# Conceptual design of the Guard Lock for Strandeiland Flood Defence

## Master Thesis

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TU Delft

Gemeente Amsterdam

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by


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# Preface

Over the past fourteen months, I have undertaken a comprehensive study intersecting hydraulic engineering and professional sports. I aimed to meet the requirements for a Master's degree in Hydraulic Engineering at Delft University of Technology, a journey for which I am thankful.

This research, initiated by the Engineering Department of Amsterdam and jointly refined by Dr. Ing. M.Z. Voorendt, Dr. Ir. Cong Mai Van, and me, allowed me to deepen my understanding of my discipline. The majority of this work took place at the Engineering Department of Amsterdam, under the guidance of Ir. J. Jansen & Ir. T. Post. Their insight and expertise have been integral to my thesis.

Balancing a detailed thesis with my responsibilities as a professional field hockey player has been demanding. Competing in the Dutch Hoofdklasse league and the national squad while concurrently focusing on my research posed a set of challenges.

In this period, beyond my academic pursuits, I represented my country at the World Indoor Championship in South Africa, where we secured a commendable position. Additionally, I made my debut in the national field team, aiding in our success in the FIH Pro League.

I appreciate the support and guidance from TU Delft throughout this process. I extend my gratitude to Dr. Ir. Cong Mai Van, Dr. Ing. M.Z. Voorendt, and Dr. P. Taneja for their consistent assistance. I'm also grateful to Ir. J. Jansen, Ir. T. Post, and Ir. K. Ko for their patience and significant contributions to my research.

Looking back, these experiences have contributed to my personal and professional development. The journey has been demanding, yet the sense of achievement has been rewarding.

*B. Burkhardt*
*Delft, September 2023*

# Abstract

In response to Amsterdam's housing shortage, IJburg on the east side of Amsterdam, is being developed. IJburg consists of six artificial islands in total on which about 20,000 homes for 50,000 residents are being built. Strandeiland (Beach island in English), located in the lake called IJmeer, is one of these artificial islands which is currently being developed to accommodate approximately 20,000 residents. This project aims to introduce approximately 8,000 homes. However, the establishment of Strandeiland presents hydraulic safety concerns, especially with the presence of extreme low water levels and fluctuations due to wind set-up and set-down.

Strandeiland will feature an inland water with high recreational value for residents. This inland water will also provide access for recreational boating. Wind set-down and set-up can create extreme water level differences in this inner water. These water level differences are unfavorable and create danger to the stability of the island. These fluctuations pose a severe challenge to the stability and functionality of the island's water infrastructure. Two primary solutions were evaluated: the adaptation of the inner quay wall or the implementation of a guard lock. Making the inner quay wall suitable for the water level differences brings several implications:

- **Restrictions on utilities:** Because the inner quay wall would be marked as primary flood defence, no pipes and cables would be allowed inside the wall.
- **Design constraints:** When the quay wall serves as the primary flood defence, construction on its inner slope and tree planting is prohibited. This would negatively impact the aesthetics of the waterfront and limit the housing construction space.
- **Increased height requirements:** The inner wall has to be higher according to primary flood defence regulations, this would escalate construction costs.
- **Higher strength sheet piles:** A higher strength of sheet piles has to be used to withstand extremely low water levels which would also lead to escalating construction costs.

Adapting the Inner Quay Wall, while feasible, introduces significant design and functional constraints. Because of these constraints, this option is less desirable and implementing a guard lock is the favorable solution to solve the water level fluctuations.

The guard lock as part of the primary flood defence controls extreme water level variations, ensuring Strandeiland's water infrastructure's integrity. The Guard Lock not only modifies the primary flood defence location, reducing its length considerably but also mitigates the constraints associated with an adapted inner quay wall. The water level spread is set from the program of requirements at NAP -0.6 meters and NAP +0.1. The current estimate is that this will require the lock to close 10 times a year, which is considered acceptable. A movable bridge is integrated which functions both as a neighborhood connector and a part of the beachside boulevard.

The main objective during the design phase is to develop a Guard Lock concept for Strandeiland that facilitates the management of water level fluctuations while accounting for potential failure mechanisms. Dimensions for the Guard Lock were determined based on the standard vessel, leading to lock chambers measuring 22 meters in length and 7.6 meters in width. Anticipating approximately ten annual guard lock closures in extreme scenarios, the F/E system is not included initially. To facilitate a possible future inclusion of a F/E system, the core dimensions are based on two lock chambers resulting in an overall structural length of 36.46 meters.

Different gate designs were evaluated for the guard lock design: Mitre Gates, Rolling/Sliding gates, Lift gate (submersible), Lift gate (upward direction), Radial gates and Single-leaf gates. The first step was to check whether the different gate types were suitable for the situation in which Strandeiland is, looking at space and and vertical clearance. The lift gate in upward direction and the rolling sliding gate

were considered unsuitable for this purpose. The elevator gate limits vertical clearance and the rolling sliding gates takes to much space besides to the lock. The remaining gate types were measured in a multi-criteria analysis, which resulted in a double set of mitre gates ensuring both-way retention. Because of safety and energy efficiency, but especially because of local knowledge and possibilities for maintenance, electromechanical driving mechanisms were chosen for the gates instead of an hydraulic driving mechanism.

Key design elements of the core construction of the guard lock consist of the concrete structure consisting of the walls and the floor slab. After performing the stability checks for the bearing capacity, overturning and piping the strength calculations are done for the floor slab and the walls of the concrete structure. The reinforcement calculations are providing detailed reinforcements for the floor slab and walls.

The pile foundation is another main component of the structure. The vertical bearing capacity check indicates that no pile foundation is needed, but because non uniform settlement is expected a pile foundation will be included. Three different layers were evaluated to check which one was the best fit for the guard lock of Strandeiland. As the first layer is determined to be suitable to bear the load of the structure, this layer has been used. This is the most cost effective for the pile foundation design, as there is less material needed for the pile foundation design. The calculations show that 45 piles with an diameter of 0.8 m each satisfies the need to prevent non-uniform settlements under the structure.

The last component which has been determined is the gate height. The process for determining gate height utilized Reliability-Based Design (RBD) principles. Reliability-Based Design (RBD) ensures the gate height is optimal in terms of cost and safety. In this way an effective design for the gate height is obtained. A comprehensive fault tree analysis, combined with a Monte Carlo analysis, established the final gate height at 5.05 meters corresponding to NAP + 1.55 m, as shown in Figure 1.

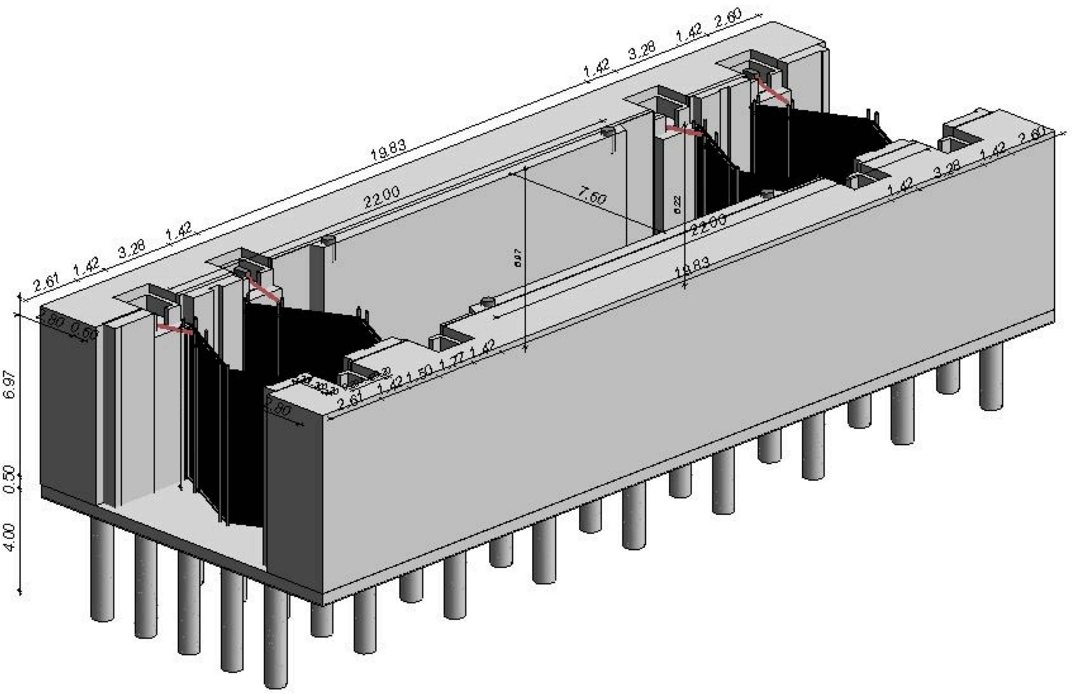



**Figure 1:** 3D Conceptual Model of the Guard Lock Design for Strandeiland

# Contents

# 1
# Introduction

## 1.1. Background

In 1996, the municipality of Amsterdam decided to create living area to partly solve the housing shortage in the city by creating artificial islands. IJburg, on the east side of Amsterdam, consists of six islands on which about 20,000 homes for 50,000 residents are being built. The first phase with Steigereiland, Rieteiland and Haveneiland is nearing completion. The second phase is now under development and consists of Centrumeiland, Strandeiland and Buiteneiland. Strandeiland is located in the Markermeer, a controlled lake which is separated from the North Sea by the Afsluitdijk The location of the Strandeiland compared to Markermeer and Amsterdam are shown in Figure 1.1.


**Figure 1.1:** Location of Strandeiland and the lock (Google Maps, 2022)

The reason for constructing artificial islands was the lack of space to realise this inside the city. This growing demand did not stop after constructing these islands so in 2010 the plan was made for further expansion which led to the idea of IJburg 2. One of the islands which is part of the plans from 2010 is Strandeiland. Strandeiland will consist of a total of around 8,000 homes for around 20,000 residents. All the islands that are part of IJburg are indicated in Figure 1.2.

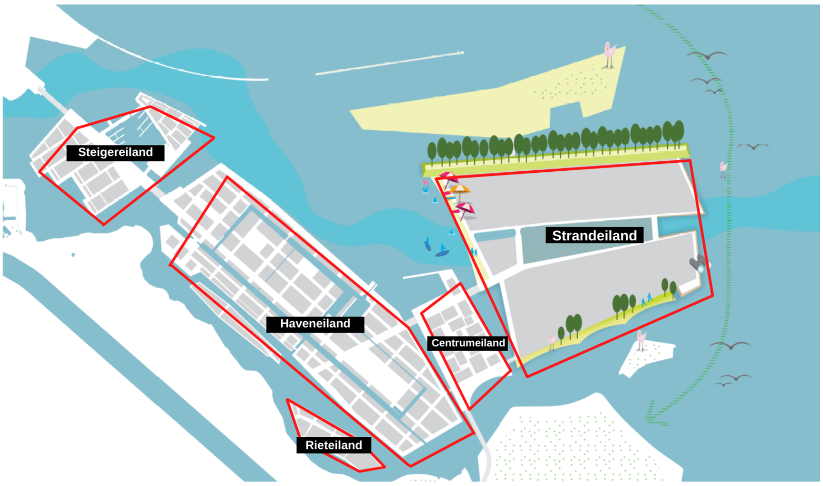



**Figure 1.2:** Indication of islands of IJburg 1 & 2

Due to the financial crisis in 2008, this plan was not executed. Due to its popularity, Amsterdam continued to grow by an average of 11,000 inhabitants each year. The municipality of Amsterdam wants to facilitate this growth and at the same time relieve the pressure on the housing market. The ambition is to enable the construction of 50,000 homes within the city boundaries by 2025. The locations where this can happen are described in Koers 2025, the municipal development strategy for the medium term. This plan put pressure behind the implementation of IJburg 2.

In the spring of 2017 the land reclamation of Strandeiland started and the first residential construction will start in spring 2024. The expectation is that the construction of the first homes will start on the southern part of Strandeiland, the Muidenbuurt. In 2025 the first people should start to live on the island. The island will feature a harbor linked to an inland water body, providing significant recreational opportunities for its future residents.

## 1.2. Problem Analysis

The Amsterdam Engineering Department is currently seeking solutions to address significant water level variations affecting the inland water system of Strandeiland. The inland waterway commences with a harbor known as de Havenkom, situated directly behind the lock. This harbor creates a series of compartments, including the IJburgbaai, Havenkom, and het Oog, as illustrated in Figure 1.3.
Wind set-up and set-down events are causing significant water level fluctuations at Strandeiland, especially extremely low water levels in the inland water due to wind set-down. These fluctuations pose a severe challenge to the stability and functionality of the island's water infrastructure.

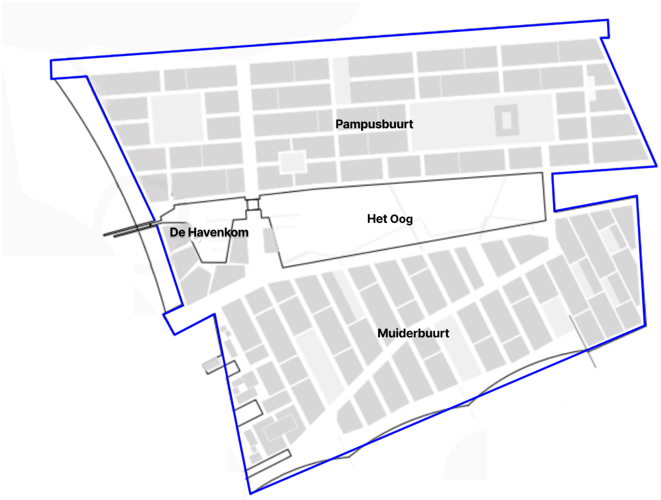



**Figure 1.3:** Locations of Het Oog, De Pampusbuurt, De Muiderbuurt, & De Havenkom(Waterrecreatie Advies BV, 2021)

To mitigate these water level differences, two primary options are being considered: Adapting the Inner Quay Wall and Implementing a (Guard) Lock.

**Adapting the Inner Quay Wall**

If the quay wall of the inner water is dimensioned to withstand these water level differences, it would be categorized as a primary flood defence structure. In Figure 1.4 the red line indicates how the primary flood defence of Strandeiland would look like. This situation would provide an open connection between the outer water and the inner water of Strandeiland without the possibility of closure.

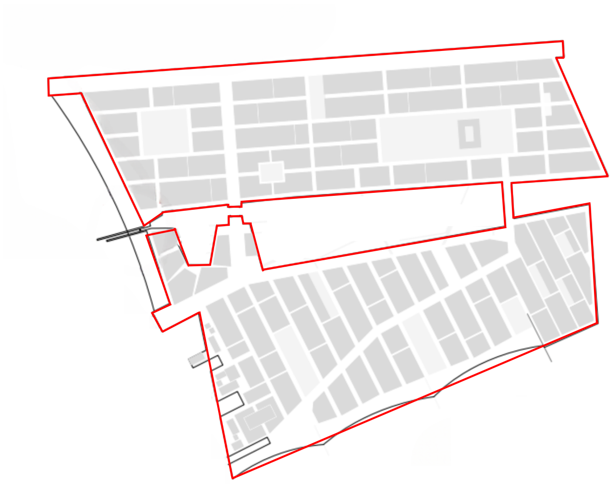


**Figure 1.4:** Solution where the quay wall of the inner water acts as primary flood defence (red line)

This designation would lead to several implications, which are further elaborated below.

1. **Restrictions on Utilities:**
   The structure does not permit the inclusion of cables and pipes, leading to challenges for utility planning and routing in the adjacent areas. This restriction is in line with the NEN 3651:2020, which specifies additional requirements for pipelines in or near crucial hydraulic structures. According to this standard, laying a pipeline longitudinally in or on a water barrier, or within the theoretical profile of such a barrier, is not permissible. (NEN3651, 2020)

2. **Design Constraints:**
   The main objective of constructing Strandeiland is to address the acute housing shortage in Amsterdam. As such, maximizing the space on the island for housing units and related infrastructure is paramount. However, there are several limitations to realizing this when the quay wall is marked as primary flood defence. Construction on the inner slope of the primary flood defence is not allowed and imposes significant spatial constraints. This not only limits design versatility but could also negatively impact the aesthetics and user experience of the waterfront.
   The prohibition on planting trees, further compounds these limitations. Trees play a vital role in urban areas, providing not only aesthetic value but also ecological benefits like air purification, shade, and habitat for birds and insects. Their absence could make the environment less appealing and less resilient to urban heat islands. Het Oog, recognized as a valuable recreational green area, will contain less green space than the situation when the quay wall is not marked as primary flood defence. Given the constraints, there might also be a challenge in providing sufficient amenities and green spaces for residents, impacting the overall livability of Strandeiland.

3. **Increased Height Requirements:** The quay wall is required to be higher in the situation when it is marked as primary flood defence, affecting the visual and spatial aspects of the adjacent areas and possibly escalating construction costs. An estimate would be a 65 cm height increase of the full length of the inner quay wall, which corresponds to a NAP + 1,65 m. This height increase is illustrative, provided by the Engineering Department of Amsterdam. Important to note that this is not the definitive height applied throughout the report. (Oord et al., 2023).

4. **Higher Strength Sheet Pile:**
   The wall would need to be constructed with a stronger type of sheet pile to ensure stability during extreme low water levels. This would likely increase construction complexities and costs significantly. As confirmed in an internal email correspondence with ir. Timor Post, using a heavier sheet piling that would be required to make the inner quays suitable would mean a price increase of 18.3%. This does not account for the 65 cm height increase required since it acts as a primary flood defence. Excluding the height increase, deeper excavation, and other modifications, this 18.3% increase translates to an added cost of €5 million euros. However, a rough estimate that includes these additional factors could lead to a price surge of €10-15 million euros. This is in contrast to the scenario with the inner quay wall not acting as primary flood defence, which would not necessitate a height increase or the use of stronger sheet pile types.[1]

Given these stringent conditions and implications, designating the inner quay wall as a primary flood defence is not considered desirable.

#### Implementing a (Guard) Lock

As an alternative to adapting the inner quay wall, implementing a guard lock at the entrance of the inland water would also serve as a flood defence measure. This approach would alter the location of the primary flood defence, significantly shortening its length, as illustrated by the red line in Figure 1.5.

[1] Email titled "FW: Kostenindicatie kademuren binnenwater" from Groot, Frits (Frits.Groot@sweco.nl) to ir. Timor Post, Technisch Manager, Waterbouw, Ingenieursbureau, Gemeente Amsterdam. The email, dated 14 March 2022, discusses cost estimates for the aforementioned adaptations.

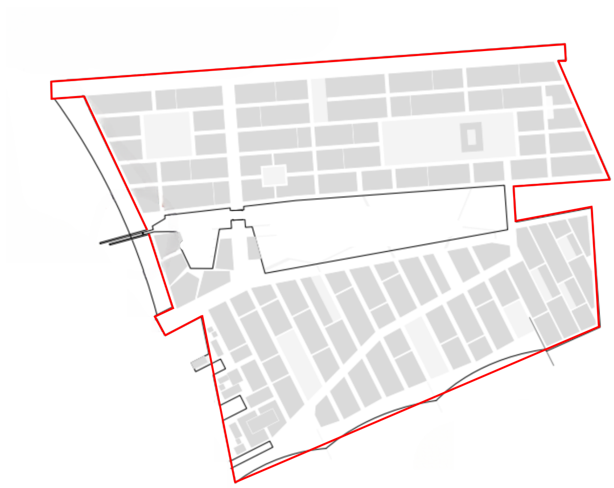


**Figure 1.5:** Solution where the guard lock acts as primary flood defence (red line)

The guard lock, as part of the water defence system, would effectively manage and mitigate extreme water level fluctuations caused by wind set-up and set-down, thereby safeguarding both the structural and functional integrity of Strandeiland's water infrastructure without the substantial constraints that come with designating the inner quay wall as a primary flood defence. The feasibility of incorporating a filling and emptying system in the guard lock could be further explored later in this report, assessing its necessity and potential benefits.

## 1.3. Problem Statement

The Amsterdam Engineering Department is confronted with the challenge of managing significant water level variations in Strandeiland's inland water system, originating from events such as wind set-up and set-down. These fluctuations threaten the structural and functional integrity of the island's water infrastructure. The department is evaluating two primary solutions:

- Adapting the Inner Quay Wall to act as a primary flood defence. While this approach could provide an open connection between the outer and inner waters of Strandeiland, it presents substantial constraints. These include restrictions on utilities, design constraints that potentially limit the island's housing and recreational capabilities, heightened requirements for the quay wall, and increased construction costs.
- Implementing a (Guard) Lock at the entrance of the inland water. This alternative aims to serve as a flood defence measure, potentially shortening the length of the primary flood defence and managing water level fluctuations without the extensive limitations associated with the first solution.

The central problem is to determine the most effective and efficient solution to counteract the extreme water level variations, ensuring the stability and functionality of Strandeiland.

## 1.4. Design objective

As outlined in the problem analysis, designating the inner quay wall as a primary flood defence is not considered to be a desirable solution. Therefore, the primary purpose of this thesis is to facilitate the development of a conceptual design for the guard lock. The design should not only handle potential failure mechanisms but also provides access to the inner water of Strandeiland. A special emphasis will be placed on addressing potential failure mechanisms like overtopping, utilizing tools such as probabilistic analysis and a top-down fault tree analysis. The design objective can be stated as follows:

> "To develop a conceptual guard lock design for Strandeiland that effectively mitigates extreme water level variations"

## 1.5. Approach

This section describes the methodological approach used to obtain the conceptual design. The standard design method is used, which systematically transforms a problem statement into a practical solution. This method, inspired by the applied engineering design cycle from General Hydraulics Engineering Design Principles (Molenaar and Voorendt, 2022), is crucial in accomplishing the final design and structuring the report.

### 1.5.1. Design Method

The systems engineering design method as described by Molenaar and Voorendt (2020) presents a well-structured strategy to unearth the optimal design solution, consistent with the outlined design objectives. Given the unique and intricate nature of civil engineering infrastructures, this method offers a meticulous process of evolution from a broad conceptualization to a more granulated design, which aligns with the trajectory of this report.

The aforementioned methodology by Molenaar and Voorendt (2020) has been modified to better serve the needs of the design objective. The method unfolds through seven key phases which are connected to the chapters in this thesis, as demonstrated in Figure 1.6.

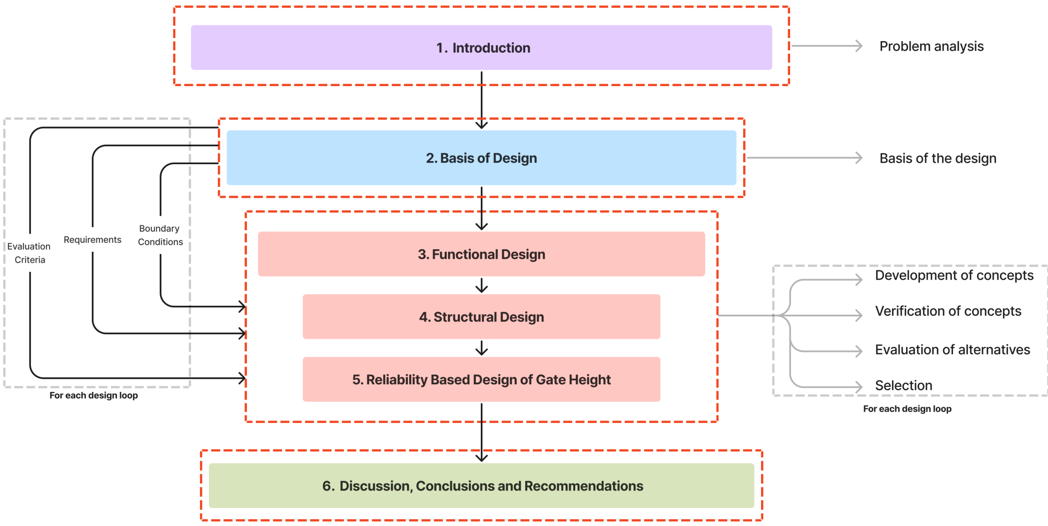



**Figure 1.6:** Thesis Outline in combination with the design method

- **Chapter 2 - Defining the Basis of the Design:** In this phase, the basic parts of the design are set up to create a clear conceptual design. All details are laid out, including the requirements for the Guard Lock, boundary conditions, and hydraulic conditions.
- **Chapter 3 - Functional Design Elaboration:** Presented in Chapter 3, this chapter delves into the design criteria of the Strandeiland guard lock. Detailing preliminary dimensions aligned with vessel specifications emphasizes the progression of the basis of the design to the functional design. By outlining choices like gate selection and potential integrations, the chapter is part of the phase of evolving concepts.
- **Chapter 4 - Establishing the Structural Design:** Chapter 4 centers on the core structural considerations of the guard lock's design. The focus on constructability, stability, and the crucial aspects of strength and stiffness directly correlates with the phase of verifying and evaluating design concepts.
- **Chapter 5 - Determining the Gate Height through Reliability Based Designing:** Chapter 5 establishes the gate height for the guard lock using a thorough reliability-based design approach.

This design loop is an iterative process that optimizes the balance between safety and efficiency considerations. This phase directly pertains to the evaluation of design concepts and the selection stage.

### 1.5.2. Design Loops

To further refine the approach, a set of design loops is employed. The design process begins with comprehensive problem analysis, generating essential questions about objectives, functions, operational aspects, constraints, and assumptions, leading to well-defined design criteria. The generation of alternatives requires a systematic workflow and adequate creativity. Multi-criteria analyses are common to assess different design options for example during the gate selection. The design loops used for further specification include the functional design, structural design, and a detailed design addressing potential failure mechanisms. The functional design considers the boundary conditions to identify critical components in the system that align with the client's requirements. The structural design involves choosing an appropriate main construction method, defining construction steps, providing stability, and performing stability and strength checks. Lastly, a detailed design is created that addresses potential failure mechanisms, using probabilistic analysis and top-down fault tree analysis for reliability budgeting, aiming to mitigate the risk level associated with various components of the structure.

# 2
# Basis of Design

This chapter discusses the requirements and boundary conditions guiding the guard lock's design. section 2.1 unpacks the guard lock's functional, performance, and structural requirements, along with the criteria used for evaluating the design. Boundary conditions, another essential aspect of the design process, are covered in section 2.3. The chapter concludes with section 2.4, dedicated to outlining the specific hydraulic boundary conditions, vital for maintaining the functionality and safety of the lock.

## 2.1. Guard Lock Requirements

Guard lock specifications fall into two broad categories: functional and structural. These aspects are interdependent and constitute the core of the design process. Functional requirements indicate the essential functions and performance criteria of the lock, while structural requirements pertain to the lock's physical traits.

### 2.1.1. Functional Requirements

The guard lock's main function is to safeguard Strandeiland from flooding by managing 'Het Oog's water levels to ensure hydraulic safety. Additionally, the lock should guarantee safe passage of vessels between IJmeer and the inner water marina, allow safe crossing for pedestrians and slow traffic like bicycles, and accommodate infrastructure like cables, mains, and pipes. The lock should also facilitate its own maintenance and repair, visually and audibly display its status and operation, protect against potential ship collisions, and ensure safety for both its operating personnel and the residents and users of Strandeiland.

### 2.1.2. Performance Criteria

To ensure the efficacy of Strandeiland's guard lock, several performance criteria have been derived from the main requirements. The lock should maintain a water level between NAP -0.6 meters and NAP +0.1 meters, endure maximum wave height, period, and wind speed (to be specified later), accommodate maximum vessel dimensions based on the normative vessel, and maintain a clearance distance of 2.50 meters between lock gates and passing vessels. Other parameters include a stipulated maximum time to open and close lock gates, minimum system reliability, and compliance with regulations like *Ontwerp Instrumentarium 2014v4 2017* and *Werkwijze Ontwerpen Waterkerende Kunstwerken (WOWK) 2018*. During power failures, manual gate closure should be possible.

### 2.1.3. Structural Requirements

Structural requirements specify the necessary characteristics that the guard lock structure should possess to function effectively and safely. This includes load considerations, material selection, and requirements related to the foundation and superstructure. These requirements ensure that the lock structure can withstand environmental and operational conditions and safeguard those who operate and maintain it. Notably, the primary function of the guard lock, the water retention, demands sufficient system reliability, strength, and stability (micro & macro). These requirements extend to the lock's constructibility, taking into account material availability, site conditions, and construction techniques.

It must be able to withstand hydrodynamic loads caused by wind-generated waves and loads from passing vessels. The lock's foundation should support the structure, resist hydrodynamic and other loads, prevent settlement and differential settlement, and counteract the uplift forces caused by the structure's buoyancy. The lock's superstructure should be able to resist hydrodynamic loads caused by wind-generated waves and tidal currents, provide enough clearance for vessels, house necessary mechanical and electrical components for operation, and bear loads from lock gate operations. Lastly, materials used should endure environmental conditions such as exposure to salt water, wind, and temperature fluctuations, be durable and resistant to corrosion, erosion, and other forms of degradation, and endure loads and stresses without deformation or failure.

**Material specifications and requirements**
The following applies to the use of materials:

- The materials to be used should preferably be recyclable;
- The principles of sustainable building will be a point of attention in the design.
- Using wood with the FSC quality mark.

### 2.1.4. Additional Requirements

While the functional, performance, and structural requirements provide a comprehensive framework for the guard lock's design, there are several supplementary aspects that need to be addressed. These additional requirements ensure that the design is not only technically sound but also meets broader objectives that encompass safety, reliability, environmental considerations, and more.

- **Safety:** The guard lock design must adhere to relevant safety standards and regulations.
- **Reliability:** The design should have a high degree of reliability and availability.
- **Ease of operation and maintenance:** The lock design should be maintenance-friendly with simple operational procedures.
- **Cost-effectiveness:** The design should be economically viable considering construction, operation, and maintenance costs.
- **Environmental impact:** The design should have minimal negative environmental effects.
- **Adaptability:** The lock design should be flexible to future changes and technological advancements.
- **Aesthetic:** The design should be visually compatible with its surroundings and enhance the visual appeal of the area.

Addressing these requirements will ensure that the guard lock is a suitable solution to address the extreme water level fluctuations.

## 2.2. Evaluation Criteria for the Design

After defining the intricate requirements for the guard lock, it's essential to establish clear criteria to evaluate the design solution. These criteria could also be used to evaluate potential solutions or components of the guard lock. Using these criteria ensures the design surpasses expectations in performance, safety, and efficiency.

