of the cone resistance is less (as this is a factor) than the reduction of the effective stress. Therefore it is likely that the change on liquefaction decreases as well, as this is depended on the cone resistance over the effective stress, see equation E.1.

E.3 Macro stability in D-stability

The stability calculations are made for CPT 9, which is one of the two CPT’s for pier 3. To simplify and the magnitude of the calculations, the CPT is summarised into a set of 9 or 10 layers, which are depicted in table E.3.

E.3.1 Input D-stability

In general, there are three options in D-stability to apply loads in the model. The first option is as a line load at a certain depth, the second option is a distributed load at the soil surface and the third option is no load at all. For the case of the Haringvliet bridge piers in the case study design, an uniform load \( (193.5 \, kN/m^2, \text{approximation from chapter 6.2}) \) is applied for both piers. To include the retaining character of the pile foundations, forbidden lines are applied at the boundaries of the bridge piers.
In figure E.4 the input of for the D-stability model is included. The input for the D-stability calculations summarised below. The loads on the slope by the bridge piers were applied as distributed load on the soil surface.

- Water level: 0 meter NAP
- Bottom levels: -8 to -38 meter NAP
- Slope inclination of 1:5
- Slope height of 30 meters
- Soil level according to the conditions of CPT 9 (see chapter 3.2.6)
- Uniform distributed loads: 193.5 kN/m² over 14.8 meters (see chapter 6.2)
- Forbidden lines at the locations of the bridge piers til a depth of -30 meters NAP

<table>
<thead>
<tr>
<th>Layer</th>
<th>Type soil</th>
<th>γsat</th>
<th>c’</th>
<th>φ</th>
<th>Coordinates top of the layer</th>
</tr>
</thead>
</table>
| 10    | Loose Sand        | 19   | 0  | 30.00 | X 0.00 50.00 200.00 250.00  
|       |                   |      |    |      | Y -38.00 -38.00 -8.00 -8.00 |
| 9     | sand silty        | 21   | 0  | 27.00 | X 0.00 50.00 199.30 250.00  
|       |                   |      |    |      | Y -38.00 -38.00 -8.14 -8.14  |
| 8     | clay very sandy    | 19   | 2  | 30.00 | X 0.00 50.00 193.80 250.00  
|       |                   |      |    |      | Y -38.00 -38.00 -9.24 -9.24  |
| 7     | clay slightly sandy| 18   | 10 | 22.50 | X 0.00 50.00 180.95 250.00  
|       |                   |      |    |      | Y -38.00 -38.00 -11.81 -11.81|
| 6     | clay very sandy    | 19   | 2  | 30.00 | X 0.00 50.00 172.70 250.00  
|       |                   |      |    |      | Y -38.00 -38.00 -13.46 -13.46|
| 5     | sand silty        | 21   | 0  | 27.00 | X 0.00 50.00 145.45 250.00  
|       |                   |      |    |      | Y -38.00 -38.00 -18.91 -18.91|
| 4     | Sand              | 20   | 0  | 32.50 | X 0.00 50.00 122.75 250.00  
|       |                   |      |    |      | Y -38.00 -38.00 -23.45 -23.45|
| 3     | Stiff Clay        | 19   | 3  | 20.00 | X 0.00 50.00 86.00 250.00   
|       |                   |      |    |      | Y -38.00 -38.00 -30.80 -30.80|
| 2     | Sand              | 20   | 0  | 32.50 | X 0.00 50.00 82.30 250.00   
|       |                   |      |    |      | Y -38.00 -38.00 -31.30 -31.30|
| 1     | Sand              | 20   | 0  | 32.50 | X 0.00 50.00 250.00          
|       |                   |      |    |      | Y -42.18 -42.18             |

Table E.3: Input soil layers D-stability CPT 9.

Figure E.4: Input D-stability CPT 9.

E.3.2 Limitations of D-stability
A limitation of D-stability is that it initially meant to evaluate the stability of dikes and embankments. In a situation of excavation other processes are present as well, like the redistribution of soil particles. D-stability is not able to take this redistribution into account. However, based on
expert judgement (Raymond van der Meij, contacted via the Deltares Helpdesk) the redistribution due to excavations will not have a significant influence on the stability safety factor \( F \) (see email conversation of 14-07-2016).

Another limitation of D-stability is the input of local loads on top of the slope. An uniform distributed load could only be applied at the soil surface. A line load could be applied in any arbitrary point, but is no correct or satisfying simplification of the loadings by the bridge pier. D-stability is not able to cope with 3D-effects, dynamical issues or the collapse of a structural element (like a sheet pile wall).

### E.3.3 Results D-stability calculations

In the following figures the results of the D-stability calculations are depicted. For two scenarios (see chapter 5.2.2) the stability was assessed of the subaqueous slopes of the trench that is required for the Tilting Lock. First, the overall stability of the trench slopes is reviewed and secondly a more detailed assessment of the Haringvliet bridge pier halfway down the trench slopes.

It was found in the general slope stability assessment that hardly any instabilities would occur at the general slope sections (where no top loads are present. Therefore, it was considered to be unnecessary to assess the slope stability of the trench sections that are not located underneath the Haringvliet bridge (thus no top loads by bridge piers).

#### General slope stability

In figure E.5 it can be seen that no instabilities are to be expected at the top of the subaqueous trench slopes. The green areas represent sliding circles with a safety factor over 1.5. Orange areas are safety factors between 1 and 1.5, while red areas represent safety factors lower than 1.0. The normative slip circle is depicted in figure E.6. This normative circle has a safety factor of 1.21 and a radius of 22.99 meter.

![Figure E.5: The overview of the safety factor for the macro stability of the subaqueous slopes of the trench for the Tilting Lock.](image)
Figure E.6: The normative slip circle for the macro stability of the subaqueous slopes of the trench for the Tilting Lock.

Slope stability at the Haringvliet bridge pier halfway the trench slope

As in figure E.5 was found that slope stability can occur at the Haringvliet bridge pier half way the trench slopes, this area was studied into more detail by refining the calculation grid. In figure E.7 the overview of the slope stability is depicted. The normative circle (figure E.8) has a safety factor 1.15 and a radius of 19.63 meter.

Figure E.7: The overview of the safety factor for the macro stability of the subaqueous slopes of the trench for the Tilting Lock.
Figure E.8: The normative slip circle for the macro stability of the subaqueous slopes of the trench for the Tilting Lock.
Appendix F

Stability of the Haringvliet bridge piers under excavation

To assess the stability of the foundation, the loads on the foundation need to be known. This appendix elaborates on the actual loads on the Haringvliet bridge piers and the normative conditions.

The loads as used in the original design calculations are depicted in figure F.1. In the initial design calculations, no safety factors are taken into account on the load parameters, but safety is included due to overestimating the self weight of the pier (load A). Therefore, it was considered to be necessary to assess the loads on the Haringvliet bridge piers into more detail and according to the current applicable standards.

(a) Load simplification

<table>
<thead>
<tr>
<th>Vertical loads</th>
<th>Pier 3</th>
<th>Pier 4</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>Self weight pier</td>
<td>3470 ton</td>
</tr>
<tr>
<td></td>
<td></td>
<td>4240 ton</td>
</tr>
<tr>
<td>B</td>
<td>Self weight deck</td>
<td>1222 ton</td>
</tr>
<tr>
<td>C</td>
<td>Distributed traffic load</td>
<td>1081 ton</td>
</tr>
<tr>
<td>D</td>
<td>Point load by traffic</td>
<td>60 ton</td>
</tr>
</tbody>
</table>

(b) Characteristic loads

<table>
<thead>
<tr>
<th>Horizontal loads</th>
<th>Pier 3</th>
<th>Pier 4</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>Braking force</td>
<td>5 ton</td>
</tr>
<tr>
<td>K</td>
<td>Ship collision</td>
<td>100 ton</td>
</tr>
<tr>
<td>L</td>
<td>Ice pressure</td>
<td>400 ton</td>
</tr>
</tbody>
</table>

Figure F.1: Loads on bridge piers 3 and 4 of the Haringvliet bridge. Pier 3 in tons (original) and pier 4 in kN (converted).

The scope of the research on the bridge pier stability is discussed in F.1. Subsequently the listed loads are elaborated into more detail in F.2. In F.2.9 the load conditions for an individual foundation pile under the Haringvliet bridge piers are determined.

- Load A: Self weight bridge pier (F.2.1)
- Load B: Weight prefabricated bridge deck (F.2.2)
- Loads C & D: Traffic loads (F.2.3)
- Load E: Wave loads (F.2.4)
- Load I: Horizontal loads from the bridge deck (F.2.5)
APPENDIX F. STABILITY OF THE HARINGVLIEI BRIDGE PIERS UNDER EXCAVATION

- Load K: Ship collision loads (F.2.6)
- Load L: Ice pressure loads (F.2.7)

F.1 Scope of the stability checks

For the assessment of the stability of the bridge pier, boundary conditions are set to test the results from the calculations. The boundary conditions are separated in several types and steps. First of all, there are the Ultimate Limit State (ULS) and the Serviceability Limit State (SLS). ULS is related to the safety of persons and the safety of the structures and covers the loss of stability or collapse due to extraordinary deformations. SLS is related to the functioning of the structures or structural elements under normal conditions.

The boundary conditions are based on rules of thumb by NEN 9997 and are elaborated on in section F.1.1. The load conditions in SLS and ULS differ due to the application of safety factors, which will be elucidated in section F.1.2. In F.1.3 the different load combinations used in the calculations are discussed.

The focus of this research is on the bearing capacity of the subsoil in relation to the working loads. The scope of the research is discussed into more detail in the following statements:

- The working loads on the bridge piers of the Haringvliet bridge were considered to be transferred to the subsoil through the foundation piles only.
- The connection between the foundation and the pier was considered to be rigid, as the bottom of the in-situ build pier is poured around the same foundation piles and thus will work as one piece.
- The redistribution of the bridge deck weight over multiple bridge piers, due to the settlement of a single bridge pier, was not taken into account.
- Deformations and damages in the bridge piers, the bridge pier foundation and the bridge deck due to (uneven) settlements of the bridge pier foundation were not taken into account.

F.1.1 Boundary conditions

The boundary conditions are the limits of loads or deformations, to which the structure should be tested to assess its safety. In general two states are assessed in the design of a structure: Serviceability Limit State (SLS) and Ultimate Limit State (ULS).

It was searched for the critical values of the bearing capacity of the pile foundation of the Haringvliet bridge ($R_{ck}$ in SLS and $R_{cd}$ in ULS), which were not exceeded by the loads on the individual piles of the foundation of the bridge piers, see equation F.1. The design values for the load and resistance are determined according to equations F.2 and F.3. The magnitude of the safety factors $\gamma$ depend on the specific situation and the material. These safety factors are elaborated on in appendix F.1.2.

\[
F_k \leq R_k & & F_d \leq R_d \\
F_d = \gamma \cdot F_{rep} & & F_{rep} = \psi \cdot F_k \\
R_d = \frac{R_k}{\gamma} & & R_k = \frac{R_{cal}}{\xi}
\]  

In which:

- $F_k$ Characteristic load (SLS) kN
- $F_d$ Design load (ULS) kN
- $F_{rep}$ Representative load kN
- $R_k$ Characteristic resistance (SLS) kN
- $R_d$ Design resistance (ULS) kN
- $R_{cal}$ Calculated resistance kN
- $\gamma$ safety factor (see appendix F.1.2)
- $\psi$ Load factor (assumed 1.0)
- $\xi$ Reduction factor for unfavourable permanent load
The main reason to review the stability of the piers was to prevent the Haringvliet bridge from becoming unserviceable. It was considered in this thesis that the serviceability of the bridge mainly depends on the stability of the bridge piers. When bridge piers are settling, the bridge deck was expected to deform accordingly to the settlements, which would induce a rotation of the bridge deck. The rules of thumb regarding the allowable rotation of the bridge deck are [Nederlands Normalisatie-instituut, 2012]:

- In ULS the allowable rotation of the bridge deck is $\beta = 1 : 100$
- In SLS the allowable rotation of the bridge deck is $\omega = 1 : 300$
- The structural bearings of the bridge deck allow for $\delta_{wh} \leq 0.05$ meter of horizontal deformations.

**Activation of the bearing capacity**

The required settlements to obtain the maximum bearing capacity differ for the positive shaft friction and the tip bearing capacity. For the shaft friction a deterministic value of 10 millimetre is common [Nederlands Normalisatie-instituut, 2012]. The required settlement for the maximum tip bearing capacity depends on the dimensions of the foundation piles. In the case of concrete, driven piles, line number 1 in figure F.2 should be used. According to the figure, the maximum pile tip resistance is activated by 10% of $D_{ef} = 40$ mm settlements of the pile relative to the bearing layers [Nederlands Normalisatie-instituut, 2012].

For the full activation of the bearing capacity, it is therefore required that the pile foundation of the Haringvliet bridge piers are allowed to settle the required 40 mm relative to the bearing soil layers. When these settlements are not allowed, a lower value for the bearing capacity ($R_k$) should be taken into account.

Figure F.2: Relations between the load on the pile tip ($R_{b,i}$), the shaft friction ($R_{s,i}$), the maximum loads ($R_{b,max,i}$ & $R_{s,max,i}$) in the ULS of SLS and the settlement of the pile tip ($s_{b,i}$) [Nederlands Normalisatie-instituut, 2012].

**Rotation of the bridge deck in the longitudinal cross section**

As could be seen in figure F.3, is the settlement of a single pier normative over the settlement of two piers for the assessment of the bridge deck rotation in the longitudinal direction.

**Rotation of the bridge deck in the lateral cross section**

In figure F.4 the uneven settlements in the lateral direction of the bridge deck are depicted. As the lateral direction is considered to be out of the scope, this will not be a normative boundary condition for this thesis.
APPENDIX F. STABILITY OF THE HARINGVLIET BRIDGE PIERS UNDER EXCAVATION

Figure F.3: Allowed settlements for piers in longitudinal direction of the bridge in SLS and ULS.

Figure F.4: Allowed settlements for piers in the lateral direction of the bridge in SLS and ULS.

Horizontal deformations at the spherical bearings

For horizontal deformations a deterministic value is used in the Dutch standards, which for SLS is set on $\delta w_h = 50$ millimetre [Nederlands Normalisatie-instituut, 2012]. This coincides with the allowable horizontal displacements of the bridge deck for the spherical bearings at the Haringvliet bridge [Civiele technieken De Boer bv, 2009].

The horizontal deformation in SLS for the top of the bridge pier will be $\delta w_h = 0.07$ meter according to the rules of thumb related to the rotation of the pier, see figure F.5. Therefore, it was concluded that the allowable rotation of the bridge piers in the lateral cross section is depending on the allowable horizontal deformation ($= 1 : 420$). Such a rotation of the foundation will occur if the settlements of the foundation piles differ more that 0.029 meter (29 mm) over the lateral cross section.

Figure F.5: Allowed horizontal displacement in SLS.
F.1. Scope of the stability checks

Conclusion boundary conditions
With respect to the deformations of the bridge deck, the maximum allowable settlements are 0.18 meter in SLS and 0.53 meter in ULS. As the required settlements of the pile foundation, for full activation of the bearing capacity of the subsoil, are \( \approx 40 \) mm, it was concluded that the complete bearing capacity of the subsoil can be taken into account.

The maximum allowable horizontal displacements at the top of the pier are limiting the rotation of the bridge pier in case of uneven settlements. For uneven settlements, for instance due to horizontal loads, these will be the normative boundary conditions.

Note: The calculations on the settlements that are to be expected were considered to be beyond the scope of this thesis. Besides the settlements that can be calculated (settlements of the pile tips, shortening of the foundation pile and settlement of the foundation layers), the settlements of the Haringvliet bridge piers due to the swell of the subsoil, related to the excavation of the trench that is required for the Tilting Lock, were considered to be out of the scope of this thesis.

F.1.2 Safety factors
To come to a safe design, safety factors will be applied in the calculations as discussed in F.1.1. Safety factors are applied on the loads (\( \gamma_G \) and \( \gamma_Q \)) and the resistances (\( \gamma_b \) and \( \gamma_s \)). The safety factor in SLS is 1.0 for all loads and resistances [Nederlands Normalisatie-instituut, 2012].

For ULS, the Dutch norm describes three types of design approaches, which are depicted below in tab F.1 [Nederlands Normalisatie-instituut, 2012]. In each combination, another set of safety factors is available for the loads (A), the resistances (R) and the soil parameters (M). Each component in a combination coincides with a set of safety factors that should be applied, which are included in the tables F.2 and F.3. The “+” means in combination with, so does not say anything about positive or negative values.

| Approach 1 | Combination 1: | A1 ”+” M1 ”+” R1 |
| Approach 2 | Combination 2: | A2 ”+” (M1 or M2) ”+” R4 |
| Approach 3 | Combination 3: | A1 ”+” M1 ”+” R2 |
| Approach 3 | Combination 4: | A1 ”+” M2 ”+” R3 |

Table F.1: Design approaches by the Dutch norm [Nederlands Normalisatie-instituut, 2012].

Note: According to the Dutch standards (NEN 9997), bridge piers normally are in the geotechnical category 2 (RC2) unless it is a very big or unusual structure [Nederlands Normalisatie-instituut, 2012]. The latter is the case for the Haringvliet bridge piers under the required excavations for the Tilting Lock trench. Therefore, the bridge piers are classified in category 3 (RC3).

Design loads
To include a margin in the allowable loads, safety factors should applied to increase the representative loads (\( F_{rep} \)) to the design loads (\( F_d \)) according to equation F.4 [Nederlands Normalisatie-instituut, 2012]. The safety factors are \( \gamma_G \) for permanent loads and \( \gamma_Q \) for temporary loads. The representative loads are found by multiplying the representative loads by the factor \( \psi \), see equation F.4 [Nederlands Normalisatie-instituut, 2012]. In this thesis \( \psi \) was assumed to be 1.0. The magnitude of the safety factors depends on the type and direction of the loads, see table F.2.

\[
F_d = (\gamma_G; \gamma_Q) \cdot F_{rep} \\
F_{rep} = \psi \cdot F_k
\] (F.4)
APPENDIX F. STABILITY OF THE HARINGVLIET BRIDGE PIERS UNDER EXCAVATION

<table>
<thead>
<tr>
<th>Load</th>
<th>Symbol</th>
<th>A1</th>
<th>A2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Permanent</td>
<td>Unfavourable</td>
<td>γG</td>
<td>1.2</td>
</tr>
<tr>
<td></td>
<td>Favourable</td>
<td>0.9</td>
<td>1.0</td>
</tr>
<tr>
<td>Temporary</td>
<td>Unfavourable</td>
<td>γQ</td>
<td>1.5</td>
</tr>
<tr>
<td></td>
<td>Favourable</td>
<td>0</td>
<td>0</td>
</tr>
</tbody>
</table>

Table F.2: Factors for loads [Nederlands Normalisatie-instituut, 2012]

Design resistances
The resistance of a foundation pile ($R_{c,d}$) depends in general on two main factors: the pile tip bearing design resistance ($R_{b,d}$) and the positive design shaft friction ($R_{s,d}$) along the pile (see equation F.5). These two design resistances should be determined from the characteristic resistances ($R_{b,k}$ & $R_{s,k}$) according to equation F.6 [Nederlands Normalisatie-instituut, 2012]. The characteristic values will be calculated with equation F.7 from the calculated resistances ($R_{b,cal}$ & $R_{s,cal}$) [Nederlands Normalisatie-instituut, 2012].

The values for $\gamma_b$ & $\gamma_s$ for driven concrete piles are depicted in table F.3. For rigid structures, two available CPT’s and the minimum calculated resistance, a factor of $\xi_3 = 1.20$ should be applied [Nederlands Normalisatie-instituut, 2012]. When more CPT’s are available, the $\xi$’s will reduce. Therefore, the bearing capacity that can be charged will be increased for more site investigations.

\[
R_{c,d} = R_{b,d} + R_{s,d} \quad \text{(F.5)}
\]
\[
R_{b,d} = \frac{R_{b,k}}{\gamma_b} \quad \text{and} \quad R_{s,d} = \frac{R_{s,k}}{\gamma_s} \quad \text{(F.6)}
\]
\[
R_{b,k} = \frac{R_{b,cal}}{\xi_3} \quad \text{and} \quad R_{s,k} = \frac{R_{s,cal}}{\xi_3} \quad \text{(F.7)}
\]

Table F.3: Factors for resistance parameters ($\gamma_R$) for driven piles [Nederlands Normalisatie-instituut, 2012]

<table>
<thead>
<tr>
<th>Resistance</th>
<th>Symbol</th>
<th>Combination</th>
<th>R1</th>
<th>R2</th>
<th>R3</th>
<th>R4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tip</td>
<td>$\gamma_b$</td>
<td>-</td>
<td>-</td>
<td>1.2</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>Shaft</td>
<td>$\gamma_s$</td>
<td>-</td>
<td>-</td>
<td>1.2</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>Total/combined</td>
<td>$\gamma_t$</td>
<td>-</td>
<td>1.2</td>
<td>-</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>Shaft (negative)</td>
<td>$\gamma_{s,t}$</td>
<td>-</td>
<td>1.35</td>
<td>-</td>
<td>-</td>
<td></td>
</tr>
</tbody>
</table>

Soil parameters
No safety factors are applied on the soil parameters, because the soil characteristics were already conservative simplified for the Haringvliet area. Using safety factors on top was considered to be too conservative.

