Improving the reliability of the Maeslant barrier in the Delta21 configuration

MSc thesis

Vyása Sewberath-Misser

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By

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DISCLAIMER

This master thesis concerns a simplified study carried out by a student in which it is investigated how the reliability of a schematized simplified Maeslant barrier changes in the Delta21 configuration. The probabilities reported in this thesis may not be declared applicable to the real Maeslant barrier.

TU Delft

Rijkswaterstaat
Ministerie van Infrastructuur en Waterstaat

DELTA 21

Preface

This report contains my Master Thesis in Hydraulic Engineering which I will obtain at Delft University of Technology. I was searching for a graduation topic when I came across this topic. It allowed me to do research on two subjects, the Maeslant Barrier, and the effect of the Delta21 project on the reliability of the barrier. This project is a new and innovative plan to decrease flood risk in the Southwestern Delta in the Netherlands. This graduation topic also allowed me to utilize the knowledge I have gained during my master.

I want to thank my committee members for their critical feedback and for keeping me on the right track. I also want to thank Delta21 initiators Huub Lavooij and Leen Berke for helping me obtain the internship at Rijkswaterstaat and their feedback on my work. I also would like to thank Hans Nederend and Jan van Oorschot for their insights which helped me to understand the subject better and gave me ideas to work on. Lastly, I would like to thank my family, friends and girlfriend for supporting me during my master thesis.

Utrecht, March 2022
Vyása Sewberath-Misser

Summary

The coastal zone is becoming more vulnerable due to growing concentrations of human population, settlements, and socio-economic activities. The Rhine-Meuse Delta is low-lying land, which makes it vulnerable to flooding. The Maeslant barrier is a storm surge barrier which should prevent flooding in the low-lying land behind the barrier. Various climate models have predicted that sea level rise will occur. The predicted sea level rise is more than what was anticipated during the design stage of the Maeslant barrier. Due to the increasing water levels, the closing frequency of the barrier will increase. This has a negative effect on the reliability of the barrier, as the components are used more often. The Maeslant barrier also needs to discharge river water during a storm closure when the water level on the rear side of the barrier becomes too high due to river inflow. This discharging process is a complex process which contributes greatly to the failure probability of certain components.

Delta21 is a spatial plan to redevelop a part of the Dutch delta to mitigate the effects of climate change which are sea level rise and increased river discharges. Delta21 claims to improve the safety of the entire Rhine-Meuse delta till a sea level rise of 1.1 m. Delta21 pleads for a central approach to focus on improving pump capacity in the main water systems instead of raising and strengthening all the dikes. The main goal of Delta21 is to reduce the flood risk in the downstream area. Due to the large pump capacity available, Delta21 can replace the discharging function from the Maeslant barrier. This simplifies the closure operation of the Maeslant barrier to only retain seawater during storms on the North Sea. The effect of the simplified closure operation has consequences on the reliability of the Maeslant barrier.

Therefore, the main objective of this master thesis is to find out how the failure probability of closure of the Maeslant barrier with the simplified closure operation changes in the Delta21 configuration.

To fulfil the objective, the first step was to find out how the Maeslant barrier itself works and how the discharging procedure impacts the failure probability of closure. Then, a hydraulic system analysis was done to understand how Delta21 impacts the current configuration of the Rhine-Meuse delta. In the next step, the new situation with Delta21 was schematized into a simple hydraulic model which has been verified and validated.

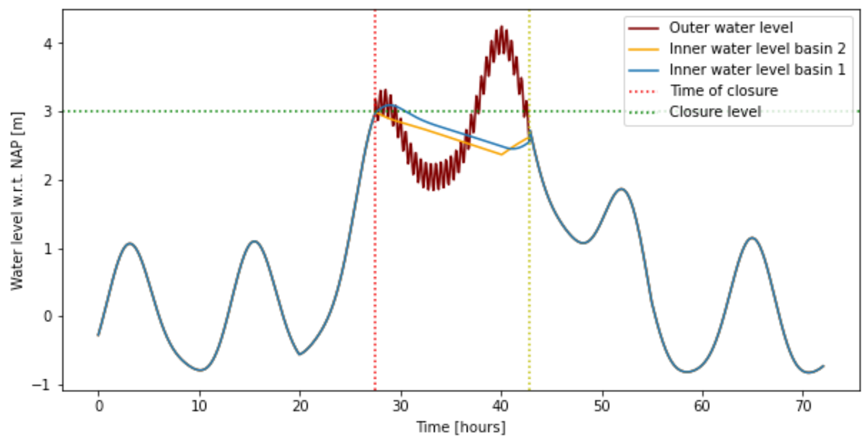

Figure 1: water levels during a simulation in which Delta21 pumps at full capacity

Figure 1 depicts the output of a run of the developed model. The blue line represents the water level behind the closed Maeslant barrier, and the red line represents the water level at sea. The red peaks are seiches. The orange line represents the water level in the Haringvliet area. The water level inside the basins decreases, because Delta21 pumps at full capacity (10,000 m^3/s) and the incoming discharge is less than the pump capacity.

By making the variables in this model probabilistic, a Monte Carlo simulation was done to find out what the probability of a negative head difference larger than 1.5 m was. The simulation was done for the situation without and with 0.6 m sea level rise.

The next step was the failure probability analysis. In the qualitative analysis of the failure probability analysis, different failure mechanisms and failure scenarios are identified for the Maeslant barrier in the current situation, and for the Maeslant barrier in the Delta21 configuration. In the quantitative analysis, the probabilities for the defined failure mechanisms and failure scenarios are calculated according to the upper bound approximation, for the current situation and with Delta21. The probability which was calculated with the Monte Carlo simulation is also used here. Ultimately, a fault tree can be composed on the basis of the qualitative and quantitative analysis for the Maeslant barrier in the current situation and the Delta21 configuration.

The failure probability of closure has been reduced by a maximum of approximately 10%. This is because many systems and operations were simplified because of the simplified closure operation.

Additionally, the failure probability analysis also provided insight in how the probability of flooding changed for Rotterdam with the Maeslant barrier in the Delta21 configuration. While the failure probability of closure was reduced by a maximum of 10% with Delta21, the probability of flooding for Rotterdam has increased by 53% when using the full pump capacity from Delta21. This increase is not significant since the probability of flooding remains very small. Due to the removal of the discharging function with Delta21, the probability of several flooding scenarios increased drastically which resulted in the increase in the probability of flooding for Rotterdam.

With a sea level rise of 0.6 m, the probability of flooding increased approximately 500% for the current situation (without Delta21). When comparing Delta21 with no sea level rise to Delta21 with 0.6 m sea level rise, the increase was also present with approximately 375%. This is less than 500%, which indicates that Delta21 is effective to mitigate the effects of sea level rise compared to the situation without Delta21.