## 2.3. Boundary Conditions

Taking into account the boundary conditions is vital during the design process of Strandeiland's guard lock. These conditions highlight the outside factors that affect the lock, such as environmental, operational, and design issues. By understanding these conditions, the lock can be designed to handle the specific challenges it will face. This section details these boundary conditions. By identifying and evaluating them, a clear understanding of the project's needs is gained, leading to a design that's effective, safe, and meets necessary standards and regulations.

| Criteria | Description |
|---|---|
| Performance | Extent to which the design meets or exceeds requirements |
| Safety | Degree to which design minimizes risks and potential hazards |
| Reliability | Consistency and predictability of the design's performance over time |
| Operational Ease | Simplicity and efficiency in operation and maintenance |
| Cost-effectiveness | Balance between the design's benefits and its costs |
| Environment | Extent to which the design respects and protects the environment |
| Future-readiness | Design's potential to adapt to future needs and technologies |
| Aesthetic Appeal | Design's contribution to the visual and sensory experience of the location |

**Table 2.1:** Evaluation criteria for assessing the success of the guard lock design.

### 2.3.1. Wind Set-up & Set-down

The wind speed at 10 meters above water level is 20 m/s found by Hydra-NL which is used by the KNMI. The KNMI-measuring station Schiphol is located on land, tens of kilometers away from the IJmeer. This means that spatial interpolation of wind information, taking transitions from land to water and vice versa, is required. The KNMI-data covers 4 decades, which means that statistical extrapolation is required.

### 2.3.2. Environmental Boundary Conditions

The environmental boundary conditions include factors such as soil conditions at the site, regulations related to water quality and marine life, and other environmental requirements. This section will detail the specific environmental boundary conditions that must be considered during the design process, ensuring that the lock is designed to meet the project's specific needs and constraints while also protecting the surrounding environment. The location of the project is explained in the problem description in section 1.1 and indicated in Figure 1.1.

**Detailed site description**

The surface area of the island is approximately 140 hectares, which is surrounded by its own dike ring. The island is provided with temporary dumping stone revetment during the construction phase, which must be upgraded to a permanent flood defence.

An offshoot of the flow channel of the Oer-IJ, the Oergeul, is found across a strip about 200 to 300 m wide. Here, moderately weak, peaty clay with a few sandy layers is found under the aforementioned clayey silt layers to a depth of approximately NAP -10.0 m to NAP -15.0 m (outside the western location of Strandeiland approximately NAP -18.0 m) were found. The global contours of the Oer-IJ current channel are shown in Figure 2.1.

**Figure 2.1:** Global contours of the Oergeul (Jansen, 2021)

**Soil conditions at the site**

The bottom of the IJmeer lies at a level of approximately NAP - 2.2 m to NAP - 3.3 m. An overall soil structure directly beneath the bed can be seen in Table 2.2.

**Table 2.2:** Soil structure Strandeiland (Jansen, 2021)

| Depth (NAP ... m) | Soil structure |
|---|---|
| Ca. -2,2 to -3,3 | Soft clay/silt layer |
| Ca. -3,3 to -4,1 | Clay sediment |
| Ca -4,1 to -6,5 | Light clayey sand |
| Ca -6,5 to -7,5 | Peat |

Below an overview shown in Figure 2.2 of Strandeiland with indicated areas of the soil structure.

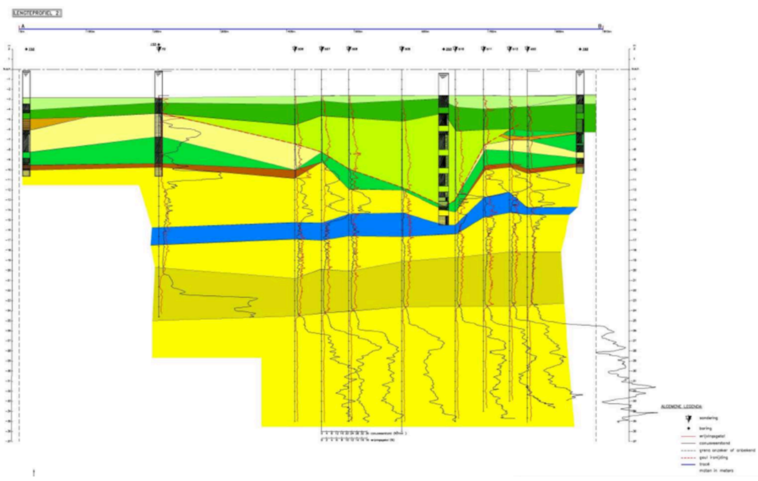


**Figure 2.2:** Soil structure overview Strandeiland (Jansen, 2021)

**Groundwater level variation within Strandeiland**

An open connection between the outer and inner water causes variations of the inner water level. Here, low and high water levels can theoretically pose a problem for groundwater levels on the island. This chapter first examines the consequence of a variable low water level and then of a variable high water level.

**Effects of low inland water level**
At low levels of the inland water level, the groundwater level will drop. Trees at the edges of the inner lake will experience more (short-term) groundwater level fluctuations.

Houses, cables, pipes and roads must meet a drainage standard to prevent groundwater flooding. Groundwater flooding is not an issue here as there are no compressible layers, or wooden piles. Most trees already rely on a hanging water profile. It does not matter whether the groundwater level is, say, 2 or 2,5 meters below groundlevel.

**Effects of high water levels**
The ground level of Strandeiland is calculated for an inland water level up to NAP -0.2 m. Long-term high inner (and outer) water levels lead to higher groundwater levels throughout Strandeiland. Short-lasting water levels have little effect, or only along the edges of Strandeiland.

Water levels at Schellingwouderbrug (see Figure 2.3) regularly show (long-lasting) water levels of NAP -0.1 and NAP +0.0 m. The island's ground level is calculated for an inland water level up to NAP -0.2 m. This means that groundwater levels on the island are expected to be higher than currently taken into account. As a result, the drains might flood and the dewatering standard would not be met. This could potentially lead to flooding in the Pampusbuuurt and the Muiderbuurt neighbourhoods.

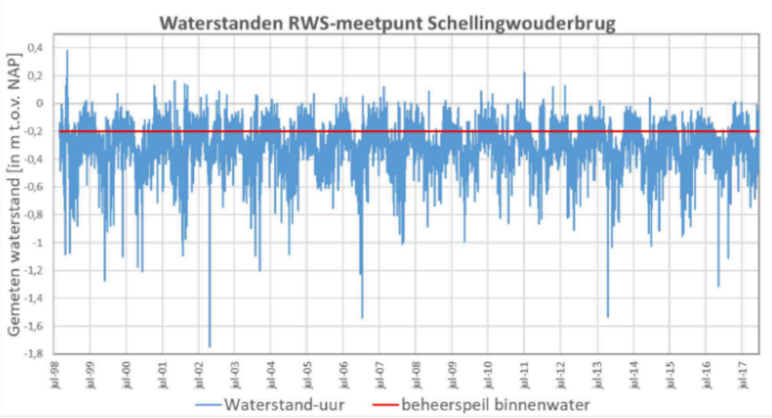



**Figure 2.3:** Water levels of the RWS measuring point Schellingwouderbrug

**Groundwater level**
Because water level differences in the inland water have negligible effects on the groundwater level, a ground water level (GWL) is assumed to be used in the structural design. This comes from the research of Fugro which is shown in Appendix A.2. The average GWL is set at + 0.9 m NAP for the calculation further in this report.

**Environmental regulations and requirements**
The IJmeer is subject to a range of environmental regulations and requirements aimed at protecting the surrounding environment and maintaining water quality. These regulations and requirements are implemented by various organizations and agencies, including the Dutch Ministry of Infrastructure and Water Management and the Dutch National Institute for Public Health and the Environment. Some of the key regulations and requirements that may apply in the IJmeer include:

- The Water Framework Directive, which sets standards for water quality and aims to protect aquatic ecosystems and promote sustainable water use
- The Birds and Habitats Directives, which aim to protect and conserve natural habitats and species within the European Union

- The Dutch Water Act, which provides a framework for managing and protecting water resources in the Netherlands
- Regulations related to water quality, including limits on pollutants and requirements for monitoring and reporting water quality data
- Regulations related to marine life, including restrictions on fishing and other activities that may harm marine ecosystems.

The specific regulations and requirements that apply in the IJmeer will depend on various factors, including the location and intended use of the area. (Ministerie van Infrastructuur en Milieu, 2015)

### 2.3.3. Operational Boundary Conditions

**Vessel traffic & vessel size**

The guard lock ensures that the inland water is accessible for ships that want to enter the harbour. The width of Lock 125 and the vertical clearance of Bridge 2061 are based on 800 vessels in Het Oog and 25,000 movements per year (50 sailing days per year with 250 vessels x 2).

Het Oog is divided into 2 compartments by Bridge 2062, as shown in Figure 1.3. In the section between Lock 125 and Bridge 2062, 250 moorings are provided. The number of vessels behind the bridge has been reduced from 600 to 400 by the designer of Space and Sustainability. Thus, a total of 650 vessels are anticipated at this time.

A mix of boats is assumed. The division of that mix into sailing boats with or without cabin, motorboats and sloops. The exact proportions are difficult to determine due to a number of developments, such as the consequences of compulsory emission-free sailing in Amsterdam (compulsory as of 2025) and an expected further increase in the harbour fees.

Based on 25,000 shipping movements and the provisions of the Richtlijn Vaarwegen, the Varen, Nautisch Beheer programme assumes a B profile. This means that the normative or size-dependent vessel for the lock has a length of 20 m and a width of 4.25 m and that at least four normative vessels must fit into the lock. Based on this, the lock has to be at least 40 x 10 m according to Nautical Management.

As indicated in the introduction in section 1.1, a guard lock is chosen because the number of ship movements could cause long delays with a navigation lock, especially on sunny days.

**Traffic**

The movable bridge 2061 is intended for slow traffic, such as cyclists and pedestrians, this bridge is indicated in Figure 1.3. Access should only be granted to emergency services and maintenance vehicles, other road traffic is excluded. This bridge connects the southern Muiderbuurt with the northern Pampusbuurt. Bridge 2062 has a public transport connection where a bus passes every 4 to 6 minutes. Active sailing yachts will be moored at the Havenkom, as they do not disrupt road traffic and/or public transport.

The Eurocode standards and the national annexes have to be applied for the elaboration of the structural design of the bridge. For the railing, a height of 1.00 metres is currently assumed. In the design phase, a check will have to be made as to whether this complies with the new regulations or whether it should be 1.30 metres.

**Figure 2.4:** View from the beach: the lock and slow traffic bridge 2061, the Havenkom, bridge 2062 and behind it Het Oog (Waterrecreatie Advies BV, 2021)

**Lock operation**
The future operator, Waternet, operates objects in Amsterdam remotely from their control centre on the Korte Ouderkerkerdijk in Amsterdam. Incidentally, it may be necessary for Waternet to operate locally. The bridge and lock are a nautical unit, which means they should be regarded as a single entity for the purpose of operation. In the event of a malfunction, it should be possible to operate the bridge and/or lock and secure the object. The objects should be provided with a mechanical hand control, so that the objects can be secured in case of a power failure. power failure.(IBA, 2021)

### 2.3.4. Design Boundary Conditions

**Building codes and regulations**
The guard lock of Strandeiland is subject to building codes and standards that are designed to ensure the safety, stability, and durability of hydraulic structures. Beklow the key building codes and standards that apply to hydraulic structures in the Netherlands include:

- Building Decree (Bouwbesluit): This code outlines the basic safety and health requirements for buildings and other structures in the Netherlands. It includes provisions related to fire safety, structural safety, accessibility, and environmental performance, among other things. (wetten.nl, 2012)
- Dutch Water Act (Waterwet): This act provides a framework for managing and protecting water resources in the Netherlands, and includes provisions related to the design, construction, and maintenance of hydraulic structures such as locks, dams, and levees. (wetten.nl, 2021)
- Dutch National Annex to the Eurocodes: The Eurocodes are a series of European standards for the design and construction of buildings and other structures. The Dutch National Annex provides additional requirements and guidance for the application of the Eurocodes in the Netherlands. (Ministerie van Infrastructuur en Milieu, 2022)
- National Hydraulic Conditions (Nationale Waterstaatswerken): These conditions provide guidelines for the design and construction of hydraulic structures in the Netherlands, taking into account local water levels, wave conditions, and other factors that may impact the structure's performance. (Ministerie van Infrastructuur en Milieu, 2022)
- Technical Guidelines for Hydraulic Structures (Technische Leidraad voor de Binnenwateren): These guidelines provide detailed technical specifications and guidance for the design, construction, and maintenance of hydraulic structures in the Netherlands.(Ministerie Verkeer en Waterstaat, 2009)

The guard lock is part of the primary water barrier. Its main function, the water retaining function of the lock is to ensure hydraulic safety. The design of the lock should be carried out in accordance with the *Ontwerp Instrumentarium 2014v4* from 2017 and *Werkwijze Ontwerpen Waterkerende Kunstwerken (WOWK)* from 2018.

**Safety factors and load combinations**
The safety factors and load combinations used in the design of the guard lock are important considerations to ensure its long-term performance and safety. The safety factors used may vary depending on the specific component being designed, with values such as 1.5 or 1.3 commonly used for foundation and gate design, respectively.

Load combinations are used to evaluate the effect of different loads on the guard lock, including the weight of the lock gates, hydrostatic pressure of the water, wind loads, and wave loads. The design must be able to withstand the most severe load combination it is likely to experience during its lifetime, which will depend on factors such as expected wave conditions, wind speeds, and water levels at the site. It is important to consider realistic safety factors and load combinations to ensure the guard lock is designed to withstand the conditions it will face over its lifetime.

## 2.4. Hydraulic Boundary Conditions

This section will delve into the specifics of the hydraulic boundary conditions that have been factored into the design process of the guard lock. The following topics will be discussed:

- Water level variations in IJmeer: This involves an examination of the fluctuations in the water levels that may be driven by various natural and man-made factors.
- Explanation of Bretschneider equations: These equations are instrumental in determining the significant wave height and peak period, which are key variables in the design of the guard lock.
- Depth of IJmeer: The depth of IJmeer, obtained from Navionics, is a critical parameter in the calculations and modeling of wave conditions.
- Determination of fetch: Fetch, the distance over which the wind blows across open water, is a crucial factor influencing the wave conditions. The method used to determine this parameter will be discussed.
- Wind speed distribution: The wind speed distribution was obtained from Meteoblue data and the wind direction distribution. The process of acquiring this information and its relevance to the wave conditions will be outlined.
- Determination of significant wave height and peak period: Lastly, the method for obtaining the significant wave height and peak period using the variables fetch, depth, wind direction, and wind speed will be elucidated.

This in-depth assessment of the hydraulic boundary conditions is crucial in guiding both the functional and structural design of the guard lock.

### 2.4.1. Water level variations in IJmeer

Water levels in the IJmeer are affected by a variety of factors, including tides, wind, and precipitation. Tides are generally not a significant factor in the IJmeer, as the lake water levels are controlled by the Houtribdijk. The Houtribdijk includes a shipping lock and a pumping station that allows water to be transferred between the Markermeer and the IJsselmeer, helping to maintain water levels and prevent flooding in the surrounding areas.
However, water levels can be influenced by storm surges, which can cause sudden and significant increases in water levels. Additionally, water levels can be influenced by wind-driven currents, which can cause skewness. The water level variations in the IJmeer are an important consideration for the design of the guard lock of Strandeiland, as they will have a significant impact on the lock's operation and performance.

The following water level assumptions are used:

- Summer levels from the outer waters fluctuate from -0.1 m to NAP -0.3 m and winter levels fluctuate from NAP -0.2 m to NAP -0.4 m according to new RWS water level ordinance. (Ministerie infrastructuur en Waterstaat, 2018)
- Water level increases according to *Het Nationale Waterplan 2016-2021* (Ministerie van Infrastructuur en Milieu, 2015) and the *Addendum to the Guideline sea- and lake-dykes*. (Ministerie Verkeer en Waterstaat, 2009)

Wind set-up and set-down due to extreme weather conditions can cause the IJmeer's water levels to deviate from target levels. The design of flood protection is dimensionalized in relation to the magnitude of deviations during normative situations. The normative deviation from target level depends on the location and way hydraulic loads are determined: whether they are combined with wave loads or not. In addition, very rapid water level changes as a result of both phenomena should be taken into account for Wind setup and down-setting. It also has to be taken into account. Water level differences of 1 metre per hour are no exception and these may pose a risk.

Extreme water levels should be considered when designing flood defenses. However, they are determined according to the safety standards (1: 300 per year). (Jansen, 2021)

### 2.4.2. Significant Wave Height

As there is no direct data available for the wave characteristics at the Strandeiland location, an analysis was conducted to deduce the significant wave height and peak period. This section delves into the processes used to discern the wave characteristics that are applicable to Strandeiland within the Markermeer/IJmeer domain. The Bretschneider method, a widely-recognized tool, provides estimates for ocean wave characteristics such as significant wave height and period, based on the wind speed and its duration. Remarkably, this method's scope isn't confined solely to ocean waves; it is equally effective for determining the characteristics of waves in lakes and other similar water bodies. This versatility stems from the method's empirical relationships which link wave characteristics to attributes of the wind field, making it universally relevant across an array of water bodies. However, the Bretschneider method operates under certain assumptions, including waves being purely wind-generated and the wind field being spatially consistent and in a steady-state.
Another notable method is the Young Verhagen equation. While it offers refinements in wave characteristic estimation, it assumes non-variable wind direction and an infinite fetch, conditions which might not be accurately representative of Strandeiland's environment. Therefore, the Bretschneider method was deemed more appropriate for this analysis.
To choose a suitable method to determine the wave characteristics, it's important to understand the underlying assumptions and constraints of each method. Table 2.3 contrasts the Bretschneider method with other widely utilized techniques, such as Pierson Moskowitz, Philips, JONSWAP, and Young Verhagen, focusing on their suitability for different water bodies and inherent assumptions.

**Table 2.3:** Comparison of wave spectra

| Spectrum | Applicable water bodies | Assumptions |
|---|---|---|
| Bretschneider | Ocean, lakes, other water bodies | Wind-generated waves homogeneous and ste wind field |
| Pierson Moskowitz | Fully-developed sea with wind duration $> 30$ hrs | Fully-developed sea, ter, narrow-banded wa |
| Philips | Fully-developed sea | Fully-developed sea, d |
| JONSWAP | Fully-developed sea | Fully-developed sea, banded waves, no c spreading |
| Young<br>Non-variable wind direction, infinite fetch | Verhagen | Fully-developed sea |

Based on the characteristics of the Strandeiland location in Markermeer/IJmeer, the Bretschneider method is the most applicable approach for estimating wave characteristics. The method's wide applicability for both ocean and lake waves makes it a suitable choice for this location. (Holthuysen, 2010)

### 2.4.3. Bretschneider Equations

The Bretschneider equations for significant wave height and significant wave period are shown below:

$$\bar{H} = 0.283 \tanh\left(0.530\bar{d}^{0.75}\right) \tanh \left\| \frac{0.0125\bar{F}^{0.42}}{\tanh\left(0.53\bar{d}^{0.75}\right)} \right\|$$

$$\bar{T} = 2.4\pi \tanh\left(0.833\bar{d}^{0.375}\right) \tanh \left\| \frac{0.077\bar{F}^{0.25}}{\tanh\left(0.833\bar{d}^{0.375}\right)} \right\|$$

$$\bar{d} = \frac{dg}{u^2}, \bar{F} = \frac{Fg}{u^2}$$

$$\bar{H} = \frac{H_s g}{u^2}, \bar{T} = \frac{T_s g}{u}$$

Where:

- $g =$ gravity acceleration $[\mathrm{m/s^2}]$
- $u =$ wind speed at $10$ m height $[\mathrm{m/s}]$
- $d =$ water depth $[\mathrm{m}]$
- $F =$ fetch length $[\mathrm{m}]$
- $H_s =$ Significant wave height $[\mathrm{m}]$
- $T_s =$ Significant wave period $[\mathrm{s}]$

The $T_s$ value that is used in the equation for the wave period corresponds to the significant wave period and is transformed to $T_p$ (peak wave period) by multiplying by a 1.08 factor.

The following subsections will consider the relevant parameters of the Bretschneider formula for the calculation of the significant wave height of the IJmeer & Markermeer. The parameters that will be taken into account are the fetch, wind direction, wind speed, and depth

**Meteoblue weather data**

Meteoblue has been selected as the weather data provider for this project over KNMI for several reasons. Firstly, the raw data files from KNMI contained a vast amount of information, making it challenging to extract the necessary data needed for the calculations. Secondly, the user interface of Meteoblue is more user-friendly, with various options for customizing the weather data based on specific user requirements. Finally, Meteoblue provides ready-to-use weather data that can be easily integrated into our models. (Meteoblue, 2023)
Here's the revised content without using "we":

**Fetch length determination**

To determine the fetch for the guard lock, four points located at the boundary of IJmeer Markermeer were identified. The fetch represents the distance over water that wind can travel to create waves. The following points are indicated in Figure 2.5:

- Guard lock: r = 0, $\theta$ = 0°
- Point A: r = 150 m, $\theta$ = 190°
- Point B: r = 450 m, $\theta$ = 235°
- Point C: r = 3000 m, $\theta$ = 300°
- Point D: r = 340 m, $\theta$ = 340°

In polar coordinates, r represents the distance from the origin (in this instance, the guard lock), and $\theta$ signifies the angle between the positive x-axis and the line connecting the origin to the point.

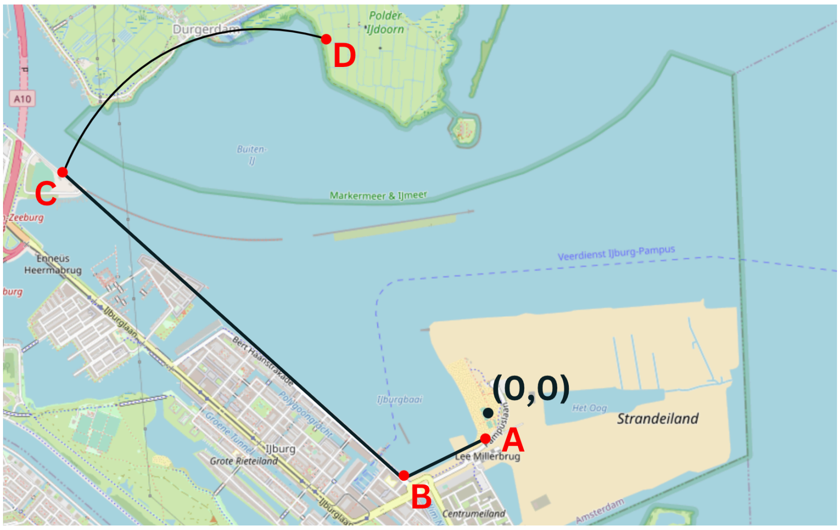



**Figure 2.5:** Identified points to determine relevant fetch for the guard lock

In this context, the fetch depends on the direction of the wind. For instance, if the wind originates from the north, the fetch becomes the distance between the guard lock and the northern boundary of IJmeer Markermeer. Similarly, if the wind blows from the west, the fetch becomes the distance between the guard lock and the western boundary of IJmeer Markermeer. By pinpointing these boundaries, it becomes feasible to determine the maximum possible fetch for any wind direction.
To ascertain the fetch of the guard lock, a line emanating from the guard lock (0,0) extending 5000 meters, and rotating from point A (at 190 °) to point D (at 340 °) was considered. This line exemplifies potential wind directions. The intersection of this line with the boundary condition lines, drawn between the points, was then identified. Using these intersection points, the lines between the guard lock and the intersections were established, facilitating the determination of the fetch from those specific directions.

**Wind direction distribution**
Although wind direction is not directly used as an input parameter in the Bretschneider formula for wave height calculation, it is still an important factor to consider in determining the fetch. It is important to note that obtaining the correct mean wind direction and distribution is crucial when conducting a Monte Carlo analysis with a large number of samples, as this can have a significant impact on the resulting fetch values.

The wind direction data was obtained from Meteoblue, which provides historical weather data based on climate models. The data was analyzed to determine the frequency distribution of wind direction in the Amsterdam area. This distribution was then used to determine the fetch to obtain the wave height for different wind directions using the Bretschneider formula. The wind direction data was organized into 16 categories based on the compass rose, ranging from N, NE, E, SE, S, SW, W, to NW. The Python script utilizing Meteoblue data was used to generate the wind distribution, which is visualized in Figure 2.6. The wind distribution is superimposed on the map. It considers all 16 wind categories but is limited to the relevant wind directions coming from angles between point A (at 190 °) to point D (at 340 °).

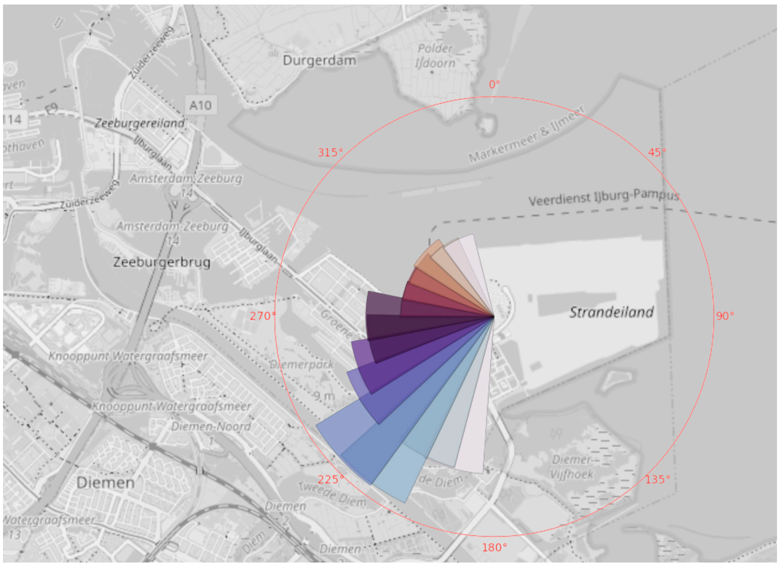



**Figure 2.6:** Wind Distribution plot for Strandeiland Meteoblue, 2023

**Wind speed mean and standard deviation**

Wind speed plays a pivotal role as an input parameter for the Bretschneider formula. Meteoblue, the weather data provider in consideration, provided both the wind directions and their distribution along with the wind speeds and the number of hours per year during which the wind maintained that speed. From this data, the mean and standard deviation of the wind speed were derived. The average wind velocity was identified to be **5.31 m/s**, and the standard deviation of the mean wind velocity was **2.91 m/s**. These figures regarding the mean and standard deviation of wind speed are set as inputs for the Bretschneider formula. In the context of a Monte Carlo analysis, the formula will be executed multiple times using randomly generated inputs.

**Depth IJmeer**

For accurate modeling of the significant wave height in bodies of water like IJmeer/Markermeer, incorporating precise depth data is crucial. Depth data from Navionics, a Garmin Company, was utilized to obtain data points for the depth in IJmeer. (Navionics, 2023)

### 2.4.4. Results of Wave Height and Peak Period

The simulation used to derive the results of significant wave height ($H_s$) and peak wave period ($T_p$) is based on the following input data shown in Table 2.4.

**Table 2.4:** Input Parameters for the Bretschneider Wave Spectrum Formula

| Parameter | Value |
|---|---|
| Wind Speed Mean | 5.31 m/s |
| Wind Speed Standard Deviation | 2.91 m/s |
| Wind Direction Mean | 210° |
| Wind Direction Standard Deviation | 60° |
| Fetch Mean | 1000 m |
| Depth | depth data from Navionics |
| Samples in Monte Carlo Simulation | 10,000 |

Initially, the Bretschneider wave function is defined. This function computes the $H_s$ and $T_p$ values taking into consideration the water depth, fetch, and wind speed inputs. The Bretschneider wave spectrum formula links the spectral density of wave energy to the wave height parameters, effectively facilitating the calculation of $H_s$ and $T_p$. Subsequently, a Monte Carlo simulation is executed. This simulation estimates the distribution of $H_s$ and $T_p$ values derived from random samples of wind speed and direction data. Parameters such as the mean and standard deviation of wind speed and direction are set to generate a dataset of 10,000 samples following a normal (Gaussian) distribution. The next stage involves converting wind speed data into the wind speed in the direction of wave propagation (wind speed u) by utilizing the cosine of the wind direction. The $H_s$ and $T_p$ values are then calculated for each sample through the previously defined Bretschneider wave function. Employing the Monte Carlo simulation with 10,000 samples, alongside the Bretschneider method, input values for mean wind and standard deviation, as well as variable fetch according to wind direction and water depth, the following results are obtained:

| Parameter | Value |
|---|---|
| $H_s$ mean | 0.91 m |
| $H_s$ standard deviation | 0.75 m |
| $T_p$ mean | 1.50 s |
| $T_p$ standard deviation | 1.29 s |

**Table 2.5:** Table presenting estimates for the mean and standard deviation of the significant wave height and wave period for the specific location of the guard lock

With this data determined, all hydraulic boundary conditions are now mapped out. These conditions will be used in the Structural Design (Chapter 4) and the Reliability-Based Design of Gate Height (Chapter 6).

### 2.4.5. Vessel-generated waves

While the focus has primarily been on wind-generated waves, one cannot overlook the potential for waves induced by vessels, especially in areas with significant maritime traffic. Vessel-generated waves can arise from the movement of boats and ships, and their characteristics can be influenced by factors such as the vessel's speed, size, hull shape, and the water depth in which it operates.

However, in the context of Strandeiland's guard lock design, vessel-generated waves have not been considered as a governing factor for a few reasons. First, the frequency and magnitude of such waves are typically less predictable than wind-generated waves, making them harder to model reliably. Second, the potential impact of vessel-generated waves is often localized and transient, meaning that while they may cause short-term disturbances, they are less likely to result in sustained, problematic water level fluctuations. Lastly, measures such as speed restrictions, designated navigation channels, and vessel traffic management can mitigate the effects of these waves.

# 3
# Functional Design

This chapter provides a comprehensive overview of the functional design aspects of Strandeiland's guard lock. section 3.1 delineates the functional requirements of the guard lock and their incorporation into the overarching design. section 3.2 focuses on the necessary waterway dimensions for the lock's operation and hydraulic safety. In section 3.3, an inventory of guard lock components is outlined, further detailed in the subsequent subsections focusing on gate selection and arrangement, the lock head, and the lock chamber design. Overall, this chapter encapsulates the key design aspects that ensure the guard lock's optimal performance and flooding risk mitigation.