Conclusion safety factors
In conclusion can be said that the normative combination for the maximum compression will be obtained by applying load combination 4 (table F.1). The largest safety is included in this combination, as the magnitude of the factors related to A1 and R3 are the largest. Therefore, the safety factors according to the fourth combination will be used in this thesis.

F.1.3 Load combinations
The applied load combinations for the assessment of the stability of the Haringvliet bridge piers were subdivided into two parts: compression and tension in the pile foundation.
F.2. Bridge pier loadings

Tension in the foundation
The most unfavourable combination regarding potential floating up of the bridge pier structure (tension in the foundation piles), includes the unfavourable working self weight loads (Loads A and B) and favourable working horizontal loads. The horizontal loads from the bridge deck (Load I) were considered to be permanent, the loads by waves (Load E) or ship collision (Load K) were not. Therefore, the load combinations related to tension in the foundation were:

• ABIE (ULS)
• ABIK (ULS)

Compression in the foundation
The most unfavourable combinations regarding settlements of the bridge piers (compression in the foundation piles), includes the favourable working self weight loads (Loads A and B) and favourable working horizontal loads. The temporary loads by the traffic (Loads C and D) were taken into account in these combinations, as they induce additional compression in the foundation. The horizontal loads are included in the load combinations as well. Therefore, the load combinations related to compression in the foundation were:

• ABCDIE (SLS and ULS)
• ABCDIK (ULS)

F.2.1 Load A: Self weight bridge pier
The self weight of the bridge pier is subdivided into three parts, for which the dimensions are determined according to the discussed geometry of the Haringvliet bridge piers (appendix C).

• Upper structure of the Haringvliet bridge piers ($\approx 1182 \ m^3$, $\rho_c = 2400 \ kg/m^3$)
• Under water concrete ($\approx 1080 \ m^3$, $\rho_{uwcd} = 2300 \ kg/m^3$)
• Foundation piles ($\approx 590 \ m^3$, $\rho_c = 2400 \ kg/m^3$)

In excel a calculation is made to check the original calculation of the self weight and to include the missing parts. In table F.4 the overview of the representative loads are given, together with the design forces for the upward and downwards favourable situations in the piles.

As the structure is partly submerged an upward water pressure is present. The most unfavourable situation is taken into account, which coincides with a water level of -0.4 meter NAP. The most unfavourable situation in that case would be the water level of +2.6 meter NAP.

Note: In the calculation of the self weight of the pier the sheet pile walls are not taken into account, as is assumed that the sheet pile walls are an independent element, which stability does not depend on the stability of the bridge pier.

F.2.2 Load B: Weight prefabricated bridge deck
For the weight of the prefabricated bridge deck, the value of the initial design calculation (1222 tonnes $\approx 12.047 \ kN$) is used as the characteristic load.

Note: In the initial calculations the environmental loads, like current pressures, wind and snow loads, were not taken into account. These environmental loads are also considered to be out of the scope in this thesis.

Note: As the trench slope will be inclined, the soil will cause a horizontal load onto the foundation piles. The horizontal loading due to the inclined excavations is considered to be out of the scope of this thesis.
APPENDIX F. STABILITY OF THE HARINGVLIET BRIDGE PIERS UNDER EXCAVATION

<table>
<thead>
<tr>
<th>Source of load</th>
<th>Downward</th>
<th>Upward</th>
</tr>
</thead>
<tbody>
<tr>
<td>Upper structure</td>
<td>26.675</td>
<td>32.010</td>
</tr>
<tr>
<td>Underwater concrete</td>
<td>24.315</td>
<td>29.178</td>
</tr>
<tr>
<td>Foundation piles</td>
<td>13.959</td>
<td>16.751</td>
</tr>
<tr>
<td>Water pressure (-0.4 m NAP)</td>
<td>-18.403</td>
<td>-16.563</td>
</tr>
<tr>
<td>Water pressure (+2.6 m NAP)</td>
<td>-21.021</td>
<td>-25.255</td>
</tr>
<tr>
<td>Total</td>
<td>61.377</td>
<td>33.230</td>
</tr>
</tbody>
</table>

Table F.4: Self weight of the bridge pier (load A) in [kN].

F.2.3 Loads C and D: Traffic loads

Over the years the rules for determining the representative loads of the traffic on a bridge have changed. The Dutch standards (NEN-EN 1991-2+C1) are applied to calculate the currents loads by traffic [Normcommissie "Technische Grondslagen voor Bouwconstructies", 2015]. The used type of load model is applicable for bridge spans over less than 200 meters and therefore applicable for the Haringvliet bridge (span of 106 meters).

The load model consists of two parts: a concentrated double axle load \( Q_k \) and an equally distributed load \( q_k \), both multiplied by their safety factor \( \alpha_Q \) and \( \alpha_q \), see figure F.6. The magnitude of the loads are depending on the classification of the theoretical driving lanes, see table F.5. Therefore, the road is divided in theoretical driving lanes according to the rules as presented in the Dutch standards, with \( w \) as the width of the road, measured from kerb to kerb.

Each theoretical driving lanes should be classified 1 to 4. The most unfavourable driving line is assigned as 1, the next 2, and the third most unfavourable lanes gets 3. The remaining lanes will be assigned as 4. As the Haringvliet bridge consists of only one deck, which contains all the lanes, each number is allowed to occur only once, except for 4. The numbers and the loads should be assigned such, that the load combination is the most unfavourable, which is depicted in figure F.7. The representative and design vertical traffic loads (Loads C and D) at one bridge pier are calculated in table F.6.

Note: For the determination of the safety factors is assumed that the number of trucks passing the Haringvliet bridge each year, is above 2,000,000, which is the most severe condition. The safety factors \( \alpha_{q1}, \alpha_{q2} \) and \( \alpha_Q \) for bridges with more than two driving lanes are used.

<table>
<thead>
<tr>
<th>Theoretics</th>
<th>Width road ( w )</th>
<th>No. theoretical lanes</th>
<th>Width theoretical lane</th>
<th>Remaining width ( w_r )</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>( w &lt; 5.4 ) m</td>
<td>( n = 1 )</td>
<td>( w_l = 3 ) m</td>
<td>( w_r = w - 3 ) m</td>
</tr>
<tr>
<td></td>
<td>( 5.4 ) m &lt; ( w ) &lt; ( 6 ) m</td>
<td>( n = 2 )</td>
<td>( w_l = w/2 ) m</td>
<td>( w_r = 0 )</td>
</tr>
<tr>
<td></td>
<td>( 6 ) m &lt; ( w )</td>
<td>( n = w/3 )</td>
<td>( w_l = 3 ) m</td>
<td>( w_r = w - 3 \cdot n )</td>
</tr>
<tr>
<td>Haringvliet bridge</td>
<td>( w = 8.25 ) m</td>
<td>( n = 3 )</td>
<td>( w_l = 3 ) m</td>
<td>( w_r = 2.25 ) m</td>
</tr>
<tr>
<td></td>
<td>( w = 5.5 ) m</td>
<td>( n = 2 )</td>
<td>( w_l = 2.75 ) m</td>
<td>( w_r = 0 )</td>
</tr>
</tbody>
</table>

Table F.5: Number of and width \( (w) \) of the theoretical driving lanes. [Normcommissie "Technische Grondslagen voor Bouwconstructies", 2015].
F.2. Bridge pier loadings

Figure F.7: Lateral cross section of the Haringvliet bridge, with the original and theoretical driving lanes [Rijkswaterstaat - Directie bruggen, 1961]

Point load (load D)  \[ 2 \cdot \alpha Q_i \cdot Q_i \cdot w_i \cdot length \]

\[ \begin{array}{c|c|c|c}
\hline
\text{Point} & \text{Distributed} & \text{Combined} \\
\text{load (load D)} & Q_i \cdot \alpha q_i \cdot w_i \cdot length & \\
\hline
1 & 3 \text{ m} & 2 \cdot 1 \cdot 300 = 600 \text{kN} & 9 \cdot 1.15 \cdot 3 \cdot 106 = 3291 \text{kN} & 3891 \text{kN} \\
2 & 3 \text{ m} & 2 \cdot 1 \cdot 200 = 400 \text{kN} & 2.5 \cdot 1.40 \cdot 3 \cdot 106 = 1113 \text{kN} & 1513 \text{kN} \\
3 & 3 \text{ m} & 2 \cdot 1 \cdot 100 = 200 \text{kN} & 2.5 \cdot 1.40 \cdot 3 \cdot 106 = 1113 \text{kN} & 1313 \text{kN} \\
4 & 13 \text{ m} & 0 \text{kN} & 2.5 \cdot 1.4 \cdot 13 \cdot 106 = 4823 \text{kN} & 4823 \text{kN} \\
R & 3.5 \text{ m} & 0 \text{kN} & 2.5 \cdot 1.4 \cdot 3.5 \cdot 106 = 1299 \text{kN} & 1299 \text{kN} \\
\hline
\end{array} \]

\[ F_{\text{rep}} = 1200 \text{kN} \]

\[ \text{Upward } F_D = 1080 \text{kN} \]

\[ \text{Downward } F_D = 1800 \text{kN} \]

Table F.6: Vertical characteristic and design traffic loads transferred on one bridge pier.

F.2.4 Load E: Waves

Since the Haringvliet estuary is closed off from the sea, no swell waves will be able to enter the area. The waves in the Haringvliet will mainly be generated by the wind or by passing by vessels, see table F.7. The actual wave height was estimated by the formula of Young and Verhagen [Young and Verhagen, 1996]. The working height of the wave loads was assumed to be below the water surface (-3 meter NAP), as the impact force will be the largest at that point.

\[
\begin{align*}
H &= 4 \cdot \sqrt{E} \\
\epsilon &= \frac{g^2 \cdot E}{u_{10}^2} \\
\delta &= \frac{g \cdot d}{u_{10}^2} \\
\chi &= \frac{g \cdot F}{u_{10}^2} \\
\epsilon &= 3.64 \cdot 10^{-3} \cdot \left\{ \tanh(A_1) \cdot \tanh\left( \frac{B_1}{\tanh(A_1)} \right) \right\}^{1.74}
\end{align*}
\]

(F.8)

(F.9)

The formula’s by Young and Verhagen are included in equations F.8 to F.10. The normative fetch is measured from the southern bank of the Haringvliet to the Haringvliet bridge and estimated on 11 kilometers. The found wave height is \( H = 1.51 \text{ meter} \). It was concluded that the waves generated by wind are normative for the case study.

The upper boundary value for forcing by waves on the Haringvliet bridge piers is based on the rule of thumb for wave forcing as depicted in equation F.11 [Vrijling et al., 2016]. With the assumed wave height of 1.51 meters, the maximum force by waves is estimated on \( F_{\text{max}} = 120 \text{kN/m} \). Over the width of the Haringvliet bridge pier (12 meters) this results in a total load of 1560 kN, which was considered to be the normative value for the wave loads (Load E) in this case study.

\[
\text{Table F.7: Rules of thumb for waves in the Netherlands (CUR 197; Breuksteen in de praktijk) cited from [Schiereck and Verhagen, 2012].}
\]

<table>
<thead>
<tr>
<th></th>
<th>Wind waves</th>
<th>Ship waves</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lakes</td>
<td>0.25 - 1.0</td>
<td>0.10 - 0.50</td>
</tr>
<tr>
<td>Canals</td>
<td>0.10 - 0.25</td>
<td>0.25 - 0.75</td>
</tr>
<tr>
<td>Rivers</td>
<td>0.25 - 1.0</td>
<td>0.25 - 0.75</td>
</tr>
</tbody>
</table>

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APPENDIX F. STABILITY OF THE HARINGAVLIET BRIDGE PIERS UNDER EXCAVATION

\[ A_1 = 0.493 \cdot \delta^{0.75} \]

\[ B_1 = 3.13 \cdot 10^{-3} \cdot \chi^{0.57} \]  

(F.10)

\[ F_{\text{max}} = \frac{1}{2} \cdot \rho \cdot g \cdot H^2 + d \cdot \rho \cdot g \cdot H \]  

(F.11)

In which:
- \( H \): Wave height
- \( E \): Total wave energy
- \( \epsilon \): Non-dimensional energy
- \( d \): Water depth
- \( \delta \): Non-dimensional depth

\( A_1 = 0.07181 \)

\( B_1 = 0.04452 \)

\[ F_{\text{max}} = 129 \cdot 10^3 \]  

\( N/m \)

**F.2.5 Load I: Loads from the bridge deck**

The bridge pier is subjected to several horizontal loads. One of these loads are the ones induced by the bridge deck. These loads consist of a component of the braking forces of the traffic and a component of the deformations due to the differences in temperature of the bridge deck. Both these forces are transferred from the bridge towards the bridge pier by bearings. So if the maximum allowable horizontal force of the bearings is known, the information will be sufficient to test the pier conditions in a satisfying way.

Although the horizontal longitudinal deformation is allowed by the spherical bearings, some force will be present due to friction. This friction coefficient can be estimated by equation F.12. The friction factor \( f \) in the spherical bearings is assumed to be around 0.025 (2.5%). For maximum loading the bearings bear 33.715 kN, which results in a factor (0.025), thus 843 kN horizontal loadings. See NEN 1337-2, table 11.

\[ H = f \cdot V \]

\[ H = 0.025 \cdot 33.715 kN \approx 850 kN \]  

(F.12)

In which:
- \( H \): Horizontal force \( kN \)
- \( V \): Vertical force \( kN \)
- \( f \): Friction factor

**F.2.6 Load K: Ship collision**

The ship collision force at inland waterways is depending on the ship class that is allowed in the shipping lane. Per class indicative values for the impact force on the bridge pier in longitudinal \( (F_{dx}) \) and lateral direction \( (F_{dy}) \) are given [Nederlands Normalisatie-instituut, 2012], see table F.8 The assumed working height of the ship collision force is at 1.5 meter above the normative water level [Nederlands Normalisatie-instituut, 2012] (+2.6 meter NAP + 1.5 meter = +4.1 meter NAP).

In the Hollandsch Diep, the CEMT class VIc is allowed to use the eastern locks of the Volkerak lock system and class Va is allowed to use the western locks, while for the Haringvliet the CEMT class VIa is applicable [Rijkswaterstaat, 2009]. It was decided to work with the shipping class of the Haringvliet.

For the assessment of the pier stability in the longitudinal direction relative to the bridge deck, only the impact force \( F_{dy} \) should be taken into account. The other loads two are directed perpendicular to the bridge deck and therefore they do not work in the direction of the reviewed plane, see figure.
F.2. Bridge pier loadings

F.8. Hollandsch Diep

<table>
<thead>
<tr>
<th>Va</th>
<th>&quot;Large Rhine&quot;</th>
<th>$F_{dx}$</th>
<th>$F_{dy}$</th>
<th>$F_r$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Vlc</td>
<td>2<em>3 or 3</em>2 convoy</td>
<td>8000 kN</td>
<td>3500 kN</td>
<td>1400 kN</td>
</tr>
<tr>
<td></td>
<td></td>
<td>17000 kN</td>
<td>8000 kN</td>
<td>3200 kN</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Haringvliet</th>
</tr>
</thead>
<tbody>
<tr>
<td>Vla</td>
</tr>
<tr>
<td>2*1 convoy</td>
</tr>
</tbody>
</table>

Table F.8: CEMT classes shipping lanes and accompanying indicative values for the impact forces

Note: Whether ship collision might take place, depends on the presence of protection measures (like fenders or dolphins) to prevent the ships from coming near the bridge pier.

F.2.7 Load L: Ice pressure

When the ice pressures as used in the initial design calculations are compared to the ship collision loads, it was concluded that the shiploads are of the same magnitude, but the shiploads are more acting at a higher level and therefore cause a large moment. In the Dutch standards, the presence of ice loading is mentioned, but no guidelines to quantify these loads are given [Nederlands Normalisatie-instituut, 2012]. Therefore the ice pressures were considered to be beyond the scope of this thesis.

F.2.8 Overview loads

The overview of the found characteristic and design loads on the fourth pier of the Haringvliet bridge are given in Table F.9. The loads for the third bridge pier will be slightly lower, due to the lower vertical position of the bridge deck, which leads to some less self weight of the upper structure and smaller momentum arm. For this thesis these differences were considered to be insignificant.

<table>
<thead>
<tr>
<th>SLS</th>
<th>Compression</th>
<th>Tension</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$\gamma G/\gamma Q$</td>
<td>$F_d$</td>
</tr>
<tr>
<td>A Total weight pier</td>
<td>46.546</td>
<td>-</td>
</tr>
<tr>
<td>Upper structure</td>
<td>26.675</td>
<td>1.2</td>
</tr>
<tr>
<td>Underwater concrete</td>
<td>24.315</td>
<td>1.2</td>
</tr>
<tr>
<td>Foundation piles</td>
<td>13.959</td>
<td>1.2</td>
</tr>
<tr>
<td>Upward water pressure (0.4 m NAP)</td>
<td>-18.403</td>
<td>0.9</td>
</tr>
<tr>
<td>Upward water pressure (2.6 m NAP)</td>
<td>-21.021</td>
<td>1.2</td>
</tr>
<tr>
<td>B Self weight deck</td>
<td>12.047</td>
<td>1.2</td>
</tr>
<tr>
<td>C Traffic loads</td>
<td>11.639</td>
<td>1.5</td>
</tr>
<tr>
<td>D Point load traffic</td>
<td>1.200</td>
<td>1.5</td>
</tr>
<tr>
<td>E Wave forces (-3 m NAP)</td>
<td>1.560</td>
<td>1.5</td>
</tr>
<tr>
<td>I Braking/temperature (+18.5 m NAP)</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>K Ship collision (4.1 m NAP)</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>L Ice pressure</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

Table F.9: Overview of the loads on pier 4.
APPENDIX F. STABILITY OF THE HARINGVLIET BRIDGE PIERS UNDER EXCAVATION

Small rotation of the bridge piers

When the bridge pier is subjected to an unequal settlement, an additional moment will be initiated by the small shift of the vertical loadings with respect to the point of rotation of the whole pier, see figure F.9. During the elaboration of the thesis, the interim situation with a maximum allowable horizontal displacement at the top of the pier was evaluated.

The rotation of the Haringvliet bridge piers will cause a moment somewhere between 1700 kNm and 2500 kNm. With respect to the moments caused by the horizontal load I (21.285 kNm) and K (64.000 kNm) the additional moments due to the rotation of the piers relatively small. Therefore, the contributions of the small rotations to the moment equilibrium were considered to be beyond the scope of this thesis.

F.2.9 Individual pile loads

Individual pile loads

The loads as calculated in table F.9 are for the whole foundation of the pier. In the following paragraphs, the loads on the individual pile loads will be calculated. The results are depicted in table F.10 for the case without any excavation.

The resultant pile loads are calculated according to equation F.13 for the vertical loads. The horizontal forces were considered to only induce a force through moments. The resultant pile loads due to moments caused by the horizontal loads are calculated according to equation F.14. The length of the moment arm was determined from the fixating depth of the foundation piles ($6 \cdot D_{eq}$) the working height of the load as depicted in table F.9.

$$F_{d,v;i,pile} = \frac{F_{d,pier,vert}}{\text{number of piles}}$$  \hspace{1cm} (F.13)

$$F_{d,m;i,pile} = \frac{(F_{H} \cdot c_{e}) \cdot e_{H,i}}{\sum e_{H,i}^{2} \cdot n_{i}}$$  \hspace{1cm} (F.14)

\textbf{Note:} The piles will be subjected to axial and lateral forces under inclinations of 1:10. This inclination was considered to be sufficiently small to consider that the loads are working in vertical and horizontal direction instead of under an angle. This is justified as the focus is on the bearing capacity of the soil and not on the structural strength of the foundation piles.

\textbf{Note:} In contrast to the original design calculations, the self weight of the pile foundation is completely taken into account. According to the Dutch standard NEN 9997, only the stretches above the soil surface should be taken into account, but by including all the piles, an even more conservative approach is used.
<table>
<thead>
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Table F.10: Individual pile loads per load case in kN, without excavations.
Loading tables per load combination and pile row

The maximum and minimum pile loads in the different loading combinations are included in table F.11 for the several characteristic excavation levels.

<table>
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<tr>
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<td>511 211</td>
<td>565 157</td>
<td>605 117</td>
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<td>659 63</td>
<td>739 -17</td>
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Table F.11: Maximum and minimum pile loads in [kN] for several excavation levels with respect to NAP in several load combinations.

Tension force

In case subsoil will be excavated, the negative shaft friction and thus the tension capacity of the subsoil will reduce significantly. This because the negative friction is mainly caused by the weaker, upper layers around the foundation piles. These soil layers will be the first to be removed during excavations.

As a rule of thumb for the available tension force, equation F.15 can be used. With the found maximum bearing capacity of the subsoil of 789 kN for excavations of -20 meter NAP, it was concluded that the occurring maximum tension force (-381 kN), will not be reached. However, this should be verified in future research.

\[
R_{st,tension} = \frac{R_{d,compression}}{1.5} = \frac{789}{1.5} = 526kN
\]  

(F.15)

As the maximum found tension force in the foundation piles is -184 kN (see table F.11), it was concluded that failure due to an exceedance of the tension bearing capacity of the subsoil was negligible.
F.3 Pile bearing capacity theory

The bearing capacity of a foundation pile under compression consists of three main components, see figure F.10. The bearing capacity of a single pile consists of three aspects: the tip bearing capacity, the positive shaft friction and the negative shaft friction. The tip bearing capacity is obtained by the surface of the foundation pile. The shaft frictions work over the complete length of the foundation piles. It was considered that the inclination of the foundation piles does not have significant influences on the bearing capacity of the pile foundation.