Based on the conclusion, the most important recommendation is that more detailed modelling should be done where the dependencies between variables is modelled more according to reality, and that a model is being used which contains the actual bathymetry of the Rhine-Meuse delta. Furthermore, in order to determine if a closure is needed or not, it should be investigated how the location (Hoek van Holland or Rotterdam) at which the water level is measured affects the closure frequency in the situation with Delta21.

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1 Introduction

This chapter introduces the subject of this master thesis. The essence of the Maeslant barrier in the Rhine-Meuse Delta, the effect of sea level rise on the Maeslant barrier and the effect of Delta21 on the functioning of the Maeslant barrier will be elaborated. Due to Delta21, the closure operation of the Maeslant barrier can be simplified. What this means will be explained later in this chapter.

1.1 Motivation for the present research

1.1.1 Rhine-Meuse delta: vulnerable area

The Rhine-Meuse delta is located in the coastal zone in the Netherlands. The coastal zone is becoming more vulnerable due to growing concentrations of human population, settlements and socio-economic activities (Mooyaart & Jonkman, 2017). The delta is also low-lying land, which makes it vulnerable to flooding (Voorendt & Timmermans, 2020). Flooding can happen when the dikes or barriers fail, and the water level is higher than the land. The dikes (and barriers) need to be improved regularly to withstand sea level rise due to climate change. The most trivial measure to counter sea level rise would be to raise the existing dikes. However, in urban areas the space is limited, and the social impact could be considerable (Mooyaart & Jonkman, 2017). Therefore, it was decided that a storm surge barrier in combination with limited raising of dikes would serve as an alternative for continued strengthening of dikes (Rijkswaterstaat, 2012). The Maeslant barrier is designed as a storm surge barrier which has been realized between 1991 and 1997. It is part of the Europoort barrier (depicted in Figure 20), which consists of the Hartel barrier, the Maeslant barrier and the dike between the two barriers. The Europoort barrier is part of the Delta Works, which protects the land in the Southwestern (Rhine-Meuse-Scheldt) delta from the sea.

Figure 2: Location of the Maeslant barrier (Google Maps, 2021).

Figure 3: the Maeslant barrier in closed position (Het Keringhuis, 2021)

The Maeslantkering is classified as a storm surge barrier. According to Mooyaart & Jonkman (2017), a storm surge barrier is defined as follows: “A storm surge barrier is a fully or partly moveable barrier which can be closed temporarily to limit water levels in the basin behind the barrier and so prevent flooding of the area surrounding the inner basin”. According to Rijkswaterstaat (2012), the main purpose of the barrier is to lower the water level in the areas behind it. This is in accordance with the definition which was provided by Mooyaart & Jonkman (2017). Besides the main function, the Maeslant barrier can also discharge river water in closed position during certain conditions. These conditions will be explained in Section 1.2. The Maeslant barrier with its main components is depicted in Figure 4.

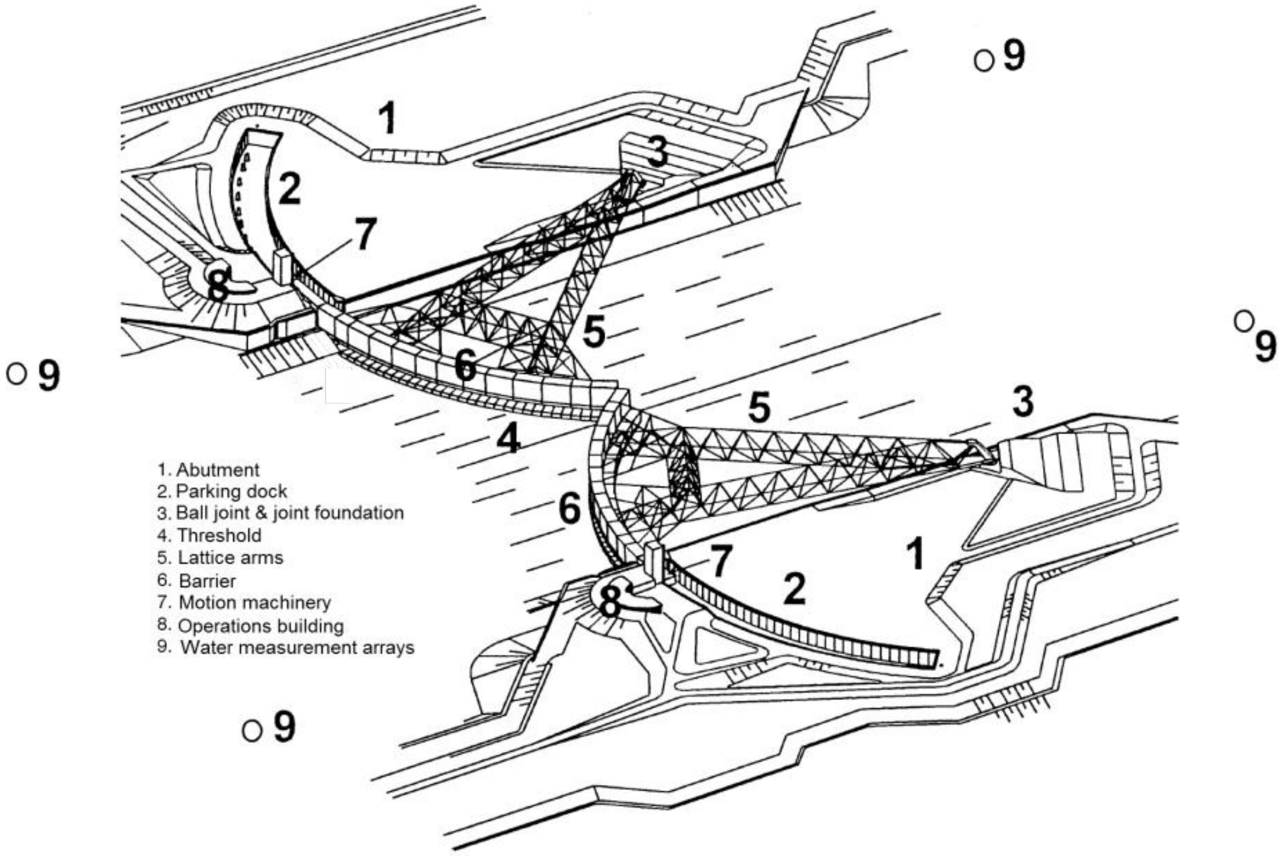


Figure 4: the Maeslant barrier and its components (Rijkswaterstaat, 2012)

1.1.2 Motive for improvements

In the design stage of the barrier, a sea level rise of 0.25 m every 50 years (so 0.50 m in 100 years) was taken into account (Rijkswaterstaat, 2012). However, different climate scenarios (which have not been validated) indicate that climate change could accelerate sea level rise a lot more than expected. Deltares (2018) provided an overview of how the different climate scenarios predict sea level rise. This can be seen in Figure 5. In this figure, the difference in uncertainty between the scenarios is notable. The Deltascenario's are based on the KNMI'14 scenarios. This model predicts a sea level rise of 0.4 m in 2050 with a maximum of 1.0 m in 2100 (with reference year 1995). Other models, RCP4.5 and RCP8.5 are models which have been developed by The Intergovernmental Panel on Climate Change (IPCC). RCP4.5 assumes a reasonably successful implementation of the Paris agreement and a worldwide temperature rise of 2^{o}C in 2100. RCP8.5 assumes that emissions continue to rise throughout this century and a temperature rise of 4^{o}C in 2100 (Deltares, 2018). When comparing the current predictions to the 0.50 m in 100 years from the design stage, the rise could occur sooner than anticipated in the design stage.