## 3.1. Addressing Functional Requirements in Design

The main aim is to design a system that addresses the primary requirements outlined in chapter 2. This process consists of several steps, which are described as sequential and interconnected actions. Firstly, the key functional requirements that the guard lock must fulfill are identified. Based on the program of requirements from chapter 2, the principal functional requirement is to prevent flooding of Strandeiland by fulfilling the water retaining function to ensure hydraulic safety by controlling water levels in the inner water 'Het Oog'. This involves the closure of the gates when the water level reaches NAP + 0.1 m, and also when the water level decreases to NAP -0.6 m. Secondly, preserving functions that need to be considered in the functional design are identified. These functions are critical for maintaining the guard lock's overall effectiveness and usability. They involve providing access for vessels to the inner water of Strandeiland, including the normative vessel and normative combination vessels. Additionally, the lock should enable dry crossing of the water separating the Pampus neighbourhood from the Muiderbuurt, which involves roads for slow traffic like bikes and pedestrians, as well as the integration of cables, mains, and pipes. The functional requirements identified are summarised in Table 3.1.

| Functional Requirement | Details |
|---|---|
| Maintain the water level of Strandeiland | Closure of the gates when the water level reaches NAP + 0.1 m and NAP -0.6 m |
| Provide access for vessels to the inner water of Strandeiland | Accommodate the normative vessel and normative combination vessels |
| Enable dry crossing of the water | Include roads for slow traffic (bikes and pedestrians), and integration of cables, mains, and pipes |

**Table 3.1:** Functional Requirements of the Guard Lock

An analysis is then performed on how each of these functional requirements can be addressed in the design. This includes aspects such as lock dimensions, gate design, operational procedures, and the integration of supportive infrastructure. Next, alternative functional designs that fulfill the main functional requirement, sub-functions, and supportive functions are developed and evaluated. This evaluation involves assessing the design alternatives against the evaluation criteria outlined in chapter 2, considering factors such as performance, safety and reliability. Finally, based on the evaluation of functional design alternatives, the optimal design that best meets the requirements and evaluation criteria is selected for further development in the subsequent design stages. This selected functional design serves as the basis for the structural design and detailed design phases of the project.

## 3.2. Waterway Dimensions

This section focuses on determining the appropriate dimensions for the guard lock and waterways. Establishing suitable dimensions is a crucial step towards ensuring the efficient operation of the lock system and facilitating the passage of vessels. The discussion includes the dimensions of the normative vessel, special requirements for the guard lock, and other contributing factors that influence the sizing of the lock and waterways. Functional design demands the establishment of appropriate dimensions for the guard lock and waterways to ensure the efficient passage of different types of vessels. This section leverages the normative vessel dimensions to guide the design of the guard lock and waterways in Het Oog. Two different passage profiles, B and D, are considered to accommodate various vessel sizes.

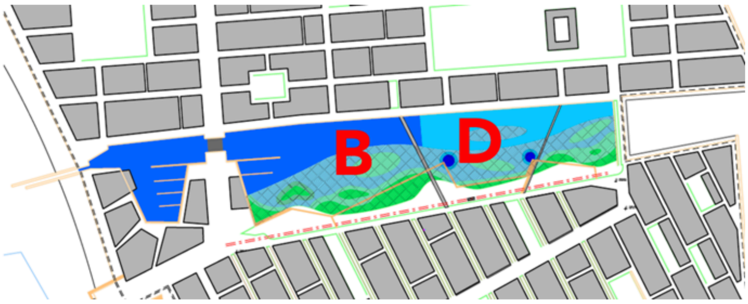



**Figure 3.1:** Vessel profiles indication in Het Oog (IBA, 2021).Maximum dimensions of vessels for passage profile B: 20 x 4.25 x 1.90 meters. Maximum dimensions of vessels for passage profile D: 14 x 3.75 x 1.50 meters

Based on these normative vessel dimensions, the guard lock and waterways will be designed to provide sufficient clearance and facilitate efficient vessel passage. It is important to note that a width of 4.5 meters is used for the normative ship, as it is the minimum width for new waterways to be constructed according to the guidelines for designing waterways (Koedijk, 2020). The guard lock design consists of several components, such as the fairway, lead-in structure, and lock chamber.

The chosen design for the guard lock and outer harbor accommodates a single-lane profile, which enables one normative vessel to pass through the lock at a time. This choice is based on the low frequency of normative ships, the recreational character of the inland waterway, and the expectation that ships often sail in the same direction (morning departure, afternoon, and evening entry). Vessels from the opposite direction will have to wait, and running up and meeting in the turning lock is avoided by means of signals (VRI) and possible VHF radio contact. Taking into account the normative vessel dimensions, the lock chamber dimensions can be determined. The lock chamber should have a minimum length of 22 meters to accommodate a sizeable vessel, and a width of 7.6 meters, which includes a surcharge of 0.4 meters for ease of navigation in a longer passage. These dimensions are determined based on the guidelines for waterways from (Koedijk, 2020).

In the functional design of the guard lock and waterways, determining appropriate water depths for the different sections is crucial. The depths are influenced by factors including the minimum water level (NAP -0.45 m to -0.60 m), the depth of the normative vessel (1.90 m), storage for dredge accretion (0.30 m, not applicable to the lock chamber), and a safety margin between the vessel bottom and the waterway bottom (0.40 m in the lock chamber, 0.20 m in the fairway for a tight profile). For the lock chamber, the recreational navigation guideline requiring a 0.4 m margin is used, differing from the guard lock-specific guideline. The fairway uses a 0.2 m mark-up for a tight profile, while dredge accretion, per Waternet's guidance, is considered at 0.30 m. The resulting depths are presented in Figure 3.2.

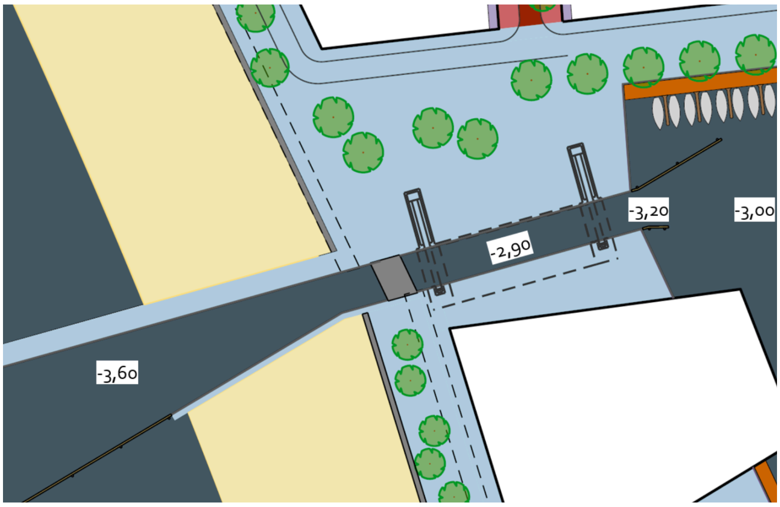



**Figure 3.2:** Sketch of depths at NAP -0.60 m for the various locations. Depth at the west of the lock (-3.60) was determined using the existing outer water plan IBA, 2021

Table 3.2 summarizes the critical dimensions derived from the normative vessel specifications and relevant guidelines. Additionally, adhering to certain pivotal requirements is paramount for the functionality and safety of the guard lock, as detailed in Table 3.3, which have been extracted from the Guidelines for Waterways.

**Table 3.2:** Waterway and Guard Lock Dimensions

| **Parameter** | **Value** |
|---|---|
| Maximum dimensions for passage profile B | 20 x 4.25 x 1.90 m |
| Maximum dimensions for passage profile D | 14 x 3.75 x 1.50 m |
| Width of normative ship | 4.5 m |
| Lock chamber minimum length | 22 m |
| Lock chamber width | 7.6 m (includes 0.4 m surcharge) |
| Minimum water level | NAP -0.45 m to -0.60 m |
| Depth of normative vessel | 1.90 m |
| Storage for dredge accretion | 0.30 m (not for lock chamber) |
| Safety margin (lock chamber) | 0.40 m |
| Dredge accretion (Waternet's guidance) | 0.30 m |

| No. | Special Requirement |
|---|---|
| 1 | Ensure a closed guard lock serves as a full-fledged (primary) water defense. |
| 2 | Provide a straight clearance before and after the guard lock, with a minimum length of 1.5 times the length of the normative vessel, to facilitate navigation. |
| 3 | Align the axis of the turning lock with the waterway section's axis to offer a clear view of oncoming traffic and enable passage in a straight line. |
| 4 | Gradually transition between the lock profile and the connecting waterway section to limit excessive flow gradients, such as return flow and flow caused by wind and water level effects. |
| 5 | Designate waiting areas at a guard lock without an adjacent navigation lock to accommodate vessels during times of obstruction. |
| 6 | Ensure the depth of the navigable part of the outer harbor is at least equal to that of the connecting waterway (see Figure 3.2). To prevent sedimentation on the lock sill, the depth of the outer harbor should be greater than the depth of the threshold. |

**Table 3.3:** Special requirements for the guard lock as outlined in the Guidelines for Waterways (Koedijk, 2020). These requirements are integral to ensuring the lock's functionality and safe and efficient vessel passage. They address a range of considerations, including water defense, navigation facilitation, lock alignment, flow gradient management, vessel accommodation during obstructions, and depth management to prevent sedimentation.

**Vertical clearance**
The free vertical clearance at the bridge is at least 2.50 m relative to the highest target level; NAP -0.20 m + 0.10 m water level rise in the future. (i.e. 2.40 m + NAP) For the bridges The target level for the inland waterways is between NAP -0.20 m and NAP - 0.40 m. As a result of lock management, inland water levels may fluctuate between NAP - 0.60 m and NAP 0.00. The outside water level may fluctuate strongly.
The lock also has a minimum passage width of 10 m. The construction depth is NAP -3.00 m. (taking into account that the lowest water level outside the lock occurs twice a year).

In deviation from the NEN 6786 (VOBB 2017) a maximum unavailability applies for shipping traffic of the bridge of an average of 2 days per year. This concerns the unavailability due to wind. As a basis for the operation a wind measurement installation must be placed at the object.

## 3.3. Inventory of Guard Lock Components

The guard lock at Strandeiland primarily serves two functions: **Water Retention** and the **Passage of Vessels**. These dual functionalities often result in a functional conflict, as the more critical function (water retention) potentially inhibits the secondary function (vessel passage). In standard operational circumstances, the guard lock's gate remains open to facilitate maritime traffic, temporarily suspending its water retention capabilities. However, certain circumstances, such as excessively high or low water levels, may necessitate the closure of the gate, thereby interrupting vessel movement. These extreme conditions could occur due to a storm surge leading to unusually high water levels, or a drought causing critically low water levels. The guard lock is a both-way retaining guard lock. Under regular conditions, considering the water level variations, the lock remains open. However, during extreme situations, such as during a storm or a severe drought, the lock retains water in both directions. Once closed, the gates become an integral part of the flood defense system, which also includes adjacent dikes or other flood defense structures.

## 3.4. Gate Type Variant Study

In the pursuit of an optimal gate design for the guard lock, it's essential to thoroughly evaluate the different gate types available. Each gate type comes with its unique set of characteristics, advantages, and potential constraints. This section aims to provide a comprehensive analysis, starting with an inventory of the various gate types, followed by a verification of their suitability based on distinct project requirements. Subsequent sections delve deeper into a multi-criteria analysis, providing a thorough evaluation to assist in making an informed decision about the most suitable gate type variant for the guard lock of Strandeiland.

### 3.4.1. Inventory of Concepts

The following table provides a detailed inventory of the different gate types, encompassing their characteristics, advantages, and constraints.

| Gate Type | Characteristics | Advantages | Constraints |
|---|---|---|---|
| Mitre gates | • Retains water in one direction<br>• Moved in horizontal direction<br>• Half the size of chamber width results in lighter lock head structures | • Requires lesser machinery power | • Cannot be opened or closed under water head difference<br>• Vulnerable to ship impact, especially supports |
| Rolling/sliding gate | • Can span the full width of lock chamber<br>• Moved in horizontal direction<br>• Can be set dry in recesses | • Resistant to vessel impact | • Cannot be opened or closed under water head difference<br>• Requires a dry recess for maintenance |
| Lift gate (submersible) | • Requires significant vertical movement<br>• Sits below water level when open | • Low horizontal footprint<br>• Frequent lifting offers inspection and maintenance opportunities | • Limitation on air draught for passing vessels<br>• Considerable machinery for vertical movement |
| Lift gate (upward direction) | • Moves vertically upwards<br>• Requires significant vertical movement | • Can span the full width of lock chamber<br>• Regular lifting provides maintenance and inspection ease | • High machinery power needed<br>• Landmark appearance might be out of place |
| Radial gates | • Curved gates rotating around a horizontal axis<br>• Might not retain water effectively in both directions | • Can control water flow rates effectively | • Specific structural needs |
| Single leaf gate | • Similar to mitre, but a single gate structure | • Simplicity in design | • Might require more maintenance than double leafed counterparts<br>• Vulnerable to certain impacts |

**Table 3.4:** Comparison of Different Gate Types

The selected gates for further consideration should be based on the specific requirements of the project and the characteristics of each gate type as described in the table above.

### 3.4.2. Verification

Lock gates serve multiple purposes, chiefly to retain water and ensure a navigable passage. The suitability of a gate type depends on specific conditions like spatial constraints, water retention, equipment requirements, and potential vulnerabilities.
The table in Table 3.5 evaluates the suitability of each gate type. Each gate's suitability is primarily judged based on spatial constraints, though other factors, such as operation, maintenance, and vulnerability, also play crucial roles.

| Gate type | Suitability | Reasoning |
|---|---|---|
| Mitre gates | + | Suitable for both water retention directions with added length or specific adjustments. |
| Rolling/sliding gate | + | Suitable due to its capability to span the lock chamber width and its resistance to ship impact. |
| Lift gate (submersible) | + | Vertical footprint advantage, but can be costly in construction and maintenance. |
| Lift gate (upward direction) | -(C) | Restricts vertical clearance due to its upward movement, impacting air draught of vessels. |
| Radial gates | -(A) | Requires additional space beside the area of interest, leading to potential spatial constraints. |
| Single leaf gate | + | Simplified version of mitre gates, reducing some complexities but may have vulnerabilities. |

**Table 3.5:** Detailed assessment of different gate types based on their suitability. '+' indicates compatibility, '-' indicates a specific issue with suitability: (A) Requires excessive space beside the area of interest, (B) Not suitable for the lock width, (C) Restricts vertical clearance.

From this assessment, it is clear that lift gates moving in an upward direction and radial gates do not meet the criteria for further consideration. The primary concerns are the spatial constraints posed by radial gates and the vertical clearance restrictions of upward-moving lift gates. Conversely, Mitre gates, Rolling/sliding gates, Lift gates (submersible), and Single leaf gates show promise for the project, meeting the necessary criteria. Each of these gates, while having their distinct advantages, will also have particular challenges to address in the subsequent phases of the project.

### 3.4.3. Evaluation

The multi-criteria analysis below evaluates each gate type based on a set of essential project criteria. Each gate is examined for its strengths and weaknesses across the criteria, ensuring a comprehensive assessment. By the end of this evaluation, stakeholders will be equipped with a clear understanding of which gate type best aligns with project goals and constraints. (Volkart, 2014)

**Mitre Gates:**

- **Retaining Water:** Ensures excellent water retention due to its traditional design, especially when closed under hydrostatic pressure.
- **Reliability & Maintenance:** Historically proven reliability. Yet, being a traditional design, regular maintenance is necessary to keep the gate functioning optimally.
- **Future Proof:** Can handle moderate future changes, especially those in water levels. However, significant structural or mechanical upgrades may prove challenging.
- **Costs:** Moderate initial construction costs. Over the gate's life cycle, maintenance, particularly for the seal and hinge mechanisms, can increase costs.

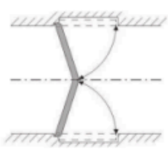
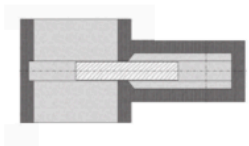
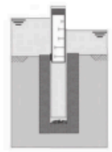
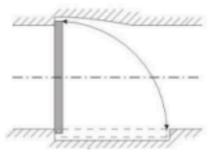
- **Constructability & Technical Feasibility:** Given its long history, construction techniques are well established, although it requires skilled labor familiar with the intricacies of the design.

**Rolling/Sliding Gate:**
- **Retaining Water:** Effective seal overall but may have some vulnerabilities at the bottom seal due to the sliding mechanism, which might let in minor seepages.
- **Reliability & Maintenance:** Mechanical parts, especially those associated with the sliding mechanism, could necessitate more regular check-ups to prevent jamming or misalignment.
- **Future Proof:** Adaptable to future requirements, especially with advancements in sliding and rolling technologies.
- **Costs:** While initial construction might be higher due to the complexity of the sliding mechanism, it might pay off in terms of reduced long-term maintenance.
- **Constructability & Technical Feasibility:** Relatively complex due to the sliding mechanism but feasible with the right equipment and skilled labor.

**Lift Gate (submersible):**
- **Retaining Water:** Highly efficient at water retention when submerged and in the closed position.
- **Reliability & Maintenance:** Given that many of its components are submerged, it may require frequent maintenance checks to ensure all parts function correctly.
- **Future Proof:** Moderately adaptable. Its vertical movement provides flexibility against varying water levels, but mechanical upgrades might be limiting.
- **Costs:** Likely higher initial cost due to its unique mechanism, but this might balance out with efficiency gains in water management.
- **Constructability & Technical Feasibility:** Requires specialized construction knowledge due to its submersion mechanism and potential challenges with installation.

**Single Leaf Gate:**
- **Retaining Water:** Efficient at water retention, but the sealing mechanism requires frequent checks to ensure no leakage.
- **Reliability & Maintenance:** While the design is simple, the sheer size of individual components can make maintenance tasks challenging, especially if replacements are needed.
- **Future Proof:** Limited adaptability owing to its singular, large component design. Structural changes or significant mechanical upgrades would be challenging.
- **Costs:** Benefits from a lower initial construction cost, but potential future costs could arise from component replacements or extensive maintenance.
- **Constructability & Technical Feasibility:** While the design is straightforward, the construction process might pose challenges due to the handling and installation of large components.

| Project | | Option 1 | | | Option 2 | | | Option 3 | | | Option 4 | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Gate Type MCA | | Mitre Gates | | | Rolling Gates | | | Submersible Lift Gate | | | Single Leaf Gate | | |

| Criteria | Weight | Remarks | score | * weight | Remarks | score | * weight | Remarks | score | * weight | Remarks | score | * weight |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| **Quality** | **#REF!** | | 20 | **105** | | 10 | **73** | | 11 | **73** | | 18 | **98** |
| **Retaining water** | **#REF!** | | 1 | **4** | | 4 | **16** | | 4 | **16** | | 3 | **12** |
| Both way retaining | 4 | Mitre gates are not both way retaining so a double set of doors is necessary | 1 | 4 | Rolling gate is both way retaining | 4 | 16 | Submersible lift gate is both way retaining | 4 | 16 | Single leaf gate is both way retaining when using a locking system | 3 | 12 |
| **Durability** | **7** | | 5 | **17** | | 6 | **21** | | 5 | **18** | | 4 | **15** |
| Lifetime of structure | 4 | In case of wood material, the lifetime of the gates is less than other gate options | 2 | 8 | Steel material because of width of gate | 3 | 12 | Steel material because of width of gate | 3 | 12 | Steel material because of width of gate | 3 | 12 |
| Future proof | 3 | Can handle moderate future changes, especially those in water levels. | 3 | 9 | Adaptable to future requirements, especially with advancements in sliding and rolling technologies. | 3 | 9 | Moderately adaptable. Its vertical movement provides flexibility against varying water levels, but mechanical upgrades might be limiting | 2 | 6 | Structural changes or significant mechanical upgrades would be challenging | 1 | 3 |
| **Reliability & maintenance** | **11** | | 7 | **22** | | 6 | **16** | | 6 | **18** | | 8 | **22** |
| Closure reliability | 5 | Possibility of obstruction by objects between the doors | 2 | 10 | Possibility of obstruction by objects between the doors | 2 | 10 | High closure reliability | 3 | 15 | Possibility of obstruction by objects between the doors | 2 | 10 |
| Maintainability | 3 | Doesn't need much maintenance | 4 | 12 | Rolling gates need frequent cleaning of the rails | 2 | 6 | Submersible lift gate needs cleaning of slot | 1 | 3 | Single leaf gate doesn't need much maintenance | 4 | 12 |
| Maintenance requirements | 3 | More frequent maintenance | 1 | 3 | Less frequent maintenance | 2 | 6 | Less frequent maintenance | 2 | 6 | Less frequent maintenance | 2 | 6 |
| **Sustainability** | **6** | | 4 | **12** | | 1 | **3** | | 1 | **3** | | 1 | **3** |
| $CO_2$ footprint during construction | 3 | No difference | 0 | 0 | No difference, slightly more emissions | 0 | 0 | No difference, slightly more emissions | 0 | 0 | No difference | 0 | 0 |
| Sustainable materials used | 3 | Wood material | 4 | 12 | Steel material | 1 | 3 | Steel material | 1 | 3 | Steel material | 1 | 3 |
| **Constructability** | **18** | | 15 | **50** | | 5 | **17** | | 6 | **18** | | 14 | **46** |
| Width suitability | 5 | No difference | 0 | 0 | No difference | 0 | 0 | No difference | 0 | 0 | No difference | 0 | 0 |
| Impact on surroundings during construction | 2 | Low impact, relatively easy construction in the lock chamber | 3 | 6 | Higher impact, because of space for rolling gate in the side wall | 1 | 2 | Low impact, as the gate will be constructed in the lock chamber | 2 | 4 | Low impact, relatively easy construction in the lock chamber | 3 | 6 |
| Technical feasibility | 4 | Relatively easy construction, also much knowledge in the area about these type of gates | 4 | 16 | Relatively complex due to the sliding mechanism, but there is also knowledge in the area about this gate type | 2 | 8 | Requires specialized construction knowledge due to its submersion mechanism and potential challenges with installation | 1 | 4 | Challenges due to the handling and installation of large components | 4 | 16 |
| Construction time | 4 | Lower construction time as this is a common gate type used in Amsterdam | 4 | 16 | Higher construction time by building opening for rolling gate | 1 | 4 | Higher construction time by excavating the opening for the gate | 1 | 4 | Moderate construction time, large components could take more time | 3 | 12 |
| Construction space needed | 3 | Gates need little space in side wall | 4 | 12 | More space needed on the side of the structure | 1 | 3 | Space needed under the structure | 2 | 6 | Gates need little space in side wall | 4 | 12 |
| **Costs** | **20** | | 5 | **50** | | 2 | **20** | | 2 | **20** | | 5 | **50** |
| Construction costs | 10 | Moderate initial construction costs, but higher for a double set of doors. | 3 | 30 | Initial construction might be higher due to the complexity of the sliding mechanism | 1 | 10 | Higher initial cost due to its unique mechanism | 1 | 10 | Lower initial construction cost | 3 | 30 |
| Maintenance costs | 10 | Maintenance cost are higher in case of wood doors and double sets of doors | 2 | 20 | Maintenace costs are high because the big size of the doors, the sliding mechanism also needs to stay clean | 1 | 10 | Maintenace costs are high because the big size of the doors, the slot needs to stay clean | 1 | 10 | Large components bring high maintenance costs | 2 | 20 |
| **Totaal** | **100** | | 155 | **155** | | 93 | **93** | | 93 | **93** | | 148 | **148** |

**Figure 3.3:** Multi-criteria Analysis for the gate type selection (Volkart, 2014) & Molenaar, 2011

### 3.4.4. Selection

Following the multi-criteria analysis, Mitre gates were identified as the most suitable choice. In addition to their excellent performance across various criteria, they particularly stand out due to their double-sided water retention capability and compatibility with a lock width of 7.6 m. Moreover, when compared to single-leaf Mitre gates, double-leaf Mitre gates present various advantages like faster opening and closing times, lesser hydraulic resistance, and better symmetry during motion. Hence, the final selection favours a double set of two Mitre gates for the guard lock design.

## 3.5. Gate Arrangement

The design of the mitre gates is based on the fundamental requirement for two-way water retention. Given the primary role of the gates in water defense, the objective is to maintain the water level within the inner water, Het Oog, between NAP + 0.1 m and NAP -0.6 m, as detailed in Table 3.1. This ensures the protection of Strandeiland against extreme low and high water levels resulting from wind set up and set down. This requirement informed the decision to use a double gate set, designed to contain high waters from both sides. For internal water retention, the preliminary design also includes a double gate set. This decision arises from ongoing uncertainties about the potential impacts of gate failure due to operational issues. As discussed in the problem statement and in the evaluation of options (**??**),

designating the inner quay wall as a primary flood defense is not recommended. Consequently, the inner quay wall is not fortified to resist extreme low water levels. A solitary mitre gate, resilient against extreme low water levels from the exterior, poses a risk to the integrity of the inner quay wall in the event of a failure. Therefore, a double door set in both directions is deemed advantageous.
The guard lock dimensions are crafted to potentially incorporate a filling and emptying (FE) system in the future, even if it's not part of the initial phase. Such a design not only offers flexibility for future modifications but also precludes potential costly structural changes. Therefore, even if an FE system isn't integrated initially, the guard lock is readied for its inclusion, enhancing its longevity, adaptability, and future utility. The guard lock comprises two distinct lock chambers, each measuring 22 meters. Consequently, the entire structure spans 36.46 meters, encompassing lock heads with gates and driving mechanisms. An illustrative layout is presented in Figure 3.4, which depicts the gate arrangement and the lock structure's dimensions.

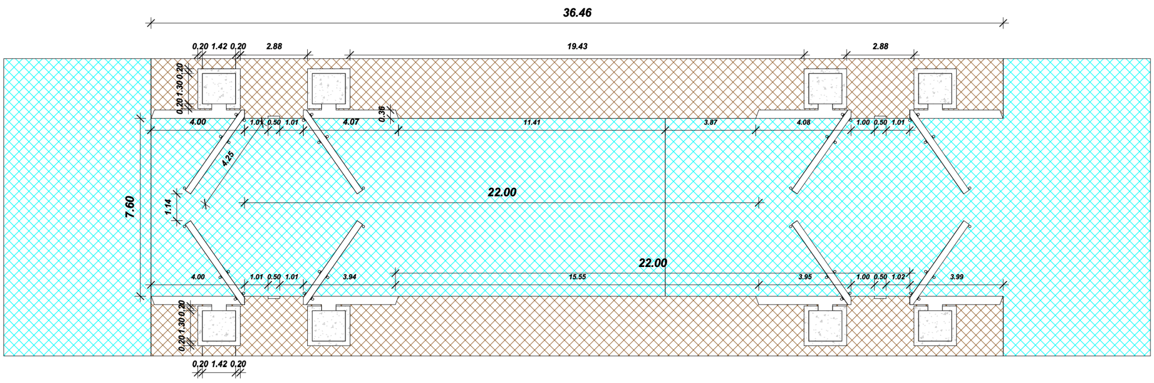



**Figure 3.4:** Diagram illustrating the gate arrangement of the guard lock, which features two sets of double mitre gates allowing both-way water retention. The figure also provides indications of the guard lock's main dimensions.

## 3.6. Functional Design of the Lock Head

The functional design of the Strandeiland guard lock head revolves around the selection of a mitre gate configuration. The design aims to serve three primary functions:

1. Accommodating the gate and facilitating its operations.
2. Ensuring water retention, sealing, and prevention of seepage.
3. Managing hydrostatic load transfer.

### 3.6.1. Gate Accommodation and Operation

The lock head incorporates recesses designed specifically for the mitre gates when they are in the open position. This design aspect minimizes the risk of collisions with passing vessels. Furthermore, the recesses are tailored to match the dimensions of the selected mitre gates. Embedded within the lock head structure is the necessary gate operating equipment to ensure the gates function efficiently.

### 3.6.2. Water Retention and Seepage Prevention

The lock head plays a pivotal role in water retention. The design of the seal between the mitre gate and the lock head structure ensures a watertight fit, preventing leakage and maintaining the lock's efficacy. Groundwater flow around and beneath the lock head is also accounted for, with the lock head and chamber interconnected in a watertight manner. To further prevent groundwater seepage, which could lead to structural erosion or piping, cut-off screens are proposed for construction under and beside the lock head.

### 3.6.3. Load Transfer Management

The lock head is designed to effectively transmit the hydrostatic load when the mitre gates are closed. Its walls and floor distribute this load to the foundation, ensuring minimal deformation under pressure.

This design takes into consideration local soil conditions and the expected load from the retained water, yielding a robust structure resilient to extreme conditions.
In addition to these core functions, the design process also factored in operational requirements, boundary conditions, and local conditions, leading to a comprehensive and efficient structure for the Strandeiland guard lock.

### 3.6.4. Additional Features

The design integrates stop-log recesses, vertical slots located both in front of and behind the gate. These slots enable the insertion of horizontal beams, or stop-logs, which can be stacked to form a retaining wall. Steel stop-logs, preferred for their resilience, serve three main purposes:

1. Acting as a backup to the primary gate.
2. Facilitating the emptying of the lock chamber.
3. Offering protection to the closing structures, such as against floating ice.

For the gate's movement, an electromechanical driving mechanism is favored over a hydraulic one. The reasons behind this choice are detailed in Table 3.6.

| **Consideration** | **Advantage of electromechanical system** |
|---|---|
| Energy Efficiency | Require less energy to function |
| Maintenance Costs | Often incur lower maintenance costs |
| Increased Control | Ensure the smooth and accurate operation of the gate |
| Noise | Produce less noise |
| Environmental | Do not require hydraulic fluid, preventing potential pollution |

**Table 3.6:** Table providing considerations that led to the choice of an electromechanical driving mechanism over a hydraulic system.

A depiction of a monolithic U-shaped lock structure, inclusive of the driving mechanism, is showcased in Figure 3.5. This illustrates the driving mechanism's integration into the lock head. A comprehensive design of the lock head, including the driving mechanism, can be found in chapter 4.

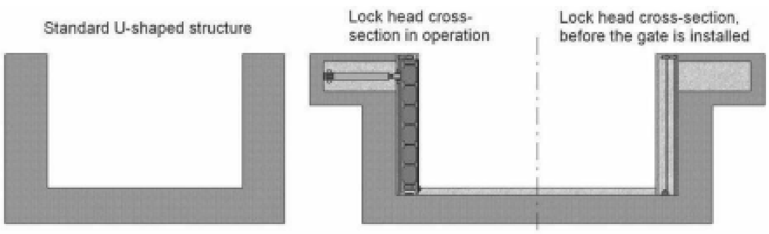



**Figure 3.5:** Illustrative cross-section representation of a monolithic U-shaped lock structure, demonstrating the integration of the driving mechanism within the structure. (Molenaar, 2011)

## 3.7. Design of the Lock Chamber

The lock chamber's design is pivotal to the overall functionality of the guard lock. Although a three-dimensional structure, the lock chamber's design predominantly revolves around analyzing its two-dimensional cross-section, simplifying the design process. The primary emphasis is on the lock chamber walls and the lock chamber floor.