To check whether sufficient bearing capacity is present in the subsoil of the Haringvliet bridge piers, the bearing capacity of a single pile is determined. An assessment of the pile group was not required, as a check on the bearing capacity of a single pile is sufficient for rigid structures\(^1\)[Nederlands Normalisatie-instituut, 2012].

F.3.1 Tip bearing capacity

The main part of the resistance against settlements of a foundation pile is in general obtained by the tip resistance, which can be calculated with the method of Koppejan, see equation F.17 [Van Tol, 2006] & [Nederlands Normalisatie-instituut, 2012].

\[
q_{b,\text{max};i} = \alpha_p \cdot \beta \cdot s \cdot \frac{1}{2} \left( \frac{q_{c;I,\text{av}} + q_{c;II,\text{av}}}{2} + q_{c;III,\text{av}} \right) \\
R_{b,\text{cal};\text{max};i} = A_p \cdot q_{b,\text{max};i} \tag{F.16}
\]

\[
R_{b,\text{cal};\text{max};i} = A_p \cdot q_{b,\text{max};i} \tag{F.17}
\]

In which:
- \(q_{b,\text{max};i}\) = Maximum pile tip resistance in MPa, knotted off when exceeding 15 MPa for cases where CPT’s were used as source.
- \(\alpha_p\) = Pile class. In case of soil displacing piles, \(\alpha_p = 1\) [Nederlands Normalisatie-instituut, 2012]. In the near future, this pile class factor will be possibly lowered to 0.7 [Nederlands Normalisatie-instituut, 2015].
- \(\beta\) = Factor for the shape of the foot of the pile into account. For straight piles \(\beta = 1\).
- \(s\) = Factor for the shape of the pile. For square and circular piles \(s = 1\).
- \(q_{c;I,\text{av}}\) = Average value of the cone resistances along section I, from the level of the tip of the pile to a level at least 0.7 \(\cdot D_{eq}\) and at most 4 \(\cdot D_{eq}\) deeper. The bottom of section I must be selected between these boundaries so that \(q_{b,\text{max}}\) is minimum.
- \(q_{c;II,\text{av}}\) = Average value of the cone resistance along section II, from the bottom of section I to the level of the tip of the pile. The value of the cone resistance to be used in calculations never may exceed that of a lower level.
- \(q_{c;III,\text{av}}\) = Average value of the cone resistance along section III, which runs up from the pile tip level to a level 8 \(\cdot D_{eq}\) above. As for section II, the value used for the cone resistance may never exceed that of a lower level, starting with the value of cone resistance at the end of section II.
- \(D_{eq}\) = equivalent pile tip diameter. For a circular pile: \(D = D_{eq}\). For a square pile: \(D = \sqrt{4/\pi \cdot \text{pile width}}.\)

\(^1\)As the foundation of the Haringvliet bridge piers consist of a 3 meter thick concrete layer, in which all the pile heads were embedded during the casting of the concrete, the bridge pier was considered to be rigid.
APPENDIX F. STABILITY OF THE HARINGVLIET BRIDGE PIERS UNDER EXCAVATION

F.3.2 Pile shaft friction

Three types of pile shaft friction are present. The positive and negative shaft friction add resistance and load to the foundation piles subjected to compressive forces. For foundation piles under tension, due to potential lift of the structure, friction between the foundation piles and the subsoil is the only source of resistance. In this thesis was assumed that a failure of the foundation piles on tension was negligible.

Positive shaft friction

Positive shaft friction is activated when the foundation pile settles more than the surrounding soil layers. In this case, the friction between the pile and the soil will increase the bearing capacity of the foundation pile.

Positive shaft friction only applies for soil layers with a \(q_c > 2\) MPa, which are not interrupted by softer soil layers [Nederlands Normalisatie-instituut, 2012]. The shaft friction can be calculated with equation F.19 [Nederlands Normalisatie-instituut, 2012]. To full activate the positive shaft friction, a foundation pile should have settled 10 millimetres or more [Nederlands Normalisatie-instituut, 2012].

\[
q_{s;\text{max};z} = \alpha_s \cdot q_{c;z;\text{a}} \quad (F.18)
\]

\[
R_{s;\text{cal};\text{max}} = O_p \cdot \Delta L \cdot q_{s;\text{max};z} \quad (F.19)
\]

In which:

- \(O_p\): Circumference foundation pile (0.4 \cdot 4 = 1.6 m)
- \(\alpha_s\): Factor for the execution method.
- For soil displacing piles and sand layers: \(\alpha_s = 0.01\).

In which: \(q_{c;z;\text{a}}\): Cone resistance. Values above 12 MPa should be reduced to 12 MPa, unless:

- The \(q_c\) values are higher than 12 MPa over more than 1 meter, then:
- The lowest \(q_c\) value over the stretch should be used, limited to 15 MPa.

\(\Delta L\): Working length along the pile shaft of the positive shaft friction.

\(z\): Depth, relative to NAP.

Negative shaft friction

Due to potential settlements of softer layers, downwards directed (negative) shaft friction might be induced on the piles, which reduces the bearing power of the pile. In general the negative shaft friction is only applied in SLS and in case the expected settlements are larger than 0.1 meter [Nederlands Normalisatie-instituut, 2012]. In case of ULS the piles are likely to settle more than the surrounding soil layers. Therefore, no negative shaft friction should be taken into account in ULS [Nederlands Normalisatie-instituut, 2012]. The value of the negative shaft friction should be calculated according to equation F.20 [Nederlands Normalisatie-instituut, 2012].

\[
F_{st;d} = \gamma_{st} \cdot F_{st;\text{rep}} \quad (F.20)
\]

\[
F_{st;\text{rep}} = O_p \cdot \sum F_{j;\text{rep}} \quad F_{j;\text{rep}} = \Delta h_j \cdot K_{0;j} \cdot \tan \delta_j \cdot \sigma'_{j;\text{avg}} \quad (F.21)
\]

\[
\sigma'_{j;\text{avg}} = \frac{\sigma'_{j;\text{rep}} + \sigma'_{j-1;\text{rep}}}{2} \quad \sigma'_{j;\text{rep}} = \Sigma \gamma_{j;\text{sat}} \cdot \delta h_j - p_j
\]

In which:

- \(F_{st;\text{rep}}\): Representative negative friction kN
- \(F_{j;\text{rep}}\): Representative negative friction per soil layer j kN/m
- \(O_p\): Circumference foundation pile m
- \(\Delta h_j\): Thickness layer j m
- \(K_{0;j}\): Neutral horizontal soil pressure factor
- \(\delta_j\): Friction angle between pile and soil
- \(\sigma'_{j;\text{avg}}\): Average value of the vertical effective stress in layer j kN/m²
- \(\sigma'_{j;\text{rep}}\): Representative value of the vertical effective stress in layer j kN/m²
- \(p_j\): Water pressure in layer j kN/m²
F.3. Pile bearing capacity theory

The values for $K_{0,j,k} \cdot \tan \delta_{j,k}$ were simplified to general values per type of soil. For clay and peat layers this was $\approx 0.25$, for soil displacing piles in sand layers 0.5 was more appropriate, also when clay was present in the sand layer [Van Tol, 2006]. In layers with mainly clay and a small partition of sand, 0.3 was considered as a representative value [Van Tol, 2006].

Tension resistance foundation piles

In the case the resulting load tend to lift the bridge pier, tension forces will occur in the foundation piles. The resistance against these tension forces is generated by shear stresses along the pile shaft. The amount of tension resistance is depending on the amount of friction between the pile shaft and the subsoil. In general, the friction due to tension is lower than the friction due to compression. If the foundation piles are placed in a group, they might affect each other unfavourable as the zones of influences might overlap. The influence of the pile group on the resistance against tension could be checked by the clump weight criterion [Vrijling et al., 2016].

It was assumed that the tension force is no critical aspect in the feasibility of the designs made in this thesis. In chapter 3.3, table 6.3, is shown that tension forces up to -111 kN might occur, in case a ship collides with a pier of the Haringvliet bridge and the soil is excavated to a level of -25 meter NAP which is only 6 meters above the pile tip level. As the tension forces in the pile foundation are only found for the loading case including ship collision, it was concluded that the tension force only occurs for a very short period of time. The influence of the tension forces were assumed to be allowed, as long as the tension forces were very small (below 100 kN).

F.3.3 Cone resistance reduction due to excavation

The excavation of soil layers, will unfavourable affect the situation. The loads will increase due to the larger arm of the momentum and the soil resistances reduce. The reduction of the resistance is mainly caused by the reduced cone resistance due to relaxation of the soil [Nederlands Normalisatie-instituut, 2012].

In the Dutch standards guidelines methods are available to calculate the cone resistance reduction for existing pile foundations, see equation F.22 [Nederlands Normalisatie-instituut, 2012]. In addition to the cone resistance reduction, the length over which the positive shaft friction might work decreases with more excavations. The reduced cone reduction has its influences on both the pile tip resistance and the positive shaft friction of a single foundation pile.

With respect to the initial design calculations for the Haringvliet bridge as made in 1962, the bearing capacity of the foundation was already reduced, as several soil layers around the Haringvliet bridge have been eroded after the closing of the Haringvliet dam.

\[
q_{c,z;exc} = q_{c,z;0} \cdot \sqrt[\sigma_{v,z;exc}^{'} / \sigma_{v,z;0}']
\]

\[
\sigma_{v,z;exc}^{'} = \sigma_{v,z;0}^{'} - \Delta \sigma_{v,z;exc}^{'}
\]

In which:
- $q_{c,z;exc}$ Corrected cone resistance at depth $z$ [MPa]
- $q_{c,z;0}$ Initial Cone Resistance at depth $z$ [MPa]
- $\sigma_{v,z;exc}^{'}$ Effective vertical soil stress at depth $z$ after excavation $[kN/m^2]$ 
- $\sigma_{v,z;0}^{'}$ Initial effective vertical soil stress at depth $z$ $[kN/m^2]$ 
- $\Delta \sigma_{v,z;exc}^{'}$ Reduction of the effective vertical soil stress $[kN/m^2]$
F.4 Bearing capacity foundation piles of the Haringvliet bridge

In this appendix it is elaborated on the bearing capacity of the Haringvliet bridge pier foundation. In figures F.11 and F.12 the interpretations of the original CPT of figure B.2 are included.

In appendix F.4.1 the bearing capacity of an individual pile is calculated for the initial situation according to CPT 9 (see figure B.2).

Figure F.11: Tip bearing capacity interpretation of CPT 9 for a pile depth of -29.5 m NAP. The \( q_c \) values on the horizontal axis should be multiplied by 2 to have the right dimensions.
F.4. Bearing capacity foundation piles of the Haringvliet bridge

F.4.1 Bearing capacity calculation for the initial situation

Tip bearing capacity

In figure F.11 and table F.12 the determination of the tip bearing capacity CPT 9 are depicted according to section F.3.1. The full length of \( q_{cI,av} \) for CPT 9 should be taken into account, as this results in the lowest \( q_{b,\text{max,}9} \). The calculated pile tip resistance of \( q_{b,\text{max,}9} = 5.5 \) MPa will result in a tip bearing capacity of \( R_{b,\text{cal,}max,9} = 873 \) kN per pile, as can be seen in equation F.24.

**Note:** When the complete positive pile shaft friction is absent in the initial situation, the characteristic bearing capacity of the pile tips (\( R_{b,k} = R_{b,\text{cal}}/\xi = 873/1.2 = 727.5kN \)) is still sufficient to withstand the maximum SLS individual pile loads. However, this does not hold for any other case.

Figure F.12: Positive shaft friction interpretation of CPT 9 for a pile depth of -29.5 m NAP. The \( q_c \) values on the horizontal axis should be multiplied by 2 to have the right dimensions.
APPENDIX F. STABILITY OF THE HARINGVLIET BRIDGE PIERS UNDER EXCAVATION

\[ q_{b,\text{max},i} = \alpha_p \cdot \beta \cdot s \cdot \left( \frac{q_{c,1\text{av}} + q_{c,11\text{av}}}{2} + q_{c,111\text{av}} \right) \]

\[ q_{b,\text{max},9} = 1 \cdot 1 \cdot \frac{1}{2} \left( \frac{11.1 + 3.9}{2} + 3.4 \right) = 5.5 \text{ MPa} \]

\[ R_{b,\text{cal},\text{max},9} = A_p \cdot q_{b,\text{max},i} = 0.4^2 \cdot 5.5 = 873 \text{ kN} \]

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<th>( q_{c,11\text{,z}} )</th>
<th>( q_{c,111\text{,z}} )</th>
<th>( q_{c,\text{max},z} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>[m]</td>
<td>[( \frac{1}{2} \text{ kg/cm}^2 )]</td>
<td>[MPa]</td>
<td>[( \frac{1}{2} \text{ kg/cm}^2 )]</td>
<td>[MPa]</td>
</tr>
<tr>
<td>0.25</td>
<td>6</td>
<td>2.5</td>
<td>3.4</td>
<td>1.75</td>
</tr>
<tr>
<td>0.14</td>
<td>5.4</td>
<td>10.59</td>
<td>1.59</td>
<td>3.43</td>
</tr>
<tr>
<td>0.21</td>
<td>5.6</td>
<td>10.99</td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.20</td>
<td>6.6</td>
<td>12.90</td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.23</td>
<td>8.5</td>
<td>16.68</td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.56</td>
<td>5.1</td>
<td>10.01</td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.25</td>
<td>3.4</td>
<td>6.67</td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>1.84</strong></td>
<td><strong>11.10</strong></td>
<td><strong>1.84</strong></td>
<td><strong>3.87</strong></td>
<td><strong>3.49</strong></td>
</tr>
</tbody>
</table>

Table F.12: Calculation of the average cone resistances for sections I, II and III for CPT9 pier 3, according to figure F.11.

Positive shaft friction

In figure F.12 and table F.13 the calculations of the required parameters according to equation F.25 (chapter F.3.2) are depicted. As the positive shaft friction only applies for soil layers with a \( q_c > 2 \text{ MPa} \), only the section between -19 meter NAP and -29.5 meter is taken into account.

\[ q_{s,\text{max},z} = \alpha_s \cdot q_{c,z} \]

\[ R_{s,\text{cal},\text{max}} = O_p \cdot \Delta L \cdot q_{s,\text{max},z} \]

\[ \alpha_s = \frac{\sigma_{\text{sat}}}{\sigma_{\text{sat}} + \gamma \cdot \delta h_j} \]

\[ \sigma_{\text{avg}} = \frac{\sigma_{j,\text{avg}} + \sigma_{j-1,\text{avg}}}{2} \]

\[ \sigma_{j,\text{avg}} = \frac{\sigma_{j,\text{avg}} + \sigma_{j-1,\text{avg}}}{2} \]

Negative shaft friction

The negative shaft friction is calculated according to equation F.26, as discussed in chapter F.3.2. The lowest soft soil layer \( (q_c \leq 2 \text{ MPa}) \) reaches to approximately -19 meter NAP. In table F.14 the values for the initial situation (soil surface at -3.8 meter NAP) and the eroded situation are depicted. The accompanying design resistances are given in equation F.27.

\[ F_{st,\text{rep}} = O_p \cdot \Sigma F_{j,\text{rep}} \]

\[ F_{j,\text{rep}} = \Delta h_j \cdot K_{0,j} \cdot \tan \delta_j \cdot \sigma_{j,\text{avg}} \]

\[ \sigma_{j,\text{avg}} = \frac{\sigma_{j,\text{avg}} + \sigma_{j-1,\text{avg}}}{2} \]

\[ \sigma_{j,\text{rep}} = \gamma_{j,\text{sat}} \cdot \delta h_j - p_j \]
F.4. Bearing capacity foundation piles of the Haringvliet bridge

Table F.14: Calculation negative shaft friction for initial and eroded situation at CPT 9 pier 3.

<table>
<thead>
<tr>
<th>Layer</th>
<th>$\delta h_j$ [m NAP]</th>
<th>$\gamma_{j, sat}$ [kN/m$^3$]</th>
<th>$\sigma_{j, rep}$ [kN/m$^2$]</th>
<th>$p_j$ [kN/m$^2$]</th>
<th>$\sigma'_{j, rep}$ [kN/m$^2$]</th>
<th>$\sigma'_{j, avg}$ [kN/m$^2$]</th>
<th>$K_0$ tan((\delta))</th>
<th>$F_{j, rep}$ [kN/m]</th>
<th>$F_{s, rep}$ [kN]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Water</td>
<td>3.8</td>
<td>10</td>
<td>38.0</td>
<td>38.0</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td>Sand</td>
<td>4.3</td>
<td>19.0</td>
<td>120.5</td>
<td>81.4</td>
<td>39.1</td>
<td>19.5</td>
<td>0.5</td>
<td>42.4</td>
<td></td>
</tr>
<tr>
<td>Clay</td>
<td>1.1</td>
<td>20.0</td>
<td>142.5</td>
<td>92.4</td>
<td>50.1</td>
<td>44.6</td>
<td>0.3</td>
<td>14.7</td>
<td></td>
</tr>
<tr>
<td>Sand/clay</td>
<td>9.7</td>
<td>18.0</td>
<td>316.3</td>
<td>189.0</td>
<td>127.3</td>
<td>88.7</td>
<td>0.4</td>
<td>342.7</td>
<td></td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td><strong>18.9</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table F.15: Initial soil stresses for the soil layers $j$ for CPT 9, pier 3.

<table>
<thead>
<tr>
<th>Layer</th>
<th>Layer</th>
<th>$\Delta h_j$ [m]</th>
<th>$\gamma_{j, sat}$ [kN/m$^3$]</th>
<th>$\sigma_{j, sat}$ [kN/m$^2$]</th>
<th>$\sigma_{j, avg}$ [kN/m$^2$]</th>
<th>$\sigma_{j, rep}$ [kN/m$^2$]</th>
<th>$\sigma_{j, rep}$ [kN/m$^2$]</th>
<th>$p_j$ [kN/m$^2$]</th>
<th>$\sigma'_{j, rep}$ [kN/m$^2$]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Water</td>
<td>3.8</td>
<td>3.8</td>
<td>38.0</td>
<td>38.0</td>
<td>38.0</td>
<td>38.0</td>
<td>38.0</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td>Sand</td>
<td>8.14</td>
<td>4.34</td>
<td>19</td>
<td>82.5</td>
<td>120.5</td>
<td>81.4</td>
<td>39.1</td>
<td>39.1</td>
<td></td>
</tr>
<tr>
<td>Sand</td>
<td>8.14</td>
<td>1.1</td>
<td>19</td>
<td>22.0</td>
<td>142.5</td>
<td>92.4</td>
<td>92.4</td>
<td>50.1</td>
<td>50.1</td>
</tr>
<tr>
<td>Clay</td>
<td>9.24</td>
<td>18.9</td>
<td>18</td>
<td>173.9</td>
<td>316.3</td>
<td>189.0</td>
<td>189.0</td>
<td>189.0</td>
<td>127.3</td>
</tr>
<tr>
<td>Sand</td>
<td>18.9</td>
<td>7.4</td>
<td>18</td>
<td>148.0</td>
<td>464.3</td>
<td>263.0</td>
<td>263.0</td>
<td>263.0</td>
<td>263.0</td>
</tr>
<tr>
<td>Sand</td>
<td>26.7</td>
<td>8.0</td>
<td>20</td>
<td>8.0</td>
<td>472.3</td>
<td>267.0</td>
<td>267.0</td>
<td>267.0</td>
<td>267.0</td>
</tr>
<tr>
<td>Sand/clay</td>
<td>26.7</td>
<td>4.1</td>
<td>20</td>
<td>82.0</td>
<td>554.3</td>
<td>308.0</td>
<td>308.0</td>
<td>308.0</td>
<td>246.3</td>
</tr>
<tr>
<td>Clay</td>
<td>30.8</td>
<td>0.5</td>
<td>18</td>
<td>9.0</td>
<td>563.3</td>
<td>313.0</td>
<td>313.0</td>
<td>313.0</td>
<td>250.3</td>
</tr>
<tr>
<td>Sand</td>
<td>31.3</td>
<td>10.7</td>
<td>20</td>
<td>214.0</td>
<td>777.3</td>
<td>420.0</td>
<td>420.0</td>
<td>420.0</td>
<td>357.3</td>
</tr>
</tbody>
</table>

F.4.2 CPT 9: Erosion

Due to the erosion of 4.2 meters of soil to a soil surface level of -8 meters NAP the cone reduction should be applied. In table F.16 the effective stresses and the accompanying reduction ratios are calculated. With this cone reduction the tip bearing capacity and the positive shaft friction are calculated in tables F.17 and F.18.