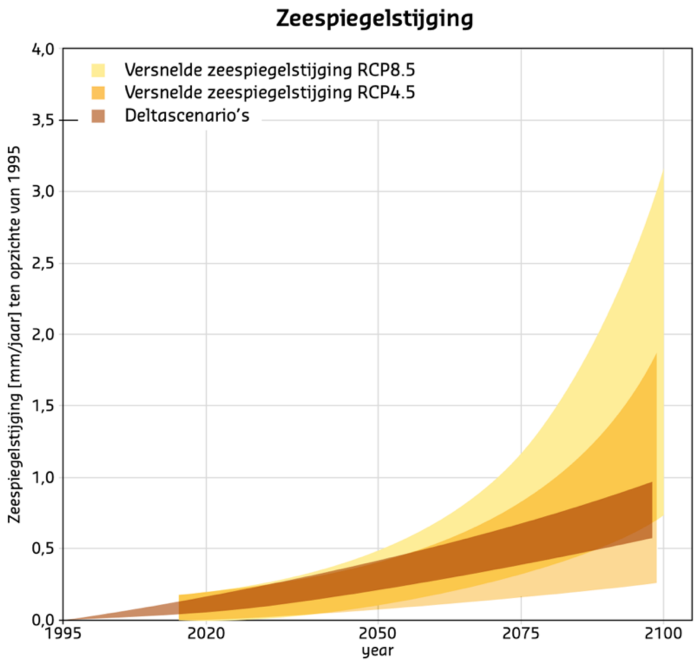


Figure 5: sea level rise predictions according to the Deltascenario's, RCP4.5 and RCP8.5 (Deltares, 2018).

As a result of the sea level rise, the closing frequency of the barrier increases. This has been demonstrated by Deltares (2018) in Figure 6. Each component in the barrier has a certain probability of failure. If the barrier is used more often, the components of the barrier are used more often, which results in a higher chance that one of the components can fail. Therefore, the failure probability of the Maeslant barrier also increases. Not only does the increase in closing frequency have impact on the reliability of the barrier, but it also has impact on the area behind (the hinterland) the barrier. Since the barrier is open most of the time, the water level in the hinterland will also rise together with the sea level because of the backwater effect. This has consequences for the river dikes as the hydraulic load increases on those dikes. Figure 6 gives an overview of the closing frequency of the Maeslant barrier as a function of sea level rise. With higher sea level rise, the closing frequency increases. Climate change does not only increase sea level rise, but it also increases the river discharge. This is relevant, because when the Maeslant barrier operates in closed position the water accumulates behind the barrier.

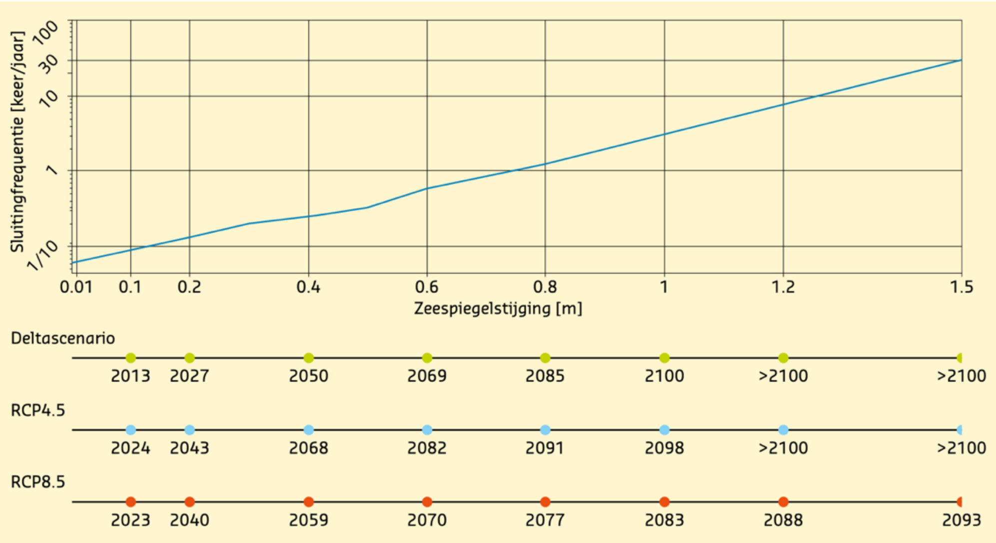


Figure 6: closing frequency of the Maeslant barrier on the y-axis as a function of sea level rise on the x-axis (Deltares, 2018).

Oerlemans (2020) simulated 1.0 m sea level rise in the Rhine-Meuse Delta in combination with a correct functioning Europoort barrier. This means that the Hartel- and Maeslant barrier close when they are supposed to close, and open when they are supposed to open. The failure probabilities of closure for the Maeslant- and Hartel barrier are equal to their failure probabilities of closure in the current situation, which is 10^{-2}. The result is depicted in Figure 7. The water levels would significantly increase around Rotterdam and Dordrecht.

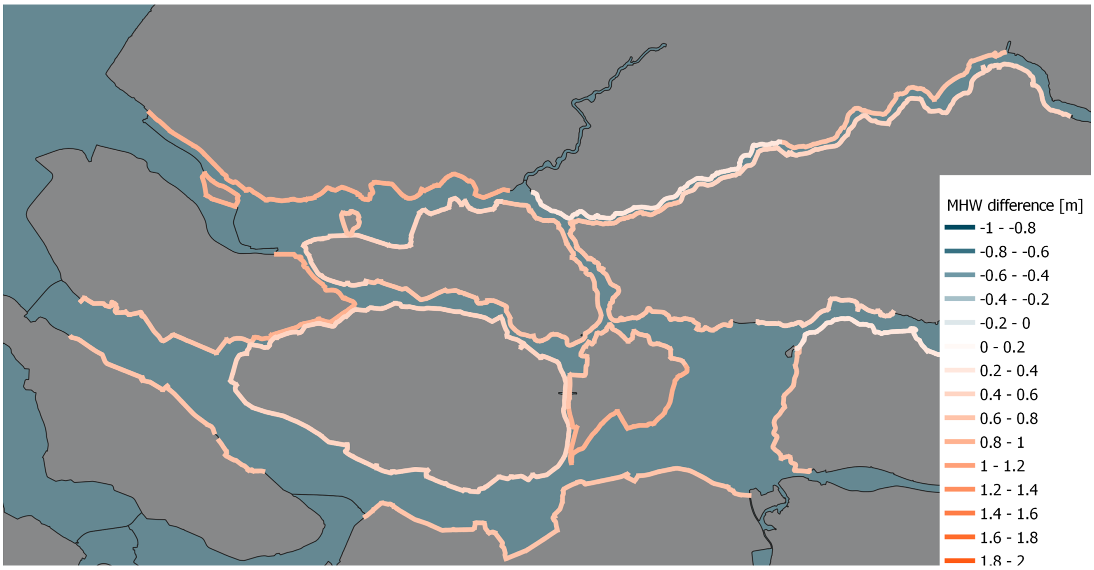


Figure 7: water level difference of 1,0 m sea level rise compared to MHW in the current situation and a correct functioning Maeslant- and Hartel barrier with failure probability of 10^{-2} (Oerlemans, 2020).