### 3.7.1. Functionality of the Lock Chamber Components

| Component | Function | Description |
|---|---|---|
| Lock Chamber Walls | Soil and Water Retention | Structurally robust to retain the surrounding soil and water, preventing unwanted influx. |
| | Guiding Vessels | Serve as guides for vessels, ensuring smooth and safe navigation through the lock. |
| | Mooring Vessels | Provide mooring points during the filling and emptying process, ensuring vessel stability. |
| Lock Chamber Floor | Water Retention | Designed to prevent water leakage, maintaining the lock's operational efficiency. |
| | Load Transfer | Distributes and transfers the load of water and vessels to the underlying foundation. |

**Table 3.7:** Design considerations for the lock chamber walls and floor.

### 3.7.2. Operational Considerations

Considering the projected operational conditions, the lock is expected to close about ten times per year. These closures often align with high water conditions, adverse weather, and a decrease in pleasure boat usage. As a result, the lock will not incorporate a filling and emptying (F/E) system to enhance operational efficiency and cost-effectiveness. Instead, it will operate solely as a guard lock, restricting access to inland water for vessels during closures.

### 3.7.3. Groundwater Seepage and Countermeasures

A potential hydraulic gradient could induce groundwater flow under and around the lock. If seepage becomes significant enough to transport soil particles, it could lead to erosion threatening the lock's stability. A proposed countermeasure involves vertical seepage cut-off screens under and beside the lock. These screens aim to disrupt groundwater flow, mitigating seepage. A detailed analysis during the piping check will determine the necessity of these screens.

### 3.7.4. Approach Structures and Berthing Facilities

The design also encompasses approach structures and berthing facilities, tailored to the area's specific requirements:

- **Approach Structures:** These are primary entry and exit points for vessels, ensuring their safe and efficient navigation and minimizing risks.
- **Berthing Facilities:** Designed to cater to various vessel types, they ensure safe mooring, especially for larger vessels requiring specialized equipment during the lock operation.

While these components are integral to the lock's overall functionality, this report's main focus remains on the guard lock's functional and structural design. Detailed analyses of the approach structures and berthing facilities will be addressed in subsequent project stages.

## 3.8. Summary

This chapter outlines the design considerations for Strandeiland's guard lock. The structure, totalling 36.46 meters in length and 11.90 meters in width, houses two lock chambers each measuring 22 meters in length. These dimensions are tailored to accommodate the normative vessel size of 20 x 4.25 x 1.90 meters. In this conceptual design, the lock does not incorporate a filling and emptying system; however, it has been designed to accommodate the potential future integration of such a system. This decision was informed by the expected lock closure frequency, estimated at around ten times per year, predominantly during storm events when ship traffic is likely to be minimal. The lock's waterway depth is -2.90 meters. Four sets of mitre gates, selected through a multi-criteria analysis, are integrated into the design to enable two-way water retention. The aim is to regulate the water level in 'Het Oog' between NAP +0.1 meters and NAP -0.6 meters, as detailed in Figure 3.6. The chosen gate driving mechanism is electromechanical, offering advantages such as energy efficiency, reduced maintenance costs, precise control, and quieter operation compared to hydraulic systems. Lastly, the potential requirement for seepage cut-off screens will be examined further in the upcoming piping checks chapter. Collectively, this chapter presents the integral design considerations shaping the guard lock of Strandeiland, ensuring it efficiently retains water and facilitates safe vessel passage.

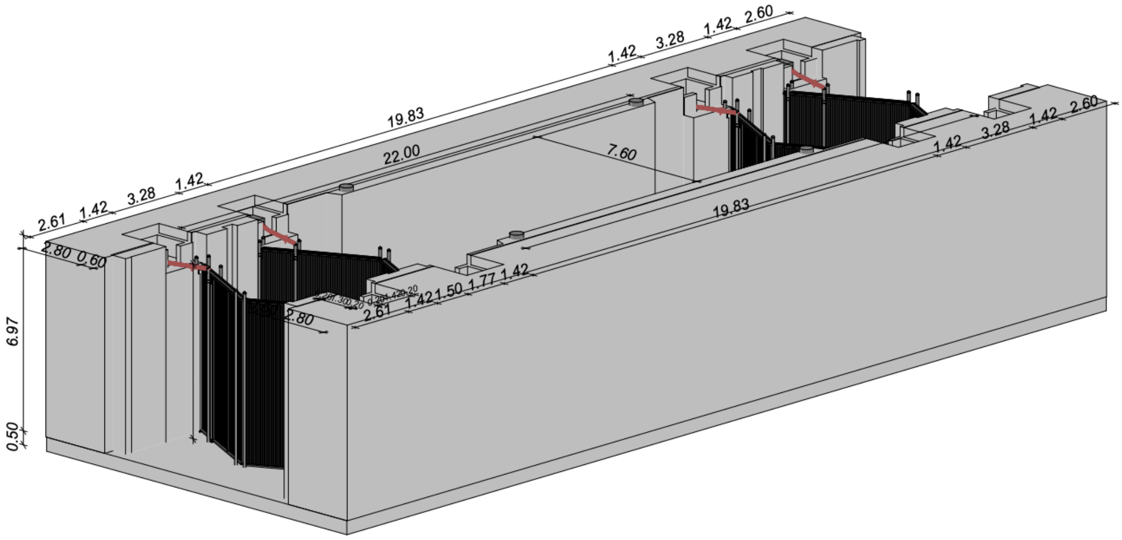



**Figure 3.6:** Main dimensions and components of the guard lock

# 4 Structural Design

This chapter delves into the core structural considerations of the guard lock's design. section 4.1 delves into constructability, providing a detailed sequence outlining the construction phases of the guard lock. Stability, a key aspect of the design, is examined in section 4.2. Both micro and macro stability checks for the structure are conducted to ensure its long-term resilience.Lastly, section 4.3 discusses strength and stiffness. This part of the chapter is dedicated to the design aspects of the pile foundation and reinforcement, ensuring the structure's robustness and durability.

## 4.1. Constructability

**Step 1: Installing Dewatering**
To ensure a dry working condition within the construction pit, a dewatering system is put in place. This involves setting up appropriate mechanisms, such as wellpoints or deep wells, to lower the groundwater level temporarily. This step is crucial in facilitating the subsequent excavation and construction processes.

**Step 2: Preparation of an Open Construction Pit with Drainage**
After the dewatering system is in place, the next step is to prepare the site for construction. Initially, the island is built up entirely with sand. This provides a stable foundation for the subsequent steps. Once this foundational layer is established, an open construction pit for the guard lock is excavated. Given the spacious nature of the area designated for the guard lock, the site is free from constraints posed by housing or other existing structures. To maintain a stable construction environment and effectively address potential water infiltration challenges, a comprehensive drainage system is implemented around the pit's perimeter. This measure ensures a dry workspace, setting the stage for the subsequent construction phases.

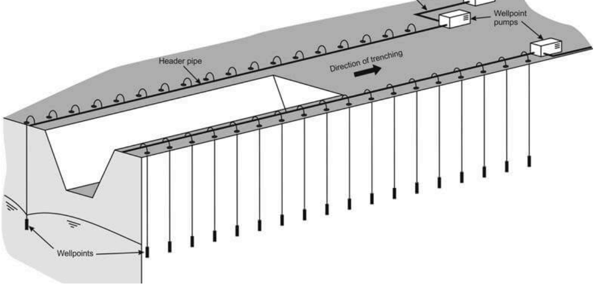



**Figure 4.1:** Step 2: Preparation of an Open Construction Pit with Drainage (Deloney, 2023)

**Step 3: Excavation**
With the foundational layer and drainage system in place, the excavation process begins. Soil is removed to reach the desired depth for the guard lock. The excavated region will be the site where

structural concrete is directly cast onto the subsoil, a common practice to ensure stability.

**Step 4: Pile Construction**
The decision to employ a pile foundation hasn't been finalized. This assessment will be conducted in subsection 4.2.3. However, when examining constructability, the pile construction is considered. For the construction process, prefabricated concrete piles are employed. They are driven into the ground using specialized machinery to reach deeper, more stable soil layers. Prefabricated concrete piles offer advantages like consistent quality, elimination of curing time on-site, and accelerated construction.

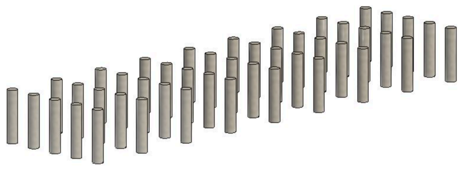


**Figure 4.2:** Step 4: Pile Construction

**Step 5: Formation of a 50 cm Concrete Slab**
Once the piles are adequately set and cured, a 50 cm thick concrete slab is cast over the area. Formwork is strategically positioned to hold the concrete during its curing process. Typically reinforced with steel bars, this slab is designed for increased strength and durability.

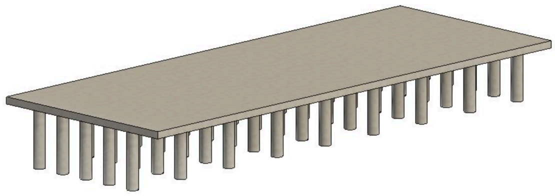


**Figure 4.3:** Step 5: Formation of a 50 cm Concrete Slab

**Step 6: Casting the 60 cm Concrete Wall**
After the slab is cured, formwork is established, and a 60 cm thick concrete wall is cast directly onto the subsoil. Reinforced with steel, this wall is designed for resilience and strength.

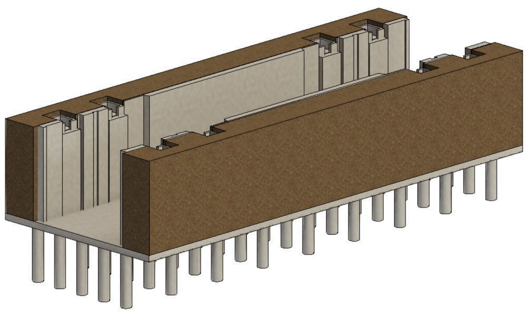


**Figure 4.4:** Step 6: Casting the 60 cm Concrete Wall

**Step 7: Installation of the Gate Opening Mechanism**
Once the concrete wall and slab are fully cured, the gate opening mechanism installation follows. This involves creating mounting brackets within the concrete wall, establishing necessary electrical or hydraulic circuits, and integrating the actual gate opening mechanism.

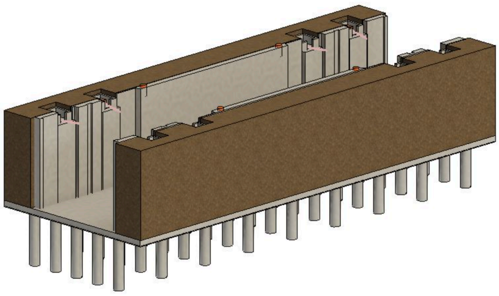


**Figure 4.5:** Step 7: Installation of the Gate Opening Mechanism

**Step 8: Installation of Steel Gates**

In the final step, steel gates are transported over water to the construction site. A crane, situated on location, is utilized to hoist and install the gates onto the concrete walls using the previously installed opening mechanism. After installation, the gates are fine-tuned for seamless operation, and any remaining electrical or hydraulic connections are established.

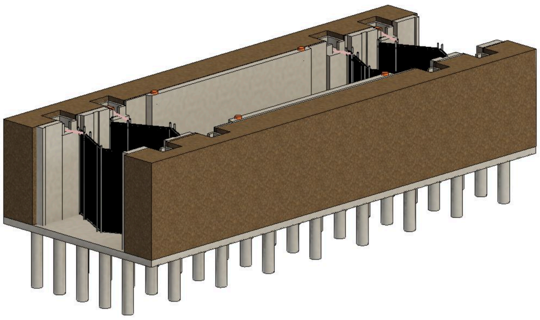


**Figure 4.6:** Step 8: Installation of Steel Gates

## 4.2. Stability

The stability of any hydraulic structure is of utmost importance to ensure its longevity and safe operation. Therefore, in the structural design of the guard lock of Strandeiland, the first step is to perform various stability checks. These checks will determine the bearing capacity of the soil, both vertically and laterally, to ensure that the structure is stable under the applied loads. The checks will also include over-turning analysis and piping checks to assess the stability of the structure under the influence of water pressure and seepage. This section delves into the detailed methodologies used for performing these stability checks, along with the results obtained.

Scour has not been considered in the stability analysis due to the expectation of low water velocities around the guard lock. However, specific data on site conditions is currently unavailable since the island construction is ongoing. Further investigation into this aspect could be beneficial once more information is accessible.

For overall stability the following failure mechanisms are checked:

**Micro stability:**

1. Piping

**Macro stability:**

2. Bearing capacity (vertical) $\sum V = 0$
    - Uplift
    - Settlements

Given the size of the structure, overturning is not expected to pose an significant risk. We start with the vertical bearing capacity check also containing uplift and settlements. This stability check will show the necessity of using a pile foundation. For example, if the settlements are not within acceptable margins, a pile foundation is used because a shallow foundation would not suffice

### 4.2.1. Water Levels and Dimensions for Stability Checks

The stability checks for the guard lock structure require specific water level data, which is based on the information provided in the Basis of the Design in Chapter 2. To determine the upper boundary of the inner water, a limit of NAP + 0.1 m is set, which results in a depth of 3.60 meters in the lock chamber. Conversely, the lower boundary of the inner water is limited to NAP - 0.6 m, which corresponds to a depth of 2.90 meters in the lock chamber. The crest height is set at +1.1 m NAP. This corresponds to a water depth of 4.60 meters within the guard lock. The establishment of this level is derived from the considerations detailed in Section 2.4.1, and it is graphically represented in Figure A.3.
The average ground water level (GWL) is determined to be NAP + 0.9 m and the gathering of this data is explained in Section 2.3.2. The ground level (GL) is at + 3.7 m NAP. The specific water levels used for the stability checks are shown in Figure 4.7. Critical water levels and circumstances are determined for each stability check.

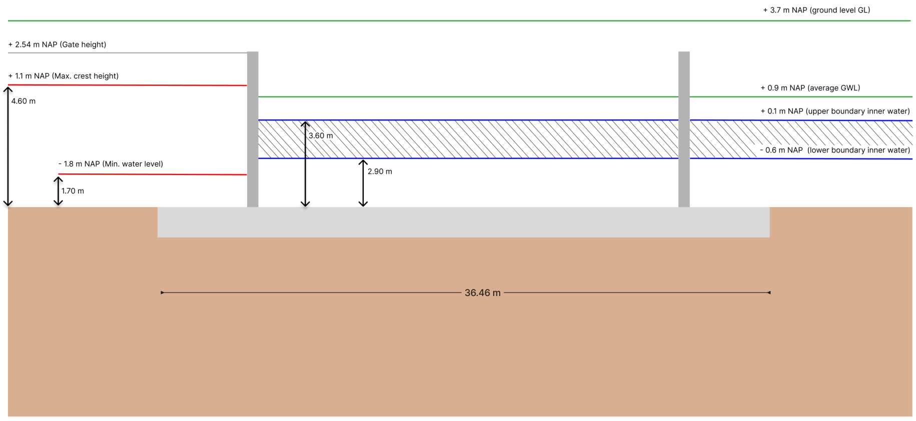



**Figure 4.7:** Water levels for the stability checks

In Appendix A.3, all the soil data input required for the stability checks can be found, which was obtained from the geotechnical research conducted by Fugro and further elaborated upon in the appendix.

### 4.2.2. Vertical Bearing Capacity

While an earlier design consideration assumed the use of a pile foundation, this analysis aims to evaluate the feasibility of employing a shallow foundation. This shift in foundation type necessitates a thorough check to guarantee that the foundation can sustainably support the weight of the lock and any additional loads that might be imposed.

The guard lock's foundation is subjected to a vertical bearing capacity check to confirm the foundation soil's competence in supporting the structure's load without significant settlement or bearing failure. The chosen method for this analysis is the Brinch Hansen approach, a well-established method in soil mechanics to gauge the bearing capacity of foundations Arora, 2004.

Figure 4.8 provides a visual representation of the guard lock structure, highlighting the components contributing to the vertical load.

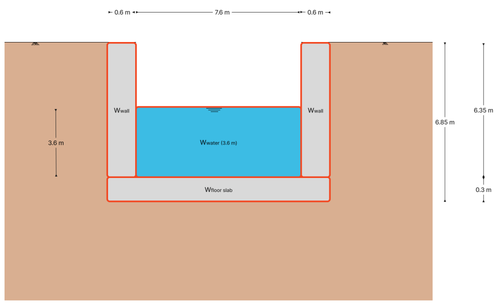



**Figure 4.8:** Components of the guard lock adding to the vertical load.

The reference scenario for this assessment assumes the guard lock at its maximum water level, resulting in the peak vertical load on the foundation. Using the Brinch Hansen method, the soil's ultimate bearing capacity is determined through Equation 4.1:

$$q_v = cN_c s_c d_c i_c b_c g_c + qN_q s_q d_q i_q b_q g_q + 0.5\gamma B N_\gamma s_\gamma d_\gamma i_\gamma b_\gamma g_\gamma \tag{4.1}$$

The terms in the equation arise and are rationalized as:

- $N_c, N_q, N_\gamma$: Hansen Bearing capacity factors, derived from soil properties (see Appendix E).
- $s_c, s_q, s_\gamma$: Shape factors, based on the shape of the foundation.
- $d_c, d_q, d_\gamma$: Depth factors, based on the depth of foundation below ground level.
- $i_c, i_q, i_\gamma$: Inclination factors, relevant if the load is not vertical.
- $\gamma$: Unit weight of the soil, obtained from geotechnical investigation.
- $C$: Shear strength of the soil, obtained from geotechnical investigation.
- $\phi$: Friction angle of the soil, obtained from geotechnical investigation.
- $q_0$: Effective stress at the bearing level, calculated based on soil unit weight and depth of foundation.

Employing these parameters, the soil's ultimate bearing capacity is determined to be **334.0** Kn/m$^2$.

The guard lock's vertical load is deduced by aggregating the reinforced concrete structure's weight with the weight of water at its highest level within the lock. The reinforced concrete's weight is ascertained using its dimensions and material specifics. The comprehensive vertical load, when averaged across the lock's expanse, results in a load per unit area, which is **119.8** $kN/m^2$.
Comparing this per unit area load with the ultimate bearing capacity yields:

$$q_{\text{vertical\_load}} < q_{\text{allow}} \;\rightarrow\; 119.83\;kN/m^2 < 334.0\;\text{ Kn/m}^2 \quad \checkmark \tag{4.2}$$

This inequality signifies the structure's applied load is beneath the soil's bearing capacity. Hence, the soil can adequately support the structure, affirming the guard lock design's adequacy in terms of vertical bearing capacity.

A reflection on these results indicates that the derived bearing capacity is nearly threefold the vertical load. Given the guard lock's sturdy design coupled with geotechnical design methodologies' inherent conservatism, this finding is credible. A substantial safety margin is evident, suggesting the lock's design is poised to function optimally under the anticipated maximum load.

As indicated in the constructability illustration, the design will incorporate a pile foundation in anticipation of non-uniform settlement, which might induce structural deformations. The integration of a pile foundation undoubtedly enhances the vertical bearing capacity. The subsequent calculation showcases the vertical bearing capacity check for the pile foundation (for an exhaustive calculation, see Appendix E):

$$\text{q}_{\text{vertical load}} < q_{\text{allow}} \quad \rightarrow \quad 29\text{ Kn/m}^2 < 1728\text{ Kn/m}^2 \quad \checkmark$$

**Uplift**
The buoyant force exerted by the groundwater is a potential source of uplift, threatening the stability of the structure. To ensure stability against this uplift force, the total downward force from the structure must surpass it. This downward force is a combination of the weight of the structure and the weight of the soil situated above the groundwater level.
Parameters considered in this calculation include:

- Groundwater level relative to the foundation base: $4.9$ meters
- Foundation dimensions:
  - Length: $36.46$ m
  - Width: $8.8$ m
  - Depth: $0.5$ m
- Unit weight of soil: $20$ kN/m$^3$
- Unit weight of water: $10$ kN/m$^3$

From these, the uplift force due to groundwater is calculated to be approximately $15,721.55$ kN.
Based on the Vertical Bearing Capacity calculations (refer to Section 4.2.2), the combined downward force is estimated to be $38,421.68$ kN:

$$F_{\text{uplift}} < F_{\text{downward}} \rightarrow 15,721.55 \text{ kN} < 38,421.68 \text{ kN} \quad \checkmark$$

Considering that the downward force substantially surpasses the uplift force, the structure is assessed to be stable against the threat of uplift. Detailed calculations supporting this assertion can be found in Listing E.3 within the Appendix.

**Settlements**
For the settlement analysis, the first layer, comprised of gravel-sand-mud, has a depth of 5.7 meters. The high bearing capacity of this layer suggests minimal settlement. However, a comprehensive assessment requires evaluating both immediate and consolidation settlement.
Given the composition of the first layer, immediate settlement is determined using the theory of elasticity. For more compressible layers, like the fifth layer of siltstone, consolidation settlement becomes more relevant.

The immediate settlement $\delta$ is computed as:

$$\delta = \frac{q \times B}{E} \times I_s$$

Where:

- $q$ is the applied pressure on the soil. This pressure is determined by the vertical stress imposed by the structure. Specifically, $q$ is calculated as the total weight of the structure divided by the area of the foundation, representing the average pressure or stress the structure imparts on the soil.
- $B$ is the width of the foundation, given as 8.8 meters.
- $E$ is the modulus of elasticity of the soil. For the gravel-sand-mud composition, it is assumed to be $40,000$ kPa, based on typical values.
- $I_s$ is the influence factor, dependent on the shape and depth of the foundation and the soil type. An average value of 0.7 is assumed for this calculation.

For the given conditions and assumptions, the calculated immediate settlement is approximately $0.0111$ meters or $11.1$ mm. The calculations are shown in Listing E.4 in the Appendix.
As this size of settlement is considered acceptable, the settlement is expected to not be uniform over the length of the structure. This will cause deformations which are not desirable, therefore it is still decided to apply a pile foundation.

### 4.2.3. Pile Foundation Design
This section presents the design considerations for the foundation of the guard lock at Strandeiland, influenced by several parameters such as bearing capacity, soil behavior under load, compressibility, permeability, and economic feasibility. Figures displaying the cross-sectional view of pile foundation design, length cross-section view of pile foundation, foundation plan (top view), and an overview of the whole structure, alongside data input and calculations presented in Appendix A.3, support these considerations.

**Selection of Foundation Layer**
Three layers were evaluated as potential foundation layers.

- The first layer, comprising of gravel-sand-mud, possesses high bearing capacity (Q0 19 $MN/m^2$), but is relatively shallow at 5.7 meters. The length of the piles might necessitate deeper layers for improved stability.
- The fifth layer of siltstone, bearing a capacity of 5 $MN/m^2$, is deeper at 14.5 m with a unit weight (range) of 1800 $kg/m^3$. While silty clay is generally less favorable due to its compressibility and low shear strength, its capacity is quite substantial.

- The seventh layer, consisting of Silty Sand, is the deepest at 22 meters with a moderate bearing capacity of 6 $MN/m^2$. This layer offers optimal stability, especially if piles are sufficiently long, but the process of driving piles to this depth might be economically disadvantageous.

Below in Figure 4.9 is a schematization shown of the different layers. Soil layer schematization is obtained the cone penetration testing (CPT) done by Fugro which is shown in Appendix A.2

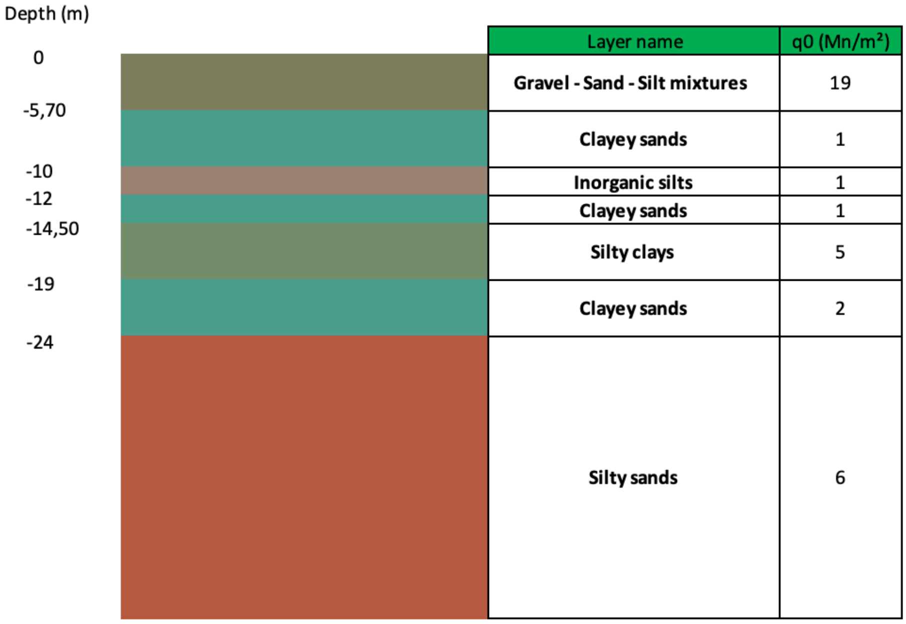



**Figure 4.9:** Soil layers schematization of Fugro CPT in Appendix A.2

**Enhancing Pile Stability**
Several strategies can be employed to improve the stability of piles in the first layer:

- **Increase pile diameter:** Larger diameter piles offer a more expansive bearing surface, thus distributing load more evenly and reducing the risk of failure.
- **Adopt a pile cap or mat foundation:** This foundation type distributes the load over a larger area, consequently enhancing individual pile stability.
- **Grouping of piles:** Implementing several closely-arranged piles to carry the same load can increase the safety factor and stability.
- **Soil improvement techniques:** These involve enhancing the soil's strength and stability, possibly through compaction, grouting, or use of geosynthetics.

### 4.2.4. Assessing the First Layer Suitability

**Overview of Foundation Design**
The foundation design was a meticulous process that evaluated three potential layers: the first (gravel-sand-mud), fifth (siltstone), and seventh (Silty Sand). Both engineering requirements and economic implications influenced the choice of foundation layer. Moreover, the selected layer would determine the strategies for enhancing pile stability. These could include increasing pile diameters, adopting a pile cap or mat foundation, grouping the piles, or using soil improvement techniques.
The first layer was identified as the most apt to bear the structure's load, primarily because of its capacity to mitigate non-uniform settlements. As a result, the pile foundation design was conceived with 45 piles, each spanning a diameter of 0.8 m.

**Load Distribution Among Piles:** The structure's total vertical load is $38,421.68$ kN. With 45 piles, the load distributed to each pile is calculated as:

$$\text{Load per pile} = \frac{\text{Total vertical load}}{\text{Number of piles}} = \frac{38,421.68\,\text{kN}}{45} = 853.81\,\text{kN}$$

**Pile Bearing Capacity:** The bearing capacity for each pile is a product of its cross-sectional area and the soil's safe bearing capacity. For a pile of 0.8 m diameter:

$$A = \pi \times \left(\frac{\text{diameter}}{2}\right)^2 = \pi \times \left(\frac{0.8\,\text{m}}{2}\right)^2 = 0.502\,\text{m}^2$$

Hence, each pile's bearing capacity is:

$$\text{Bearing capacity per pile} = A \times \text{Safe bearing capacity of the first layer} = 0.502\,\text{m}^2 \times 19\,\text{MN/m}^2 = 9.54\,\text{MN or } 9540\,\text{kN}$$

The calculations confirm that each pile can safely bear a load of 9540 kN, a value notably higher than the 853.81 kN required per pile. This robust capacity endorses the first layer as an optimal foundation, qualified to manage both the vertical load and the specified horizontal load of 24.85 kN.
The final design not only meets the engineering specifications but also presents an economical solution. A graphical depiction of the pile design, through longitudinal and cross-sections, is available in Figure 4.10.

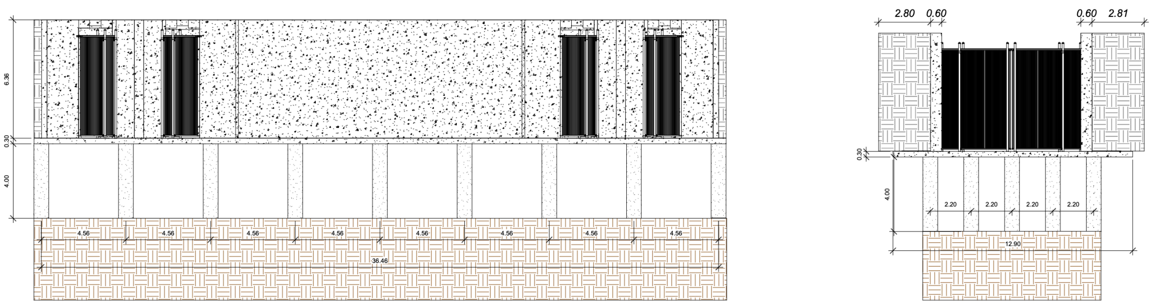



**Figure 4.10:** Longitudinal- and cross-section overview pile design

Figure 4.11 presents the plan view of the pile foundation, offering a top-down perspective. It illustrates the spatial arrangement of the piles under the structure, clearly marking their locations and distances from each other.

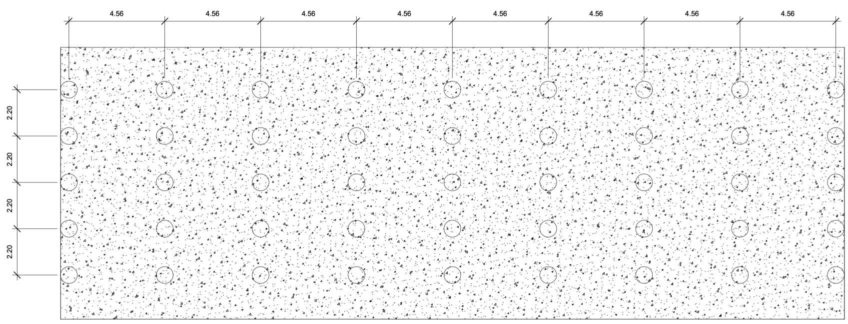



**Figure 4.11:** Pile foundation plan overview

Figure 4.12 below provides a comprehensive view of the structure, specifically highlighting the inclusion of the pile foundation. It serves as a visual guide to better understand the placement and role of the pile foundation within the overall structure.