In equation F.28:

$$q_{c;exc} = q_{c;0} \cdot \frac{\sigma'_{v;exc}}{\sigma'_{v;0}}$$

$$\sigma'_{v;exc} = \sigma'_{v;0} - \Delta \sigma'_{v;exc}$$  

In table F.15 the values for the initial situation are included. With these values, the cone reductions for the excavated situations are determined in the following paragraphs.
The used reduction ratio was based on the soil layer between -19.2 meter and -29.5 meter NAP. Over this section the most unfavourable cone reduction factor is 0.9. The total resistance of a single pile under erosion is depicted in equation F.31.

\[
R_{b,k} = \frac{R_{b,cal}}{\xi_3} = \frac{785.9}{1.2} = 654.9 \text{ kN} \quad \text{and} \quad R_{b,d} = \frac{R_{b,k}}{\gamma_b} = \frac{654.9}{1.2} = 545.8 \text{ kN} \quad \text{(F.29)}
\]

\[
R_{s,k} = \frac{R_{s,cal}}{\xi_3} = \frac{1348.8}{1.2} = 1124.0 \text{ kN} \quad \text{and} \quad R_{s,d} = \frac{R_{s,k}}{\gamma_s} = \frac{1124.0}{1.2} = 936.7 \text{ kN} \quad \text{(F.30)}
\]

\[
R_{c,d} = R_{b,d} + R_{s,d} = 545.8 + 936.7 \text{ kN} = 1482.5 \text{ kN} \quad \text{(F.31)}
\]

<table>
<thead>
<tr>
<th>Layer [m NAP]</th>
<th>(\Delta h_j) [m]</th>
<th>(\gamma_{sat,j}) [kN/m²]</th>
<th>(\sigma_{sat,j}) [kN/m²]</th>
<th>(\sum \sigma_j) [kN/m²]</th>
<th>(p) [kN/m²]</th>
<th>(\sigma_{v,0}) [kN/m²]</th>
<th>(\sqrt{\sigma_{v,0}^2 + \sigma_{v,exc}^2}) [kN/m²]</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>8</td>
<td>8</td>
<td>80.0</td>
<td>80.0</td>
<td>80.0</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td>8</td>
<td>8.14</td>
<td>0.14</td>
<td>19</td>
<td>2.7</td>
<td>82.7</td>
<td>1.3</td>
<td>0.18</td>
</tr>
<tr>
<td>8.14</td>
<td>9.24</td>
<td>1.1</td>
<td>20</td>
<td>22.0</td>
<td>104.7</td>
<td>12.3</td>
<td>0.49</td>
</tr>
<tr>
<td>9.24</td>
<td>18.9</td>
<td>9.66</td>
<td>18</td>
<td>173.9</td>
<td>278.5</td>
<td>189.0</td>
<td>89.5</td>
</tr>
<tr>
<td>18.9</td>
<td>26.3</td>
<td>7.4</td>
<td>18</td>
<td>148.0</td>
<td>426.5</td>
<td>263.0</td>
<td>163.5</td>
</tr>
<tr>
<td>26.3</td>
<td>26.7</td>
<td>0.4</td>
<td>20</td>
<td>8.0</td>
<td>434.5</td>
<td>267.0</td>
<td>167.5</td>
</tr>
<tr>
<td>26.7</td>
<td>30.8</td>
<td>4.1</td>
<td>18</td>
<td>8.0</td>
<td>516.5</td>
<td>308.0</td>
<td>208.5</td>
</tr>
<tr>
<td>30.8</td>
<td>31.3</td>
<td>0.5</td>
<td>18</td>
<td>9.0</td>
<td>525.5</td>
<td>313.0</td>
<td>212.5</td>
</tr>
<tr>
<td>31.3</td>
<td>42</td>
<td>10.7</td>
<td>20</td>
<td>9.0</td>
<td>730.5</td>
<td>420.0</td>
<td>319.5</td>
</tr>
</tbody>
</table>

Table F.16: Initial soil stresses vs. erosion for CPT 9, pier 3, -8 meter NAP

<table>
<thead>
<tr>
<th>Initial situation</th>
<th>Erosion</th>
</tr>
</thead>
<tbody>
<tr>
<td>(q_{c,I,\text{av}})</td>
<td>11.1 [MPa]</td>
</tr>
<tr>
<td>(q_{c,II,\text{av}})</td>
<td>3.9 [MPa]</td>
</tr>
<tr>
<td>(q_{c,III,\text{av}})</td>
<td>3.4 [MPa]</td>
</tr>
<tr>
<td>(q_{b,\text{max},9})</td>
<td>5.5 [MPa]</td>
</tr>
<tr>
<td>(R_{b,\text{cal,max},9})</td>
<td>873 [kN]</td>
</tr>
</tbody>
</table>

Table F.17: Calculation tip bearing capacity in CPT 9 with erosion, cone reduction factor of 0.9.

<table>
<thead>
<tr>
<th>(\Delta L) [m]</th>
<th>(q_{c,0}) [MPa]</th>
<th>(q_{c,exc}) [MPa]</th>
<th>(q_{b,\text{max},z}) [MPa]</th>
<th>(R_{b,\text{cal,max},9,\text{ero}}) [kN/m²]</th>
</tr>
</thead>
<tbody>
<tr>
<td>7.5</td>
<td>7.65</td>
<td>6.9</td>
<td>0.07</td>
<td>68.9</td>
</tr>
<tr>
<td>0.8</td>
<td>11.77</td>
<td>10.6</td>
<td>0.11</td>
<td>106.1</td>
</tr>
<tr>
<td>0.4</td>
<td>12.16</td>
<td>11.0</td>
<td>0.11</td>
<td>109.6</td>
</tr>
<tr>
<td>1.4</td>
<td>13.93</td>
<td>12.6</td>
<td>0.13</td>
<td>125.5</td>
</tr>
<tr>
<td>0.2</td>
<td>12.56</td>
<td>11.3</td>
<td>0.11</td>
<td>113.2</td>
</tr>
<tr>
<td>Total</td>
<td>0.52</td>
<td>522.6</td>
<td>1348.8</td>
<td></td>
</tr>
</tbody>
</table>

Table F.18: Calculation positive shaft friction CPT 9 pier 3 with the erosion to -8 m NAP, with cone reduction ratio 0.9

**F.4.3 CPT 9: Excavations**

For the calculations for the several excavation depths the same approach was used as for the eroded situation. The values for several the excavations depths are depicted in the coming tables. The overview of the pile resistance under several situations is depicted in table F.19. These values are calculated in the paragraphs in this section, according to the methods as described in the previous.
F.4. Bearing capacity foundation piles of the Haringvliet bridge

### Table F.19: Overview pile resistances for CPT 9 in [kN].

<table>
<thead>
<tr>
<th>Layer [m NAP]</th>
<th>( R_{b,cal} ) [kN]</th>
<th>Ration</th>
<th>( R_{b,cal} ) [kN]</th>
<th>( R_{d,cal} ) [kN]</th>
<th>( R_{s,cal} ) [kN]</th>
<th>( R_{c,d} ) [kN]</th>
</tr>
</thead>
<tbody>
<tr>
<td>-10 m NAP</td>
<td>742</td>
<td>0.833</td>
<td>618</td>
<td>429</td>
<td>1273</td>
<td>884</td>
</tr>
<tr>
<td>-15 m NAP</td>
<td>631</td>
<td>0.792</td>
<td>500</td>
<td>347</td>
<td>1083</td>
<td>752</td>
</tr>
<tr>
<td>-20 m NAP</td>
<td>488</td>
<td>0.750</td>
<td>366</td>
<td>254</td>
<td>770</td>
<td>535</td>
</tr>
<tr>
<td>-25 m NAP</td>
<td>222</td>
<td>0.729</td>
<td>162</td>
<td>113</td>
<td>194</td>
<td>135</td>
</tr>
</tbody>
</table>

Excavation -10 meter NAP

\[
R_{b,k} = \frac{R_{b,cal}}{\zeta_3} = \frac{741.5}{1.2} = 617.9 \text{ kN} & R_{b,d} = \frac{R_{b,k}}{\gamma_b} = \frac{617.9}{1.2} = 515.0 \text{ kN} \quad (F.32)
\]

\[
R_{s,k} = \frac{R_{s,cal}}{\zeta_3} = \frac{1272.7}{1.2} = 1060.6 \text{ kN} & R_{s,d} = \frac{R_{s,k}}{\gamma_s} = \frac{1060.6}{1.2} = 883.8 \text{ kN} \quad (F.33)
\]

\[
R_{c,d} = R_{b,d} + R_{s,d} = 515.0 + 883.8 \text{ kN} = 1398.8 \text{ kN} \geq F_{c,d} \quad (F.34)
\]

Table F.20: Initial soil stresses vs. excavation for CPT 9, -10 meter NAP

<table>
<thead>
<tr>
<th>Layer [m NAP]</th>
<th>( \Delta h_j ) [m]</th>
<th>( \gamma_{sat,j} )</th>
<th>( \sigma_{sat,j} ) [kN/m²]</th>
<th>( \Sigma \sigma_j ) [kN/m²]</th>
<th>( p ) [kN/m²]</th>
<th>( \sigma'_{v,z:0} ) [kN/m²]</th>
<th>( \sqrt{\sigma'<em>{v,z:exc} / \sigma'</em>{v,z:0}} ) [-]</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>10</td>
<td>10</td>
<td>100.0</td>
<td>100.0</td>
<td>100.0</td>
<td>0.0</td>
<td>0.00</td>
</tr>
<tr>
<td>10</td>
<td>10</td>
<td>0</td>
<td>0.0</td>
<td>100.0</td>
<td>100.0</td>
<td>0.0</td>
<td>0.00</td>
</tr>
<tr>
<td>10</td>
<td>10</td>
<td>19</td>
<td>0.0</td>
<td>100.0</td>
<td>100.0</td>
<td>0.0</td>
<td>0.00</td>
</tr>
<tr>
<td>10</td>
<td>18.9</td>
<td>8.9</td>
<td>160.2</td>
<td>260.2</td>
<td>189.0</td>
<td>71.2</td>
<td>0.75</td>
</tr>
<tr>
<td>18.9</td>
<td>26.3</td>
<td>7.4</td>
<td>148.0</td>
<td>408.2</td>
<td>263.0</td>
<td>145.2</td>
<td>0.85</td>
</tr>
<tr>
<td>26.3</td>
<td>26.7</td>
<td>0.4</td>
<td>8.0</td>
<td>416.2</td>
<td>267.0</td>
<td>149.2</td>
<td>0.85</td>
</tr>
<tr>
<td>26.7</td>
<td>30.8</td>
<td>4.1</td>
<td>82.0</td>
<td>498.2</td>
<td>308.0</td>
<td>190.2</td>
<td>0.88</td>
</tr>
<tr>
<td>30.8</td>
<td>31.3</td>
<td>0.5</td>
<td>9.0</td>
<td>507.2</td>
<td>313.0</td>
<td>194.2</td>
<td>0.88</td>
</tr>
<tr>
<td>31.3</td>
<td>42</td>
<td>10.7</td>
<td>214.0</td>
<td>721.2</td>
<td>420.0</td>
<td>301.2</td>
<td>0.92</td>
</tr>
</tbody>
</table>

### Table F.21: Calculation positive shaft friction CPT 9 with the excavation to -8 m NAP, with cone reduction ratio 0.85.

<table>
<thead>
<tr>
<th>( \Delta L ) [m]</th>
<th>( q_{v,z:0} ) [MPa]</th>
<th>( q_{v,z:exc} ) [MPa]</th>
<th>( q_{v,max:z} ) [MPa]</th>
<th>( R_{s,cal,max:9.0} ) [kN/m]</th>
<th>( R_{c,cal,max:9.0} ) [kN]</th>
</tr>
</thead>
<tbody>
<tr>
<td>7.5</td>
<td>7.65</td>
<td>6.5</td>
<td>0.07</td>
<td>65.0</td>
<td>779.6</td>
</tr>
<tr>
<td>0.8</td>
<td>11.77</td>
<td>10.0</td>
<td>0.10</td>
<td>100.0</td>
<td>127.9</td>
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<tr>
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</tr>
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<td>0.12</td>
<td>118.3</td>
<td>265.0</td>
</tr>
<tr>
<td>0.2</td>
<td>12.56</td>
<td>10.7</td>
<td>0.11</td>
<td>106.7</td>
<td>34.1</td>
</tr>
</tbody>
</table>

Total | **0.49** | **493.1** | **1272.7** |
APPENDIX F. STABILITY OF THE HARINGVLIEI BRIDGE PIERS UNDER EXCAVATION

Initial situation

<table>
<thead>
<tr>
<th>$q_{c,1,av}$</th>
<th>$q_{c,11,av}$</th>
<th>$q_{c,III,av}$</th>
<th>$q_{b,max,9}$</th>
<th>$R_{b,cal,max,9}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>11.1</td>
<td>3.9</td>
<td>3.4</td>
<td>5.5</td>
<td><strong>873</strong></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>$q_c$</th>
<th>$I_{av}$</th>
<th>$R_b$</th>
<th>$R_s$</th>
<th>$R_c$</th>
</tr>
</thead>
<tbody>
<tr>
<td>3.9</td>
<td>3.3</td>
<td>2.9</td>
<td>873.0</td>
<td></td>
</tr>
<tr>
<td>3.4</td>
<td>2.9</td>
<td>2.9</td>
<td>741.5</td>
<td></td>
</tr>
<tr>
<td>5.5</td>
<td>4.6</td>
<td>4.6</td>
<td>438.0</td>
<td></td>
</tr>
</tbody>
</table>

Table F.22: Calculation tip bearing capacity in CPT 9 with the excavation to -10 m, with cone reduction ratio 0.85.

Excavation -15 meter NAP

\[
R_{b,k} = \frac{R_{b,cal}}{\xi_3} = \frac{631.2}{1.2} = 526.0 \text{ kN} \quad \text{and} \quad R_{b,d} = \frac{R_{b,k}}{\gamma_b} = \frac{526.0}{1.2} = 438.3 \text{ kN} \quad (F.35)
\]

\[
R_{s,k} = \frac{R_{s,cal}}{\xi_3} = \frac{1083.3}{1.2} = 902.8 \text{ kN} \quad \text{and} \quad R_{s,d} = \frac{R_{s,k}}{\gamma_s} = \frac{902.8}{1.2} = 752.3 \text{ kN} \quad (F.36)
\]

\[
R_{c,d} = R_{b,d} + R_{s,d} = 438.3 + 752.3 = 1190.6 \text{ kN} \geq F_{c,d}
\]

<table>
<thead>
<tr>
<th>Layer</th>
<th>$\Delta h_j$ [m]</th>
<th>$\gamma_{sat,j}$</th>
<th>$\sigma_{sat,j}$ [kN/m²]</th>
<th>$\Sigma \sigma_j$ [kN/m²]</th>
<th>$p$ [kN/m²]</th>
<th>$\sqrt{\sigma'<em>{v,z,exc}/\sigma'</em>{v,z,0}}$ [-]</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>15</td>
<td>15</td>
<td>150.0</td>
<td>150.0</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td>15</td>
<td>15</td>
<td>0</td>
<td>150.0</td>
<td>150.0</td>
<td>0.0</td>
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<tr>
<td>15</td>
<td>15</td>
<td>0</td>
<td>150.0</td>
<td>150.0</td>
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<td>4.1</td>
<td>8.0</td>
<td>376.2</td>
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<td>109.2</td>
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<tr>
<td>30.8</td>
<td>31.3</td>
<td>0.5</td>
<td>9.0</td>
<td>467.2</td>
<td>313.0</td>
<td>154.2</td>
</tr>
<tr>
<td>31.3</td>
<td>42</td>
<td>10.7</td>
<td>214.0</td>
<td>681.2</td>
<td>420.0</td>
<td>261.2</td>
</tr>
</tbody>
</table>

Table F.23: Initial soil stresses vs. erosion for CPT 9, pier 3, -15 meter NAP

<table>
<thead>
<tr>
<th>$\Delta L$ [m]</th>
<th>$q_{c,1,av}$ [MPa]</th>
<th>$q_{c,11,exc}$ [MPa]</th>
<th>$q_{c,III,av}$ [MPa]</th>
<th>$R_{s,cal,max,9,exc}$ [kN/m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>7.5</td>
<td>7.65</td>
<td>5.5</td>
<td>0.06</td>
<td>55.3</td>
</tr>
<tr>
<td>0.8</td>
<td>11.77</td>
<td>8.5</td>
<td>0.09</td>
<td>85.1</td>
</tr>
<tr>
<td>0.4</td>
<td>12.16</td>
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<td>87.9</td>
</tr>
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<td>1.4</td>
<td>13.93</td>
<td>10.1</td>
<td>0.10</td>
<td>100.7</td>
</tr>
<tr>
<td>0.2</td>
<td>12.56</td>
<td>9.1</td>
<td>0.09</td>
<td>90.8</td>
</tr>
<tr>
<td>Total</td>
<td><strong>0.42</strong></td>
<td><strong>419.8</strong></td>
<td><strong>1083.3</strong></td>
<td></td>
</tr>
</tbody>
</table>

Table F.24: Calculation positive shaft friction CPT 9 pier 3 with the erosion to -15 m NAP, with cone reduction ratio 0.72
F.4. Bearing capacity foundation piles of the Haringvliet bridge

Initial situation

<table>
<thead>
<tr>
<th>Layer</th>
<th>∆h_j [m NAP]</th>
<th>( \gamma_{sat,j} ) [kN/m³]</th>
<th>( \sigma_{sat,j} ) [kN/m²]</th>
<th>( \Sigma \sigma_j ) [kN/m²]</th>
<th>( p ) [kN/m²]</th>
<th>( \sigma_{w,z,0}' ) [kN/m²]</th>
<th>( \sqrt{\sigma_{w,z,exc}/\sigma_{w,z,0}'} ) [-]</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>20</td>
<td>10</td>
<td>200.0</td>
<td>200.0</td>
<td>200.0</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td>20</td>
<td>20</td>
<td>0</td>
<td>200.0</td>
<td>200.0</td>
<td>200.0</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
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<td>20</td>
<td>0</td>
<td>200.0</td>
<td>200.0</td>
<td>200.0</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td>20</td>
<td>20</td>
<td>0</td>
<td>200.0</td>
<td>200.0</td>
<td>200.0</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td>20</td>
<td>26.3</td>
<td>6.3</td>
<td>126.0</td>
<td>326.0</td>
<td>263.0</td>
<td>63.0</td>
<td>0.56</td>
</tr>
<tr>
<td>26.3</td>
<td>26.7</td>
<td>0.4</td>
<td>82.0</td>
<td>416.0</td>
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<td>108.0</td>
<td>0.66</td>
</tr>
<tr>
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<td>0.5</td>
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<td>242.0</td>
<td>313.0</td>
<td>112.0</td>
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<td>31.3</td>
<td>42</td>
<td>10.7</td>
<td>214.0</td>
<td>639.0</td>
<td>420.0</td>
<td>219.0</td>
<td>0.78</td>
</tr>
</tbody>
</table>

Table F.26: Initial soil stresses vs. erosion for CPT 9, pier 3, -20 meter NAP

Excavation -20 meter NAP

\[
R_{b,k} = \frac{R_{b,cal}}{\xi_3} = \frac{488.4}{1.2} = 407.0 \text{ kN} & R_{b,d} = \frac{R_{b,k}}{\gamma_b} = \frac{407.0}{1.2} = 339.2 \text{ kN} \quad (F.38)
\]

\[
R_{s,k} = \frac{R_{s,cal}}{\xi_3} = \frac{769.9}{1.2} = 641.6 \text{ kN} & R_{s,d} = \frac{R_{s,k}}{\gamma_s} = \frac{641.6}{1.2} = 534.6 \text{ kN} \quad (F.39)
\]

\[
R_{c,d} = R_{b,d} + R_{s,d} = 339.2 + 534.6 \text{ kN} = 873.8 \text{ kN} \geq F_{c,d} \quad (F.40)
\]
APPENDIX F. STABILITY OF THE HARINGVLIET BRIDGE PIERS UNDER EXCAVATION

<table>
<thead>
<tr>
<th>Layer</th>
<th>$\Delta h_j$ [m NAP]</th>
<th>$\gamma_{sat,j}$ [kN/m³]</th>
<th>$\sigma_{sat,j}$ [kN/m²]</th>
<th>$\sum \sigma_j$ [kN/m²]</th>
<th>$p$ [kN/m²]</th>
<th>$\sigma_{v,z:0}$ [kN/m²]</th>
<th>$\sqrt{\sigma_{v,z:exc}/\sigma_{v,z:0}}$ [-]</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
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<td>25</td>
<td>250.0</td>
<td>250.0</td>
<td>0.0</td>
<td>0</td>
<td>0.0</td>
</tr>
<tr>
<td>25</td>
<td>25</td>
<td>0</td>
<td>0</td>
<td>19</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
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<td>25</td>
<td>0</td>
<td>0</td>
<td>20</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td>25</td>
<td>25</td>
<td>0</td>
<td>0</td>
<td>18</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td>25</td>
<td>26.3</td>
<td>1.3</td>
<td>26.0</td>
<td>276.0</td>
<td>263.0</td>
<td>13.0</td>
<td>0.25</td>
</tr>
<tr>
<td>26.3</td>
<td>26.7</td>
<td>0.4</td>
<td>8.0</td>
<td>284.0</td>
<td>267.0</td>
<td>17.0</td>
<td>0.29</td>
</tr>
<tr>
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<td>30.8</td>
<td>4.1</td>
<td>20</td>
<td>366.0</td>
<td>308.0</td>
<td>58.0</td>
<td>0.49</td>
</tr>
<tr>
<td>30.8</td>
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<td>313.0</td>
<td>62.0</td>
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</tr>
<tr>
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<td>42</td>
<td>10.7</td>
<td>20</td>
<td>214.0</td>
<td>589.0</td>
<td>420.0</td>
<td>169.0</td>
</tr>
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</table>

Table F.29: Initial soil stresses vs. erosion for CPT 9, pier 3, -25 meter NAP

<table>
<thead>
<tr>
<th>$\Delta L$ [m]</th>
<th>$q_{c,cal}$ [MPa]</th>
<th>$q_{c,exc}$ [MPa]</th>
<th>$q_{b,max}$ [MPa]</th>
<th>$R_{b,cal,max}$ [kN]</th>
</tr>
</thead>
<tbody>
<tr>
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<td>7.65</td>
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<td>3.0</td>
<td>0.03</td>
<td>29.9</td>
</tr>
<tr>
<td>0.4</td>
<td>12.16</td>
<td>3.1</td>
<td>0.03</td>
<td>30.9</td>
</tr>
<tr>
<td>1.4</td>
<td>13.93</td>
<td>3.5</td>
<td>0.04</td>
<td>35.4</td>
</tr>
<tr>
<td>0.2</td>
<td>12.56</td>
<td>3.2</td>
<td>0.03</td>
<td>31.9</td>
</tr>
<tr>
<td>Total</td>
<td>0.15</td>
<td>147.6</td>
<td>194.2</td>
<td></td>
</tr>
</tbody>
</table>

Table F.30: Calculation positive shaft friction CPT 9 pier 3 with the erosion to -25 m NAP, with cone reduction ratio 0.25
F.5. Bearing capacity checks based on rules of thumb

F.5.1 Settlements of pile foundations

The settlements of the foundation piles related to the determined loadings can have several causes. The following settlement causes are briefly studied in the following paragraphs.