1.1.3 Delta21

Delta21 is a spatial plan to redevelop a part of the Dutch delta to mitigate the effects of climate change which are sea level rise and increased river discharges. Delta21 claims to improve the safety of the entire Rhine-Meuse delta until a sea level rise of 1.1 m. Delta21 pleads for a central approach to focus on improving pump capacity in the main water systems instead of raising and strengthening all the dikes. The main goal of Delta21 is to reduce the flood risk in the downstream area whereby the maximum target water level at Dordrecht is NAP + 2.5 m (Berke, Lavooij, Tonneijck, de Boer, & Vrijling, 2018).

Delta21 consists of an Energy Storage Lake, a Tidal Lake, and a new closable barrier. The Haringvliet sluices which are currently present will be in fully open position. The water retaining function will be replaced by the new closable barrier. The Energy Storage Lake has a large pump capacity of 10,000 m^3/s to ensure a water level lower than NAP + 2.5 m at Dordrecht, but not higher than NAP + 3.0 m once in 10,000 years (Lavooij & Berke, 2019). The Energy Storage Lake can also produce green energy, in which the world is in desperate need of to reduce carbon emissions. An overview of the design of Delta21 is given in Figure 9.

During normal conditions, river water can be discharged via the tidal lake through the new closable barrier. During high river discharge, the river water cannot only be discharged through the tidal lake and the new barrier, but also through a siphon between the Tidal Lake and the Energy Storage Lake. The river water can then be discharged from the Energy Storage Lake into the sea using pumps. In storm conditions, when the barrier is closed, the river water can be discharged trough the Energy Storage Lake using pumps. The operation of Delta21 in the described situations is depicted in Figure 8, along with Table 1 which includes the legend for Figure 8.

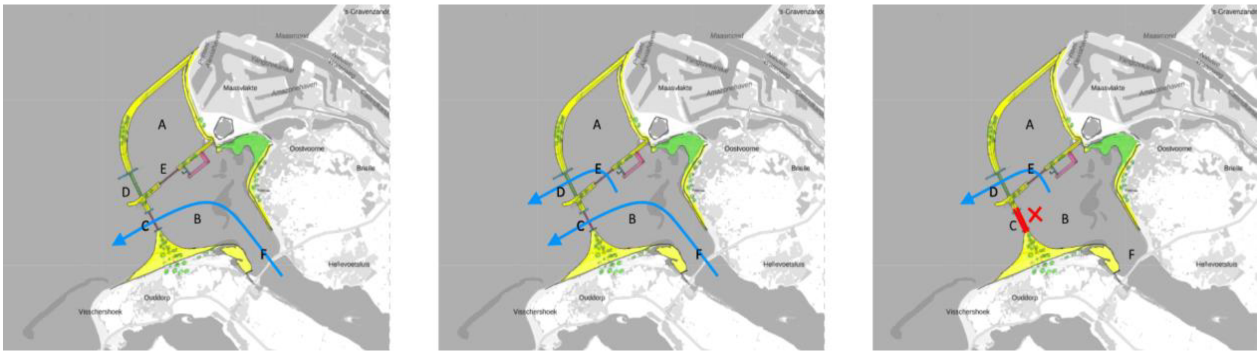


Figure 8: Delta21 operation in normal conditions (left), high discharge (middle) and extreme conditions at North Sea (right) (IJntema, 2021)

Legend
A = Energy storage lake
B = Tidal Lake
C = New closable barrier
D = Pumps
E = Siphons
F = Haringvliet barrier

Table 1: legend for Figure 8

Due to the large pump capacity available, Delta21 can replace the water discharging function from the Maeslantkering (explained in Section 1.2.3 and 1.2.4). This simplifies the closure operation of the Maeslantkering to only retain seawater during storms on the North Sea. The effect of the simplified closure operation has consequences on the reliability of the Maeslantkering (Lavooij & Berke, 2019). The magnitude of these effects will be investigated in this master thesis.

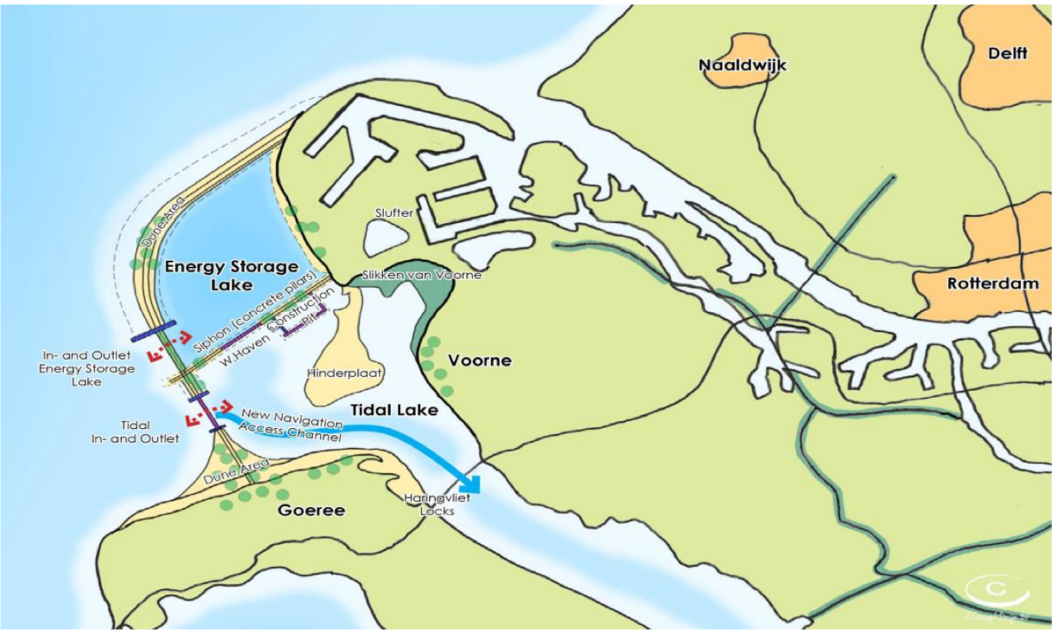


Figure 9: overview of Delta21 (Lavooij & Berke, 2018)