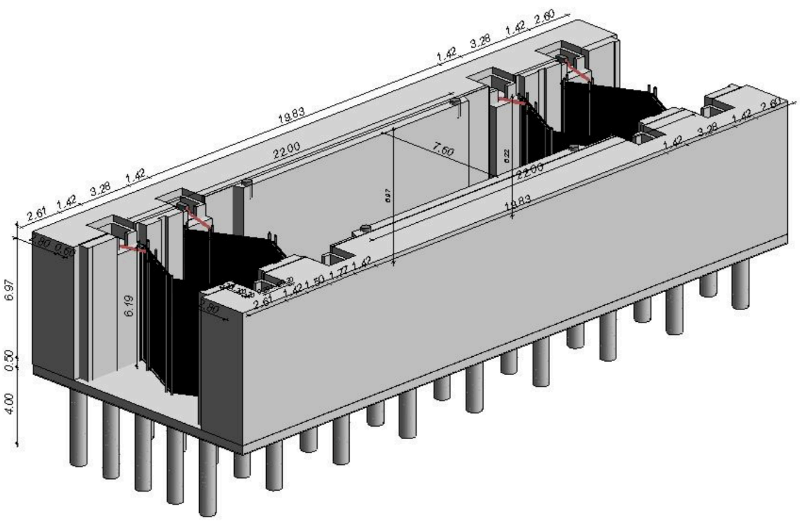


**Figure 4.12:** Overview of structure with the pile foundation included, some dimensions are illustrative

### 4.2.5. Omission of Overturning and Lateral Bearing Capacity checks

In foundation engineering, the selection of a particular type of foundation often alleviates some concerns while highlighting others. With pile foundations, there's a shift in emphasis from horizontal stability checks (such as overturning and lateral bearing capacity) to vertical bearing capacity checks. Here's why:

**Distribution of Loads in Pile Foundations**
Pile foundations distribute loads from the structure deep into the ground, reaching layers of soil or rock that have much higher load-bearing capacities. Due to the depth and the manner in which piles transfer loads, they inherently have a better resistance against lateral forces and overturning moments.

**Overturning**
Overturning is a phenomenon where the structure attempts to rotate about a point due to unbalanced moments, typically resulting from horizontal forces. In shallow foundations, where the base of the structure is close to the ground surface, overturning is a genuine concern due to the leverage effect. However, in pile foundations:

1. The depth of the foundation means that any horizontal force has to create a moment significant enough to overturn the structure deep from the ground surface, which is inherently difficult.
2. Piles, being driven deep, have considerable resistance against rotation due to the soil's or rock's grip along the length of the pile.

Let's illustrate this with a simple calculation. If we assume a shallow foundation subjected to a horizontal force $F_h$ at a height $h$ from the base, the overturning moment $M_o$ would be:

$$M_o = F_h \times h$$

For a pile foundation, this force would have to act at a much greater effective height (depth of the pile + above-ground height) to generate a similar moment. In most real-world scenarios, it's unlikely for such a significant horizontal force to act at that depth.

**Lateral Bearing Capacity**
While shallow foundations primarily rely on the breadth and depth of the foundation to resist lateral forces, pile foundations leverage their depth. The frictional resistance along the length of the pile in the soil provides a significant counter to lateral forces. Furthermore, the deeper layers of soil, where the pile tip often rests, generally have a higher shear strength, providing additional lateral resistance.
For example, the lateral resistance $R_l$ offered by a pile can be roughly estimated as:

$$R_l = \mu \times \gamma \times h \times A$$

Where:

- $\mu$ = coefficient of friction between the pile and soil (often taken as a fraction of the soil's angle of internal friction)
- $\gamma$ = unit weight of the soil
- $h$ = effective depth of the pile in the soil
- $A$ = surface area of the pile in contact with the soil

This resistance is significant and often surpasses the lateral loads that the structure might experience.

**Conclusion**
Given these factors, the design and stability checks for pile foundations prioritize vertical bearing capacity. While it's essential to ensure the pile design can handle vertical loads, the concerns related to overturning and lateral bearing capacity are inherently addressed due to the nature and depth of pile foundations.

### 4.2.6. Piping

Piping is a phenomenon that threatens the structural stability of hydraulic structures, such as guard locks. In pile-founded structures, piping becomes especially critical due to the potential for the subsoil to settle while the structure, anchored to a deeper stable layer, remains unaltered. This differential settling can result in voids beneath the structure, promoting unimpeded water flow and potential internal erosion.

| **Piping method:** | **Bligh** | | **Lane** | |
|---|---|---|---|---|
| criterion | $L \geq \gamma \cdot C_B \cdot \Delta H$ | | $L \geq \gamma \cdot C_L \cdot \Delta H$ | |
| used seepage length | $L = \sum L_{vert} + \sum L_{hor}$ | | $L = \sum L_{vert} + \sum \frac{1}{3} L_{hor}$ | |
| | $C_B$ | $i_{max}$ | $C_L$ | $i_{max}$ |
| **Soil type:** | | | | |
| Very fine sand / silt /sludge | 18 | 5,6 % | 8,5 | 11,8 % |
| Fine sand | 15 | 6,7 % | 7,0 | 14,3 % |
| Middle fine sand | - | - | 6,0 | 16,7 % |
| Coarse sand | 12 | 8,3 % | 5,0 | 20,0 % |
| (fine) gravel (+sand) | 5-9 | 11,1 – 20,0 % | 4,0 | 25,0 % |

**Figure 4.13:** Methods for Piping check from Bligh and Lane (Molenaar, 2011)

Figure 4.14 represents the potential piping path for the guard lock design, illustrated by black arrows.

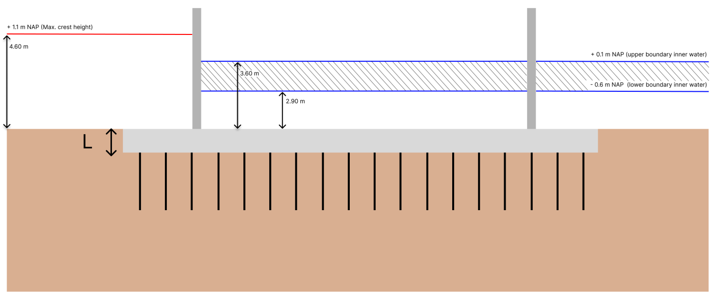



**Figure 4.14:** Sketch showing the horizontal piping length and head difference

Utilizing Bligh's formula, the risk of piping is assessed. The critical seepage distance according to Bligh is calculated as:

$$L > \gamma \times C_B \times H$$

Where:

- $L$ is the floor thickness, which is 0.5 meters.
- $\gamma$ is the safety factor, taken as 1.5.
- $C_B$ is Bligh's constant, which is 15 for the given soil type.
- $H$ is the differential head across the structure, 1.7 meters in this case.

Including the values:

$$0.5 > 38.25 (pipingrisk)$$

In conclusion, based on the above calculations, the guard lock design might be at risk for piping according to Bligh's method. It is recommended to incorporate a seepage screen to prevent piping and enhance the seepage length.

### 4.2.7. Results of Stability Checks

Ensuring the stability of the guard lock is paramount in the design process. Various stability checks were conducted to validate the design's safety. A summary of these checks, their outcomes, and the recommended safety measures is provided below:

| Stability Check | Status | Safety Measures |
|---|---|---|
| Piping | Not safe | Seepage screen inclusion |
| Vertical Bearing Capacity | No uniform settlement | Pile foundation |
| Lateral Bearing Capacity | No risk expected | No measure |
| Overturning | No risk expected | No measure |

**Table 4.1:** Summary of stability checks and safety measures for the guard lock foundation.

While most stability checks indicate that the design of the guard lock structure is in alignment with safety criteria, the piping check suggests potential vulnerability. Thus, the recommendation is to include a seepage screen as a safety measure. Following these stability checks, the next step is to proceed with strength calculations to further ascertain the design's robustness.

## 4.3. Strength and Stiffness

Having determined the definitive loads caused by waves, the next step involves calculating the dimensions for various structural components. These components encompass the concrete U-shaped walls and floor slab, as well as the reinforcement detailing for these. The design and dimensioning of these components are pivotal to ensuring the guard lock can endure the forces and loads it will face throughout its service life. This section outlines the necessary design calculations and detailing to achieve the required strength and stiffness for the guard lock.

### 4.3.1. Reinforcement Design

This section provides a brief overview of the reinforcement design implemented for the core structural components of the guard lock: the columns (or piles), walls, and the floor slab. In Figure 4.15 the locations of each reinforcement detailing are indicated.

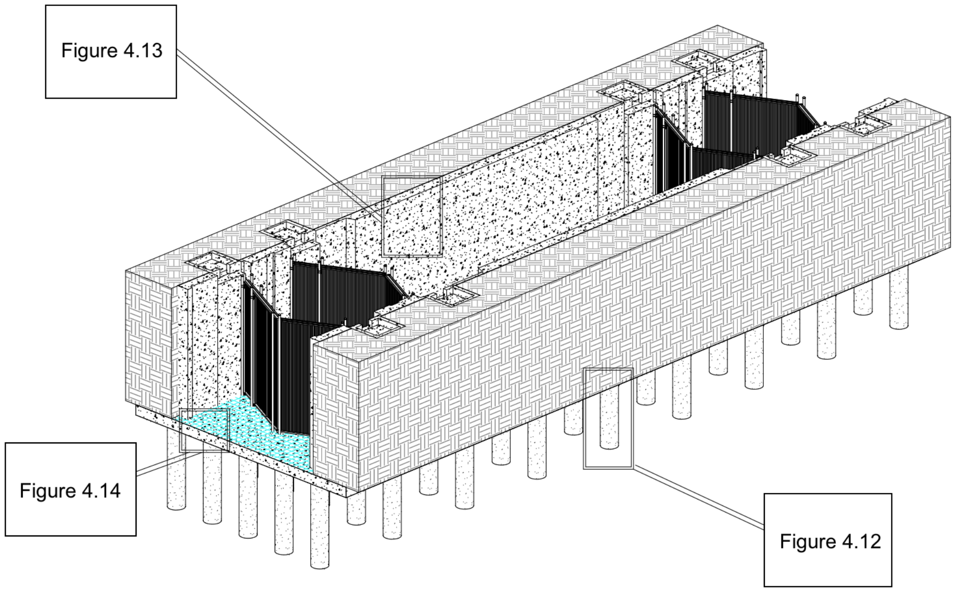



**Figure 4.15:** Overview of the structure and reinforcement detailing

**Columns Reinforcement**
The columns of the structure have been designed and reinforced following the design specifications outlined in EN 1992-1-1:2004/A1:2014 (European Committee for Standardization, 2014). Each column is constructed from C35/45 concrete and is fortified with FeB 500 HKN reinforcement. For a comprehensive understanding of the design details, refer to Table 4.2.

| Property | Detail |
|---|---|
| Material | C35/45 concrete, FeB 500 HKN reinforcement |
| Height | 4.01 meters |
| Diameter | 80 centimeters |
| Load Combinations | COMB1 (447.82 kN), COMB2 (319.91 kN) with Load Factor (g) of 1.00 |
| Safety Factor | Rd/Ed = 24.55 |
| Longitudinal Reinforcement | 10 bars of FeB 500 HKN (14 mm diameter, 3.97 m length) |
| Transverse Reinforcement | 26 stirrups of FeB 500 HKN (8.3 mm diameter, 2.43 m length) |

**Table 4.2:** Reinforcement details for the pile columns.

The cover of the column is measured at 4 cm. The column is classified as a short column to account for its slenderness, and after undergoing ULS/ALS Analysis, it is found to be safe with a safety factor of Rd/Ed = 24.55 > 1.0. Buckling analyses in the Y and Z directions have also been performed with satisfactory outcomes. The reinforcement ratio is measured at 0.31%, which results in an actual area of 15.39 cm$^2$. In total, the column reinforcement requires 1.94 m$^3$ of concrete, 9.68 m$^2$ of formwork, and 74.63 kg of steel with a density of 38.57 kg/m$^3$ and an average steel diameter of 10.4 mm.
Below in Figure 4.16 is an overview shown of the reinforcement detailing.

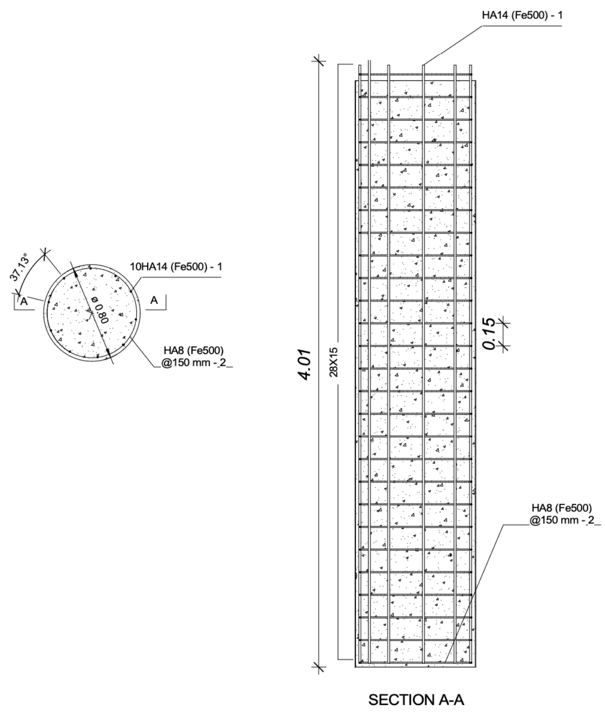



**Figure 4.16:** Overview of pile foundation reinforcement detailing

**Walls Reinforcement**
The walls are constructed as intermediate-level reinforced concrete walls, designed according to the EN 1992-1-1:2004/A1:2014 standard. For a quick reference to the design details, consult Table 4.3.

| Property | Detail |
|---|---|
| Material | C30/37 concrete, FeB 500 HKN reinforcement |
| Dimensions | 36.48 m (L) x 0.6 m (T) x 6.36 m (H) |
| Load Combination | ULS (COMB1) |
| Slenderness Ratio (l) | 31.21 |
| Max Permitted Compressive Force (Nulim) | 309513.83 kN |
| Reinforcement Area (Af) | 3.14 cm$^2$ |
| Main Reinforcement Bars | 8 mm and 12 mm (Vertical and Horizontal) |

**Table 4.3:** Reinforcement details for the walls

The concrete used in the construction has a characteristic cylinder strength (fck) of 30 MPa. The longitudinal and transverse reinforcements, both of type FeB 500 HKN, have a yield strength (fyk) of 500 MPa. The wall is designed under the ultimate limit state (ULS) load combination (COMB1), which accounts for 1.35 times the dead loads and 1.5 times the live loads. The buckling length of the wall (Lo) is 5.41 m. The wall is stable under the prescribed loading conditions, and even without distributed reinforcement, it can withstand the forces applied. However, edge reinforcements and stiffeners against bending are implemented on both sides for added strength. In terms of materials, the wall reinforcement requires 471.66 m$^2$ of formwork, 139.21 m$^3$ of concrete, and 5260.39 kg of steel, with an average diameter of 10.7 mm.
Below in Figure 4.17 is an overview of the wall reinforcement detailing.

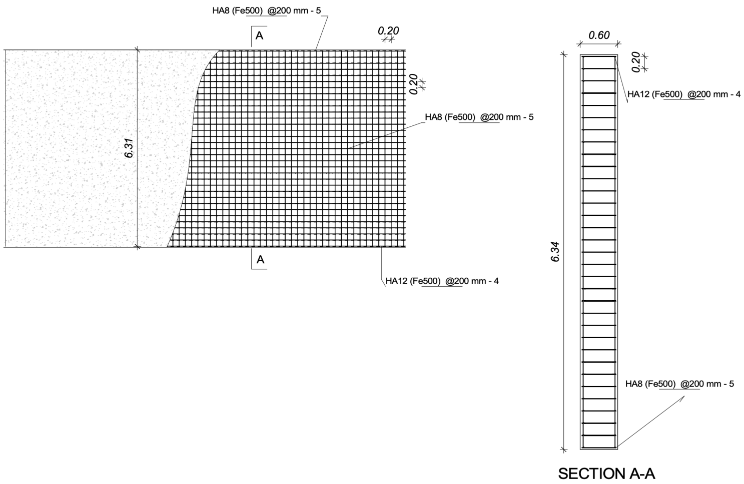



**Figure 4.17:** Overview of the wall reinforcement detailing

**Slab Reinforcement**

The floor slab is a reinforced concrete slab designed in alignment with a 0°primary reinforcement direction. For a rapid reference of the design details, refer to Table 4.4.

| Property | Detail |
|---|---|
| Material | C25/30 concrete, FeB 500 HKN reinforcement |
| Thickness | 0.5 m |
| Reinforcement (design) | 11.31 cm$^2$/m |
| Reinforcement (required) | 24.47 - 40.26 cm$^2$/m |
| Moments (SLS) | 36.11 - 538.45 kNm/m |
| Moments (ULS) | 24.23 - 716.22 kNm/m |
| Max Deflection | 0.4 cm (permissible: 3.0 cm) |
| Load Capacity | Up to -10.00 kN/m$^2$ |

**Table 4.4:** Concise reinforcement details for the slab.

The slab is constructed using C25/30 grade concrete, with a characteristic strength of 25 MPa, and reinforced with FeB 500 HKN steel bars, boasting a yield strength of 500 MPa. Supported at its ends by linear supports, the slab is designed to withstand an assortment of moments and forces, all within acceptable deflection limits. These load considerations factor in both self-weight and uniform loads, and have been examined under both ultimate load and serviceability limit state combinations. With regards to construction materials, the slab necessitated a total of 321.02 m$^2$ of formwork and 160.51 m$^3$ of concrete. The total reinforcement used is approximately 11424.37 kg of FeB 500 HKN steel. Below in Figure 4.18 is a detailed representation of the slab reinforcement.

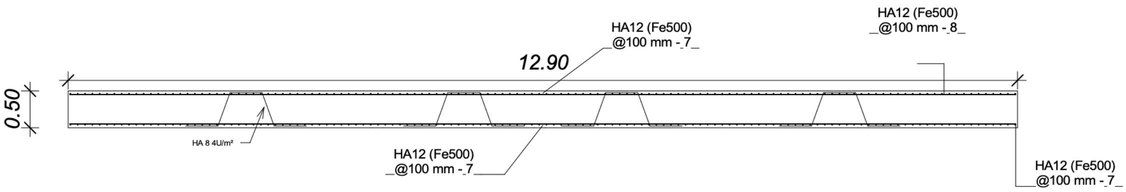



**Figure 4.18:** Overview of the slab reinforcement detailing

# 5

# Reliability Based Design of Gate Height

While Strandeiland is generally elevated above the New Amsterdam Peil (NAP), the design of its guard lock is of paramount importance to counteract potential overtopping risks. This chapter embraces the principles of Reliability-Based Design, an approach that emphasizes both safety and efficiency, especially relevant when designing structures in areas with seemingly lower inherent risks. Through reliability-based design, the gate design of the Strandeiland guard lock specifically targets optimal gate height. This chapter applies the principles of reliability-based design to the gate design of the Strandeiland guard lock, focusing specifically on gate height to mitigate overtopping risks. The chapter starts with a top-down fault tree analysis, detailed in section 5.1, followed by defining the overtopping limit state function, as described in section 5.2. In section 5.3, the limit state function and the allowable probability of failure are used to determine the gate height. Section 5.4 shows us the final gate height design with the obtained results and **??** concludes with an analysis of the loads acting on the gates and walls.

## 5.1. Top Down Fault Tree Analysis

To determine the permissible failure probability for overtopping, a top-down fault tree analysis is conducted. The top event in this analysis is the Flooding of Strandeiland, which is further decomposed into failure events for the Dike section 13-A, Pumping Station, Guard Lock, Aquifer Thermal Energy Storage (ATES) Outlet, and ATES Inlet. The fault tree diagram is depicted in Figure 5.1.
The focal point of this analysis is the guard lock, particularly regarding the overtopping failure mechanism. This aspect is elaborated further, identifying the following potential failure mechanisms: Overtopping, Stability/Strength, Non-Closure, and Failure Connection Structure. These mechanisms are interconnected with an OR gate, signifying that any one of these failures can trigger the guard lock's failure. The percentages provided indicate the distribution of failure probability budget as prescribed in the design guide for hydraulic structures, represented in Table 5.1 (Bree et al., 2018). Following careful examination, the decision has been made to adhere to the failure probability budget as prescribed by the Default Probability of Failure in WBI2017 and OI2014. This decision aligns with the established reliability and safety standards, thus ensuring structural integrity and long-term viability. Adherence to this failure probability budget also aligns the project with accepted best practices and industry norms, effectively minimizing potential risks and complications.

| Type of Structure | Failure Mechanism | Sandy Coast | Other (Dikes) |
|---|---|---|---|
| Dike or Structure | Overflow or Wave Overtopping | 0% | 24% |
| Dike | Uplift and Piping | 0% | 24% |
| | Macro Instability | 0% | 4% |
| | Damage to Cover and Erosion of Dike Body | 0% | 10% |
| Structure | Not Closing | 0%* | 4% |
| | Piping | 0%* | 2% |
| | Structural Failure | 0%* | 2% |
| Dune | Dune Erosion | 70% | 0% |
| Other | | 30% | 30% |
| Total | | 100% | 100% |

**Table 5.1:** Default Probability of Failure from WBI2017 and OI2014. (Bree et al., 2018)

The flooding probability of the island is once every 100 years, corresponding to a failure probability P(f) of approximately 0.01. By leveraging the failure probability budget, the failure probabilities for the specific mechanisms that can lead to the guard lock's failure are ascertained. Consequently, the allowable failure probability for Overtopping is deduced to $P(f) = 0.0024$

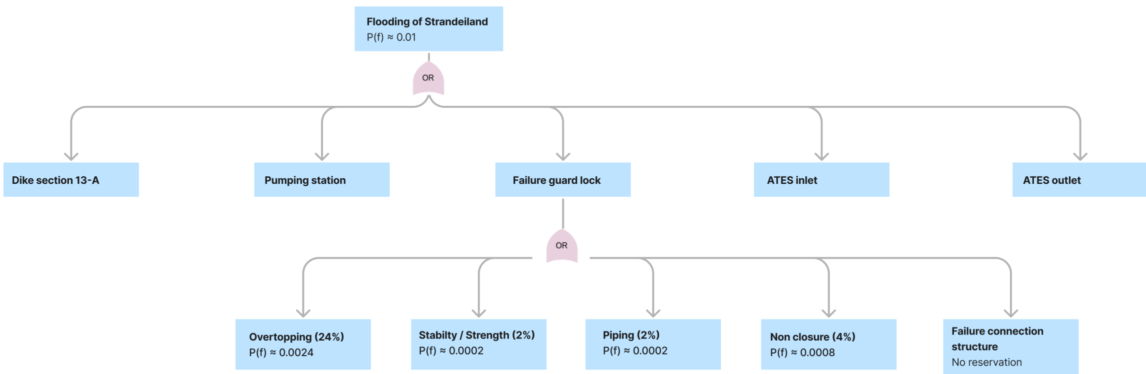



**Figure 5.1:** Fault tree Strandeiland

## 5.2. Overtopping Limit State Function

Calculation of Lock Gate Height The calculation for the height of the lock gate is done based on the article "Wave overtopping at vertical and steep structures" by van der Meer and Bruce (2017). In this article, a distinction is made between waves that break against the vertical structure and waves that do not. This is calculated by the following formula:

$$h_* = 1.3 \frac{d}{H_s} \frac{2\pi d}{g t_p^2}$$

Where:

- $h_*$= "impulsiveness" parameter
- d= depth in front of lock gate (m)
- $H_s$= significant wave height (m)

- g=9.81 $m/s^2$
- $t_p$= peak period (s)

When $h_* > 0.3$, it indicates that the waves do not break against the lock gate. This is applicable to all situations in this study. When this is the case, the following formula can be used to determine the wave overtopping at a certain height of the lock gate:

$$\frac{q}{\sqrt{gH_s^3}} = 0.04\,\text{exp}\left(-2.6\frac{R_c}{H_s}\right) \quad \text{only when } 0.1 < R_c/H_s < 3.5$$

Where:

- $R_c$= Height of lock gate above waterline (m)
- q= Overtopping ($m^3/s/m)$)

The results of applying this formula to the different situations assume a wave overtopping of $10, l/m/s$. The water level calculations took into account an additional allowance for oscillations (0.26 m) and an additional allowance for knowledge uncertainty from Waternet (0.30 m). For the wave height and wave period, a safety factor of 1.1 was used. With an understanding of the gate height calculation and the overtopping phenomenon, the construction of the limit state function is possible. This function is crucial in the context of a reliability-based design approach, as it establishes the boundary between failure and non-failure conditions, thereby allowing the evaluation of the system's performance relative to the acceptable failure probability. The limit state function ($G$) related to overtopping can be defined as follows:

$$G = q_{\text{limit}} - a\sqrt{gH_s^3}\,\text{exp}\left(-\frac{bR_c}{\gamma H_s}\right) \tag{5.1}$$

where:

- $q_{\text{limit}}$ is the maximum allowable overtopping discharge ($m^3/s/m$), which is set to prevent failure. It is the design limit that our overtopping discharge should not exceed.
- $a$, $b$, and $\gamma$ are constants associated with wave overtopping.
- $g$ is the acceleration due to gravity ($m/s^2$).
- $H_s$ is the significant wave height (m).
- $R_c$ is the height of the lock gate above the waterline (m).

The limit state function is based on the crucial requirement that the overtopping discharge should be less than the allowable limit $q_{\text{limit}}$ for a given gate height $R_c$ and wave height $H_s$ to avoid failure. In our case, the system is considered to have failed when $G \leq 0$, that is, when the overtopping discharge calculated from the right-hand side of the equation surpasses the maximum allowable limit $q_{\text{limit}}$. This limit state function lays the foundation for the reliability analysis that will be undertaken in the next section, aimed at ensuring that our design complies with the allowable failure probability for overtopping, as derived from the fault tree analysis.

## 5.3. Height Determination with Allowable Failure Probability

In this study, a Monte Carlo simulation was employed to design a lock gate system that satisfies the required safety standards. This approach accounted for the random nature of wave heights and the risk of overtopping while meeting an allowable failure probability. In this simulation, it was determined from previous calculations and historical data, as shown in section 2.4.4, that the significant wave height $H_s$ follows a normal distribution with a mean of 0.9072 m and a standard deviation of 0.7479 m. For each wave height drawn from this distribution, the corresponding gate height above the waterline ($R_c$) was calculated using an overtopping formula for non-breaking waves. The limit state function for overtopping was defined as $G = q - q_{\text{des}}$, where $q$ is the wave overtopping rate and $q_{\text{des}}$ is the design overtopping discharge, specified as 10 l/s/m and expressed in $m^3/s/m$. The solution of this function provides the gate height above the waterline that results in the design overtopping discharge.

The specific input parameters utilized in this simulation are provided in Table 5.2, the full code is shown in Appendix E.9.

| Parameter | Value | Standard Deviation | Distribution |
|---|---|---|---|
| $g$ (acceleration due to gravity) | 9.81 m/s$^2$ | - | Constant |
| $a$ | 0.192 | - | Constant |
| $b$ | 4.3 | - | Constant |
| $\gamma$ | 1 | - | Constant |
| $q$ (flow rate) | 0.01 m$^3$/s/m | - | Constant |
| $n\_simulations$ | $10^7$ | - | Constant |
| $P_f$ (allowable failure probability) | 0.0024 | - | Constant |
| $Hs\_mean$ | 0.9072 m | 0.7479 m | Normal |

**Table 5.2:** Input parameters used in the simulation.

A Monte Carlo simulation was then conducted with 10 million iterations. For each significant wave height sampled from the assumed distribution, the corresponding gate height above the waterline was computed. Exceedance probabilities were then calculated using these gate heights. These probabilities provide an estimation of the likelihood that the actual gate height will be lower than a given value. The final step in the analysis involved identifying the gate height above the waterline that corresponded to an exceedance probability equal to the allowable failure probability, which was specified as 0.0024. This gate height serves as the design criterion, implying that the lock gate should be built at this height to ensure that the risk of overtopping does not exceed the allowable failure probability. Figure 5.2 depicts a flowchart that outlines the step-by-step procedure employed to estimate the crest level (Rc) using Monte Carlo simulations.

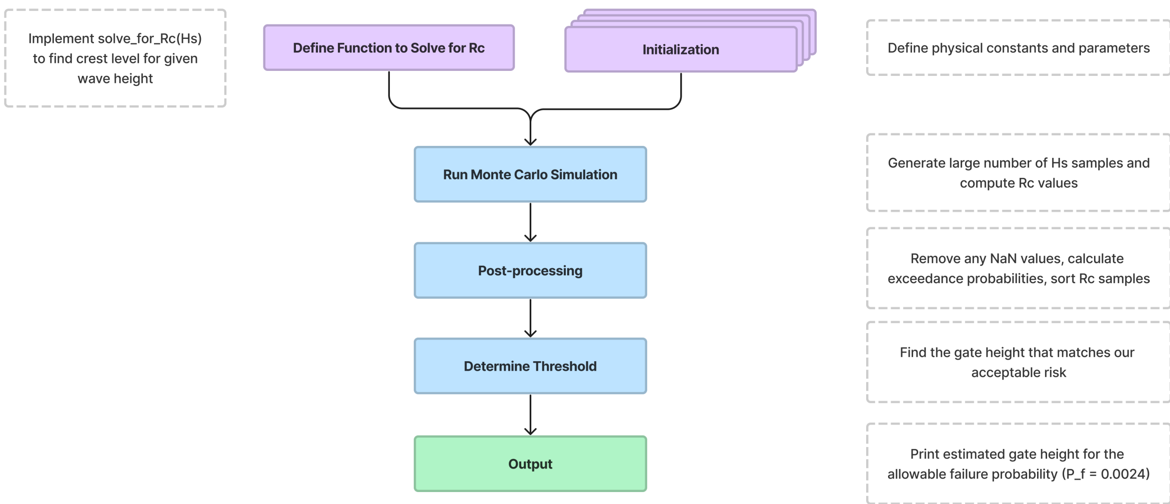



**Figure 5.2:** Flowchart outlining the process of calculating the crest level (Rc) using Monte Carlo simulations for a given allowable failure probability. The process begins with initialization of parameters and includes detailed steps like defining the Rc function, running the simulations, post-processing, determining the threshold, and finally providing the output. (Full code in Appendix E.9)

The gate height above the waterline was determined to be approximately 1.1452 m for the specified allowable failure probability of 0.0024.

## 5.4. Determination of Gate Height

Based on the reliability analysis and the methodical consideration of potential failure mechanisms, the design height of the guard lock's gate is determined. The gate height is primarily based on the design height of the gate above the waterline ($R_c$) obtained from the height determination with allowable failure probability in the section before. In reliability-based design, surcharges and knowledge uncertainties are excluded to avoid over-conservatism. This approach assumes that primary components inherently capture most uncertainties, ensuring that designs are both safe and efficient.

- The maximum water level in the guard lock was 3.6 meters, measured from the bottom, as indicated in previous analyses. A water level increase and a water level surcharge are taken into account of respectively 0.18 and 0.4 meters.
- A water level rise of 0.30 m is expected after 2050 which is taken into account.
- A settlement requirement is left out.
- Surcharges for oscillations and knowledge uncertainty are left out.
- The design height of the gate above the waterline, $R_c$, was estimated as 1.15 m based on the reliability analysis.