- Firstly the settlements of the piles them self into the soil,
- secondly the piles deformations under the loads
- thirdly the settlement of the bearing soil layers.

Settlement of a pile

The settlements of the pier relative to the soil could be estimated from the loads-settlements diagram in figure F.2. According to this figure, the maximum settlement is reached for the maximum load on the pile. This maximum settlement of the foundation pile is 40 millimetre for the piles size of $D_{eq} = 0.4$ meter.

Shortening of a pile

The shortening of the concrete pile is determined by the rule of thumb in equation F.44 \cite{Vrijling_2016}. Rewriting these equations to a simple formula for $s_{el}$. Filling in the several parameters, leads to a settlement of approximately 7.7 millimetre.

\[
\sigma = E \cdot \epsilon \quad \sigma = F/A \quad \epsilon = \Delta L/L \quad \rightarrow \quad \Delta L = \frac{L \cdot F}{A \cdot E} \quad \text{(F.44)}
\]

\begin{tabular}{|c|c|c|}
\hline
$q_{c: I, \text{av}}$ & 11.1 & 2.8 [MPa] \\
$q_{c: I, \text{av}}$ & 3.9 & 1.0 [MPa] \\
$q_{c: I, \text{av}}$ & 3.4 & 0.9 [MPa] \\
$q_{b, \text{max.9}}$ & 5.5 & 1.4 [MPa] \\
$R_{b, \text{cal}, \text{max.9}}$ & 873 & 221.9 [kN] \\
\hline
\end{tabular}

Table F.31: Calculation tip bearing capacity in CPT 9 pier 3 with the excavation to -25m.

Settlement of soil layers beneath the pile group

In the Dutch norm a calculation guideline is given for the settlements of the bearing layer underneath the pile tips, which is depicted in equation F.45 \cite{Nederlands_Normalisatie-instituut_2012}. The loaded surface in the calculation is determined by extending the surface at the pile tip level under an angle of 45° to a depth of 4 $\cdot$ $D_{eq}$ beneath the pile tips. see figure F.13.

It was considered that only the vertical forces will lead to plastic settlements of the soil layers beneath the pile foundation. For $F_{fund}$ the vertical design loads are used (ABCD), leading to an overall downward pressure of $\approx 95,000$ kN. The loaded surface area $A_{LP}$ is calculated according to figure F.13. The factor $m^*$ for a ratio of $b_1/b_2 = 17.6/36 \approx 2$ is 0.92 \cite{Nederlands_Normalisatie-instituut_2012}. The soil layer beneath the pile tips is generalised to a sandy, slightly silty soil layer, which has an elasticity of approximately 35,000 kN/m$^2$ according to \cite{Nederlands_Normalisatie-instituut_2012}. With the assumed values, the settlement of the bearing layers is estimated on 89.3 millimetre.
APPENDIX F. STABILITY OF THE HARINGVLIELT BRIDGE PIERS UNDER EXCAVATION

\[ s_2 = \frac{m^* \cdot \sigma'_{v:4D} \cdot 0.9 \cdot \sqrt{A_{4D}}}{E_{ea:gem}} \]  
\[ \sigma'_{v:4D} = \frac{F_{fund}}{A_{4D}} \]  

in which:
- \( s_2 \): Settlement due to compression of bearing layers
- \( m^* \): A factor for the shape of the loaded area 0.92 -
- \( \sigma'_{v:4D} \): The effective soil stress due to \( F_{fund} \)
- \( A_{4D} \): The loaded surface at \( 4 \cdot D_{eq} \) beneath the pile tips 633.6 \( m^2 \)
- \( E_{ea:gem} \): The average elasticity of the soil 35.000 \( kN/m^2 \)
- \( F_{fund} \): Sum of the loadings on the piles of the group 95.000 \( kN \)

Figure F.13: Sketch of the stress redistribution from the foundation piles to the subsoil. \( b_1 = 17.6 \) meter and \( b_2 = 36 \) meter.

Uneven settlement

The potential uneven settlement of the bridge pier should be treated with great care, because of the large horizontal deformation which it will cause at the top of the bridge pier. It was assumed that the horizontal loads will be mainly dynamic and temporary. It was expected that the high loads by the piles will lead to plastic deformations. The uneven settlements were assumed to be sufficiently covered by the reduction of the allowed settlement of the pile tips.

F.5.2 Vertical bearing capacity subsoil

To check the vertical bearing capacity, the subsoil underneath the Haringvlies bridge piers was checked on the vertical stability of the soil layers. The considered area is the same as for the settlements of the foundation layer, see figure F.13. The vertical bearing capacity is determined with equation F.47 [Vrijling et al., 2016]. In the subsequent equations (F.48 to F.52) the several parameters are included.

The found maximum bearing capacity of 238.600 \( kN \) is significant larger than the sum of all the vertical loads (ABCD) of 95.000 \( kN \). Therefore, it was concluded that the subsoil underneath the Haringvlies bridge piers have sufficient bearing capacity in case of the realisation of the open trench.

\[ F_{max} = p'_{max} \cdot A \]  
\[ p'_{max} = c' \cdot N_c \cdot s_c \cdot i_c + q' \cdot N_q \cdot s_q \cdot i_q + 0.5 \cdot \gamma' \cdot N_{\gamma} \cdot s_{\gamma} \cdot i_{\gamma} \]  
\[ q' = \frac{1}{2} \cdot (1 + 1.6) \cdot (22 - 10) = 15.6 kN/m^2 \]
The bearing capacity factors are:

\[ N_c = (N_q - 1) \cdot \cot(\phi') \quad N_q = \frac{1 + \sin(\phi')}{1 - \sin(\phi')} \cdot e^{\pi \tan(\phi')} \quad N_\gamma = 2 \cdot (N_q - 1) \cdot \tan(\phi') \]  

(F.50)

The shape factors are:

\[ s_c = 1 + 0.2 \cdot \frac{B}{L} \quad s_q = 1 + \frac{B}{L} \cdot \sin(\phi') \quad s_\gamma = 1 - 0.3 \cdot \frac{B}{L} \]  

(F.51)

For drained soil the inclination factor are:

\[ i_c = i_q \cdot \frac{N_q - 1}{N_q - 1} \quad i_q = \left(1 - \frac{0.7 \cdot H}{F + A \cdot c' \cdot \cot(\phi')}\right)^3 \quad i_\gamma = \left(1 - \frac{H}{F + A \cdot c' \cdot \cot(\phi')}\right)^3 \]  

(F.52)

In which:

- \( F_{\text{max}} \) Maximum bearing capacity 238,600 kN
- \( p'_{\text{max}} \) Maximum average effective stress 376.5 kN/m²
- \( A \) Effective foundation area 633.6 m²
- \( H \) Horizontal shear force 7190 kN
- \( F \) Exerted force on foundation area 95,091 kN
- \( L \) Length of the foundation area 36 m
- \( B \) Width of the foundation area 17.6 m
- \( c' \) Cohesion 30 kN/m²
- \( \phi' \) Effective angle of internal friction 30°
- \( q' \) Effective stress at depth of the foundation surface 15.6 kN/m²
- \( \gamma' \) Effective volumetric weight soil 12 kN/m²
- \( N_c \) Bearing capacity factor 30
- \( N_q \) Bearing capacity factor 18
- \( N_\gamma \) Bearing capacity factor 20
- \( s_c \) Shape factor 1.10
- \( s_q \) Shape factor 1.24
- \( s_\gamma \) Shape factor 0.85
- \( i_c \) Inclination factor 0.84
- \( i_q \) Inclination factor 0.85
- \( i_\gamma \) Inclination factor 0.79

F.5.3 Rotational stability

For rotational stability of shallow foundations, it was required that the line of action of the resulting force is situated inside the core of the structure, see equation F.53 [Vrijling et al., 2016]. The most unfavourable scenario regarding the rotational stability, would be the one with large horizontal forces and small vertical force (scenarios ULS ABIK, see table 6.3). The line of action of the resulting forces in the discussed load scenario was found the be within the moment center of the subsoil for excavations depths to subsoil levels of -16 meter NAP and less. It was concluded that the rotational stability was sufficient.

\[ e_R = \frac{\sum M}{\sum V} \leq \frac{1}{6} \cdot w \quad e_R = \frac{153,400}{75,800} = 2.02 \ m \leq \frac{1}{6} \cdot 13 = 2.17 \text{meter} \]  

(F.53)

In which:

- \( e_R \) Distance from the moment center to the intersection point m
- \( \sum V \) Sum of the acting vertical forces (Loads A and B) 75,800 kN
- \( \sum M \) Sum of the acting moments (Loads I and K \cdot \text{arm}) 153,400 kNm
- \( w \) Width of the shallow foundation 13 m
APPENDIX F. STABILITY OF THE HARINGVLIET BRIDGE PIERS UNDER EXCAVATION
Appendix G

Cofferdam foundation Haringvliet bridge piers

G.1 Reference projects for deep excavations near existing structures

To create more insight in the feasibility of the cofferdam as a foundation of the Haringvliet bridge piers, two recent building projects are used as reference projects. Firstly to the cofferdam built around the excavated, shallow founded bridge pier of the railroad bridge in the Waal near Lent. Secondly to the North-South line in Amsterdam, where large excavations near pile founded buildings took place. Both projects are discussed in some more detail in the following paragraphs.

Railroad bridge in the river Waal near Lent

The feasibility of a cofferdam as a foundation for bridge piers was already proven in the Room for the River project near Nijmegen. To provide for more room for the river Waal, an ancillary channel was dredged alongside the river. Both the river and the ancillary channel cross the local railroad bridge. To secure the stability of the railroad bridge, the piers of the bridge were encased by diaphragm walls of approximately 23 meters wide and 1.5 meters thick [I-Lent, 2013]. During the construction of the diaphragm walls, the bridge was fully available for operation and the final settlements of the foundation were within the set limits [I-Lent, 2013].

The excavation for the ancillary channel resulted in a difference of approximately 9 meters between the initial and final bed level. The new bottom level is lower than the shallow foundation of the piers of the railroad bridge, as can be seen in figure G.1. No ship collision forces were taken into account in the design of the cofferdam. The main reason for this was, that the commercial ships were not physically able to come in the vicinity of the bridge. However, some wooden fenders are attached to the pier to prevent damages, if recreation yachting collides with the pier.

The main difference between the cofferdam foundation of the bridge piers of the Waal bridge and the Haringvliet bridge is the existing type of foundation. The piers of the Waal bridge have a shallow foundation and while the Haringvliet bridge piers are founded on piles. The second large difference is due to the initial situation. The piers of the Waal bridge were initially land based, which made the installation of the diaphragm walls relatively easy.
APPENDIX G. COFFERDAM FOUNDATION HARINGVLIET BRIDGE PIERS

Figure G.1: Diaphragm wall around the bridge piers of the Waal bridge near Nijmegen to allow for a 9 meter deep excavations [Ingenieursbureau gemeente Rotterdam, 2012]. Left: initial situation; Right: new situation.

North-South line: Deep excavations adjacent to buildings
With the construction of the North-South metro line in Amsterdam, a lot of knowledge was gained about deep excavations adjacent to buildings and the response of the buildings to these deep excavations.

For the metro line in Amsterdam, multiple tunnel sections were constructed near existing building founded on wooden piles (length $\approx$ 14 meter) [Korff, 2012]. The tunnel sections were constructed with the wall-roof method, with excavations up to 30 meter below the initial soil level, see figure G.2a [Korff, 2012]. The walls of these tunnel sections were constructed as diaphragm walls, in soft soils with a high ground water table, at very short distance from existing buildings [Korff, 2012].

By measuring and analysis the deflections of the buildings next to the construction site, a lot of knowledge was gained about the settlements of pile founded buildings near large excavations.

The deflection of a structure depends on the stiffness of the building, the depth of the excavation and the stiffness of the soil according to Goh and Mair (2011) [Korff, 2012]. The deflection of the buildings are related to the soil surface settlements and the settlements at the level of the foundation layers, see figure G.2a. It was found that buildings with deep foundations are more sensitive to intolerable displacements than shallow foundations [Korff, 2009]. However, for buildings with deep foundations, the relative rotations and settlements are about half the deformations of shallow founded buildings [Korff, 2009].

It was found that the presence of a foundation in the soil influences both the horizontal and
the vertical deformations of a soil layer. The presence of a foundation determines the vertical soil settlements and the flexibility of the foundation the horizontal soil deformation \[\text{Korff, 2009}\]. Settlements near deep excavations and adjacent buildings are never two-dimensional \[\text{Korff, 2009}\]. Three dimensional effects can cause either an increase or a reduction of the settlements depending on local conditions \[\text{Korff, 2009}\].

**Conclusion reference projects cofferdam**

From both the reference projects, it was concluded that a design of a cofferdam as a stabilising element of the bridge piers is feasible. The construction of the North-South metro line proves that large excavations near pile founded structures do not necessarily lead to excessive deformations. In both projects, diaphragm walls were applied as the construction material for the retaining walls. However, it was not expected that cofferdams made out of steel are unfeasible, as the loads in the case study of the Haringvliet bridge piers differ significantly from the reference projects. In contradiction to the railroad bridge near the river Waal, the presence of the foundation piles will add both vertical and horizontal stability to the enclosed soil body of the cofferdam. Opposite to the situation at the North-South metro line, no differences in the hydraulic head are present.

**G.2 Required length of the cofferdam walls**

In this appendix, the calculations on the length that is required for the cofferdams is elaborated on. First, to determine the loads, the soil pressure factor $K$ is discussed.

**G.2.1 Soil pressure factor $K$**

For the soil pressure coefficient, multiple methods are described in the Dutch norm, like depicted in the equations G.1 till G.6 \[\text{Nederlands Normalisatie-instituut, 2012}\]. These coefficients also depend on the wall friction angle $\delta$, the internal friction angle $\phi$ and the angle of the soil surface $\beta$.

In general, the used factors are $K_0 = 1$, $K_a = 1/3$ and $K_p = 3$. In a more detailed phase, these factors should be redefined. Filling in the assumed values, gives the factors as depicted in table G.1. It was concluded that the passive soil deformations will be less in the case of the inclined bottom. The factors for the surface loads are the same as for the soil, as the wall inclination ($\alpha$) is zero. $\delta$ is assumed to be zero as well.

The factors used in the calculations are based on the absence of wall friction ($\delta = 0$) and the conditions as depicted in figure G.3.

\[
K_0 = (1 - \sin(\phi')) \quad \text{(G.1)}
\]

\[
K_{0;\beta} = K_0 \cdot (1 + \sin(\beta)) \quad \text{(G.2)}
\]

\[
K_{\gamma;a;k} = \frac{\cos^2(\phi' + \alpha)}{\cos^2(\alpha) \cdot \left(1 + \sqrt{\frac{\sin(\phi' + \beta_a) \cdot \sin(\phi' - \beta_a)}{\cos(\alpha - \beta_a) \cdot \cos(\alpha + \beta_a)}}\right)} \quad \text{(G.3)}
\]

\[
K_{\gamma;p;k} = \frac{\cos^2(\phi' - \alpha)}{\cos^2(\alpha) \cdot \left(1 - \sqrt{\frac{\sin(\phi' + \beta_p) \cdot \sin(\phi' + \beta_p)}{\cos(\alpha - \beta_p) \cdot \cos(\alpha + \beta_p)}}\right)} \quad \text{(G.5)}
\]

\[
K_{\gamma;p;k} = \frac{\cos(\alpha) \cdot \cos(\beta_p)}{\cos(\alpha + \beta_p)} \quad \text{(G.6)}
\]
In which:

- $K_0$: Neutral
- $K_{0,\beta}$: Neutral inclined
- $K_{\gamma,a,k}$: Active for soil pressure
- $K_{\gamma,a,k}$: Active for surface load
- $K_{\gamma,p,k}$: Passive for soil pressure
- $K_{\gamma,p,k}$: Passive for surface load
- $\alpha$: Inclination of the retaining wall
- $\beta$: Inclination of the soil in front of or on top of the retaining wall

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<td>0.50 0.50 0.50 0.50</td>
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<td>0.50 0.40 0.60</td>
</tr>
<tr>
<td>$K_{\gamma,a,k}$</td>
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<td>$K_{\gamma,p,k}$</td>
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<td>1.55 1.32 1.79</td>
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</table>

Table G.1: Soil pressure factors $K$

Figure G.3: Designation of the parameters in a retaining wall calculations [Nederlands Normalisatie-instituut, 2012].

In step 1. of the method the embedded depth and the required length of the retaining walls of the cofferdam are determined, by assuming that the deflection in point $E$ (see figure 7.7) is zero. The deflection of point $E$ is given in equation G.7. Each loading results in a different type of deflection of the retaining cofferdam wall. The contributions of each load to this deflection in point $E$ are determined according to the standard deflection formula’s [Vrijling et al., 2016], and differ per considered load type.

In the following paragraphs the first steps calculation methods by Blum are performed, see chapter
7.2.3. \[ u_1 + u_2 + u_3 + u_4 = 0 \] (G.7)

In which:
- \( u_1 \) Deflection due to anchor force \( T \)
- \( u_2 \) Deflection due to horizontal active soil pressure
- \( u_3 \) Deflection due to horizontal passive soil pressure
- \( u_4 \) Deflection due to distributed load by the Haringvliet bridge piers

G.2.2 Conservative approach: equally distributed top load

In figure G.4 the simplifications for the deflections are depicted. The contributions of each load to this deflection in point \( E \) are depicted in equations G.8 to G.11 according to the standard deflection formula’s [Vrijling et al., 2016].

![Figure G.4: Deflections of the retaining wall at point E due to the several loads.](image)

\[
\begin{align*}
  u_1 &= \frac{T \cdot l^3}{3 \cdot EI} = \frac{T \cdot (h + d - a) \cdot (h + d - a)^2}{3 \cdot EI} \\
  u_2 &= -\frac{q \cdot d^4}{30 \cdot EI} = -\frac{K_{\gamma_\alpha} \cdot \gamma_s \cdot (h + d - a)^5}{30 \cdot EI} \\
  u_3 &= \frac{q \cdot d^3 \cdot (h - a)}{24 \cdot EI} = \frac{K_{\gamma_{\beta \cdot \gamma_s}} \cdot d^5}{24 \cdot EI} + \frac{K_{\gamma_{\beta \cdot \gamma_s}} \cdot d^4 \cdot (h - a)}{24 \cdot EI} \\
  u_4 &= -\frac{q \cdot l^4}{8 \cdot EI} = \frac{q \cdot (l)^3 \cdot (h - a - b)}{6 \cdot EI} = -\frac{q_{\text{pier}} \cdot K_{\text{sur},a} \cdot (d + b)^4}{8 \cdot EI} - \frac{q_{\text{pier}} \cdot K_{\text{sur},a} \cdot (d + b)^3 \cdot (h - a - b)}{6 \cdot EI}
\end{align*}
\] (G.8 - G.11)

In which:
- \( a \) Height from anchor point E to top of the retaining wall
- \( EI \) Bending stiffness retaining wall

The anchor force of equation G.12, followed from the equilibrium of moment around the toe of the sheet pile wall (point \( D \)). After multiplication equations G.8 till G.11 with \( \frac{EI}{K_{\gamma_\beta \cdot \gamma_s}} \) and substituting this in equation G.7, resulted in equation G.13.

\[
\sum M_D = 0 \\
T \cdot (h + d - a) = \frac{1}{6} \cdot K_{\gamma_\alpha} \cdot \gamma_s \cdot (h + d)^3 - \frac{1}{6} \cdot K_{\gamma_{\beta \cdot \gamma_s}} \cdot d^3 + \frac{1}{2} \cdot q_{\text{pier}} \cdot K_{\text{sur},a} \cdot (d + b)^2
\] (G.12)
APPENDIX G. COFFERDAM FOUNDATION HARINGVLIEFT BRIDGE PIERS

\[(u_1 + u_2 + u_3 + u_4) \cdot \frac{EI}{K_{\gamma_p} \cdot \gamma_s} = \frac{K_{\gamma,a} (h + d)^3 \cdot (h + d - a)^2}{18} - \frac{d^4 \cdot (h + d - a)^2}{18} + \frac{K_{\gamma,p}}{K_{\gamma,a}} \cdot q_{pier} \cdot (d + b)^2 \cdot (h + d - a)^2}{6 \cdot \gamma_s}
\]  
\[\frac{K_{\gamma,a} \cdot (h + d - a)^5}{30} + \frac{d^5}{30} + \frac{d^4 \cdot (h - a)}{24} - \frac{K_{\gamma,p}}{K_{\gamma,a}} \cdot q_{pier} \cdot (d + b)^4}{8 \cdot \gamma_s}
\[\frac{K_{\gamma,a} \cdot q_{pier} \cdot (d + b)^3 \cdot (h - a - b)}{6 \cdot \gamma_s} = 0 \quad (G.13)

In which:

- \(K_{\gamma,a,h}\) Active soil pressure coefficient 0.33 [-]
- \(K_{\gamma,p,h}\) Passive soil pressure coefficient 2.11 [-]
- \(K_{\gamma,a}\) Active soil pressure coefficient top load 0.33 [-]
- \(\gamma_s\) Volumetric weight of the soil 21 kN/m³
- \(\gamma_w\) Volumetric weight of water 10 kN/m³
- \(\gamma_d\) Effective volumetric weight of the soil 11 kN/m³
- \(h\) Retaining height cofferdam walls 22 m
- \(a\) Height from anchor point \(E\) to top of the retaining wall 0 m
- \(b\) Height from point \(D\) to action point of \(q_{pier}\) 5.5 m

Equation G.13 was put in an excel-file, together with the known values for the parameters. By iteration, the embedded depth \(d\) and thus the total length of the sheet-pile wall \(L_T\) were determined.