1.2 Problem analysis

1.2.1 Control systems of the Maeslant barrier

During a storm on the North Sea, the Maeslant barrier must presently retain seawater and it must be able to discharge river water under certain conditions. These conditions will be described later in this section. In the design stage of the MLK, the failure probability “to close or halt water” was designed to be “less than once per every 1,000 requests to close” (Rijkswaterstaat, 2012). After an analysis, Rijkswaterstaat (2012) concluded that the failure probability was higher than 10^{-3} per closure in reality. After further analysis, the failure probability for closing was estimated to be only to 10^{-2} per closure (Rijkswaterstaat, 2012). This is much higher than the design failure probability of closure, but this has been accepted as it is the most feasible failure probability of closure which can be achieved for the barrier in its current state. To find new ways to reduce the current failure probability of closure (10^{-2}) is always appreciated.

To understand the causes of the higher failure probability better, first several definitions will be provided. The barrier is operated using control systems. According to Rijkswaterstaat (2012), the control systems named BOS, BESW and BESH are defined as follows:

- **BESW:** “Besturingssysteem Nieuwe Waterweg” (Control System New Waterway): This is a system that uses a network to start up and shut down devices to complete (partial) activities. Measurement data which influences the operation of these devices are requested and interpreted by BESW.
- **BOS:** “Beslis- en Ondersteunend Systeem” (Decision and Support System): This system sends out commands which puts processes in motion. These processes are then worked out in detail by BESW. BOS receives external information about weather predictions, water levels on the Nieuwe Waterweg and other locations in the Netherlands trough a measurement network.

- **BESH:** “Besturingssysteem Stormvloedkering Hartelkanaal” (Control System Hartel Barrier). This system also receives commands from BOS.
- **BESS:** “Besturingssyteem Hartelsluis” (Control System Hartel Sluice). This system also receives commands from BOS.

An overview of the described systems is given in Figure 10. It indicates the relations between BOS, BESH, BESS, BESW and the external systems. BOS receives external information and Figure 11 provides a detailed overview which external information is provided by which organization (depicted in yellow) to BOS. Note that this figure does not specify how the information is provided to BOS and the organizations (depicted in yellow), but only which information.

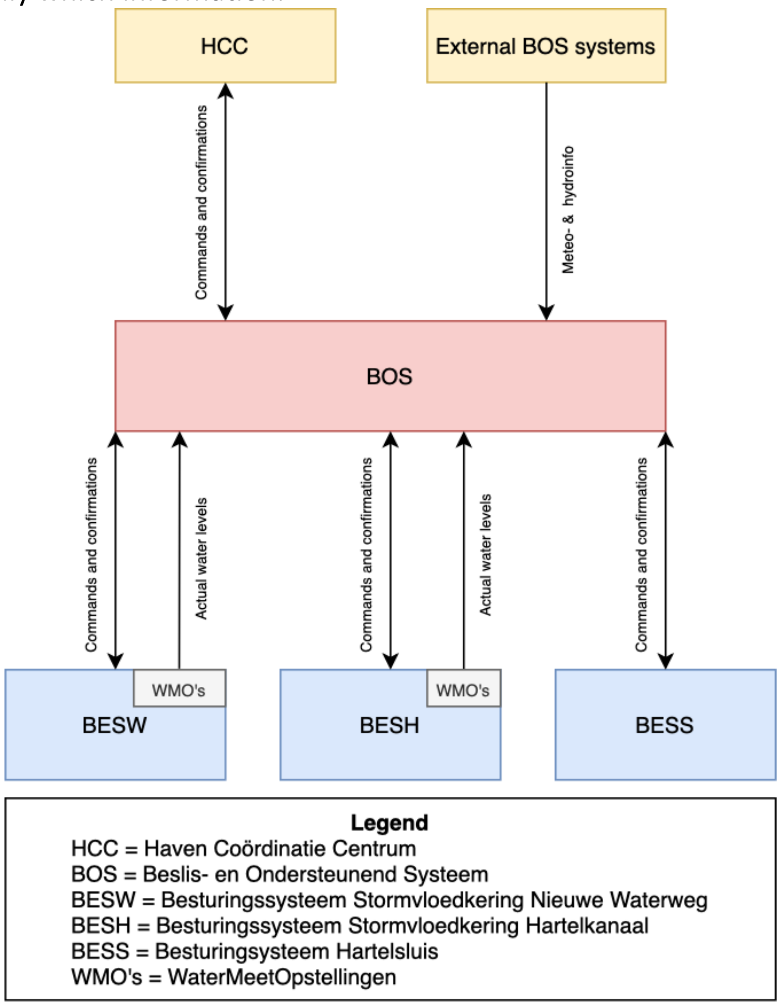


Figure 10: Overview of BOS, BESW, BESH, BESS. Simplified and adapted from Rijkswaterstaat (2005).

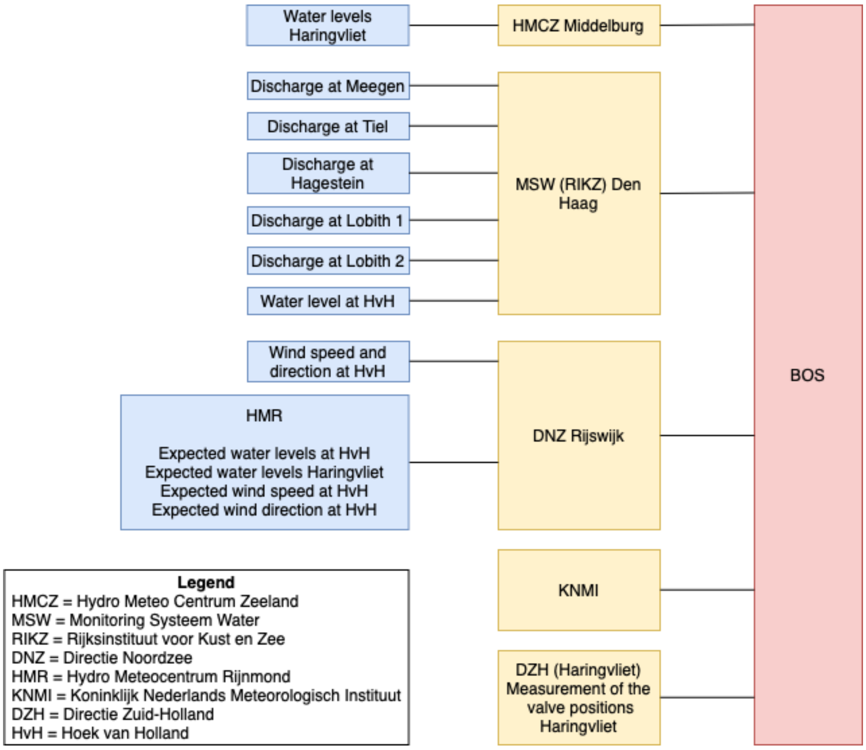


Figure 11: External BOS systems. Simplified and adapted from Rijkswaterstaat (2005).