Hence, the total gate height, measured from the bottom, is 5.05 meters. This gate height corresponds to a water level of + 1.55 m NAP.

Figure 5.3 provides a schematic overview of these components contributing to the gate height.

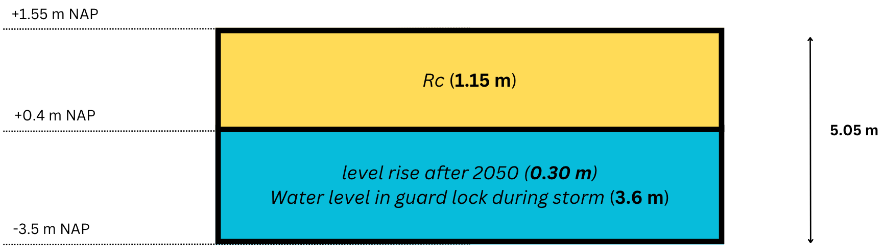



**Figure 5.3:** Gate height specification

Figures 5.4 and 5.5 further illustrate the design of the gate in both opened and closed situations, respectively.

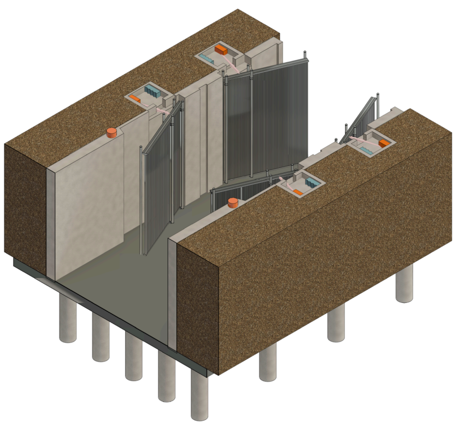


**Figure 5.4:** Gate design in opened situation

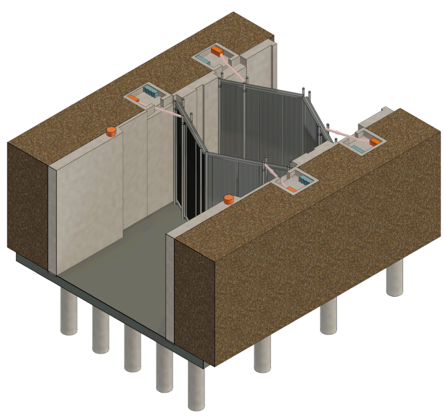


**Figure 5.5:** Gate design in closed situation

In conclusion, the design height of the guard lock's gate has been established through a rigorous reliability-based design process. With a total gate height of 5.05 meters from the bottom, the design adheres to the safety requirements and failure probabilities established in the preliminary stages of the design process. The distribution of this height among the various components further ensures that the risk of overtopping is adequately accounted for and mitigated.This design height not only ensures the functional reliability of the guard lock under normal and extreme conditions but also reflects a balance between safety, cost, and efficiency considerations.

# 6

# Discussion, Conclusions and Recommendations

## 6.1. Discussion

The conceptual design of the Strandeiland guard lock was influenced by significant considerations. Requirement specifications, boundary conditions, and prevailing hydraulic conditions played pivotal roles in shaping the design. Employing the functional design approach was notably effective in ensuring alignment with the specified requirements and in addressing the functional necessities of the guard lock. The adoption of a reliability-based design method further enhanced the design by enabling the determination of the gate height based on an acceptable failure probability.

A critical analysis of the design process reveals that it was primarily anchored in fixed and predefined hydraulic conditions. Such an approach, while systematic, might introduce potential rigidity, thereby potentially constraining the guard lock's adaptability to evolving hydraulic scenarios.

Another observation is the design process's heavy reliance on theoretical modeling and reliability analysis. While such a methodological approach is typical of initial design phases, it underscores the intrinsic challenges of ensuring that theoretical models align seamlessly with real-world conditions.

In conclusion, while the design process has been thorough and the resultant guard lock design is both comprehensive and functional, continuous refinements informed by evolving conditions and data could ensure that the guard lock remains resilient, efficient, and perfectly aligned with its intended purpose.

## 6.2. Conclusions

Reflecting on the primary objective:

> "To develop a conceptual guard lock design for Strandeiland that effectively mitigates extreme water level variations."

This study presents a conceptual guard lock design meticulously tailored to the unique challenges and requirements of Strandeiland, as depicted in Figure 6.1. The conclusions drawn are:

1. **Design Suitability:** The guard lock adeptly addresses diverse boundary conditions, from environmental challenges to regulatory mandates. This approach emerges as a more advantageous solution than adapting the inner quay wall to serve as the primary flood defence.
2. **Flexibility:** The design caters to a range of operational scenarios. Its modular nature implies that components can be individually adapted, offering flexibility for future modifications.
3. **Reliability and Longevity:** The guard lock design showcases resilience and adaptability, suggesting a potential for sustained durability. Leveraging reliability-based design, particularly in determining gate height, bolsters its safety and efficiency.

4. **Financial Perspective:** Although the guard lock design is intricate and upholds high standards, it's notably more cost-efficient than retrofitting the inner quay wall. The latter option would usher in substantial financial, spatial, ecological, and aesthetic challenges, thereby rendering the guard lock a judicious investment.

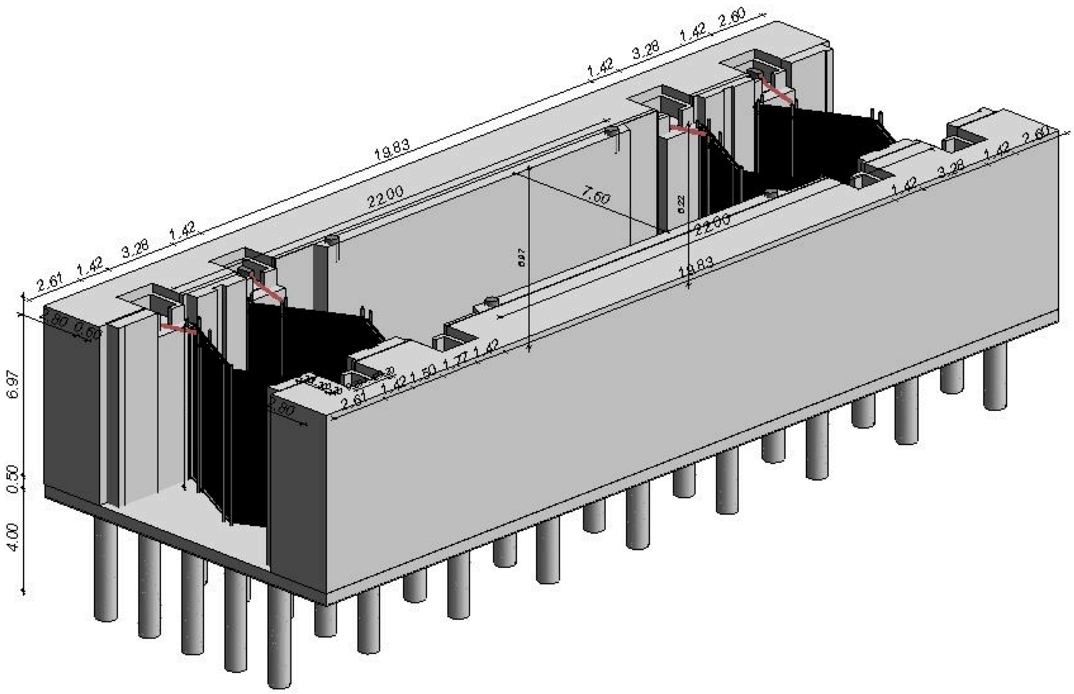



**Figure 6.1:** 3D Conceptual Model of the final design of the Guard Lock of Strandeiland

**Final Design Features:**

- The guard lock efficiently manages extreme water level fluctuations between NAP +0.1 m and NAP -0.6 meters, safeguarding Strandeiland's water infrastructure and curtailing the length of the primary flood defence.
- The design circumvents the challenges tied to adapting the inner quay wall.
- It incorporates two double sets of mitre gates, operated by electromechanical driving mechanisms.
- The foundational aspect of the design encompasses a pile foundation with 45 piles, each 0.8 m in diameter, extending into the first layer to counteract non-uniform settlement and obviate deformations in the floor slab.
- A seepage screen is integrated to augment the seepage length and preclude piping.
- The Guard Lock's dimensions, predicated on the standard vessel, result in lock chambers of 22 meters in length and 7.6 meters in width, culminating in an aggregate length of 36.46 meters.
- The design is future-proof, accommodating the prospective inclusion of a filling and emptying system.
- The definitive gate height, ascertained through reliability-based design principles, is established at 5.05 meters or NAP + 1.55 m.

## 6.3. Recommendations

Based on the initial conceptual design for the guard lock at Strandeiland, there are several recommendations and considerations that can be proposed for further investigation:

- **Vessel-Generated Waves:** In the analysis, only wind-generated waves were considered. The potential impact of vessel-generated waves, which could be governing for the design, should also be explored in subsequent design phases.

- **Design of Surrounding Structures:** In addition to the guard lock itself, other structures in the surrounding area should also be designed meticulously. These may include berthing facilities, collision protection, and scour or bottom protection of the structure. Other elements, such as cable management and dry crossing, could also be further elaborated on in future stages of the project.

- **Scour as a Failure Mechanism:** While the design addresses several potential failure mechanisms, scour could also be a significant concern. Future design iterations should take into account the potential for scour and its implications for the guard lock's stability.

- **Review the Gate Design:** A comprehensive cost analysis could be beneficial in determining the optimal gate design. By taking into account both the initial construction costs and the long-term maintenance costs, a different type of gate may be identified as more suitable over the course of the structure's lifetime.

- **Inclusion of a Filling and Emptying (F/E) System:** Although the initial design considerations saw the F/E system as potentially unnecessary due to the infrequency of guard lock closures, the design should still allow for its potential future inclusion. This aspect of the design should be reevaluated periodically to account for changes in environmental and operational circumstances.

- **Analysis of Environmental Impact:** Continual review and assessment of the project's impacts on the surrounding environment are recommended. This analysis should encompass local flora and fauna, water quality, and sediment transport.

- **Community Engagement:** Open and ongoing dialogue with the local community and stakeholders should be maintained throughout the project. Regular updates on the project's progress, addressing queries and concerns, and ensuring that the development of the lock is aligned with the community's needs and values are all important aspects of this engagement.

- **Risk Management Plan:** Developing and updating a risk management plan throughout the design, construction, and operational phases of the guard lock is essential to ensure the effective handling of potential issues.

- **Cost Analysis:** A detailed cost analysis of the project should be conducted, which includes initial construction costs, long-term maintenance costs, potential environmental mitigation strategy costs, and potential savings from reduced flood damage and increased safety. Understanding the project's economic feasibility is crucial to determining its viability and securing the necessary funding.

- **CO2 Emission Reduction:** Conducting an analysis of how the project can minimize CO2 emissions, or even achieve CO2-neutral construction, could be extremely beneficial. The construction and operation of the guard lock should adhere to environmental best practices to minimize the carbon footprint. Strategies to achieve this might include the use of energy-efficient materials and processes, the utilization of renewable energy sources, and the integration of carbon offsetting initiatives.

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# A

# Water levels at IJburg

In this Appendix an extensive analysis of water levels at the IJburg region, specifically the Strandeiland, is presented. The sections delve into critical aspects such as:

1. The **Peilbesluit 2018** ordinance, which outlines the new water level regulations adopted for the IJsselmeer region by the Ministry in June 2018. This ordinance emphasizes a shift from a fixed target level to a bandwidth wherein the water level may fluctuate, to adapt better to meteorological conditions and freshwater needs in the Netherlands. A graphical representation of this bandwidth can be viewed in Figure A.1.
2. A **proposal on water levels** from the Ministry of Infrastructure and Water Management, suggesting changes to the future lake levels of the IJsselmeer region. The reasons for not taking this policy change into account are explained in detail.
3. An **overview** of margins and surcharges in the design assumptions from different sources is shown in Figure A.2.
4. The **Peilbesluit IJsselmeer area** section gives a deeper understanding of how water levels in the Markermeer are regulated. A graphical representation of the proposed and current water levels in Markermeer can be found in Figure A.4.
5. The section titled **Expected level at IJburg** outlines the potential water level variations at the IJburg region, discussing the implications of various meteorological and infrastructural factors.
6. Lastly, the **Average closure frequency** section, referenced in A.1, provides a table (Table A.1) that details the closure frequency per year and boating season at different water levels.

This appendix serves as a comprehensive source of information on the water levels, policies, and expectations at IJburg. Researchers, policymakers, and stakeholders can refer to it for a deeper understanding of the region's water level management.

## A.1. Water levels at Strandeiland and Peilbesluit 2018

**Peilbesluit 2018**

A new water level ordinance for the IJsselmeer region was adopted by the Ministry in June 2018 to maintain the freshwater supply in the lake area. The fixed target level of the IJsselmeer and the Markermeer-IJmeer and the Zuidelijke Randmeren will be replaced by a bandwidth within which the water level may fluctuate (Ministerie infrastructuur en Waterstaat, 2018). This *Peilbesluit* must respond better to meteorological conditions and the need for freshwater in the Netherlands.

Operational management in the winter situation will be maintained in all lakes, i.e. management at the lower end of the bandwidth. In recent decades, this has resulted in a long-term average lake level of -0.33 m NAP in the Markermeer. More detailed information about the water level ordinance can be found in Appendix A. (Ministerie infrastructuur en Waterstaat, 2018)

| Bandbreedte van de meerpeilen per periode (m NAP) | | | | |
|---|---|---|---|---|
| Meren | nov-feb | mrt | apr-sept | okt |
| IJsselmeer | -0,40 tot -0,05 | -0,40 tot -0,10 | -0,30 tot -0,10 | -0,40 tot -0,10 |
| Markermeer | -0,40 tot -0,20 | -0,40 tot -0,10 | -0,30 tot -0,10 | -0,40 tot -0,10 |
| Veluwerandmeren | -0,30 tot -0,10 | -0,30 tot -0,05 | -0,10 tot -0,05 | -0,30 tot -0,05 |

**Figure A.1:** Bandwidth of water levels per period (m NAP) (Ministerie infrastructuur en Waterstaat, 2018)

**Proposal water levels**

The Ministry of Infrastructure and Water Management has stated that a policy proposal has been made which includes a number of changes regarding the future lake levels of the IJsselmeer region (Rijkswaterstaat, 2019). This proposes a winter level rise of a maximum of 30 cm for the Markermeer (maximum rise possible after 2050). This policy change will probably be adopted in the National Water Programme in late 2021 or early 2022. This future policy change has not been taken into account. The reasons for this are mentioned below.

- The development of the Strandiland and, in particular, the primary flood defence is in the early stages of preparation for the implementation phase.
- A relatively large margin for future water level increases in the Markermeer has already been included in the design of the flood defence.

This extra margin is summarised in the following table where the design assumptions from the National Water Plan, the requirement of the water board and the design assumptions of the present document are compared to the design assumptions of the OI2014v4.(Rijkswaterstaat, 2017)

| Toeslagen (*) | OI2014v4 | Waternet/AGV | Nationale Waterplan/ Deltaplannen | Programma van Eisen | Extra marge tov OI2014v4 | |
|---|---|---|---|---|---|---|
| Waterstand | 0,4 m | - | - | 0,4 m | - | technische onzekerheden |
| Golfhoogte | 10% | - | - | 10% | - | |
| Golfperiode | 10% | - | - | 10% | - | |
| Kennisonzekerheid | - | 0,3 m | - | 0,3 m | +0,30 m | beleid onzekerheden |
| Peilverhoging Markermeer na 2050 | 0,1 m | - | 0 m | 0,18 m | +0,08 m | |
| Slingeringen | 0,1 m | 0,26 m | - | 0,26 m | +0,16 m | |

**Figure A.2:** Overview of margins and surcharges in the OI2014v4 and the PvE Strandeiland (Jansen, 2021)

The surcharges in this specification of requirements are in total an extra margin of 0.30 + 0.08 + 0.16 = 0.54 m. Compared to OI2014v4 this is 0.54 m extra. Above the above-mentioned margins and surcharges of the flood defence, an extra height of 0.30 m is applied during construction in order to meet the residual settlement requirement. Finally, the surface height above the flood defence is 0.45 m higher if a bicycle path or road is present.

The Table C.5, shows the build-up of the crest height of the water defence profile on the west side of the island in two cases:

1. applying the margins and surcharges according to OI2014v4 and
2. the design of the Strandeiland flood defence.

Figure shows in gragic way the same construction of the crest height as in table 4.

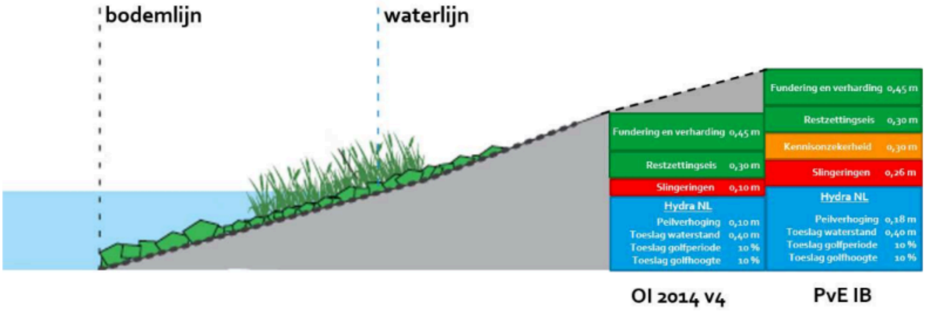


**Figure A.3:** Structure of the crest height in the PvE IB compared to OI2014 v4 in graphical form (Jansen, 2021)

**Peilbesluit IJsselmeer area**

The water level of the Markermeer is regulated to the 'Peilbesluit IJsselmeer'. Het Peilbesluit IJsselmeer was extensively discussed in previous years. Extensive research was carried out into the consequences of increasing and decreasing the current water levels. It was decided to increase the discharge capacity of the Afsluitdijk and if the sea level (expected from 2050) is so high due to climate change that insufficient IJsselmeer water can be discharged at low tide, pumps will be installed on the Afsluitdijk. The pumps will also enable the target levels newly set in 2018 to be better managed in the event of calamities. In the event of a major influx of water via the IJssel, the sluices in the IJsselmeer region will be used to anticipate and in future will be able to discharge the excess water with extra pumps.

New water levels have also been set for the Markermeer and the Randmeren:

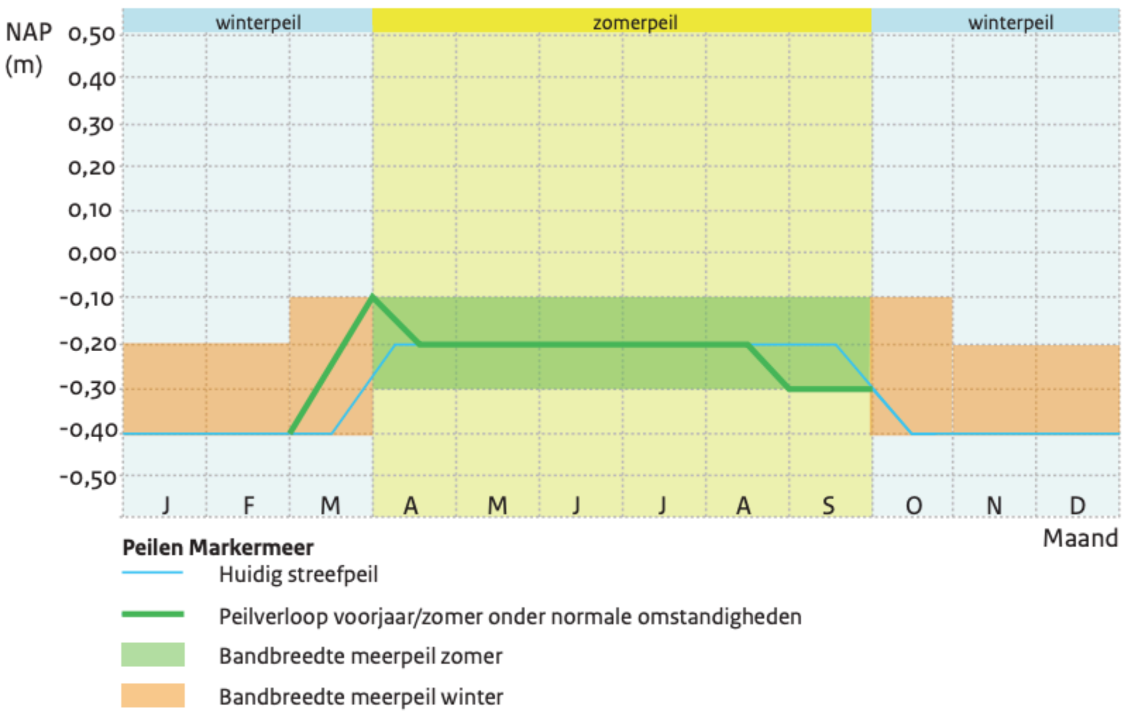


**Figure A.4:** Proposed and current summer and winter water levels in Markermeer (Rijkswaterstaat, 2018)

A change from the situation before 2018 with a summer level (target level) of NAP - 0,20 m and a winter level of NAP - 0,40 m is that a peak is created in spring and slowly worked towards the winter level from mid-August. This is called "natural level management" and is intended, among other things, to promote the development of reeds in the IJsselmeer region.

Water level fluctuations in the IJsselmeer region relative to the newly established target levels, which are easy to maintain in practice, are caused by wind set-up and set-down and the "slope" of the water in the IJsselmeer and Markermeer (incl. Gouwzee, IJmeer and Randmeren up to Nijkerk).
In the Markermeer, the water difference due to this skew is about 80 cm. During spring and autumn storms, the wind almost always comes from a south, south-west or northern direction. This means that IJburg often has to deal with set-down or limited set-up. A severe storm from the northeast with a wind blowing from the Markermeer towards Amsterdam does not (so far) occur or rarely. Rijkswaterstaat's measurements during extreme storms confirm this statistics.This means that ports in the south-western part of Markermeer (including IJburg and Muiden) may have to cope with sharp drops in water level and limited increases in water level.

**Expected level at IJburg**
It is assumed that the water level in Het Oog van het Strandeiland is the same as the target level in the Markermeer. The guard lock can be left open so that the exchange of inland water between Het Oog and IJmeer can take place and ships do not need to be locked in. During strong winds the Markermeer water surface may be skewed relative to the target level. As in the IJsselmeer region, the target level in the Markermeer can be maintained well by Rijkswaterstaat.

After the construction of the Afsluitdijk and later the Houtribdijk between Lelystad and Enkhuizen, the IJsselmeer region was compartmentalised. This meant that the ebb and flow of the tide ceased the Zuiderzee was tamed. Excess (fresh) water from the IJssel, among others, was drained off to the Wadden Sea and, if necessary, to the North Sea via the Markermeer and the North Sea Canal. The Ipenslotersluis in the Diemerzeedijk, which was restored in 2017, functions as a sluice in emergencies to drain off excess water from the hinterland into the Markermeer. An orderly situation has been created at the location in question.

Lock 125 has the function to protect the inner basin against extreme circumstances. The lock is part of the surrounding flood defence that protects the island from extreme circumstances in the Markermeer, the failure rate requirement of the flood defence is 1/830 years (Ministerie van Infrastructuur en Milieu, 2022). The lock will be exposed to waves and floods generated in the Markermeer.

**Average closure frequency**

**Table A.1:** Average closure frequency per year and per boating season for different water levels

| | 0 cm water level rise | | | 30 cm winter water level rise | | |
|---|---|---|---|---|---|---|
| Water level | per year | | | per year | | |
| m NAP | Total | nov-march | boating season | Total | nov-march | boating season |
| 0,5 | 0 | 0 | 0 | 0,3 | 0,3 | 0 |
| 0,4 | 0 | 0 | 0 | 0,7 | 0,7 | 0 |
| 0,3 | 0,1 | 0,1 | 0 | 1,9 | 1,8 | 0,1 |
| 0,2 | 0,3 | 0,3 | 0,1 | 4,6 | 4,2 | 0,4 |
| 0,1 | 1,3 | 0,7 | 0,6 | 10,5 | 8,5 | 2 |
| 0 | 6,3 | 1,8 | 4,4 | 22 | 13,8 | 8,2 |
| -0,1 | 22,7 | 4,2 | 18,5 | 36,8 | 14,7 | 22,1 |
| | | | | | | |
| -0,4 | 25,6 | 14,1 | 11,5 | 15,2 | 6,2 | 9 |
| -0,45 | 26,7 | 17,9 | 8,8 | 9,6 | 4,3 | 5,3 |
| -0,5 | 23 | 16,6 | 6,4 | 6,1 | 3 | 3,1 |
| -0,6 | 12,2 | 9,8 | 2,5 | 2,9 | 1,8 | 1,1 |
| -0,7 | 7,2 | 6,2 | 1,1 | 1,5 | 1,1 | 0,4 |
| -0,8 | 3,6 | 3 | 0,6 | 1 | 0,8 | 0,2 |
| -0,9 | 2,1 | 1,8 | 0,3 | 0,6 | 0,5 | 0,1 |
| -1 | 1,3 | 1,1 | 0,2 | 0,4 | 0,2 | 0,1 |

# A.2. Ground water level research Fugro

Below the research outcome performed by Fugro is presented in the following Figures. Figure A.5 shows the results of the calculation for the design ground level. Figure A.6 shows the groundwater fluctuation from Strandeiland, this measurement was performed at the location of the Pampusbuurt.

STRANDEILAND AMSTERDAM
RESULTATEN GEOHYDROLOGISCH GRONDWATERMODEL

E. RESULTATEN BEREKENING ONTWERPMAAIVELDNIVEAU

E.1 OPBOLLING - SCENARIO 1

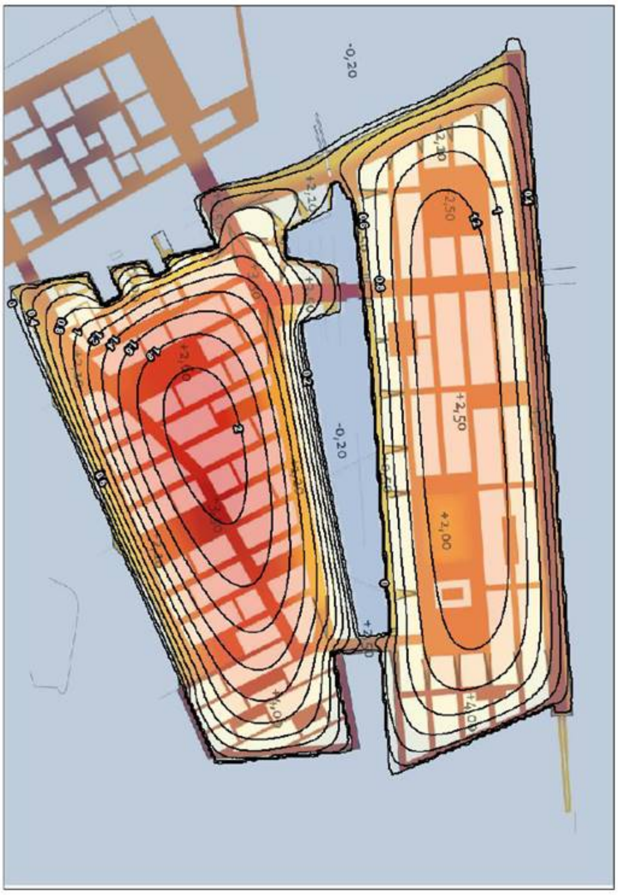


**Figure A.5:** Results calculation design ground level

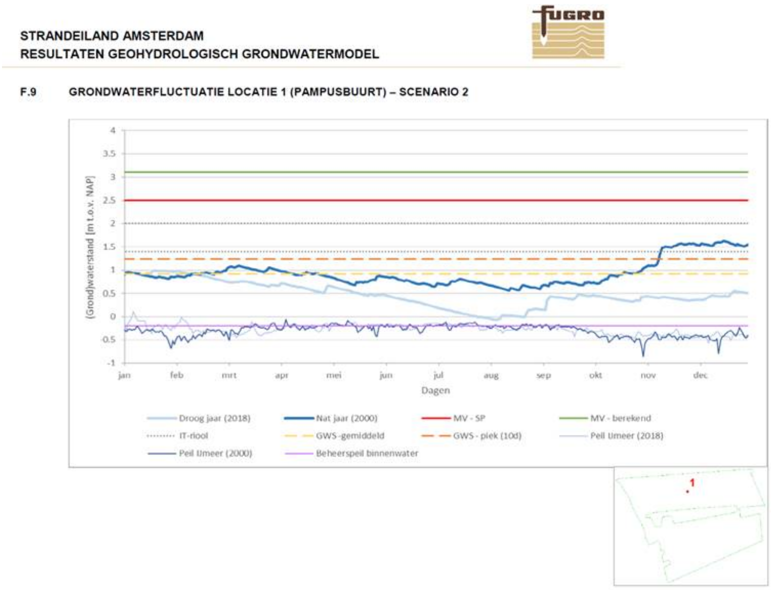


**Figure A.6:** Groundwater fluctuation measured in the pampusbuurt

## A.3. Geotechnical Investigation and Testing by Fugro

The Geotechnical Investigation and Testing appendix provides a comprehensive overview of the soil conditions at the location of the guard lock on Strandeiland. The results and documents presented in this appendix are based on the investigations carried out by Fugro, a leading geotechnical engineering company. This appendix is an essential resource for understanding the design and construction of the guard lock, as it provides crucial information on the soil properties, groundwater conditions, and other factors that can impact the stability and performance of the structure. This appendix is primarily utilized during the stability checks and foundation design.

The location of where the Cone Penetration Test (CPT) was performed is indicated in Figure A.7.