- Theoretical embedded depth cofferdam walls \((d) = 25.1\) meter.
- Total length retaining wall \((L_{tot}) = 52.1\) meter.

G.2.3 Two sided restricted top loads

The top load by the bridge pier can be treated in more detail by assessing the situation by taken the top loads into account over a restricted area. In case a two sided restricted top load is assumed, the equations as discussed in the previous paragraphs change. In figure G.5 the load simplification as presented in the Dutch standards (CUR 166) are included.

The accompanying formula’s are included in equations G.14 and G.15. For simplicity, is the horizontally distributed load was assumed to work at a single point load \(F_{eah,q}\) at a depth \(b\), see equation G.16.

\[e_{eah,q} = \frac{2 \cdot q_{pier} \cdot s \cdot k}{c - a} \quad (G.14)\]
\[k = \frac{\sin(\theta - \phi)}{\cos(\theta - \phi)} \quad (G.15)\]
\[F_{eah,q} = \frac{1}{2} \cdot e_{eah,q} \cdot (c - a) \quad (G.16)\]

Figure G.5: Horizontal soil pressure by two side restricted top load [CUR, 2005].
G.2. Required length of the cofferdam walls

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$F_{eh,q}$</td>
<td>Simplified horizontal point load</td>
<td>161.7 kN</td>
</tr>
<tr>
<td>$e_{eh,q}$</td>
<td>Horizontal soil pressure by two sided restricted top load</td>
<td>191.4 kN/m²</td>
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<tr>
<td>aa</td>
<td>Upper boundary of the horizontal pressure by top load</td>
<td>2.1 m</td>
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<tr>
<td>bb</td>
<td>Depth of the maximum horizontal pressure by top load</td>
<td>2.4 m</td>
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<tr>
<td>cc</td>
<td>Lower boundary of the horizontal pressure by top load</td>
<td>12.1 m</td>
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<tr>
<td>dd</td>
<td>Distance from retaining wall to the top load</td>
<td>3.6 m</td>
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<td>$k$</td>
<td>Auxiliary value</td>
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<td>$L$</td>
<td>Height of the retaining wall</td>
<td>47.1 m</td>
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<td>$q_{pier}$</td>
<td>Top load</td>
<td>193.5 kN/m²</td>
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<td>s</td>
<td>Width of the top load</td>
<td>14.8 m</td>
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<td>$\phi$</td>
<td>Internal soil friction angle</td>
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<tr>
<td>$\theta$</td>
<td>Auxiliary value</td>
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**Retaining wall deflections**

Each loading results in a different type of deflection of the retaining cofferdam wall. The change of the top load from a distributed load to a point load, requires a change in the equations G.8 till G.13. In figure G.6 the revised simplifications of the loads are depicted. In equations G.17 to G.22 the accompanying equations are included.

![Figure G.6: Deflections of the retaining wall at point E due to the several loads.](image)

\[
\begin{align*}
\mathbf{u}_1 &= \frac{T \cdot l^3}{3 \cdot EI} = \frac{T \cdot (h + d - a) \cdot (h + d - a)^2}{3 \cdot EI} \quad (G.17) \\
\mathbf{u}_2 &= -\frac{q^4}{30 \cdot EI} = -\frac{K_{\gamma,p} \cdot \gamma_s \cdot (h + d - a)^5}{30 \cdot EI} \quad (G.18) \\
\mathbf{u}_3 &= \frac{q \cdot d^4}{30 \cdot EI} + \frac{q \cdot d^3 \cdot (h - a)}{24 \cdot EI} = \frac{K_{\gamma,p} \cdot \gamma_s \cdot d^5}{30 \cdot EI} + \frac{K_{\gamma,p} \cdot \gamma_s \cdot d^4 \cdot (h - a)}{24 \cdot EI} \quad (G.19) \\
\mathbf{u}_4 &= -\frac{F \cdot l^3}{3 \cdot EI} - \frac{F \cdot l^2 \cdot (h - b + bb)}{2 \cdot EI} = -\frac{F_{eh,q} \cdot (cc - aa)^3}{3 \cdot EI} - \frac{F_{eh,q} \cdot (cc - aa)^2 \cdot (h - b + bb)}{2 \cdot EI} \quad (G.20)
\end{align*}
\]

The anchor force of equation G.21, followed from the equilibrium of moment around the toe of the sheet pile wall (point D). After multiplication equations G.17 till G.20 with $\frac{EI}{K_{\gamma,p} \cdot \gamma_s}$ and substituting this in equation G.7, resulted in equation G.22.

\[
\sum M_D = 0
\]

\[
T \cdot (h + d - a) = 1/6 \cdot K_{\gamma,a} \cdot \gamma_s \cdot (h + d)^3 - 1/6 \cdot K_{\gamma,p} \cdot \gamma_s \cdot d^3 + F_{eh,q} \cdot (d + b - bb) \quad (G.21)
\]
\[
\left( u_1 + u_2 + u_3 + u_4 \right) \cdot \frac{EI}{K_{\gamma:p} \cdot \gamma_s} = \frac{K_{\gamma:a} \cdot (h + d)^3 \cdot (h + d - a)^2}{18} - \frac{d^3 \cdot (h + d - a)^2}{18} + \frac{1}{K_{\gamma:p}} \cdot F_{\text{eqh},q} \cdot (d + b - bb) \cdot (h + d - a)^2 \\
\frac{K_{\gamma:a} \cdot (h + d - a)^5}{30} + \frac{d^3}{30} + \frac{d^4 \cdot (h - a)}{24} \\
\frac{1}{K_{\gamma:p}} \cdot F_{\text{eqh},q} \cdot (cc - aa)^3 \\
\frac{1}{K_{\gamma:p}} \cdot F_{\text{eqh},q} \cdot (cc - aa)^2 \cdot (h - b + bb) = 0
\] (G.22)

### G.2.4 Results calculations for equally distributed top loads

![Figure G.7: Horizontal loads onto the retaining wall of the cofferdam.](image)

<table>
<thead>
<tr>
<th>Retaining height [m]</th>
<th>Hor. stress [kN/m]</th>
<th>Shear [kN/m]</th>
<th>Moment [kNm/m]</th>
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<th>$F_p$ [kN]</th>
<th>$q_{pier}$ [kN/m]</th>
<th>$T$ [kN]</th>
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Table G.2 continues on the next page.
### G.2. Required length of the cofferdam walls

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<th>Hor. stress</th>
<th>Shear</th>
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Table G.2: Results Blum calculation
Figure G.8: Vertical stress in the retaining wall of the cofferdam.

Figure G.9: Moment stress in the retaining wall of the cofferdam.
Appendix H

Hydrodynamic changes due to the Tilting Lock

In this appendix, the changes in the hydrodynamic situation due to the implementation of the Tilting Lock are studied. The magnitude of the flow velocities is required to determine the rates of erosion and sedimentation.

H.1 Reference situations regarding the hydrodynamics

The Tilting lock is a floating object in a local, suddenly very deep trench with respect to the surroundings. Regarding a fluid mechanical view, the situation could be subdivided into three basics reference projects. The sudden increase in depths can be compared to the access channels of large ports. The blocking effect by the floating Tilting Lock in the main streamline is more complicated, as the streamlines are able to pass the structure either alongside or underneath. The redistribution of the streamlines could be approached by two simplifications: blocking the flow over the complete water depth (for instance a bridge pier) or a partly closed valve, blocking the upper part of the flow.

Access channels

An access channel is comparable to the trench that is required for the Tilting, due to the similar relatively large changes over short stretches in water depth. The blocking effect of the Tilting Lock is not taken into account in the simplification to a channel.

When flows are crossing trenches, the currents will not be able to adapt to the longitudinal velocity component instantly [T. Raaijmakers, 2005]. If the side slopes are steep, the streamlines are not able to stay attached to the bottom, which will cause turbulent recirculation zones, also known as wakes [T. Raaijmakers, 2005]. When the trench has sufficient length (or width) the streamlines will be able to find the new equilibrium, which is sketched in figure H.1. If the length (or width) of the trench is small, the current will not have sufficient length to reattach to the seabed and thus the main current will be 'blown' over the trench [T. Raaijmakers, 2005].

The sediment particles that travel with the streamlines, could be able to settle within the length of the trench, when their settling length is sufficiently short. As the flow velocity inside the trenches often is lower than in the outside zones, the heavier suspended particles would not be able anymore to travel with the streamlines. The locally increased depth of the trench can act as a sand trap.

The length required by the streamlines to reattach to the bottom can be calculated with equation H.1 [Klein, 1999]. Filling in the accompanying values, an adaption length of 4.8 to 5.8 kilometres if found. This is much larger than the length of the required trench for the Tilting Lock (≈ 500 meter).
APPENDIX H. HYDRODYNAMIC CHANGES DUE TO THE TILTING LOCK

\[ \lambda_w = \frac{C^2 \cdot h}{2 \cdot g} \]  

In which:

- \( \lambda_w \) Adaption length \( 4800-5800 \) [m]
- \( C \) Chézy coefficient \( 50-55 \) [m\(^{1/2}\)/s]
- \( h \) Water depth \( 38 \) [m]
- \( g \) Gravitation constant \( 9.81 \) [m/s\(^2\)]

Figure H.1: Velocity profiles in trenches with steep side slopes [T. Raaijmakers, 2005].

Detached bodies in a flow stream

An object that blocks the flow over the complete water depth, redirects the flow in several directions. This 3-D process is depicted in figure H.2a and H.2b. As can be seen, are the streamlines redirected in both horizontal and vertical direction. Downstream of the structure wakes will be created due to the detachments of the streamlines, which cause energy dissipation. The exact redistribution of the flow is not easily to quantify. The difference between this simplification and the situation of the Tilting Lock is the possibility for the streamlines to pass underneath.

(a) Scour around a cylinder (Breusers / Raudkivi, 1991) cited in [Schiereck et al., 2000]. (b) Simplification of the topview flows around a cylinder [Battjes and Labeur, 2009].

Figure H.2: Detached bodies in a flow stream.

Partly closed valve

A partly closed valve blocks the flow in the vertical plane, like is depicted in figure H.3. Due to the reduced cross section area underneath the valve, the flow velocity will increase to maintain the discharge equilibrium. Just like in the case of the bridge pier, turbulent flows arise at the downstream side of the valve because of the detachment of the streamlines. This simplification assumes that all the incoming flow volume will pass the obstruction underneath, which will not be the case for the Tilting Lock. The second discrepancy between this simplification and the Tilting Lock, are the differences in water depths and blocking height by the lock differ per cross section.

Conclusion on the reference projects

As shown in the previous paragraphs, no basic simplification can satisfy for the situation of the deep trench with the Tilting Lock. However, in each situation one common factor came forward: the detachment of the streamlines which will cause energy dissipation. Therefore it was decided to


**H.1. Reference situations regarding the hydrodynamics**

Figure H.3: Flow around sluice gate in canal [Vrijling et al., 2016].

use the energy dissipation over the area of the trench in relation to the initial energy dissipation to estimate the flow rates beneath the Tilting Lock.

### H.1.1 Overview of energy dissipation

To start the determination of the energy dissipation over the Tilting Lock, firstly all the potential energy losses are defined and listed, see figure H.4. Each aspect will be briefly treated in this paragraph. Later on, the two types of energy dissipation which are expected to contribute the most are elucidated in more detail.

1. Inflow losses at the upstream edge of the trench.
2. Outflow losses at the downstream edge of the trench.
3. Inflow losses at upstream edge of the Tilting Lock.
4. Outflow losses at the downstream edge of the Tilting Lock.
5. Bottom friction over the whole stretch.
6. Wall friction along the hull of the Tilting Lock.
7. In- & outflow losses due to the presence of the bridge piers in the basin.
8. In- & outflow losses due to the presence of the fixating structure in the basin.

Figure H.4: Longitudinal cross section of the Tilting Lock and the trench, including several aspects of energy losses.

1. & 2.: In- and outflow at the edges of the trench

Streamlines over steep slopes are likely to have detachment, according to the rules of thumb for inclinations of 1:8 and steeper. The detachments of the streamlines will result in local turbulent circulations of the flow, also called wakes or eddies, which cause dissipation of energy. The applied
inclination of the slopes of the trench are 1:5, which indicates the arises of wakes and thus energy dissipation is to be expected.

In the chapter about slope stability (chapter 5) is recommended that the design of the slopes could be improved by applying milder slope inclinations at the top and the bottom of the slopes. When no bottom protection is applied, it is to be expected that the slopes will redistribute, within three months, in line with the proposed design improvements [T. Raaijmakers, 2005]. With these improvements in mind, it is expected the streamlines will feel a more gradual transition from the horizontal bed to the inclined bed and are therefore less likely to detach from the bed. Therefore, it is concluded that the in- and outflow losses at the edge of the trench could be neglected.

3. & 4.: In- and outflow losses at the edges of the Tilting Lock
At the upstream edge of the Tilting Lock wakes will occur, as the streamlines will not be able to directly follow the sharp-edged sides of the Tilting Lock. A detachment of the streamlines will be initiated, with wakes and thus energy dissipation as result. However, with respect to the wakes at the downstream side, the wakes by the inflow will be relatively small. This is because the streamlines can already ‘feel’ the presence of the object by the increased pressure and therefore can adjust their direction in advance, which makes the transition more smooth.

The detachment of the flow at the downstream side will be more abrupt and a larger wake will be created. The outflow losses are considered to be more significant and are therefore taken into account. For the determination of the outflow losses, the Tilting Lock is simplified to the partly closed valve (figure H.3). In this simplification both the volume, the impulse and the energy balances between the several sections are applied to find the local water depth, energy losses and flow velocities.

5. & 6.: Bottom & Wall friction
The flow along the bottom and the hull of the Tilting Lock will feel friction by these boundaries. In normal free surface flows the friction by the bottom is often neglected for uniform flows. As the wall friction is depending on the water depth and the cross sectional surface of the flow is significantly restricted, the wall friction is expected to cause a significant amount of energy loss over the stretch of the Tilting Lock. For consistency, the bottom friction of the whole reviewed area is taken into account.

7.: Bridge piers
The presence of the bridge piers will have an influence at the total energy dissipation over the area affected by the Tilting Lock. With respect to the current situation, the amount of friction caused by the bridge piers will be increased, as they will become more exposed to the currents, due to the excavation of the trench. However, as the bridge piers are at some distance of the Tilting Lock (≈ 25 meter) and the flow rate underneath the lock is required, the presence of the bridge piers is neglected.

8.: Fixating structure
To keep the Tilting Lock in position, a fixating structure will be designed. For now is expected that the fixating structure will not have severe influences on the current patterns, as the previous design of the fixating structure entails relative slender piles, which have a small effect on the cross sectional surface. The neglecting of the friction by the fixating structure is justified, as the flow rate through the section below the Tilting Lock will be overestimated. It should be remarked that the fixating structure might cause local scour, but that phenomenon is considered to be out of the scope of this thesis.

Conclusion
It was expected that the wall friction and the outflow losses at the downstream side of the Tilting Lock cause the majority of the energy losses. To be able to quantify these aspects, both the wall friction and the outflow losses will be discussed in more detail. This simplifies the energy losses
H.1. Reference situations regarding the hydrodynamics

along a streamline through the Tilting Lock trench to the red line in figure H.4.

H.1.2 Calculations on the geometry

For the calculation of the energy losses, several parameters are required from the geometry of the Tilting Lock and the trench, which will be determined in this section. The basic geometry of the design of the trench and Tilting Lock are included in table H.1. Figure H.5 is used to determine the remaining required parameters, which is used in the calculation of the wall friction losses ($\Delta H_{w;\text{fric}}$).

The hydraulic radius is required to determine the wall friction. As the wall friction consists of a combination of the steel hull and the alluvial bed, for both partition the hydraulic radius should be determined. For the friction by the steel lock that has been done according to equation H.2. The cross sectional surface of the Tilting Lock and the wet perimeter are required, for which the equations are as well depicted in equation H.2. The angle $\alpha$ is determined in figure H.5. The hydraulic radius of the bed of the trench is determined by equations H.3. The surface of the water in the cross section is calculated by equation H.4. The results are depicted in table H.1.

$$P_{\text{lock}} = r_{\text{lock}} \cdot \alpha$$

$$P_{\text{trench}} = 2 \cdot \sqrt{r_{\text{lock}}^2 + \left(\tan(i_s) \cdot r_{\text{lock}}\right)^2}$$

$$A_{\text{lock}} = \frac{1}{2} \cdot r_{\text{lock}}^2 \cdot \left(\alpha - \sin(\alpha)\right)$$

$$A_{\text{trench}} = r_{\text{lock}}^2 \cdot \tan(i_s)$$

$$R_{\text{lock}} = \frac{A_{\text{water}}}{P_{\text{lock}}}$$

$$R_{\text{trench}} = \frac{A_{\text{water}}}{P_{\text{trench}}}$$

$$A_{\text{water}} = w \cdot d_{\text{trench}} - A_{\text{lock}} - A_{\text{trench}}$$

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<td>$i_s$ Slope inclination</td>
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<td>$w$ Width of considered area</td>
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<td>$P_{\text{trench}}$ Wet perimeter trench</td>
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<td>$A_{\text{lock}}$ Cross sectional surface of the Tilting Lock</td>
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Table H.1: Main dimensions and calculated dimensions as to be used for the wall friction losses.
APPENDIX H. HYDRODYNAMIC CHANGES DUE TO THE TILTING LOCK

Geometry required for the outflow losses calculation
To estimate the energy dissipation by the outflow ($\Delta H_{w,of}$) at the downstream edge of the Tilting Lock, the average height of the gap under the valve and the water depths in up- and downstream are required. As for the Tilting Lock and the trench those depth will differ over the width due to the inclined bed and the circular shape of the Tilting Lock, these parameters will be averaged over the reviewed width:

- $d_1$ Water depth in front of Tilting lock $(A_{water} + A_{Lock})/w$ 35.2 m
- $d_2$ Gap beneath valve / Tilting Lock $A_{water}/w$ 7.3 m

H.2 Results from the hydrodynamic calculations
H.2.1 Preliminary calculations
The found input values and the formulae drawn are used to calculate the actual energy dissipation and local flow velocities by a first calculation. The approach was discussed in chapter 8.1.1.

$$\Delta H_{w;trench} = \Delta H_{w;fric} + \Delta H_{w,of} \quad \& \quad \Delta H_{w;trench} = \Delta H_{w;bar} \quad (H.5)$$

In which:
- $\Delta H_{w;trench}$ Energy dissipation of streamline through Tilting Lock trench [m]
- $\Delta H_{w,bar}$ Energy dissipation of a streamline in the undisturbed Haringvliet [m]
- $\Delta H_{w;fric}$ Energy loss by wall friction in the Tilting Lock trench [m]
- $\Delta H_{w,of}$ Energy loss by outflow at downstream edge of the Tilting Lock [m]

Energy dissipation in undisturbed areas Haringvliet estuary $\Delta H_{w;bar}$
The reference value of the energy dissipation in the undisturbed areas of the Haringvliet estuary was determined according to the method as described in chapter 8.1.1.

$$\Delta H_{w;bar} = i_{w,bar} \cdot L \quad (H.6)$$

$$u = C \cdot \sqrt{R_{Har} \cdot i_{w,bar}} \quad \Rightarrow \quad i_{w,bar} = \frac{(u/C)^2}{R_{Har}} \quad (H.7)$$

$$C = 5.75 \cdot \sqrt{g \cdot \log(12 \cdot R_{Har}/k)} \quad (H.8)$$

In which:
- $i_{w,bar}$ Friction gradient 4.3 $\cdot 10^{-6}$ [-]
- $u$ Flow velocity 0.35 m/s
- $k$ Nikardse value for alluvial material 0.046 - 0.14
- $C$ Chézy value 51 $[m^{1/2}/s]$  
- $R_{Har}$ Hydraulic radius of the Haringvliet estuary 8 m
- $L$ Length of reviewed area 500 m
- $H_{w,bar}$ Energy dissipation in the undisturbed Haringvliet 2.9 $\cdot 10^{-3}$ m

Wall friction $\Delta H_{w;fric}$
The wall friction under the Tilting Lock is depending on the found flow velocity in equation H.13. The calculation of the applicable friction due to the bed and hull according to chapter 8.1.1, is depicted in equations H.9 and H.10.

$$\Delta H_{w;fric} = u^2 \cdot L_{lock} \left\{ \frac{c_{f,steel}}{g \cdot R_{lock}} + \frac{c_{f,bot}}{g \cdot R_{trench}} \right\} \quad (H.9)$$

$$\frac{1}{\sqrt{c_f}} = 5.75 \cdot \log \left( \frac{12 \cdot R}{k} \right) \quad c_f = \left( 5.75 \cdot \log \left( \frac{12 \cdot R}{k} \right) \right)^2 \quad (H.10)$$
In which:

According to chapter 8.1.1, the applicable equations are included in equations H.11 till H.17.