The increase in failure probability was mostly caused by (Rijkswaterstaat, 2012):

- Insufficient reliability of the software systems, namely BESW and to a lesser degree BOS.
- Knowledge of the system and keeping it in the organization.
- Quality and size of the organization charged with management.
- Not enough equipment to control the barrier with the focus on the failure probability.
- Adjustments in the area of among other things, fire, lightning strikes and related failure.

Before resolving these issues, the failure probability of closure was higher than 10^{-2} per closure. After resolving these issues, this resulted in the failure probability of closure of 10^{-2} per closure. However, there are currently still issues to maintain the required closure reliability level of 10^{-2} per closure. Next, the closing procedure will be described.

1.2.2 Closing procedure

BOS operates fully automatic, but it can be overruled by the Operational Team if this is required. BOS calculates the expected water level at Dordrecht and Rotterdam with the following input (Rijkswaterstaat, 2012):

- Actual recorded water levels
- The river discharge
- Wind data
- The expected water level at Hoek van Holland

If the calculated expected level is higher than the predetermined closure level (which is NAP + 3.0 m at Rotterdam and NAP + 2.9 m at Dordrecht), BOS sends out the command

to BESW and BESH for the closing procedure. This means that the Maeslant barrier and the Hartel barrier will be closed. Now it is known **when** the barrier closes. The closing procedure describes **how** the barrier closes.
The normal closing procedure can be summarized as follows (Rijkswaterstaat, 2012):

- 20 hours before closure: the Operational Team gets called up to the control center if the predetermined water levels at Dordrecht and/or Rotterdam will be exceeded.
- 8 hours before closure: HCC (Port Control Center) announces that shipping traffic will be suspended.
- 4 hours before closure: the water level inside the parking docks gets leveled with the water level in the river. When this process is completed, the dock doors can be opened. Traffic on the river is suspended immediately thereafter.
- 2 hours before closure: shipping traffic is suspended in the New Waterway and Hartel Canal.
- 0 hours, at closure: the sector gates travel onto the river.
- 0.5 hours after closure: the sector gates are in closed position and can be into immersed into position.
- 1.5 hours after closure: the immersion process of the sector gates is complete, the barrier is closed.

The closing procedure is visualized in Figure 12.

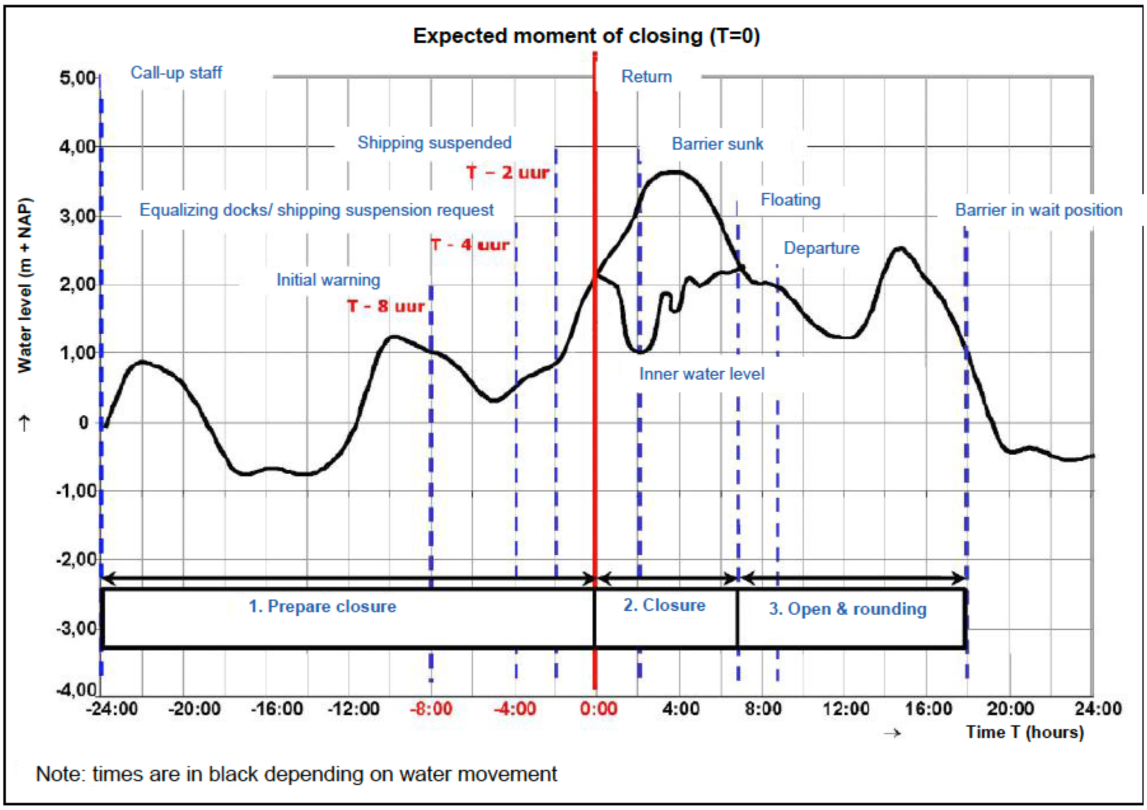


Figure 12: water levels during a storm event and the closing procedure (Rijkswaterstaat, 2012)

The closing procedure is a complex process. Apart from this complex closing procedure, the barrier can also discharge river water under certain conditions. According to Lavooij

& Berke (2019), the discharging function increases the failure probability of the barrier. The reasoning behind this assertion will be explained in the next section.

1.2.3 Discharging procedure

Three main questions will be answered in this section:

- Why is it necessary to discharge river water?
- How does the discharging process work?
- What is the role of the Haringvliet sluices in the discharging process?

Why is it necessarry to discharge river water?

It is necessarry to discharge river water for two reasons:

- To prevent flooding in the land behind the barrier due to water accumulation during long storms.
- The ball joint can withstand a limited amount of negative head difference.