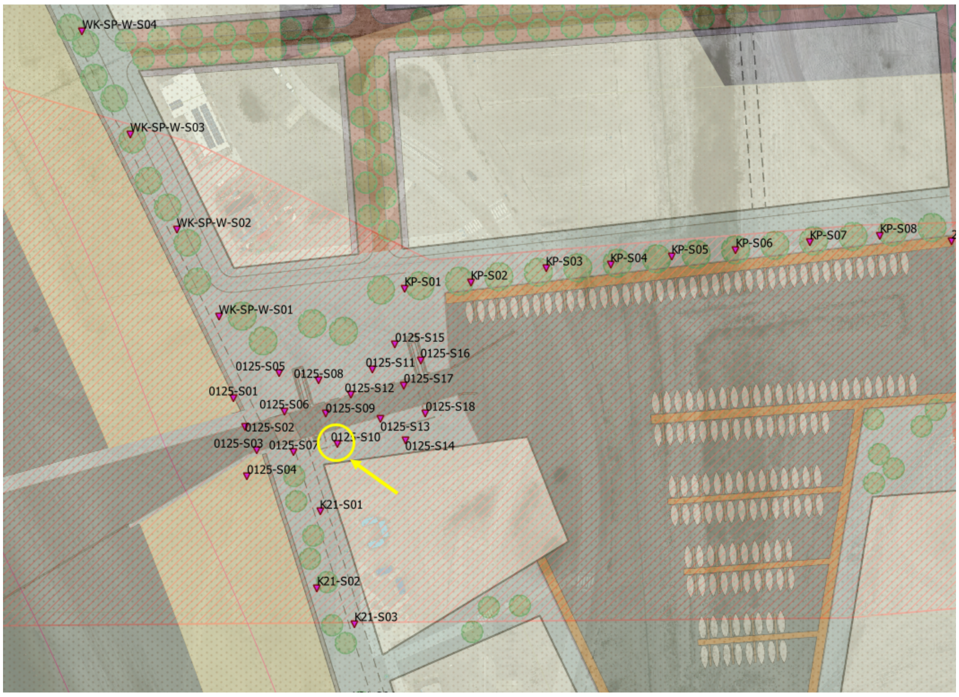


**Figure A.7:** Location in GIS where the CPT was performed

Figure A.8 below provides an overview of the results obtained from the Cone Penetration Testing (CPT) performed by Fugro. The figure displays the composition of the subsoil below the structure, including the predominant presence of sand, clay, and peat. Additionally, this figure gives us important parameters needed to calculate settlements under the structure.

**Bodemopbouw + Parameters (voor zettingen)**

| | | | |
|---|---|---|---|
| Initiele drooglegging | NAP | 3,9 | m |
| m.v. | NAP | 3,7 | m |
| GWS | NAP | -0,2 | m |

| primair | secundair |
|---|---|
| 50% | 50% |

maatgevende doorsnede: S 10

| | | | | | | | | Koppejan | | | | Isotachen | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| nr.: | van / tot [m NAP] / [m NAP] | dikte [m] | omschrijving | | g [kN/m3] | gsat [kN/m3] | cv [m2/s] | Cp [-] | Cp' [-] | Cs [-] | Cs' [-] | a [-] | b [-] | c [-] | ΔPg [kN/m2] |
| 1 | 3,70 / -5,70 | 9,40 | ZAND | Ophooglaag | 18,0 | 20,0 | 0,0E+00 | 1.000,0 | 500,0 | 2.000,0 | 1.000,0 | 1,0*10-6 | 0,0020 | 0,0001 | 0,0 |
| 2 | -5,70 / -10,00 | 4,30 | KLEI | Slap | 13,0 | 13,0 | 1,0E-08 | 36,0 | 8,0 | 182,0 | 43,0 | 0,0310 | 0,1390 | 0,0099 | 10,0 |
| 3 | -10,00 / -12,00 | 2,00 | VEEN | Hollandveen | 11,0 | 11,0 | 1,0E-07 | 31,0 | 8,0 | 119,0 | 66,0 | 0,0360 | 0,1390 | 0,0064 | 10,0 |
| 4 | -12,00 / -14,50 | 2,50 | KLEI | Hydrobiaklei | 15,0 | 15,0 | 1,0E-07 | 71,0 | 15,0 | 385,0 | 105,0 | 0,0160 | 0,0740 | 0,0040 | 10,0 |
| 5 | -14,50 / -19,00 | 4,50 | KLEI | sterkzandig | 18,0 | 18,0 | 1,0E-03 | 750,0 | 400,0 | 1.500,0 | 1.000,0 | 1,0*10-6 | 0,0020 | 0,0001 | 20,0 |
| 6 | -19,00 / -24,00 | 5,00 | KLEI | Eenklei | 17,0 | 17,0 | 1,0E-05 | 76,0 | 26,0 | 526,0 | 238,0 | 0,0150 | 0,0430 | 0,0034 | 20,0 |
| 7 | -24,00 / 0,00 | -24,00 | ZAND | 3e zandlaag | 19,0 | 21,0 | 0,0E+00 | 1.000,0 | 500,0 | 2.000,0 | 1.000,0 | 1,0*10-6 | 0,0020 | 0,0001 | 20,0 |
| 8 | 0,00 / 0,00 | 0,00 | ___ | ___ | 0,0 | 0,0 | 0,0E+00 | 0,0 | 0,0 | 0,0 | 0,0 | | | | 0,0 |
| 9 | 0,00 / 0,00 | 0,00 | ___ | ___ | 0,0 | 0,0 | 0,0E+00 | 0,0 | 0,0 | 0,0 | 0,0 | | | | 0,0 |
| 10 | 0,00 / 0,00 | 0,00 | ___ | ___ | 0,0 | 0,0 | 0,0E+00 | 0,0 | 0,0 | 0,0 | 0,0 | | | | 0,0 |

**Figure A.8:** Overview of CPT part 1

These next two Figures A.9 & A.10 show the automatically generated data from the CPT.

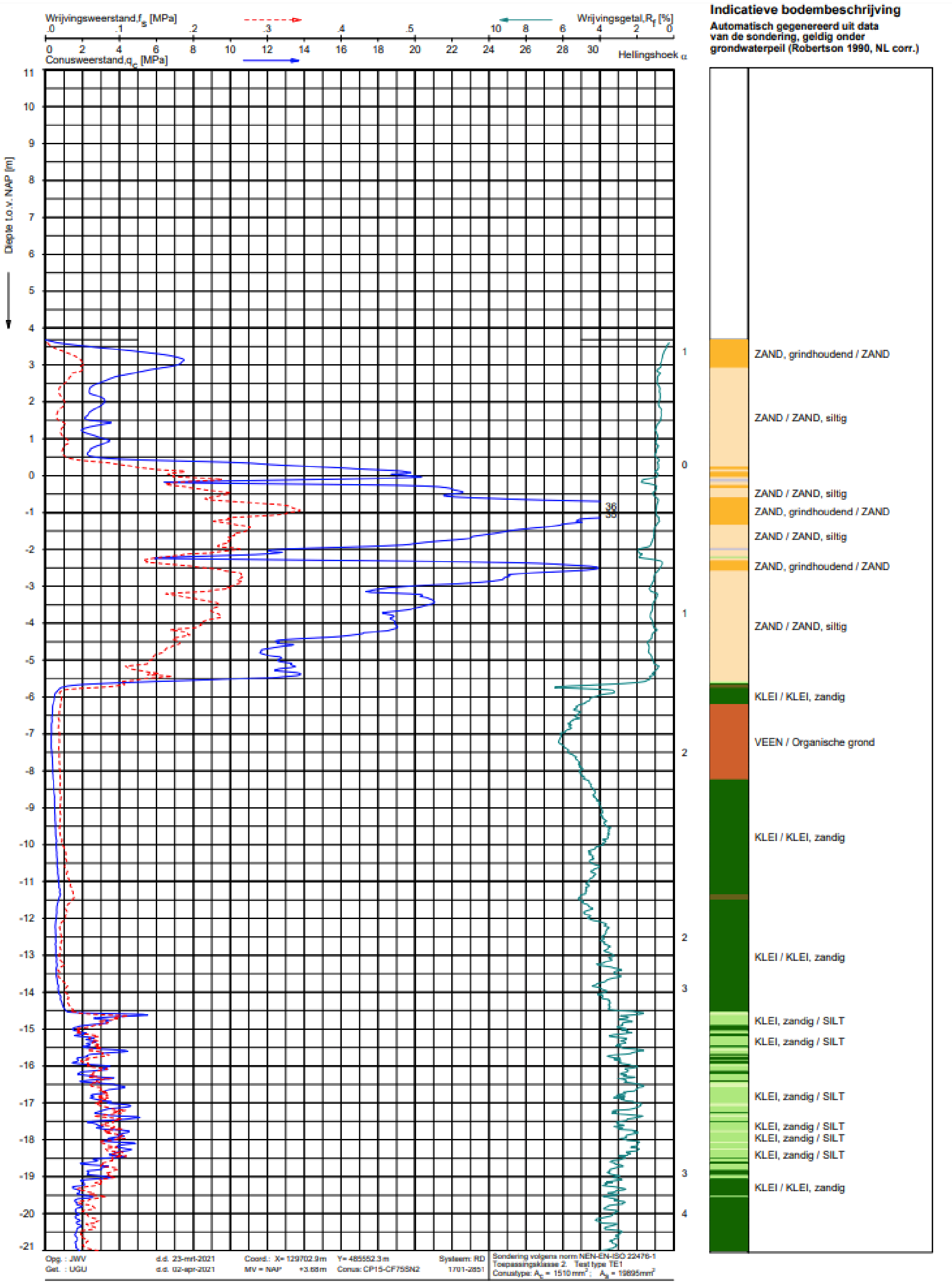


**Figure A.9:** Overview of CPT part 1

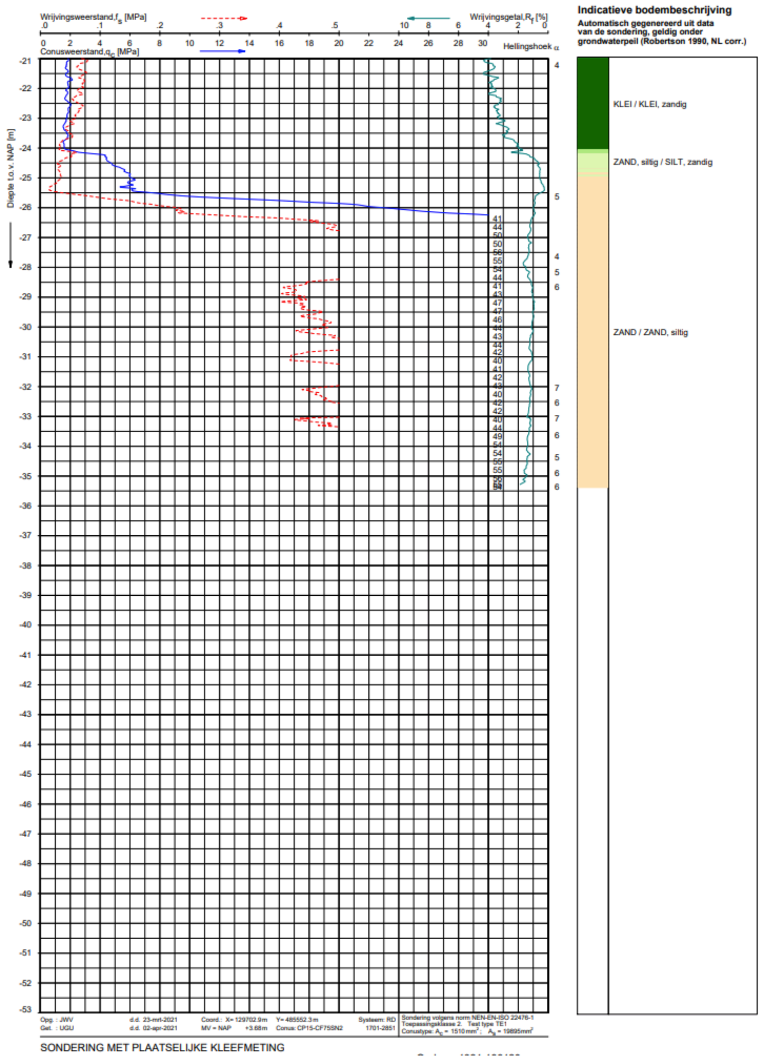


**Figure A.10:** Overview of CPT part 2

# B

# Stakeholders

In this appendix, the stakeholders are mapped out to identify and understand people with influence on the project. All stakeholders have different reasons to participate or be involved in the development of the project.

**Ingenieurs Bureau Amsterdam**
The Engineering Office is the municipality's partner in the field of procurement, technology and project realisation in civil engineering.

**Waternet**
Waternet only does the flood defence, so they want the flood defence to be closed as much as possible. The safer the water retaining structure, the better for Waternet. Waternet wants a water retaining structure with the smallest area to manage as primary flood defence. They are also responsible for the maintenance of the structure.

**Stadsdeel Oost Amsterdam**
Stadsdeel Oost regulates boat traffic in front of the lock and the harbour behind it. They don't do the maintenance directly, but they do do all kinds of things behind and around it. What the lock does affects what they have to build. For instance, the more traffic in the lock, the more mooring places. Beach island itself is managed by them and it therefore has a bearing on the lock.

**Rijkswaterstaat**
Rijkswaterstaat is responsible for the water quality outside Strandeiland. When the lock is closed, it seals off the outside waters. The water quality is then not under the management of Rijkswaterstaat. So when the sluice opens again, poor quality water could end up in outside waters. This is why Rijkswaterstaat has an interest in ensuring that the lock is open as much as possible to allow the water to interact.

# C
# HydraNL Output

The hydraulic load calculations are crucial in understanding the design and sustainability of flood defence structures, ensuring they can withstand various water-related forces and pressures. The following appendix encapsulates the output generated by HydraNL, a trusted computational tool utilized for these purposes. It provides a comprehensive insight into the various stages and components of the hydraulic load assessment for the profile of Strandeiland.

```
Hydra-NL          Versienummer: 2.8.2                 mei 2021              Berekeningsresultaten
Naam gebruiker                                          = burkha002
Gebruikersmodus                                         = Test
Datum berekening                                        = 11-07-2022 11:35:29

Invoerdatabase                                          = C:\ProgramData\App-V\CE76301F-AAA3-408D-80AF-C559407F70F8\2B55C78B-7F24-4445-
Locatie                                                 = MM_3_hy15
  X-coördinaat                                          =  128899 (m)
  Y-coördinaat                                          =  486337 (m)

De golfparameters uit de database zijn in de berekening gebruikt.

Profiel                                                 = C:\ProgramData\App-V\CE76301F-AAA3-408D-80AF-C559407F70F8\2B55C78B-7F24-4445-
  Aanwezige kruinhoogte dijk                            =          2.50 (m+NAP)
  Uitwendige dijknormaal                                =        270.00 (°N)

    Dijkprofielcoördinaten   Taludruwheids-
     Afstand     Hoogte      factor
       (m)       (m+NAP)       (-)
        0.00      -3.00       0.55
       22.00       2.50

Berekeningstype                                         = Hydraulisch belastingniveau
Faalmechanisme                                          = Golfoverslag en overloop
  Kritiek overslagdebiet                                =         10.00 (l/s/m)
  De golfoverslag is berekend met versie '19.1.1.8037' van de 'Wave overtopping at dikes'-module

Bestand met transf. van potentiële naar open-water-wind = C:\ProgramData\App-V\CE76301F-AAA3-408D-80AF-C559407F70F8\2B55C78B-7F24-4445-
Er is gerekend zonder extra steunpunten
Laagste piekwaarde meerpeiltrapezia                     =         -0.40 (m+NAP)
Hoogste piekwaarde meerpeiltrapezia                     =          1.80 (m+NAP)
Stapgrootte piekwaarde meerpeiltrapezia                 =          0.05 (m)
Meerpeiltrapezia worden niet afgetopt

Stijging meerpeil t.g.v. klimaatverandering             =          0.18 (m)

Discretisatiestap meerpeiltrapezia                      =         12.00 (uur)

Verhoging van de waterstand (b.v. als beheerruimte)     =          0.56 (m)

Bovengrens windsnelheid                                 =         50.00 (m/s)

De waterstanden en (golf)belastingen zijn voor het meerpeil gerepareerd.

Berekening met onzekerheid in de waterstand, golfhoogte én golfperioden.
De parameterwaarden van de modelonzekerheid zijn uit de database afkomstig.
  Verwachtingswaarde onzekerheid waterstand             =          0.00 (m)
  Standaarddeviatie onzekerheid waterstand              =          0.25 (m)
  Aantal gebruikte waarden onzekerheid waterstand       =             7
  Verwachtingswaarde voor onzekerheid golfhoogte        =          0.95 (-)
  Standaarddeviatie voor onzekerheid golfhoogte         =          0.11 (-)
  Aantal gebruikte waarden onzekerheid golfhoogte       =             5
  Verwachtingswaarde onzekerheid spectrale golfperiode  =          1.04 (-)
  Standaarddeviatie onzekerheid spectrale golfperiode   =          0.08 (-)
  Verwachtingswaarde voor onzekerheid piekperiode       =          0.96 (-)
  Standaarddeviatie voor onzekerheid piekperiode        =          0.05 (-)
  Aantal gebruikte waarden onzekerheden golfperioden    =             5
  Correlatiecoëfficiënt modelonz. golfhoogte en periode =          0.00 (-)

  Onzekerheid waterstand en/of golven in meerpeiltrapezia:
   percentage voor afhankelijke waterstandsblokken      =        100.00 (%)
   percentage voor onafhankelijke waterstandsblokken    =          0.00 (%)

1 gegevensblok
Som van de basisduren voor alle gegevensblokken         =        180.00 (dagen)
Totaal aantal trapezia                                  =             3

Gegevensblok 1
  Aantal keer dit gegevensblok                          =             3
  Bestand met overschrijdingskansen meerpeil            = C:\ProgramData\App-V\CE76301F-AAA3-408D-80AF-C559407F70F8\2B55C78B-7F24-4445-
  Bestand met overschrijdingskansen windsnelheid        = C:\ProgramData\App-V\CE76301F-AAA3-408D-80AF-C559407F70F8\2B55C78B-7F24-4445-
  Bestand met momentane kansen van de windrichting      = C:\ProgramData\App-V\CE76301F-AAA3-408D-80AF-C559407F70F8\2B55C78B-7F24-4445-
  Bestand met kansen op de stormduren                   = C:\ProgramData\App-V\CE76301F-AAA3-408D-80AF-C559407F70F8\2B55C78B-7F24-4445-

  Parametrische weergave van (geknikte) meerpeiltrapezia
  Tabel met topduren van de meerpeiltrapezia            = C:\ProgramData\App-V\CE76301F-AAA3-408D-80AF-C559407F70F8\2B55C78B-7F24-4445-

  Basisduur trapezium                                   =         60.00 (dagen)
  Blokduur wind                                         =         12.00 (uur)
  Insnoeringsfactor hoogte meerpeiltrapezia             =        100.00 (%)
  Insnoeringsfactor horizontale breedte meerpeiltrapez. =        100.00 (%)


Berekeningsresultaten
    Frequentie:  Hydraulisch belastingniveau:
     1/    833           1.760  (m+NAP)                  Illustratiepunten   Uitsplitsingen

  Terugkeertijd   Hydraulisch belastingniveau
     (jaren)            (m+NAP)
         10              1.270
         30              1.403
        100              1.527
        300              1.644
       1000              1.781
       3000              1.921
      10000              2.092
      30000              2.264
     100000              2.461

Illustratiepunten bij opgegeven terugkeertijden:

Illustratiepunten bij hydraulisch belastingniveau  1.76 (m+NAP) en terugkeertijd   833 (jaar)

Locatie                     = MM_3_hy15 (128899,486337)
Berekeningstype             = Hydraulisch belastingniveau, golfoverslag met kritiek overslagdebiet van 10.00 (l/s/m)
Hydraulisch belastingniveau =     1.76 (m+NAP)
Terugkeertijd               =      833 (jaar)
```

**Figure C.1:** First part of HydraNL calculation

Overschrijdingsfrequentie = 1.20E-03 (per jaar)

| r | meerp. m+NAP | -- | -- | windsn. m/s | h,teen m+NAP | Hm0,teen m | Tm-1,0,t s | golfr graden | ov. freq *0.001/whj | ov. freq % |
|---|---|---|---|---|---|---|---|---|---|---|
| 30.0 | 0.39 | -- | -- | 8.4 | 1.79 | 0.13 | 0.98 | 35.6 | 0.135 | 11.2 |
| 60.0 | 0.39 | -- | -- | 8.8 | 1.79 | 0.16 | 1.05 | 69.4 | 0.257 | 21.4 |
| 90.0 | 0.71 | -- | -- | 6.0 | 1.79 | 0.13 | 0.84 | 95.2 | 0.032 | 2.7 |
| 120.0 | 0.77 | -- | -- | 6.0 | 1.79 | 0.14 | 0.80 | 125.4 | 0.006 | 0.5 |
| 150.0 | 0.79 | -- | -- | 6.0 | 1.79 | 0.15 | 0.83 | 150.6 | 0.007 | 0.6 |
| 180.0 | 0.79 | -- | -- | 6.0 | 1.78 | 0.14 | 0.78 | 174.4 | 0.011 | 0.9 |
| 210.0 | 0.74 | -- | -- | 9.0 | 1.74 | 0.22 | 0.89 | 208.3 | 0.029 | 2.4 |
| 240.0 | 0.73 | -- | -- | 10.5 | 1.72 | 0.25 | 1.07 | 247.1 | 0.041 | 3.4 |
| 270.0 | 0.72 | -- | -- | 10.5 | 1.71 | 0.26 | 1.15 | 272.1 | 0.034 | 2.8 |
| 300.0 | 0.74 | -- | -- | 9.0 | 1.73 | 0.21 | 0.96 | 293.5 | 0.014 | 1.1 |
| 330.0 | 0.39 | -- | -- | 9.6 | 1.74 | 0.18 | 0.89 | 315.9 | 0.202 | 16.8 |
| 360.0 | -0.09 | -- | -- | 22.6 | 1.70 | 0.38 | 1.86 | 354.1 | 0.435 | 36.2 |
| som | | | | | | | | | 1.200 | 100.0 |

Onzekerheidswaarden (let op: deze zijn reeds verwerkt in de weergeven waterstanden/golfparameters)

| r | h onz. m | f_Hm0 - | f_Tm-1,0 - | f_Tp - | ov. freq % |
|---|---|---|---|---|---|
| 30.0 | 0.64 | 0.95 | 1.04 | 0.96 | 11.2 |
| 60.0 | 0.64 | 0.95 | 1.04 | 0.96 | 21.4 |
| 90.0 | 0.43 | 0.95 | 1.04 | 0.96 | 2.7 |
| 120.0 | 0.43 | 0.95 | 1.04 | 0.96 | 0.5 |
| 150.0 | 0.43 | 0.95 | 1.04 | 0.96 | 0.6 |
| 180.0 | 0.43 | 0.95 | 1.04 | 0.96 | 0.9 |
| 210.0 | 0.43 | 0.95 | 1.04 | 0.96 | 2.4 |
| 240.0 | 0.43 | 0.95 | 1.04 | 0.96 | 3.4 |
| 270.0 | 0.43 | 0.95 | 1.04 | 0.96 | 2.8 |
| 300.0 | 0.43 | 0.95 | 1.04 | 0.96 | 1.1 |
| 330.0 | 0.64 | 0.95 | 1.04 | 0.96 | 16.8 |
| 360.0 | 0.43 | 0.95 | 1.04 | 0.96 | 36.2 |

Betekenis van de gegevens:
- r = De windrichting
- meerp. = De ruimtelijk gemiddelde waterstand van het Markermeer in m+NAP
- windsn. = De potentiële windsnelheid van Schiphol in m/s
- h,teen = De waterstand op de doorgerekende locatie in m+NAP na eventuele transformatie over een voorland
- Hm0,teen = De significante golfhoogte in m na eventuele transformatie over een dam en/of voorland
- Tm-1,0,t = De spectrale golfperiode in s na eventuele transformatie over een voorland
- golfr = De golfrichting in graden t.o.v. Noord na eventuele transformatie over een voorland
- ov.freq = De overschrijdingsfrequentie van het hydraulisch belastingniveau voor de bijbehorende windrichting in gemiddeld aantal keer per winterhalfjaar en als percentage
- h onz. = De verhoging van de waterstand ten gevolge van de onzekerheid in de waterstand in m vóór een eventuele transformatie over een voorland
- f_Hm0 = De vermenigvuldigingsfactor van de golfhoogte als gevolg van de onzekerheid in de golfhoogte vóór een eventuele transformatie over een voorland
- f_Tm-1,0 = De vermenigvuldigingsfactor van de spectrale golfperiode als gevolg van de onzekerheid in de spectrale golfperiode vóór een eventuele transformatie over een voorland
- f_Tp = De vermenigvuldigingsfactor van de piekperiode als gevolg van de onzekerheid in de piekperiode vóór een eventuele transformatie over een voorland

Hoofdillustratiepunt bij hydraulisch belastingniveau 1.76 (m+NAP) en terugkeertijd 833 (jaar)

| | |
|---|---|
| windrichting r (bijdrage aan ov.freq) | 360.0 ( 36.2%) |
| Markermeerpeil m [m+NAP] | -0.09 |
| potentiële windsnelheid u [m/s] | 22.6 |
| lokale waterstand h [m+NAP] | 1.70 |
| significante golfhoogte Hm0 [m] | 0.38 |
| spectrale golfperiode Tm-1,0 [s] | 1.86 |
| golfrichting t.o.v. Noord [graden] | 354.1 |
| onz. lokale waterstand [m] | 0.43 |
| onz. significante golfhoogte [-] | 0.95 |
| onz. spectrale golfperiode [-] | 1.04 |
| onz. piekperiode [-] | 0.96 |

Uitsplitsingen van de overschrijdingsfrequentie bij opgegeven terugkeertijden:

Uitsplitsingen bij hydraulisch belastingniveau 1.76 (m+NAP) en terugkeertijd 833 (jaar)

Uitsplitsingen naar windrichtingen
Uitsplitsingen naar meerpeilen
Uitsplitsingen naar windsnelheden
Uitsplitsingen naar windsnelheden en windrichtingen

| | |
|---|---|
| Locatie | = MM_3_hy15 (128899,486337) |
| Berekeningstype | = Hydraulisch belastingniveau, golfoverslag met kritiek overslagdebiet van 10.00 (l/s/m) |
| Hydraulisch belastingniveau | = 1.76 (m+NAP) |
| Terugkeertijd | = 833 (jaar) |
| Overschrijdingsfrequentie | = 1.20E-03 (per jaar) |

Uitsplitsingen over de windrichting onafhankelijk van de stormduur over alle gegevensblokken

| richting | bijdrage |
|---|---|
| 30.0 | 11.245% |
| 60.0 | 21.380% |
| 90.0 | 2.662% |
| 120.0 | 0.482% |
| 150.0 | 0.556% |
| 180.0 | 0.896% |
| 210.0 | 2.406% |
| 240.0 | 3.420% |
| 270.0 | 2.792% |
| 300.0 | 1.138% |
| 330.0 | 16.819% |
| 360.0 | 36.204% |
| som | 100.000% |

**Figure C.2:** Second part of HydraNL calculation

```
Locatie                         = MM_3_hy15 (128899,486337)
Berekeningstype                 = Hydraulisch belastingniveau, golfoverslag met kritiek overslagdebiet van 10.00 (l/s/m)
Hydraulisch belastingniveau     =     1.76 (m+NAP)
Terugkeertijd                   =      833 (jaar)
Overschrijdingsfrequentie       = 1.20E-03 (per jaar)

Percentielen van het meerpeil (m+NAP) over alle gegevensblokken

 percentiel | waarde
------------+------------
         5% |      -0.20
        10% |      -0.17
        25% |      -0.08
        50% |       0.25
        75% |       0.57
        90% |       0.82
        95% |       0.96

Uitsplitsingen over de meerpeilen over alle gegevensblokken

 meerp. (m+NAP)| bijdrage       cond. cumul.
---------------+---------------------------
 -0.40 - -0.30 |      0.000%         0.000%
 -0.30 - -0.20 |      4.496%         4.496%
 -0.20 - -0.10 |     17.719%        22.215%
 -0.10 -  0.00 |     13.815%        36.031%
  0.00 -  0.10 |      7.282%        43.312%
  0.10 -  0.20 |      4.799%        48.111%
  0.20 -  0.30 |      4.421%        52.532%
  0.30 -  0.40 |      7.916%        60.448%
  0.40 -  0.50 |      9.345%        69.792%
  0.50 -  0.60 |      7.579%        77.371%
  0.60 -  0.70 |      6.829%        84.200%
  0.70 -  0.80 |      5.174%        89.374%
  0.80 -  0.90 |      3.664%        93.039%
  0.90 -  1.00 |      2.883%        95.921%
  1.00 -  1.10 |      1.622%        97.543%
  1.10 -  1.20 |      1.148%        98.691%
  1.20 -  1.30 |      0.618%        99.309%
  1.30 -  1.40 |      0.345%        99.654%
  1.40 -  1.80 |      0.346%       100.000%
---------------+---------------------------
 som           |    100.000%


Locatie                         = MM_3_hy15 (128899,486337)
Berekeningstype                 = Hydraulisch belastingniveau, golfoverslag met kritiek overslagdebiet van 10.00 (l/s/m)
Hydraulisch belastingniveau     =     1.76 (m+NAP)
Terugkeertijd                   =      833 (jaar)
Overschrijdingsfrequentie       = 1.20E-03 (per jaar)

Percentielen van de windsnelheid (m/s) over alle gegevensblokken

 percentiel | waarde
------------+------------
         5% |        5.8
        10% |        7.2
        25% |       10.2
        50% |       18.5
        75% |       24.0
        90% |       27.5
        95% |       29.6

Uitsplitsingen over de windsnelheden over alle gegevensblokken

 windsn. (m/s) | bijdrage       cond. cumul.
---------------+---------------------------
   0.0 -   4.0 |      1.417%         1.417%
   4.0 -   8.0 |     12.025%        13.442%
   8.0 -  12.0 |     19.619%        33.060%
  12.0 -  16.0 |     10.501%        43.561%
  16.0 -  20.0 |     11.798%        55.359%
  20.0 -  24.0 |     19.889%        75.248%
  24.0 -  28.0 |     16.102%        91.350%
  28.0 -  32.0 |      6.758%        98.109%
  32.0 -  36.0 |      1.685%        99.794%
  36.0 -  40.0 |      0.193%        99.987%
  40.0 -  50.0 |      0.013%       100.000%
---------------+---------------------------
 som           |    100.000%


Locatie                         = MM_3_hy15 (128899,486337)
Berekeningstype                 = Hydraulisch belastingniveau, golfoverslag met kritiek overslagdebiet van 10.00 (l/s/m)
Hydraulisch belastingniveau     =     1.76 (m+NAP)
Terugkeertijd                   =      833 (jaar)
Overschrijdingsfrequentie       = 1.20E-03 (per jaar)

Uitsplitsingen over de windsnelheid en de windrichting over alle gegevensblokken

             r |    30.0    |    60.0    |    90.0    |   120.0    |   150.0    |   180.0
 windsn. (m/s) |            |            |            |            |            |
---------------+------------+------------+------------+------------+------------+-----------
   0.0 -   4.0 |     0.129% |     0.175% |     0.190% |     0.103% |     0.111% |     0.106%
   4.0 -   8.0 |     1.852% |     2.651% |     1.257% |     0.314% |     0.351% |     0.496%
   8.0 -  12.0 |     3.171% |     6.051% |     0.856% |     0.061% |     0.087% |     0.248%
  12.0 -  16.0 |     1.346% |     2.067% |     0.135% |     0.003% |     0.006% |     0.041%
  16.0 -  20.0 |     1.780% |     2.619% |     0.058% |     0.000% |     0.000% |     0.003%
  20.0 -  24.0 |     2.037% |     4.740% |     0.067% |     0.000% |     0.000% |     0.000%
  24.0 -  28.0 |     0.792% |     2.687% |     0.066% |     0.000% |     0.000% |     0.000%
  28.0 -  32.0 |     0.091% |     0.471% |     0.025% |     0.000% |     0.000% |     0.000%
  32.0 -  36.0 |     0.004% |     0.029% |     0.003% |     0.000% |     0.000% |     0.000%
  36.0 -  40.0 |     0.000% |     0.001% |     0.000% |     0.000% |     0.000% |     0.000%
  40.0 -  50.0 |     0.000% |     0.000% |     0.000% |     0.000% |     0.000% |     0.000%
---------------+------------+------------+------------+------------+------------+-----------
 som           |    11.245% |    21.380% |     2.662% |     0.482% |     0.556% |     0.896%
```