With the geometric parameters known, the energy dissipation by the outflow can be determined

\[ \Delta \text{Outflow losses} \]

The energy dissipation is equal to the initial dissipation. This calculation is performed in the Matlab

the Tilting Lock, could be found by iterating the flow rate and the wall friction till the calculated

underneath, but a lot of the flow will pass alongside. The amount of water that will flow underneath

the section, it is concluded that not the complete incoming discharge will pass the Tilting Lock

As both contributions of to the friction depends on the flow velocity and thus the discharge through

the section under the Tilting Lock is assumed too high, as the caused friction is about twice the initial energy dissipation \((\Delta H_{w,in})\).

As both contributions of to the friction depends on the flow velocity and thus the discharge through

the section, it is concluded that not the complete incoming discharge will pass the Tilting Lock

underneath, but a lot of the flow will pass alongside. The amount of water that will flow underneath

the Tilting Lock, could be found by iterating the flow rate and the wall friction till the calculated

energy dissipation is equal to the initial dissipation. This calculation is performed in the Matlab

calculation of section H.2.2.

\[
\Delta H_{w,trench} = \Delta H_{w,fric} + \Delta H_{w,o.f} = 9.6 \cdot 10^{-4} + 1.7 \cdot 10^{-2} = 1.8 \cdot 10^{-2} \text{meter}
\]

\[
\Delta H_{w,trench} \gg H_{w,in} \quad \rightarrow \quad 1.8 \cdot 10^{-2} m \gg 9 \cdot 10^{-3} m
\]

**H.2. Results from the hydrodynamic calculations**

The results of the Matlab calculations for the different water levels and discharges are depicted in

tables H.2 till H.6.

**H.2.2 Results Matlab calculations**

The results of the Matlab calculations for the different water levels and discharges are depicted in

In which:

- **\( u \)** Flow velocity 0.35 m/s
- **\( L_{lock} \)** Length over which friction works 90 m
- **\( \epsilon_{f,steel} \)** Friction coefficient for the Tilting Lock 1.2 \( \cdot \) 10\(^{-3} \)
- **\( \epsilon_{f,trench} \)** Friction coefficient for the trench bottom 2.8 \( \cdot \) 10\(^{-3} \)
- **\( R \)** Hydraulic radius material i \((A/P)\) m
- **\( A \)** Area \( m^2 \)
- **\( P_i \)** Wet perimeter material i m
- **\( k_{steel} \)** Nikuradse k-value for steel 0.0005 -
- **\( k_{bottom} \)** Nikuradse k-value for alluvial material 0.046 -
- **\( \Delta H_{w,fric} \)** Energy loss by wall friction in the Tilting Lock trench 9.6 \( \cdot \) 10\(^{-4} \) m

**Outlet losses \( \Delta H_{w,o.f} \)**

With the geometric parameters known, the energy dissipation by the outflow can be determined

according to chapter 8.1.1. The applicable equations are included in equations H.11 till H.17.

<table>
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<tr>
<th>Balance</th>
<th>Equations</th>
<th>Results</th>
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<tr>
<td>( q_0 = d_0 \cdot u_0 )</td>
<td>( q_0 = 2.8 \cdot 0.35 m^2/s )</td>
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<tr>
<td>( q_1 = q_0 )</td>
<td>( u_1 = q_1 / d_1 )</td>
<td>(H.12)</td>
</tr>
<tr>
<td>( q_2 = q_0 )</td>
<td>( u_2 = q_2 / a_2 )</td>
<td>(H.13)</td>
</tr>
<tr>
<td>( H_1 = H_2 )</td>
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<tr>
<td>( F_2 = F_3 )</td>
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<td>(H.17)</td>
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<tr>
<td>( H_3 = d_3 + \frac{u_3^2}{2 \cdot g} )</td>
<td>( \Delta H_{w,o.f.} = 0.017 m )</td>
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</table>

**Result preliminary calculation**

From the preliminary calculation could be concluded that the flow rate through the section under the Tilting Lock is assumed too high, as the caused friction is about twice the initial energy dissipation \((\Delta H_{w,in})\).

As both contributions of to the friction depends on the flow velocity and thus the discharge through the section, it is concluded that not the complete incoming discharge will pass the Tilting Lock underneath, but a lot of the flow will pass alongside. The amount of water that will flow underneath the Tilting Lock, could be found by iterating the flow rate and the wall friction till the calculated energy dissipation is equal to the initial dissipation. This calculation is performed in the Matlab calculation of section H.2.2.
APPENDIX H. HYDRODYNAMIC CHANGES DUE TO THE TILTING LOCK

<table>
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<th>u₂</th>
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Ratio's: 50.6% 16.8% 75.1% 8.1%

Table H.2: Water level: -0.40 meter NAP; Keel clearance: 2.5 meter; Maximum water depth: 36.5 meter.

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Ratio's: 52.9% 16.5% 75.6% 7.9%

Table H.3: Water level: -0.20 meter NAP; Keel clearance: 2.7 meter; Maximum water depth: 36.7 meter.

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Ratio's: 60.7% 15.5% 77.1% 7.5%

Table H.4: Water level: +0.46 meter NAP; Keel clearance: 3.4 meter; Maximum water depth: 37.4 meter.
### H.2. Results from the hydrodynamic calculations

#### Table H.5
Water level: +1.63 meter NAP; Keel clearance: 4.5 meter; Maximum water depth: 38.5 meter.

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<th>$\Delta H_{w_{o.f.}}$</th>
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<td>0.272</td>
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Ratios: 75.7% 14.2% 79.2% 6.6%

#### Table H.6
Water level: +2.60 meter NAP; Keel clearance: 5.5 meter; Maximum water depth: 39.5 meter.

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<td>0.04</td>
<td>0.08</td>
<td>0.01</td>
<td>0.033</td>
<td>0.01</td>
<td>0.32</td>
<td>0.29</td>
<td>0.041</td>
<td>0.006</td>
<td>0.033</td>
<td>0.002</td>
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<td>668.0</td>
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<td>0.32</td>
<td>0.02</td>
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<td>0.02</td>
<td>0.64</td>
<td>0.57</td>
<td>0.161</td>
<td>0.022</td>
<td>0.130</td>
<td>0.009</td>
</tr>
<tr>
<td>1252.5</td>
<td>0.15</td>
<td>1.13</td>
<td>0.03</td>
<td>0.123</td>
<td>0.03</td>
<td>1.20</td>
<td>1.07</td>
<td>0.563</td>
<td>0.076</td>
<td>0.454</td>
<td>0.033</td>
</tr>
<tr>
<td>1670.0</td>
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<td>2.00</td>
<td>0.04</td>
<td>0.163</td>
<td>0.04</td>
<td>1.60</td>
<td>1.43</td>
<td>1.001</td>
<td>0.136</td>
<td>0.807</td>
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<td>3.13</td>
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<td>0.204</td>
<td>0.05</td>
<td>2.00</td>
<td>1.79</td>
<td>1.563</td>
<td>0.212</td>
<td>1.260</td>
<td>0.092</td>
</tr>
<tr>
<td>2505.0</td>
<td>0.30</td>
<td>4.50</td>
<td>0.06</td>
<td>0.245</td>
<td>0.06</td>
<td>2.40</td>
<td>2.14</td>
<td>2.251</td>
<td>0.305</td>
<td>1.814</td>
<td>0.132</td>
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<td>2922.5</td>
<td>0.35</td>
<td>6.13</td>
<td>0.07</td>
<td>0.286</td>
<td>0.07</td>
<td>2.80</td>
<td>2.50</td>
<td>3.063</td>
<td>0.415</td>
<td>2.468</td>
<td>0.180</td>
</tr>
<tr>
<td>3340.0</td>
<td>0.40</td>
<td>8.00</td>
<td>0.08</td>
<td>0.326</td>
<td>0.08</td>
<td>3.20</td>
<td>2.86</td>
<td>4.001</td>
<td>0.542</td>
<td>3.224</td>
<td>0.235</td>
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<tr>
<td>3757.5</td>
<td>0.45</td>
<td>10.13</td>
<td>0.09</td>
<td>0.367</td>
<td>0.09</td>
<td>3.60</td>
<td>3.22</td>
<td>5.063</td>
<td>0.686</td>
<td>4.080</td>
<td>0.298</td>
</tr>
</tbody>
</table>

Ratios: 89.3% 13.5% 80.6% 5.9%
APPENDIX H. HYDRODYNAMIC CHANGES DUE TO THE TILTING LOCK

Figure H.6: Plotted relations between the discharges in the undisturbed Haringvliet and the trench.

Figure H.7: Plotted relations between the flow velocities in the undisturbed Haringvliet and the trench.
Appendix I

Morphology in the Tilting Lock trench

I.1 Parameters required for the sediment balance

In this appendix multiple parameters for the sediment balance in equation I.1 [Bosboom and Stive, 2013] and figure I.1 are discussed into more detail. The objective of these elaborations was to come to a reliable and conservative estimation of these factors for the estimation of the accretion in the trench for the Tilting Lock.

\[
S_{in} - D + E = S_{out} \quad \text{(I.1)}
\]

\[
\frac{\delta z_{bed}}{\delta t} = \frac{D - E}{\rho_s \cdot A_{bed}} \quad \text{(I.2)}
\]

In which:

- \( S_{in} \) Total transport rate \( kg/s \)
- \( S_s \) Suspended transport rate \( kg/s \)
- \( S_b \) Bed load transport rate \( kg/s \)
- \( D \) Deposited sediment particles \( kg/s \)
- \( E \) Eroded sediment particles \( kg/s \)
- \( \frac{\delta z_{bed}}{\delta t} \) Change in bed level elevation \( m/s \)
- \( \rho_s \) Density of the sediment particles \( 1000 \frac{kg}{m^3} \)
- \( A_{bed} \) surface of the bed in the considered area \( m^2 \)

I.1.1 Types of sediment transport

There are roughly two types of sediment transport: bed load transport and suspended transport, see figure I.2.

**Suspended load transport**

The suspended particles will travel along the streamlines with the flow velocity. For not too high sediment concentrations and not too heavy particles, it can be assumed that at every height the particles move through a vertical plane with the horizontal water velocity and might travel for long distances [Bosboom and Stive, 2013]. Suspended particles are therefore expected to almost never touch the bed [Bosboom and Stive, 2013]. Along the streamline, the sediment particles are falling with a speed depending on the size and shape of the particle (the so-called fall velocity \( u_s \)) [Bosboom and Stive, 2013]. When the actual bed shear stress is (much) larger than the critical
APPENDIX I. MORPHOLOGY IN THE TILTING LOCK TRENCH

Figure I.2: Different modes of sediment transport. A: Bed load at small shear stresses. B: Sheet flow (often considered as bed load at higher shear stresses). C: Suspended load. From Fredsoe and Deigaard (1992). [Bosboom and Stive, 2013]

bed shear stress, the particles will be lifted from the bed and brought in suspension.

In the analysis of the morphological situation of the complete Haringvliet estuary in chapter 3.2.4, it is found that the suspended solids mainly consist of very fine, non-flocculated silt particles, which might settle in the trench of the Tilting Lock. As the settling length of these very fine particles is much longer than the length of the pit, it was expected that the main part of the suspended fine particles will be transported along the streamlines and therefore would not settle down within the stretch of the trench. However, some of the non-flocculated silt particles will be able to settle within the trench. In contradiction to the bed load transport, it is expected that those deposited silt particles will be brought in suspension again and to travel with the flow with high discharges. In this way, the trench will have a flushing character for the (very) fine sediment particles.

Bed load transport
The bed load transport of sediment particles depends on the stability of a single grain. When the actual bed shear stress exceeds a critical value, the particle will leave the bed and travel for a relatively short distance [Bosboom and Stive, 2013]. Many approaches are available to determine whether particles will move, but the majority depends on the Shields parameter. If the actual Shields parameter is below the critical value, the transport by bed load is not expected on a large scale [Bosboom and Stive, 2013]. Local scour still could be present, but is less interesting when sought after the permanent accretion into the trench. For grains on slopes the stability is less than for a horizontal bed, as particles are expected to tumble down the slope easily [Bosboom and Stive, 2013]. In contradiction, the particles are not expected to move upwards a slope.

Bed load is to be expected during high discharges, as this coincide with high flow velocities and thus with high shear stresses. It was assumed that the particles that are set in motion, will be transported by bed load transport into the trench for the Tilting Lock, where they will settle. The import of sediment particles into the trench by the bed load was considered to be an irreversible process, as the particles were not expected to leave the pit again by bed load transport.

Conclusion
Bed load transport will only occur if the flow velocities are sufficiently high. For the simplified calculations was assumed that the bed load transport of sediments was out of the scope of this thesis. This because the maximum flow velocity in the general areas of the Haringvliet area (max. 0.21 m/s, see chapter 3.2.2) were not expected to exceed the boundary value for erosion as used in the MER studies (0.4 m/s [Van Wijngaarden and Ludikhuize, 1997]). Therefore, it as considered that the in- and outgoing sediment transport rates of equation I.3 only depend on suspended sediment transport.

\[ S = c \cdot u \]  
(I.3)
I.1. Parameters required for the sediment balance

In which:

\( S \)  Total sediment transport rate  \( kg/(m^2 \cdot s) \)
\( c \)  Sediment concentration  \( mg/l \) or \( kg/m^3 \)
\( u \)  Local flow velocity  \( m/s \)

I.1.2 Suspended sediment concentration

Rouse concentration distribution

In reality, both the flow velocity and the sediment concentration differ over the height \( z \) of the water column. In this thesis is assumed that the flow velocity is distributed equally over the height. The distribution of the concentration to be used depends on the reviewed situation.

The concentration distribution over the height of the water column is determined by the Rouse distribution, see equation I.4 [Bosboom and Stive, 2013]. The reference height \( a \) is the height of measurement, which is assumed to be 1.5 meter below NAP. The Rouse number \( z^* \) is determined by equation I.5 and is used to determine which kind of sediment transport is to be expected, according to table I.1 [Bosboom and Stive, 2013].

\[
C(z) = C_a \left\{ \frac{h - z}{z} \right\} \left( \frac{a}{a - z} \right)^{z^*}
\]

\[
z^* = \frac{w_s}{\kappa \cdot u_s}
\]

As the main part of the sediment transport at the Haringvliet bridge will be suspended, the Rouse factor should be below 1.2. When the values for the fall velocity of table 3.5b are used in equation I.5, it was found that the Rouse number will never exceed 0.8 for the lower flow velocities.

The most unfavourable situation for the concentration distribution, is when the majority of the concentration is near the bottom. From equation I.4 the following could be concluded:

- Smaller Rouse numbers will lead to lower concentrations near the bottom.
- Smaller water depths will lead to lower concentrations near the bottom.
- Higher concentrations lead to higher concentrations near the bottom.
- Higher reference levels lead to higher concentrations near the bottom.

The used concentration distribution is considered to be sufficient conservative, when the values for the different parameters of equation I.4 are used as depicted in table I.2.
APPENDIX I. MORPHOLOGY IN THE TILTING LOCK TRENCH

Table I.2: Selected values for the rouse distribution parameters.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concentration $c$</td>
<td>12 mg/l</td>
</tr>
<tr>
<td>Rouse number $z_*$</td>
<td>0.8</td>
</tr>
<tr>
<td>Water depth $h$</td>
<td>8 meter</td>
</tr>
<tr>
<td>Reference height $a$</td>
<td>6.5 meter</td>
</tr>
</tbody>
</table>

I.1.3 Settling length

The required length for the particles to settle (figure I.3) is given by equation I.6 and depends on the water depth, the flow velocity of the water and the fall velocity of the suspended solids. This formula describes the length that is required for a sediment particle in the upper layer of the water column to settle at the bottom. Sediment particles which are in a lower part of the water column will settle within this length.

\[
L = u \cdot \frac{h}{w_s}
\]

In which:

- $L$: Settling length [m]
- $u$: Flow velocity [m/s]
- $h$: Water depth [m]
- $w_s$: Fall velocity [m/s]

The smallest values for the settling length are related to the highest fall velocity, the smallest flow velocity and the smallest water depth, as could be seen in figure I.4. The larger the particles, the higher the fall velocity, the shorter the settling length. This means that for similar incoming sediment concentrations, more particles are settled over the same stretch when the sediment particles are larger.

From the incoming sediment concentration, it is not likely that all the sediment particles will have sufficient time to settle within the area of the Tilting Lock. The majority of the suspended particles will be blown over the trench. When the fall velocity and the settling length are combined and compared to the length of the trench, the amount of sediment that will be able to settle within the area can be determined.
The most unfavourable situation is found for the lowest flow velocities under the Tilting Lock in combination with the length of the whole trench. The lowest flow velocities are found for the 5 percentile discharge (150 m$^3$/s) and the lowest water level, which results in a flow velocity of 0.014 m/s.

It was assumed that the flow will instantly follow the bed level. The effect of the slopes on the settling length is almost negligible for the fall velocity of the non-flocculated silt particles. This because the vector for the falling sediment particles is only marginal affected by the slope inclination (less than 0.1$^\circ$). This makes sense, as the lowest flow velocity ($= 0.014$ m/s) is significantly larger than the fall velocities (between $w_s = 0.003 \cdot 10^{-3} - 0.289 \cdot 10^{-3}$m/s). Therefore, it was considered to be justified to take into account the whole length of the area of the trench (410 meter).

**Ratio of settling length over the trench length**

In table I.3 the results are presented. It could be concluded that in the least favourable situation for the flow velocities, 15.1% of the particles will be able to settle, in case the suspended particles are indeed non-flocculated silt and with the assumed concentration distribution by the Rouse number. This value is independent of the height of the water column, as is referred to the length of the basin.

The percentage of deposited sediments will decrease for higher flow velocities, as the settling height over the fixed distance will decrease. In addition, the percentage of deposited sediments will increase for larger sediment particles. So taking the percentage for the lowest flow velocity will be a conservative approach.

<table>
<thead>
<tr>
<th>Low discharge (flow velocity of 0.014 m/s)</th>
<th>Fall velocities</th>
<th>0.003 mm/s</th>
<th>0.116 mm/s</th>
<th>0.289 mm/s</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fall velocities</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Falling heights for fixed settlement length of 410 meter</td>
<td>0.089 m</td>
<td>3.51 m</td>
<td>8.77 m</td>
<td></td>
</tr>
<tr>
<td>Percentages D of $S_{in}$ $z^* = 0.8$, $h = 8$ m, $a = 6.5$ m,</td>
<td>15.1 %</td>
<td>88.3 %</td>
<td>100 %</td>
<td></td>
</tr>
</tbody>
</table>

Table I.3: Falling heights for the lowest flow velocities, over the length of the trench (410 meter).

**I.2  Erosion**

Erosion takes place when particles are removed from the bed and is therefore closely related to the previously described bed load transport, as both are depending on the stability of a single grain particle. Multiple methods to assess the stability of the grain particles are available, but the most used is the Shields parameter.

The objective of this paragraph is to determine whether it could be expected that particles are set in motion. As the changed flow velocities in the vicinity of the Tilting Lock are overestimated, realistic values for the amount of erosion could not be determined.

**I.2.1 Erosion rate**

As stated before, the amount of erosion (or the erosion rate) is not computed in this thesis, as the flow velocities are all overestimations. However, with analysing whether erosion might occur for the overestimated velocities a first estimate on the flushing could be performed.

In the MER study by [Van Wijngaarden and Ludikhuize, 1997] a formula for calculating the erosion rate is proposed, see equation I.7. In this formula the erosion flux depends on the ratio between the actual flow velocity and the critical flow velocity. The used critical velocity for erosion for all kinds of sediments was $u_{c,cr}$ 0.5 m/s and the erosion rate constant $M$ is defined as 0.5 kg/m$^2$/day [Van Wijngaarden and Ludikhuize, 1997].

This could be related to the amount of discharge through the area and the occurrence in time of this discharge. When the lowest flow velocity is taken, the approach is conservative (lower than in


APPENDIX I. MORPHOLOGY IN THE TILTING LOCK TRENCH

reality).

\[
F_e = M \cdot \left( \frac{u}{u_{e,cr}} \right)^2 - 1 \quad \text{for } u > u_{e,cr} \\
F_e = 0 \quad \text{for } u \leq u_{e,cr}
\]  

(I.7)

\[
F_e \quad \text{erosion flux} \\
u \quad \text{Local flow velocity} \\
M \quad \text{Erosion rate parameter}
\]

In which: \( u_{e,cr} \quad \text{Critical flow velocity for erosion} \)

\[
M = 0.5 \quad \text{kg/(m}^2\cdot\text{day)} \\
M = 5.79 \cdot 10^{-3} \quad \text{kg/(m}^2\cdot\text{s)}
\]

Figure I.5: Sedimentation rates for different critical flow velocities for sedimentation for the alternative approach.