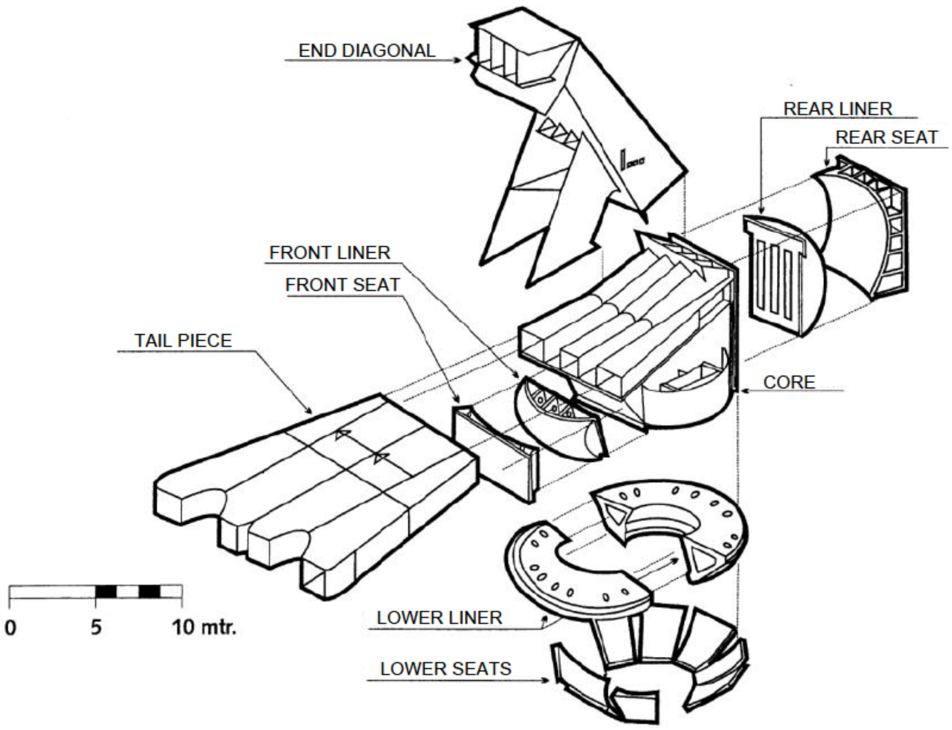


Figure 13: the ball joint and its components (Rijkswaterstaat, 2012).

The pivot point of the sector gates is the ball joint. The ball joint transfers the forces acting on the sector door to the foundation. The ball joint can only withstand a limited negative head difference, which is defined as: the water level on the river side is higher than the sea side. If the negative head difference becomes too large, the ball joints can be pushed out of their sockets (de Jong M. P., 2004). The cause for this is that the front seat is lower than the back seat. Due to this design, the rear seat can absorb more pressure than the front seat, as it provides more support. This can be seen in Figure 13. This is why it is necessarry to discharge river water. The ball joint can structurally absorb a negative head difference until a head difference of 1.5 m (Rijkswaterstaat, 2012). A negative head difference can be caused by three factors:

- A high river discharge: due to the closed barrier, the water level gradually rises behind the barrier. If there is a higher river discharge, the water rises faster than under normal conditons and the water level at Dordrecht can become too high (Rijkswaterstaat, 2012). In combination with low tide, this can cause a negative head difference as the water level on the inner side can become higher than the outer side.
- A seiche: a seiche is a standing wave in an enclosed or partially enclosed body of water. The water fluctuates with periods between 5 and 90 minutes on the North Sea. The fluctuations are higher in the port basin. The occurence seiches can be predicted, but to predict the magnitude of a seiche is still difficult (Rijkswaterstaat, 2012). The size of a seiche at the location of the barrier is a function of the layout of the harbour basin (Rijkswaterstaat, 2012). Seiches cause the water level on the seaside of the barrier to suddenly rise or drop.
- A two-peak storm: this is a storm which consists of two peaks. Between the two peaks, the water level on the sea side temporarily drops until the second peak of the storm arrives. In this period, in combination with LW, the water level on the river side can be higher than the sea side. This type of storm is depicted in Figure 14, along with the inner and outer water levels during the storm.

The effect of seiches on the negative head difference was larger than initially assumed and was discovered in a late stage of the design. Seiches will be elaborated extensively in Section 2.3. To mitigate the negative head difference caused by seiches on the ball joint, the differential head needs to be measured continuously and the ballast needs to be adjusted accordingly. Using this method, it is possible to lift the sector gates almost immediately when the water levels are equal. The differential head can then be leveled out. This method is called dynamic pre-stress management (Rijkswaterstaat, 2012).

Lavooij & Berke (2019) mentioned that the discharging function increases the failure probability of the barrier. To understand this assertion, it needs to be known how this discharging process works.

How does the discharging process work?
As mentioned in Section 1.2.2, the Maeslant barrier can also discharge river water when a negative head difference occurs as explained above. Before the barrier can discharge river water, the barrier needs to be brought into floating position. This happens if one of the following conditions is met:

- If the barrier is in closed position (sector gates closed and fully immersed) and a head difference < 0 cm is observed, BESW gives the command to start the floating process. When the floating procedure is complete, river water can be discharged.
- If the during the immersion process a head difference of < - 5 cm is observed, BESW sends out a command to stop the sinking process and start the floating process. Once the floating process is complete, river water can be discharged.

The discharging procedure stops in both cases if the head difference is ≥ 5 cm and the water levels at Rotterdam and Dordrecht exceed NAP + 3.0 m and NAP + 2.9 m respectively. An overview of when the discharging procedure starts and what happens to the water levels around the barrier during a storm event is provided in Figure 14. This figure is a result of a storm surge height of 2.5 m and a river discharge of 6,000 m^3/s (which is high compared to the average discharge of 2,200 m^3/s).

Since the ballast system and BESW are used frequently in the discharging process, the failure probability of closure increases. Rijneveld (2008) has shown that the ballast system and BESW have a large influence on the failure probability of closure. This validates the assertion made by Lavooij & Berke (2019) in Section 1.2.2.

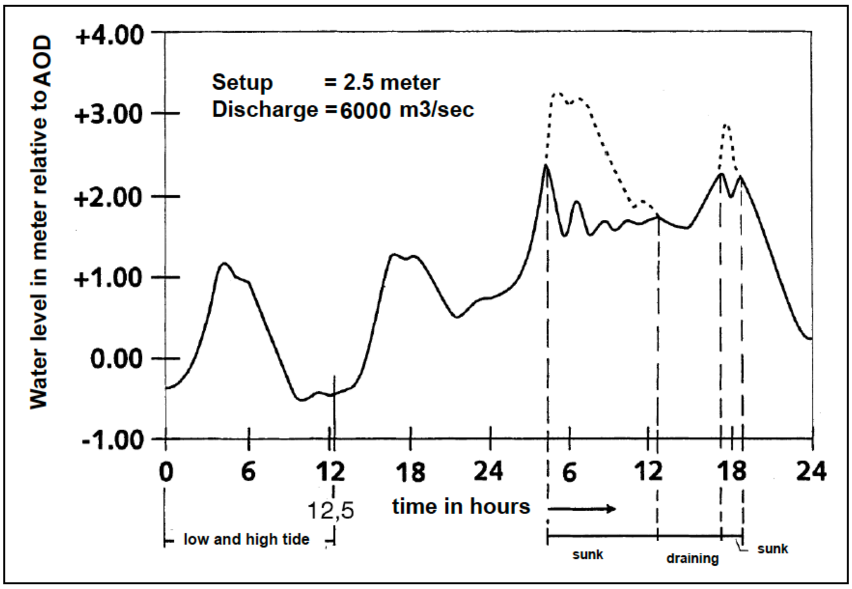


Figure 14: water levels during a storm event with discharging water during the storm (Rijkswaterstaat, 2012). The dotted lines indicate the outer water level, and the solid line the inner water level (in the period of closure).