**Figure C.3:** Third part of HydraNL calculation

Uitsplitsingen over de windsnelheid en de windrichting over alle gegevensblokken

| r / windsn. (m/s) | 210.0 | 240.0 | 270.0 | 300.0 | 330.0 | 360.0 |
|---|---|---|---|---|---|---|
| 0.0 - 4.0 | 0.087% | 0.068% | 0.047% | 0.053% | 0.130% | 0.217% |
| 4.0 - 8.0 | 0.724% | 0.531% | 0.444% | 0.281% | 1.611% | 1.513% |
| 8.0 - 12.0 | 0.920% | 0.965% | 0.821% | 0.377% | 3.734% | 2.327% |
| 12.0 - 16.0 | 0.491% | 1.024% | 0.856% | 0.233% | 2.450% | 1.849% |
| 16.0 - 20.0 | 0.126% | 0.512% | 0.355% | 0.107% | 1.721% | 4.516% |
| 20.0 - 24.0 | 0.041% | 0.227% | 0.179% | 0.061% | 1.236% | 11.302% |
| 24.0 - 28.0 | 0.011% | 0.065% | 0.064% | 0.018% | 2.203% | 10.196% |
| 28.0 - 32.0 | 0.003% | 0.016% | 0.020% | 0.005% | 2.350% | 3.778% |
| 32.0 - 36.0 | 0.001% | 0.003% | 0.005% | 0.001% | 1.194% | 0.446% |
| 36.0 - 40.0 | 0.000% | 0.000% | 0.001% | 0.000% | 0.170% | 0.021% |
| 40.0 - 50.0 | 0.000% | 0.000% | 0.000% | 0.000% | 0.012% | 0.001% |
| som | 2.406% | 3.420% | 2.792% | 1.138% | 16.819% | 36.204% |

Conditioneel cumulatieve bijdragen van de windsnelheid en de windrichting over alle gegevensblokken

| r / windsn. (m/s) | 30.0 | 60.0 | 90.0 | 120.0 | 150.0 | 180.0 |
|---|---|---|---|---|---|---|
| 0.0 - 4.0 | 1.148% | 0.815% | 7.161% | 21.383% | 20.068% | 11.889% |
| 4.0 - 8.0 | 17.687% | 13.151% | 54.460% | 86.708% | 83.178% | 67.270% |
| 8.0 - 12.0 | 45.995% | 41.308% | 86.695% | 99.362% | 98.923% | 95.019% |
| 12.0 - 16.0 | 58.008% | 50.928% | 91.763% | 99.958% | 99.965% | 99.606% |
| 16.0 - 20.0 | 73.898% | 63.115% | 93.961% | 99.995% | 99.999% | 99.972% |
| 20.0 - 24.0 | 92.086% | 85.170% | 96.472% | 99.999% | 100.000% | 99.998% |
| 24.0 - 28.0 | 99.154% | 97.672% | 98.964% | 100.000% | 100.000% | 100.000% |
| 28.0 - 32.0 | 99.965% | 99.862% | 99.889% | 100.000% | 100.000% | 100.000% |
| 32.0 - 36.0 | 99.999% | 99.997% | 99.997% | 100.000% | 100.000% | 100.000% |
| 36.0 - 40.0 | 100.000% | 100.000% | 100.000% | 100.000% | 100.000% | 100.000% |
| 40.0 - 50.0 | 100.000% | 100.000% | 100.000% | 100.000% | 100.000% | 100.000% |

Conditioneel cumulatieve bijdragen van de windsnelheid en de windrichting over alle gegevensblokken

| r / windsn. (m/s) | 210.0 | 240.0 | 270.0 | 300.0 | 330.0 | 360.0 |
|---|---|---|---|---|---|---|
| 0.0 - 4.0 | 3.633% | 2.006% | 1.688% | 4.681% | 0.773% | 0.599% |
| 4.0 - 8.0 | 33.733% | 17.574% | 17.603% | 29.420% | 10.355% | 4.783% |
| 8.0 - 12.0 | 71.996% | 45.860% | 47.014% | 62.606% | 32.564% | 11.216% |
| 12.0 - 16.0 | 92.427% | 75.872% | 77.678% | 83.117% | 47.137% | 16.328% |
| 16.0 - 20.0 | 97.664% | 90.884% | 90.379% | 92.552% | 57.376% | 28.815% |
| 20.0 - 24.0 | 99.349% | 97.526% | 96.804% | 97.900% | 64.728% | 60.066% |
| 24.0 - 28.0 | 99.820% | 99.434% | 99.100% | 99.481% | 77.832% | 88.260% |
| 28.0 - 32.0 | 99.960% | 99.897% | 99.813% | 99.900% | 91.812% | 98.707% |
| 32.0 - 36.0 | 99.996% | 99.989% | 99.977% | 99.989% | 98.917% | 99.939% |
| 36.0 - 40.0 | 100.000% | 99.999% | 99.997% | 99.999% | 99.929% | 99.998% |
| 40.0 - 50.0 | 100.000% | 100.000% | 100.000% | 100.000% | 100.000% | 100.000% |

**Figure C.4:** Fourth part of HydraNL calculation

| | | | | OI 2014 | PVE IB |
|---|---|---|---|---|---|
| **Input in Hydra NL** | | | | | |
| | *Terugkeertijd [1/jaar]* | | | 1/830 | 1/830 |
| | *Overslagdebiet* | | | 10 l/s/m | 10 l/s/m |
| | *Klimaat* | KNMI 2006 W+ | | | |
| | *Toeslagen* | | | | |
| | | Model onzekerheid waterstand | | 0,40 m | 0,40 m |
| | | Model onzekerheid golfhoogte | | 10% | 10% |
| | | Model onzekerheid golfperiode | | 10% | 10% |
| *Toekomstige peilverhogingen* | | | | | |
| | | Tot 2050 | | 0,0 m | 0,0 m |
| | | Na 2050 | | 0,10 m | **0,18 m** |
| | | | | | |
| | *Hydraulisch belastingsniveau Hydra NL* | | | 1,22 m | 1,29 m |
| **Output Hydra NL** | *Na correctie hydraulisch belastingsniveau geen strekdam* | | | **1,32 m** | **1,39 m** |
| | | | | | |
| | *Toeslag Slingeringen* | | | 0,10 m | **0,26 m** |
| | *Toeslag Kennisonzekerheid* | | | 0,0 m | **0,30 m** |
| *Extra hoogte bij aanleg restzettingeis* | | | | 0,30 m | **0,30 m** |
| | | | | | |
| **Kruinhoogte waterkering** | | | | **1,62 m** | **2,15 m** |
| | | | | | |
| *Extra hoogte ten behoeve van weg of fietspad* | | | | 0,45 m | 0,45 m |
| | | | | | |
| **Maaiveldhoogte** | | | | **2,07 m** | **2,60 m** |

**Figure C.5:** Construction of the crest height of the flood defence according to OI2014v4 and according to design of flood defence as an example for the northwestern profile of Strandeiland

# D

# Multi Criteria Analysis Gate Selection

This appendix offers a deeper examination into the Multi Criteria Analysis (MCA) that informed the gate selection for the lock system. While the main report touches upon the overarching considerations, here we delve into the nuances and specifics of each criterion. From evaluating the type of lock and lock width, to costs, durability, and operability factors, this appendix is essential for anyone seeking a comprehensive grasp of the criteria guiding our gate selection process. The details encapsulated in this segment ensure a holistic appreciation of the informed decisions made in the main report.

## D.1. Gate selection

- **Type of lock and lock width:**
  For a first 'rough' selection of gate alternatives, i.e. on a general level without considering any specific local conditions or a special ship traffic situation, Table D.1 could be used.

| Type of lock | Lock width* | Mitre gate | | | Single leaf gate | | Rolling gate | Vertical lift gate | |
|---|---|---|---|---|---|---|---|---|---|
| | | Single sided water retention | Double sided water retention | Double sided water retention | Single sided water retention | Double sided water retention | Double sided water retention | Single sided water retention | Double sided water retention |
| Single sided water retaining inland lock | Very small | | | | X | | | | |
| | Small | X | | | X | | | | |
| | Medium | X | | | | | | X | |
| | Large | X | | | | | | X | |
| Double sided water retaining inland lock | Very small | | | | | X | | | |
| | Small | | X | X | | X | | | |
| | Medium | | X | X | | X | | | |
| | Large | | X | X | | | X | | X |
| Double sided water retaining sea lock | Small | | X | X | | X | | | |
| | Medium | | X | X | | X | | | |
| | Large | | X | X | | | X | | |
| | Very large | | | | | | X | | |

* Lock width explanation:

- Very small 4-6 m
- Small 6-10 m
- Medium 10-16 m
- Large 16-24 m
- Very large >24 m

**Figure D.1:** Gate selection per type of lock and depending on lock width (Molenaar, 2011)

- **Retention of water in one or both directions:**
  Mitre gates are only able to retain water in one direction. When water retention is required in two directions a double set of mitre gates must be installed, or special measures must be taken to equip a single set for this task. A double set of mitre gates results in an increased length of the lock heads. Rolling or sliding gates, or lift gates are possibly a more suitable solution.

- **Opening and closing of gates under head difference:**
  Unless special precautions are made, mitre gates and rolling gates should not be opened or closed under a water head difference for reasons of strength and stability.

- **The area available for lock heads:**
  Often the lock head for a lift gate requires the least of area and is relatively easy to construct, however, the big disadvantage of the lift gate will always be the limitation of the air draught of the passing vessels. Due to their height lift gates often are a landmark, which is difficult to fit in harmonically with the surrounding environment. Except the submersible gate, all the other gate types have a larger horizontal footprint, relatively spoken, than the lift gate. The construction and maintenance cost disadvantage of submersible gates generally overrides the horizontal space advantage.

- **Width of the lock chamber:**
  Lift and rolling gates are able to span the full width of a lock chamber. However, the bigger lock width and larger gate go hand in hand with a larger and heavier lock head structure. Since the 2 doors of mitre gates have about half the size of the chamber width this will results in lighter lock head structures.

- **Equipment to operate or move the gate:**
  The centre of gravity of lift gates is moved vertically, this requires a large amount of power, thus considerable machinery. Although this can be reduced significantly by the use of counter weights, a certain price has to be paid with regard to machine power on one hand and the required strength and stiffness of the concrete structure on the other hand. The same will apply for a submersible gates and sector gates with a horizontal axis. Mitre, rolling gates and sector gates with a vertical rotation axis move in horizontal direction, which is less demanding for equipment. The installed power and the equipment will be smaller.

- **Vulnerability for ship impact:**
  Supports of mitre gates are vulnerable for ship impact. Usually both the gates as well as the supports are damaged severely. Even in opened condition the door might suffer from ship impact. By their more sturdy structure, rolling or sliding gates are better equipped to resist vessel impact. As an additional advantage a rolling or sliding gate cannot be damaged when it is opened because at that time the gate is completely protected in its recess.

- **Inspection, maintenance and repair:** Moving or sliding parts of a gate are subjected to quite some wear and tear, therefore regular inspection, maintenance work and repair are necessary. Lift gates are lifted out of the water frequently, which offers good opportunities for inspection, small maintenance and minor repairs. Rolling or sliding gates can temporarily be set in dry in their recesses; the recess functions as a temporary dry-dock in that case. However, to keep the navigation lock operational it will be necessary to provide two gates per lock head. For inspection and repair on the rails or sliding track the navigation lock is locally dewatered by using a steel watertight structure which is open on the bottom and functions as a kind of diving bell. Mitre gates are usually lifted out of the water for maintenance and repair; this requires a double set of doors if the lock has to remain operational. When inspecting or repairing the lock head a maintenance limpet (inspectie kolk) is used to inspect the pivot, pindle, quoins, pivot shoe or supporting frame. A limpet can have a box shape and is put in position using a mobile crane and is ready to operate after the water is pumped out. Before positioning the limpet the canal bed must be cleared of objects that might obstruct the sealing of the limpet.

## D.2. Multi Criteria Analysis

It is necessary to examine the conceptual design's many choices in order to choose the finest option for this project. In the first step, the MCA will reduce the number of possibilities from 4 to those with the greatest score. There should be a clear separation between the chosen options and the remaining alternatives in the final score. Consequently, distinct criteria are created and assessed for each option, and each alternative is assigned a score. These scores are multiplied by a weighting factor, since not all criteria are of equal value. In this chapter, a pairwise comparison will be used to establish the weighting factors. Initially, the criteria for the multi-criteria evaluation will be specified.

### D.2.1. Criteria

Each option will be assessed according to five criteria. The multicriteria analysis is based on these criteria. These five criteria are subdivided into subcriteria, and the definitions of each subcriteria and criterion are provided below. The primary factors are:

- Durability
- Usability
- Costs
- Constructability & Technical feasibility

### D.2.2. Costs

This project tries to pick a guard lock from a number of possibilities, therefore costs for the various designs might vary considerably. In the majority of cases, cost is a crucial criteria.

The following costs are considered total costs in the MCA:

- **Labor cost:** Includes the cost of every labor performed during the project's duration.
- **Material cost:** This covers all expenditures directly associated with the structure's construction.
- **Maintenance costs:**
  - Frequency of maintenance
  - Total costs of maintenance

### D.2.3. Durability

The durability is classified into the structure's durability and its life expectancy. The lifetime of the structure: The lifetime of a building is simply the expected duration of its existence. Parts may be replaced throughout this period, but the primary framework must stay intact. The structure's durability: Contains two material characteristics and a level of energy consumption:

- **Quantity of material required:** raw material/building equipment) provides an indicator of the resulting environmental pollution and greenhouse impact. A greater influence on the environment and the greenhouse effect is caused by an abundance of materials and heavy equipment.
- **Type of required material:** the primary difference between concrete and steel (maybe also plastics). Concrete is more enduring and lasts longer than (unprotected) steel.
- **Energy required to open and close a hydraulic structure:** Not only does energy consumption cost money, but it also has detrimental effects on the environment. The greater the amount of energy required to move a vessel in a lock or to open and shut a gate, the greater the expenses and environmental damage.

### D.2.4. Constructability and technical feasibility

Constructability and technical feasibility are essential in countries with little experience with big and unique projects that are relatively new to the construction industry. Constructability and technical feasibility can be subdivided into the required experience and competence, temporary structures, and structural complexity.

- **Experience/knowledge required:** Designing a guard lock requires a great deal of skill and knowledge from the consultants and the contractor. To prevent excessive execution costs, (some of) the required knowledge and competence should be accessible in Amsterdam.

- **Lock complexity:** complication of the lock's positioning and structure's installation. For instance, the building and installation of very large numbers of components or a single extremely huge element may be deemed more complicated than ordinary structures. Additionally, working on water and in-situ concrete works are examples of more complicated labor, and these complex constructions might incur extra costs

### D.2.5. Usability

The structure's usability explains the guard lock's behavior and operation after the building phase. The navigation function and the retaining/regulating function of the hydraulic structure will be evaluated for their usefulness. Additionally, the structure's operability will be highlighted.

- **Navigation function:** The navigation function defines the structure's capacity to handle the vessels that pass through Strandeiland.
- **Water retention function:** The retention function specifies the structure's capacity to retain high tidal water levels from the sea and manage water levels.
- **Operability:** The structure's operability defines the risk of failure and the structure's user-friendliness. When the structure (operational and structural) breaks during operation, the repercussions for Strandeiland may be catastrophic. There might be fatalities and a great deal of destruction. Consequently, the risk of failure of the hydraulic structure is a crucial design and operation consideration.

# E

# Calculations

## E.1. Appendix Introduction

The appendices detailed herein present an array of comprehensive calculations and Python scripts essential for understanding various aspects of the lock design and structural considerations. These calculations have been instrumental in informing the design decisions and evaluations documented within the main report. Each section addresses a distinct aspect of the lock design, from stability checks to probabilistic wave height estimations.

1. **Stability Check Calculations (Section Stability Check Calculations)**: This section encompasses a series of Python scripts aimed at understanding the stability considerations of the lock. It initiates with the Brinch Hansen method for bearing capacity analysis (refer to Listing E.1), followed by the vertical load computations for the guard lock (refer to Listing E.2). Additionally, it covers the horizontal load dynamics on the guard lock gates (refer to Listing E.5) and provides a meticulous analysis of the lateral sliding capacity (refer to Listing E.6) and the overturning stability check (refer to Listing E.7).
2. **Probabilistic Wave Height Calculations (Section Probabilistic calculations Wave Height)**: Wave dynamics play a pivotal role in the functioning of the lock, and this section, through its Python script (refer to Listing E.8), focuses on the probabilistic calculations surrounding wave height, taking into account various parameters like depth, fetch mean, and wind speed.
3. **Monte Carlo Analysis for Gate Height (Section Monte Carlo analysis gate height)**: Monte Carlo analysis serves as a robust tool for risk assessment and uncertainty evaluation. In this segment, a Python script (refer to Listing E.9) elucidates the Monte Carlo analysis employed for the precise determination of the gate height.
4. **Load Calculations for Mitre Gates (Section Load Calculations for Mitre Gates)**: The mitre gates, being pivotal components of the lock system, necessitate an intricate understanding of the loads they bear. The Python script in this section (refer to Listing E.10) provides an exhaustive breakdown of the various forces exerted on the mitre gates, considering parameters like the gate dimensions, water levels, and gate inclinations.

These scripts and calculations provide a foundational understanding of the diverse forces and dynamics at play in the design and operation of the lock system. By detailing the methods and equations used, this appendix ensures transparency, reproducibility, and a robust verification mechanism for the design considerations proposed in the main body of the report.

## E.2. Stability Check Calculations

```
import math

# Given parameters
length_lock = 22
```

```python
width_lock = 7.6
depth_lock = 3.6
unit_weight_soil = 20
cohesion_soil = 0
angle_internal_friction = 30

# Factors based on angle of internal friction
Nc = 37.3
Sc = 1.066
Ic = 0.88

# Factors based on shape of the foundation
B = 1
Nq = 17.7

# Factors based on depth of the foundation
D = 1
N_gamma = 15.7

# Calculate allowable bearing pressure
q_allow = (cohesion_soil * Nc * Sc * Ic) + (0.5 * unit_weight_soil * B * Nq) + (0.5 *
    unit_weight_soil * D * N_gamma)
q_allow = round(q_allow, 2)

print(f"Allowable bearing pressure (q_allow) is: {q_allow} kN/m^2")
```

**Listing E.1:** Python script for Brinch Hansen method

```python
# Input values
wall_height = 6.85
wall_thickness = 0.6 # m
structure_length = 22
floor_slab_thickness = 0.3 # m
lock_width = 7.6
max_water_level = 3.6

# Unit weights
unit_weight_concrete = 25 # kN/m^3 reinforced concrete
unit_weigt_water = 10 # kN/m^3

# Calculate the volume of the structure
water_volume = structure_length * structure_length * max_water_level
walls_volume = (wall_height * wall_thickness * structure_length)*2
floor_slab_volume = floor_slab_thickness * structure_length * lock_width
total_volume_concrete = walls_volume + floor_slab_volume

# Calculate the total weight
weight_water = water_volume * unit_weigt_water
weight_concrete = total_volume_concrete * unit_weight_concrete
total_weight = weight_water + weight_concrete

# Calculate the load per square meter
area = structure_length * (lock_width + 2*wall_thickness)
load_per_square_meter = total_weight / area

# Print the result
print("Total weight concrete:", weight_concrete, "kN")
print("Total weight water", weight_water, "kN")
print("Area", area, "m^2")
print("load_per_square_meter", load_per_square_meter, "kN/m^2")
```

**Listing E.2:** Vertical load of the guard lock

```python
# Given data for uplift
length_foundation = 36.46 # in meters
width_foundation = 8.8 # in meters
groundwater_level = 4.9 # in meters
unit_weight_soil = 20 # in kN/m^3
unit_weight_water = 10 # in kN/m^3

# Calculating the area of the foundation
area_foundation = length_foundation * width_foundation # in m^2

# Uplift force calculation
uplift_force = area_foundation * groundwater_level * unit_weight_water # in kN

# Downward force calculation
weight_of_soil_above_groundwater = unit_weight_soil * area_foundation * groundwater_level # in
    kN
weight_of_structure = 23199 # Total vertical load in kN
downward_force = weight_of_structure + weight_of_soil_above_groundwater # in kN

print(f"Uplift Force: {uplift_force} kN")
print(f"Downward Force: {downward_force} kN")
```

**Listing E.3:** Python script for Uplift Force Calculation

```python
# Assumed values for settlement
E = 40000 # Modulus of elasticity in kPa
Is = 0.7 # Influence factor

# Applied pressure q (vertical stress imposed by the structure)
q = weight_of_structure / area_foundation # in kPa

# Immediate settlement calculation
delta = (q * width_foundation) / E * Is

print(f"Immediate Settlement: {delta} meters")
```

**Listing E.4:** Python script for Immediate Settlement Calculation

```python
# Inputs
gate_height = 6.04 # in meters
gate_width = 7.6 # in meters
water_level_low = 3.6 # in meters
water_level_high = 4.6 # in meters
water_density = 1000 # in kg/m3
gravitational_constant = 9.81 # in m/s2

# Calculate hydrostatic pressures
hydrostatic_pressure_low = water_density * gravitational_constant * water_level_low
hydrostatic_pressure_high = water_density * gravitational_constant * water_level_high

# Calculate horizontal forces
horizontal_force_low = hydrostatic_pressure_low * gate_width * (1/3)
horizontal_force_high = hydrostatic_pressure_high * gate_width * (1/3)

# Calculate resulting force and its direction
resulting_force = abs(horizontal_force_high - horizontal_force_low)
resulting_direction = "left" if horizontal_force_low > horizontal_force_high else "right"

# Calculate the arm of the resulting force
resulting_arm = gate_height/2 if resulting_direction == "left" else gate_height - (gate_height
    /2)
```

```python

# Print outputs
print("Hydrostatic pressure on side with lower water level: {:.2f} kPa".format(
    hydrostatic_pressure_low/1000))
print("Hydrostatic pressure on side with higher water level: {:.2f} kPa".format(
    hydrostatic_pressure_high/1000))
print("Horizontal force on side with lower water level: {:.2f} kN".format(horizontal_force_low
    /1000))
print("Horizontal force on side with higher water level: {:.2f} kN".format(
    horizontal_force_high/1000))
print("Resulting force: {:.2f} kN, directing towards {}".format(resulting_force/1000,
    resulting_direction))
print("Distance of resulting force from lower edge of gate: {:.2f} m".format(resulting_arm))
```

**Listing E.5:** Horizontal load on the guard lock gates

```python
import math

# Input parameters
total_horizontal_force = 24.85 # kN
total_vertical_force = 23199 # N
angle_internal_friction = 30 # degrees

# Conversion factor for degrees to radians
deg_to_rad = math.pi / 180

# Calculation of lateral sliding capacity check
shear_strength = total_vertical_force * math.tan(angle_internal_friction * deg_to_rad)
if shear_strength >= total_horizontal_force:
    print("The lateral sliding capacity check is OK")
else:
    print("The lateral sliding capacity check has failed. Shear strength is less than total
        horizontal force.")
```

**Listing E.6:** Lateral sliding capacity check

```python
def overturning_stability_check(length, width, height, Fh, Fv, loc_Fh, loc_Fv,
    factor_of_safety):
    moment_horizontal = Fh * loc_Fh
    moment_vertical = Fv * loc_Fv
    safety_ratio = moment_vertical / moment_horizontal
    return moment_horizontal, moment_vertical, safety_ratio

length = 22
width = 8.8
height = 6.85
Fh = 24.85 * 1000 # Convert kN to N
Fv = 23199 * 1000 # Convert kN to N
loc_Fh = 0.5 * height
loc_Fv = 0.5 * length
desired_factor_of_safety = 2

moment_horizontal, moment_vertical, safety_ratio = overturning_stability_check(length, width,
    height, Fh, Fv, loc_Fh, loc_Fv, desired_factor_of_safety)

print(f"Horizontal moment: {moment_horizontal} Nm")
print(f"Vertical moment: {moment_vertical} Nm")
print(f"Factor of safety: {safety_ratio}")

if safety_ratio >= desired_factor_of_safety:
    print("The structure is stable against overturning with the desired factor of safety.")
```

```
else:
    print("The structure is not stable against overturning with the desired factor of safety.")
```

**Listing E.7:** Overturning stability check

## E.3. Probabilistic calculations Wave Height

```
import numpy as np
import math

def bretschneider_wave(depth, fetch_mean, wind_speed):
    g = 9.81 # gravitational acceleration
    d = depth
    Hs = (wind_speed ** 2) * (0.283 * np.tanh(0.0125 * ((g * fetch_mean) / wind_speed ** 2)) ** 0.42) / g
    Tp = 2.4 * math.pi * math.sqrt(d / g) * ((0.076 * (wind_speed ** 2) * fetch_mean / (g * d ** 2)) ** -0.23)
    return Hs, Tp

# Input data
wind_speed_mean = 6
wind_speed_std = 2
wind_direction_mean = 210
wind_direction_std = 60
fetch_mean = 1000
depth = 5

# Monte Carlo simulation
n_samples = 10000

wind_speed_samples = np.random.normal(wind_speed_mean, wind_speed_std, n_samples)
wind_direction_samples = np.random.normal(wind_direction_mean, wind_direction_std, n_samples)
wind_speed_u_samples = wind_speed_samples * np.cos(np.radians(wind_direction_samples))

Hs_samples, Tp_samples = bretschneider_wave(depth, fetch_mean, wind_speed_u_samples)

# Calculate mean and standard deviation
Hs_mean = np.mean(Hs_samples)
Hs_std = np.std(Hs_samples)
Tp_mean = np.mean(Tp_samples)
Tp_std = np.std(Tp_samples)

print("Hs mean:", Hs_mean)
print("Hs standard deviation:", Hs_std)
print("Tp mean:", Tp_mean)
print("Tp standard deviation:", Tp_std)
```

**Listing E.8:** Probabilistic calculations wave height Python code

## E.4. Monte Carlo analysis gate height

```


# Constants
g = 9.81 # acceleration due to gravity
a = 0.192
b = 4.3
 = 1
q = 10 / 1000 # convert l/s/m to m³/s/m
n_simulations = 10000000
```

```
P_f = 0.0024 # allowable failure probability
Hs_mean = 0.9071899554347194
Hs_std = 0.7478543699397211

# Function to solve for Rc
def solve_for_Rc(Hs):
    if Hs <= 0: # check to ensure Hs is positive
        return np.nan
    func = lambda Rc: q - (np.sqrt(g * (Hs ** 3)) * a * np.exp(-b * (Rc /  ( * Hs))) if -b * (Rc /  ( * Hs)) < 700 else float('inf'))
    try:
        sol = root_scalar(func, bracket=[0, Hs], method='brentq') # use root_scalar with the 'brentq' method
        return sol.root if sol.converged else np.nan
    except ValueError: # handle case when root_scalar fails to find a root
        return np.nan

# Monte Carlo simulation
np.random.seed(0) # for reproducibility
Hs_samples = np.random.normal(Hs_mean, Hs_std, n_simulations)
Rc_samples = np.vectorize(solve_for_Rc)(Hs_samples)

# Remove NaN values from Rc_samples
Rc_samples = Rc_samples[~np.isnan(Rc_samples)]

# Calculate exceedance probabilities for the filtered Rc_samples
prob_exceedance = 1 - norm.cdf(Rc_samples, np.mean(Rc_samples), np.std(Rc_samples))

# Sort Rc_samples in descending order
Rc_samples_sorted = np.sort(Rc_samples)[::-1]

# Determine gate height for the allowable failure probability
Rc_threshold = np.interp(P_f, prob_exceedance, Rc_samples_sorted)

print(f"Estimated gate height (Rc) for the allowable failure probability (P_f) of {P_f} is {Rc_threshold} m.")
```

**Listing E.9:** Monte Carlo analysis for determination of gate height

## E.5. Load Calculations for Mitre Gates

The Python script included in this section is utilized for computing the various forces acting on the mitre gates of a lock. It begins by defining necessary parameters such as the height and width of the lock gates, the inclination of the gates, and the high and low water levels. Following this, it calculates the angle of inclination of the lock gates with the walls and the width of each gate.

Subsequently, the script computes the wetted area of each gate, the force acting on each gate, and the resultant force on each gate. It then calculates the force acting on each hinge - both top and bottom - and finally, prints the resultant force on each gate as well as the forces acting on the top and bottom hinges.

The script provides a comprehensive analysis of the forces acting on the mitre gates, crucial for understanding the structural implications of these forces.

```
import math

# Constants
height_lock_gates = 6.04
width_lock_gates = 7.6
inclination_gates_deg = 120
```

```python
water_level_high = 4.6
water_level_low = 3.6
gravity = 9.81

# Calculations
alpha_deg = (180 - inclination_gates_deg) / 2
alpha_rad = math.radians(alpha_deg)

# width of each gate
width_gate = (width_lock_gates / 2) / math.cos(alpha_rad)

# Wetted area of each gate
A1 = water_level_high * width_gate
A2 = water_level_low * width_gate

# force acting on each gate
P1 = gravity * A1 * (water_level_high / 2)
P2 = gravity * A2 * (water_level_low / 2)

# Resultant force on each gate
P = P1 - P2

# Force acting on each hinge
RT = ((P1/2 * water_level_high/3) - (P2/2 * water_level_low/3)) / (math.sin(alpha_rad) *
    height_lock_gates)
RB = (P / (2 * math.sin(alpha_rad))) - RT

# Printing the results
print(f'Resultant force on each gate = {P:.2f} KN')
print(f'Force acting on top hinge = {RT:.2f} KN')
print(f'Force acting on bottom hinge = {RB:.2f} KN')
```

**Listing E.10:** Python Code for Load Calculations for Mitre Gates