**Erosion parameter \( M \)**

Erosion parameter \( M \) was assumed to be 0.5 kg/(m\(^2\)·day) in the studies on the effects of the Kierbesluit on the morphology of the South-Western delta [Van Wijngaarden and Ludikhuize, 1997]. As the same area was reviewed in this thesis, it was assumed that this value for the erosion parameter \( M \) was sufficiently reliable. In further research, the value of the parameter should be refined according to the specific local conditions at the Haringvliet bridge.

**I.2.2 Shields parameter**

The rate of erosion is determined by the Shields parameter \( \theta \) (equation I.8) and its critical value \( \theta_{cr} \) (equation I.9) [Bosboom and Stive, 2013]). When the shear stress caused by the flow is higher than a critical value, the particles will be set in motion and thus erode. For larger sediment particles a larger critical value for the Shields parameter will be found. The stability of particles on slopes differ from the stability for a horizontal bed, which is included in the factor \( K(\alpha) \), which is explained into more detail later on. The critical Shields parameter for particles on a slope will be less.

\[
\theta = \frac{u_*^2}{\Delta g D_{50} K(\alpha)} \\
\theta_{cr} = \frac{u_{*,cr}^2}{\Delta g D_{50} K(\alpha)}
\]  

(I.8) (I.9)

In which:

\[
\theta \quad \text{Shields parameter} \\
\theta_{cr} \quad \text{Critical Shields parameter} \\
u_* \quad \text{Shear stress velocity}^1 \\
u_{*,cr} \quad \text{Critical shear stress velocity} \quad \text{m/s} \\
\Delta \quad \text{Relative density} \quad 1.6 \\
D_{50} \quad \text{Diameter soil particles} \quad 22.1 \quad \mu m
\]

The Shields parameter depends on the dimensionless Reynolds number for grain particles \( Re_* \), see equation I.10. With both the Shields parameter and the Reynolds number calculated, it could be determined whether the sediment particles will be set in motion or not with figure I.6.

\[
Re_* = \frac{u_* D_{50}}{\nu}
\]  

(I.10)

In which:

\[
Re_* \quad \text{Dimensionless Reynolds number} \quad [\cdot] \\
\nu \quad \text{Kinematic viscosity coefficient} \quad 10^{-6} \quad \text{m/s}
\]

\(^1\)For the occurring \( u_* \) in a flow regime the following rule of thumb is assumed: \( u_* = 0.05 \cdot u \). Assumption made during a conversation with dhr. Van Prooijen, 12 February 2016.
As could be seen in the formula’s, both the Reynolds number and the Shields parameter depend on the shear stress velocity \( u_* \). Therefore an iteration is required to find the actual critical value of the Shields parameter. By iteration, the critical flow velocity can be determined.

Filling in the formula for our case (\( D_{50,avg} = 22.1 \ \mu m \)), leads to a \( \theta_{cr} = 0.44 \), which will be used as starting point for the calculation. With the assumed critical Shields parameter \( \theta_{cr} = 0.44 \) for a horizontal bed \( (K(\alpha/\gamma) = 1) \), the \( u_{*,cr} \) will be \( 0.012 \ m/s \) using equation I.8. The dimensionless Reynolds Number for these values would be \( Re_* \approx 0.27 \), which in the diagram of figure I.6 corresponds with a critical Shields parameter \( \theta_{cr} \approx 0.44 \), which validates the approach and the assumed Shields parameter.

\[ \theta_{cr} = \theta_{cr} = 0.44, \]  \( \theta_{cr} = 0.44 \) & \( \theta_{cr} = 0.44 \)

**Stability of particles on slopes**

The stability of particles on slopes are less than for a horizontal bed. The reduction factor should be determined from the graph in figure I.7. This \( K(\alpha) \) or \( K(\alpha/\gamma) \) (see equation I.11 and I.12) is the correction factor for the diameter of the sediments particles and has therefore to be applied in the denominator of equation I.8 [Schiereck et al., 2000]. As could be seen, will the flow velocity parallel to the slopes have the most reduced particle stability. So, for particles on slopes, the critical value of the Shields parameter is lower.

\[ K(\alpha) = \frac{F(\alpha)}{F(0)} = \frac{\sin(\theta - \alpha)}{\sin(\theta)} \]  \( K(\alpha) = \frac{F(\alpha)}{F(0)} = \frac{\sin(\theta - \alpha)}{\sin(\theta)} \)

\[ K(\alpha/\gamma) = \frac{F(\alpha/\gamma)}{F(0)} = \sqrt{1 - \frac{\sin^2(\alpha)}{\sin^2(\theta)}} \]  \( K(\alpha/\gamma) = \frac{F(\alpha/\gamma)}{F(0)} = \sqrt{1 - \frac{\sin^2(\alpha)}{\sin^2(\theta)}} \)

In which:

- \( K(\alpha) \) Reduction factor in direction of the flow
- \( K(\alpha/\gamma) \) Reduction factor perpendicular to the flow
- \( F(0) \) Critical flow force or the strength
- \( F(\alpha) \) Strength in the direction of the flow
- \( F(\alpha/\gamma) \) Strength perpendicular to the flow
- \( \alpha \) Angle of repose
- \( \theta \) Internal friction angle soil particles
- \( N \) Newton

It could be concluded that the particles on the slopes are more prone to erode. For this thesis the erosion of particles on the slopes is considered to be out of the scope, as this will be only part of
APPENDIX I. MORPHOLOGY IN THE TILTING LOCK TRENCH

the preliminary redistribution of the slope. It is not expected that the erosion of particles on the
slopes will significantly contribute to the permanent accretion of the trench.

![Figure I.7: Influence of slope on stability [Bosboom and Stive, 2013].](image)

I.2.3 Critical Shields parameters

The critical Shields parameters are determined by iterating the formula’s presented in equation I.13 for the two types of sediments and for both the horizontal and inclined bottom.

Filling in the formula for our case ($D_{50,avg} = 22.1 \mu m$), leads to a $\theta_{cr} = 0.44$, which will be used as starting point for the calculation. With the assumed critical Shields parameter $\theta_{cr} = 0.44$ for a horizontal bed ($K(\alpha/\uparrow) = 1$), the $u_{*,cr}$ will be 0.012 m/s using equation I.8. The dimensionless Reynolds Number for these values would be $Re_* \approx 0.27$, which in the diagram of figure I.6 corresponds with a critical Shields parameter $\theta_{cr} \approx 0.44$, which validates the approach and the assumed Shields parameter.

$$\theta_{cr} = \frac{u_{*,cr}^2}{\Delta g D_{50} K(\alpha/\downarrow)} \quad Re_* = \frac{u_{*,cr} \cdot D_{50}}{\nu} \quad u_{*,cr} = 0.05 \cdot u_{cr}$$ (I.13)

<table>
<thead>
<tr>
<th>$D_{50}$</th>
<th>$\theta_{cr}$</th>
<th>$Re_*$</th>
<th>$u_{*,cr}$</th>
<th>$u_{cr}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>22.1 $\mu m$</td>
<td>hor. bed</td>
<td>0.42</td>
<td>0.27</td>
<td>0.0121</td>
</tr>
<tr>
<td>22.1 $\mu m$</td>
<td>incl. bed</td>
<td>0.50</td>
<td>0.23</td>
<td>0.0105</td>
</tr>
<tr>
<td>110 $\mu m$</td>
<td>hor. bed</td>
<td>0.09</td>
<td>1.37</td>
<td>0.0124</td>
</tr>
<tr>
<td>110 $\mu m$</td>
<td>incl. bed</td>
<td>0.10</td>
<td>1.16</td>
<td>0.0105</td>
</tr>
</tbody>
</table>

Table I.4: Critical parameters for particles to be set in motion.

The particles that were in suspension, will have lower critical flow velocities to be set in motion again. These are 0.25 m/s at a horizontal bed and 0.2 m/s at an inclined bed. A flow velocity of 0.35 m/s occurs yearly in the area beneath the Tilting Lock, see table H.5. From this could be concluded, that the trench will have a flushing character. It is expected that the suspended particles will be sufficiently stirred up to travel with the streamlines out of the trench. The permanent bottom particles have a larger diameter and therefore a larger Shields parameter.

Reference

The critical erosion flow velocity for non-flocculated silt used in the MER studies is 0.4 m/s [Van Leeuwen et al., 2004]. The assumed critical velocities above are therefore considered to be an underestimation, as particles are expected to be set in motion earlier and therefore result in
I.2. Erosion

more erosion and thus intensify the flushing character. Therefore it might be concluded that the expectation of a flushing character is even more exaggerated. The maximum flow velocity under the Tilting Lock is 0.367 m/s in the overestimation of appendix H, which would indicate that the flushing character would not be present.

Critical erosion velocity $U_{ecr}$

The stability of a particle could be assessed by the stability parameter by Shields ($\theta$). When the flow velocity of the streamline exceeds the critical Shields parameter ($\theta_{cr}$), the bottom particles will be set in motion. Appendix I.2 describes the calculated shields stability parameters for more detailed elaboration.

The critical Shields parameters are defined for the $D_{50}$’s of both the suspended sediment particles and the particles at the current bottom, see table I.5. In this way it could be assessed if the newly accreted sediment particles are likely to be eroded again and whether it is expected that the existing bottom will erode under the changed flow regime.

The used critical velocity for erosion for all kinds of sediments was $u_{ecr} = 0.5$ m/s in the calculations for the influences of the Kierbesluit [Van Wijngaarden and Ludikhuize, 1997]. In this thesis the critical flow velocity was determined for both the suspended sediment particles and the particles in the bottom of the Haringvliet estuary. The critical flow velocity was determined by the critical Shields parameter (see appendix I.2.2) and the available data on particle sizes, see section 3.2.4.

<table>
<thead>
<tr>
<th>$D_{50}$</th>
<th>$\theta_{cr}$</th>
<th>$Re_s$</th>
<th>$u_{+cr}$</th>
<th>$u_{cr}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>22.1 $\mu m$</td>
<td>hor. bed</td>
<td>0.42</td>
<td>0.27</td>
<td>0.0121</td>
</tr>
<tr>
<td>22.1 $\mu m$</td>
<td>incl. bed</td>
<td>0.50</td>
<td>0.23</td>
<td>0.0105</td>
</tr>
<tr>
<td>110 $\mu m$</td>
<td>hor. bed</td>
<td>0.09</td>
<td>1.37</td>
<td>0.0124</td>
</tr>
<tr>
<td>110 $\mu m$</td>
<td>incl. bed</td>
<td>0.10</td>
<td>1.16</td>
<td>0.0105</td>
</tr>
</tbody>
</table>

Table I.5: Critical parameters for particles to be set in motion.

With the assumed $D_{50}$ of 110 $\mu m$ for the particles in the current bottom of the Haringvliet, the particles will show motion when the flow velocity is between 0.249 m/s. For the deposited sediments particles, a similar value is found ($D_{50}$ of 22.1 $\mu m = 0.241$ m/s appendix I.2.3). As could be seen, are both critical erosion flow velocities approximately the same. 0.24 m/s will be used in the calculations.
APPENDIX I. MORPHOLOGY IN THE TILTING LOCK TRENCH
Appendix J

Costs of the implementation of the Tilting Lock

The costs of the design of the trench for the Tilting Lock at the case study location at the Haringvliet bridge were determined according to rules of thumb based on quantities. An overview of the cost estimations are depicted in figure J.1. The construction costs were estimated on the following aspects.

- **Dredging works**
  - As contaminated soil is present in the Haringvliet estuary, it was assumed that one fourth of the dredged material will be contaminated.
  - The assembling costs of the different dredging materials were considered to be negligible in relation to the costs of the bulk quantities.

- **Bottom protection**
  - To include additional safety in the design against (local) erosion, bottom protection was assumed for 1/4 of the surface of the trench for the Tilting Lock.
  - The assembling costs of the construction equipment were considered to be negligible in relation to the costs of the bulk quantities.

- **Cofferdam: the combi walls**
  - For the cost estimation, it was assumed that a king profile (HZ 1080M C, \( w = 34.625 \text{ cm}^3/\text{m} \)) would be sufficient for the retaining wall. The weight of these combi wall elements was approximately 700 kg/m² [ArcelorMittal, 2016].
  - The total length of the retaining wall taken into account is 40 meters, which coincides with the calculations in the optimisation of the retaining wall calculations (chapter 7.2.4).
  - Labour on the cofferdam regarding the installation of the 10 meter long elements of the combi wall was taken into account. This included welding of two stretches together and the pressuring of the combi walls into the subsoil.
  - Per 1 meter stretch retaining wall, approximately 3 meter steel retaining wall will be present due to the shape of the king walls.
  - For the required material for the tension rings it was assumed that 5 tension rings were required per cofferdam, weighing approximately of 100 kg/m.
  - Coating was applied on the surface of the combi walls to increase the lifetime and to prevent and/or reduce the corrosion of the steel combi walls. The coating was applied on the outside surface of the cofferdam (≈ 25 meters of height).

- It was assumed, that the constructing works on the cofferdam from the water will lead to an increase of costs up to a maximum of 200% [CUR, 2005].
- Percentages were taken into account for uncertainties in the design and the contractors cut.
### Figure J.1: Cost estimation of the elaborated trench + cofferdam alternative

<table>
<thead>
<tr>
<th>Dimension</th>
<th>Cost (€)</th>
</tr>
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<tbody>
<tr>
<td>Total construction costs</td>
<td>340,000</td>
</tr>
<tr>
<td>Design uncertainties (10%)</td>
<td>35,400</td>
</tr>
<tr>
<td>Total costs</td>
<td>20,500,000</td>
</tr>
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</table>

<table>
<thead>
<tr>
<th>Cost Component</th>
<th>Quantity</th>
<th>Unit Cost</th>
<th>Total Cost</th>
</tr>
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<tbody>
<tr>
<td>Cofferdam</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Appendix K

Scaling of the dimensions of the Tilting Lock

This research on the scaling of the Tilting Lock provides insight in the relation between the different parameters that are important for the size of the Tilting Lock and the surrounding structures, like the trench that is required for the implementation of the Tilting Lock. In future research, these relations can be used to optimise the desired size Tilting Lock, the impacts on the surroundings and the costs of the implementation.

For smaller sized Tilting Locks, the dimensions of the required trench will decrease. The benefits of the Tilting Lock would decrease as well, as the added height to the head clearance of the Tilting Lock will be lower as well. Therefore, the influences of the radius of the Tilting Lock are studied in relation to the other different dimensional parameters of the Tilting Lock.

K.1 Approach of the scaling calculations

In the following paragraphs, the different assumptions and formulae as used in this thesis are discussed. It was expected that by analysing the effects of different radii and tilting angles, an optimal size of the Tilting Lock can be found. This in relation to the required added air draft for the vessels, the feasible excavation depths and the estimation of the costs and benefits of the Tilting Lock.

The added air draft by the Tilting Lock to the head clearance of the vessels is mainly depending on the tilting angle and the radius of the Tilting Lock. For larger angles and radii, higher elevation differences could be obtained. The different sizes of the Tilting Lock can be related to boundary conditions for the implementation of the Tilting Lock at a particular bridge location. All the parameters are eventually linked to the added air draft by the Tilting Lock ($2 \cdot \delta h$).

K.1.1 Assumed dimensions

As the Tilting Lock consists of multiple parameters and infinite variations in the layout could be thought off, the scope of this research was limited by assuming several parameters to be constant:

- Structural gauge internal channels ($w_{\text{channel}} = 6$ meter).
- Radius of the internal channel = 5.6 meter (diameter = 11.2 meter)
- Walls under the same angle as the allowed tilting of the lock ($\theta = 22^\circ$).
- Additional height of the outer hull to prevent over topping (1.5 meter).

K.1.2 Calculation formula’s

Below the different formula’s for the several dimensions of the basic variant of the Tilting Lock are depicted. These parameters are depicted in sketches of the Tilting Lock in figure K.1. The values
depicted in the figures are for a Tilting Lock with a radius of $R_{t,l.} = 28$ meters, a tilting angle of $\theta = 22^\circ$ and the assumed values in the previous paragraph.

The following parameters are of the highest interest:

- $\delta h$: Is the elevation difference in water levels in the internal water channels of the Tilting Lock between the neutral and the tilted position of the Tiling Lock ($\delta h = \sin(\theta) \cdot M$).
- $T$: is the maximum height of the Tilting Lock section underneath the bridge.
- $K$: is the maximum height of the Tilting Lock sections outside the bridge.
- $U$: is the required draft of the Tilting Lock, without the required Under Keel Clearance (UKC) of 4 meters.

\[
\begin{align*}
A &= \sin(\theta) \cdot R_{t,l.} + \delta h \cdot \cos(\theta) \\
B &= \frac{\text{Additional height}}{\cos(2 \cdot \theta)} \cdot \cos(\theta) \\
C &= \tan(\theta) \cdot A \\
D &= (R_{t,l.} - \cos(\theta) \cdot R_{t,l.}) + \sin(\theta) \cdot \delta h \\
E &= R_{t,l.} - D \\
F &= R_{t,l.} \cdot \cos(2 \cdot \theta) \\
G &= \sin(2 \cdot \theta) \cdot \delta h \\
H &= R_{t,l.} - F \\
I &= \sin(2 \cdot \theta) \cdot R_{t,l.} + \delta h \cdot \cos(2 \cdot \theta) \\
J &= \frac{\text{Additional height}}{\cos(2 \cdot \theta)} \\
L &= \frac{w_{\text{channel}} / 2}{\cos(\theta)} \quad \text{(K.1)} \\
M &= R_{t,l.} - C - D - L \quad \text{(K.2)} \\
K &= I + J - \delta h \quad \text{(K.3)} \\
N &= M - L \quad \text{(K.4)} \\
O &= \frac{N}{\tan(\theta)} \quad \text{(K.5)} \\
P &= O \cdot \cos(\theta) - \delta h \quad \text{(K.6)} \\
Q &= \sin(\theta) \cdot O \quad \text{(K.7)} \\
R &= \sin(2 \cdot \theta) \cdot (Q + D1) \quad \text{(K.8)} \\
S &= (w_{\text{channel}} / 2 + \text{Additional height}) \cdot \cos(2 \cdot \theta) \quad \text{(K.9)} \\
T &= R + s - \delta h \quad \text{(K.10)} \\
U &= \delta h + R_{t,l.} \quad \text{(K.11)}
\end{align*}
\]
K.1. Approach of the scaling calculations

Figure K.1: Designation of dimension parameters of the Tilting Lock.
K.2 Results scaling analysis

In the following paragraphs, the results of the analyses are discussed. It was found that Tilting Locks with a radius smaller than 11 meters, are not feasible for the assumed sizes for the structural gauge and the internal sailing channels. This is due to the interference of the two internal channels (negative values for parameter $Q$).

K.2.1 Parametric relations for the Tilting Lock

The two parameters that were considered to have the most influence on added air draft by the Tilting Lock (the diameter of the Tilting Lock $R_{t.l.}$ and the tilting angle $\theta$), were varied in the previously mentioned formula’s.

Radius Tilting Lock

In figure K.3 the radius of the Tilting Lock is plotted in relation to the obtained elevation difference ($\delta h$), the maximum height of the Tilting Lock underneath a bridge ($T$), the maximum height of the Tilting Lock outside a bridge ($K$) and the draft of the Tilting Lock ($U$). It was found that the relation between a larger radius of the Tilting Lock to the reviewed parameters is linear, see figure K.3.

Figure K.3: Different dimensional parameters in relation to different radii of the Tilting Lock, with a constant tilting angle ($\theta = 22^\circ$). The added air draft is $2 \cdot \delta h$. 
**K.2. Results scaling analysis**

**Tilting angle of the Tilting Lock**
In figure K.4 the rotation angles of the Tilting Lock are plotted in relation to the earlier mentioned parameters. From this graphs was concluded that there is an optimum for the tilting angle in relation to the obtained added air draft for the vessels. For the Tilting Lock with a diameter of 28 meters, this optimum will be somewhere 25°.

![Tilting angle Tilting Lock](image)

Figure K.4: Different dimensional parameters in relation to different tilting angles of the Tilting Lock, with a constant radius of the Tilting Lock ($R_{t.l.} = 28$ meters). The added air draft is $2 \cdot \delta h$.

**K.2.2 Size of the trench that is required for the Tilting Lock**
In figure K.5 the required excavations in relation to the radius of the Tilting Lock are depicted. In figure K.6 these depths are related to excavation volumes for the Tilting Lock trench. These figures are specific for the case study location of the Haringvliet bridge and the assumed slope inclination of 1:5. For other case study locations, the figures will differ due to differences in water depths and geometry of the bridge.

The green line in figure K.5 depicts the maximum depth of the trench that is required for the Tilting Lock, and coincides with the summation of the parameter $U$ (see previous paragraphs) and the required UKC of 4 meters for the Tilting Lock (see chapter 4.2.1).

In figure K.6 the relation is depicted between the radius of the Tilting Lock and the volume of the excavations required to dredge the trench for the Tilting lock. As is shown in the trend line of figure K.6, is the increase of the trench volume exponential in relation to the radius of the Tilting Lock.
APPENDIX K. SCALING OF THE DIMENSIONS OF THE TILTING LOCK

Figure K.5: The required excavations depths around the first and the second bridge piers to provide for sufficient UKC for Tilting Locks with different radii.

Figure K.6: The excavation volumes for the trench required to provide for sufficient UKC for Tilting Locks with different radii.
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