Why can the Haringvliet sluices not discharge the river water instead of the Maeslant Barrier?

The Haringvliet sluices discharge $30 \cdot 10^9$ m^3 water per year (Rijkswaterstaat, 2021). This is equal to approximately 951.3 m^3/s. The sluices can only discharge water if the water level on the river side is higher than the water level on the seaside since no pumps are present. This means that in storm conditions, the sluices can only discharge water when it is Low Water (LW) and the water elevation on the seaside is lower than on the river side. This also means that during long storms, the river water is trapped in the area behind the sluices. The short time window in combination with the limited discharge capacity under which the sluices can discharge during storms (during LW) is not enough to relieve the entire area behind the sluices including Rotterdam and Dordrecht. Due to the large storage area, it would also take a significant amount time before the water level at the location of the Maeslant barrier, Rotterdam and Dordrecht are lowered.

1.2.4 Effect of Delta21 on the functioning/operation of the Maeslant Barrier

Section 1.2.3 indicated that the discharging and floating procedure is a complex process which depends on many measurements and control systems. In the new situation with the Maeslant barrier in the Delta21 configuration, the closing procedure could be simplified. The discharging process in its current form can be omitted as a whole, because Delta21 guarantees a large pump capacity to control the water levels at Dordrecht and Rotterdam. In the Delta21 configuration, BOS would only need the water

level at Hoek van Holland as input to decide when to close. Figure 11 can be simplified considerably in the Delta21 configuration. This is depicted in Figure 15.
The effect of this simplification on the failure probability of BOS/BES system will also be addressed in this Master Thesis when the failure probability analysis will be performed. It needs to be noted that investigation is needed whether a new (simplified) control system which controls Delta21, the Maeslant- and Hartel barrier or a simplified BOS system which controls Delta21, the Maeslant- and Hartel barrier is more effective. The system which has the lowest probability of failure must be chosen.

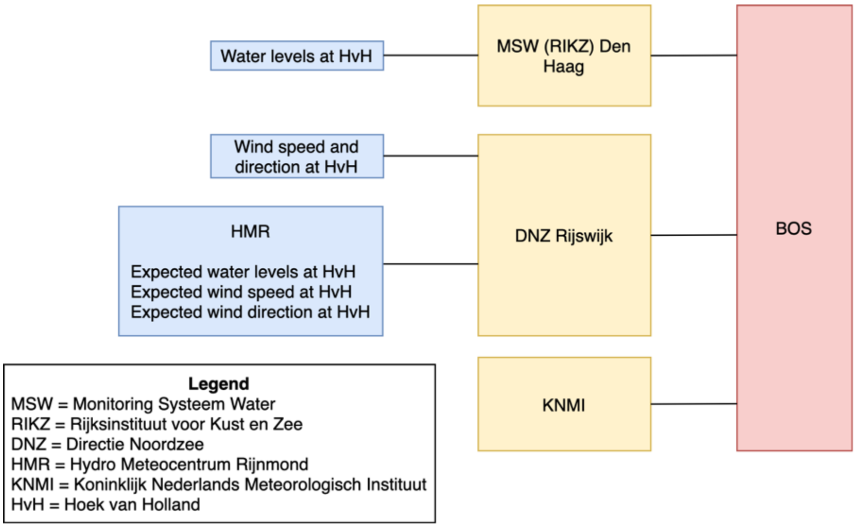


Figure 15: External information for BOS in the Delta21 configuration. Adapted and modified from Rijkswaterstaat (2005)

Since Delta21 guarantees the large pump capacity to control the water levels behind the barrier, a negative differential head can only occur due to seiches. Seiches are predictable, but their magnitude is not (as explained in Section 1.2.3). This means that even with a guaranteed low water level on the river side (due to Delta21), a negative differential head can still occur due to a seiche. To mitigate this, a solution would be to keep the water level as low as possible on the river side, using the pump capacity from Delta21. In summary, the addition of Delta21 to the system has two effects for the Maeslant barrier:

1. Delta21 adds a large pump capacity which could lower the water levels behind the closed Maeslant barrier.
2. Because of this large pump capacity, the existing discharging function of the Maeslant barrier can be omitted.

1.2.5 Problem definition

To conclude this chapter, the main problem can be formulated as follows: the failure probability of closure of the Maeslant barrier is being influenced negatively by the discharging/sluicing function. Furthermore, a consequence of sea level rise is that the barrier will need to close more often. This means that the closing frequency will increase, and the failure probability of closure will increase as well. With Delta21, the discharging function of Maeslant barrier can be omitted and the effect of this on the failure probability of closure needs to be investigated. Since the Maeslant barrier will be more vulnerable to seiches in the new situation, seiches need to be investigated in detail (which is done in Section 2.3).

1.3 Objective

Regarding the problem statement, the main research question of this master thesis is the following:

"How does the failure probability of closure of the Maeslant Barrier with the simplified closure operation change in the Delta21 configuration?"

Several questions arise from the main research question:

1. How does the hydraulic system with the Maeslant barrier work with and without Delta21?
2. How do the water levels change at the location of the Maeslant barrier with Delta21?
3. How does the reliability of the Maeslant barrier change with Delta21?
4. Does the possible change in reliability of the Maeslant barrier (with Delta21) also affect the probability of flooding for Rotterdam? How does this change with 0.6m of sea level rise?

1.4 Methodology

To be able to solve the problem, a structured methodology is needed. This section states which steps will be taken to come to a successful result. This will be done in four main steps to obtain an answer to the main objective which was defined in Section 1.3:

1. **Perform a hydraulic system analysis**
   A hydraulic system analysis will be done to describe how the water system with the Maeslant barrier works with and without Delta21. Delta21 claims to reduce the failure probability of closure of the Maeslant barrier without (structurally) modifying the barrier itself. Due to the addition of Delta21, the closure operation of the Maeslant barrier can be simplified. Delta21 claims that the effect of the simplified closure operation mentioned in research question 1 is flood risk reduction (= a lower failure probability of closure). Before calculating the new failure probability of closure, it needs to be known how the system currently works with the Maeslant barrier. By doing this, the “new” system including Delta21 can be compared to how the current system works. Information which is relevant for the failure probability of closure Maeslant barrier is how the water levels and discharge vary close to the barrier during normal conditions and during storms. Seiches also influence the water levels, and their effect could be critical in the new situation with Delta21. Therefore, seiches will be covered in depth in the system analysis. Seiches influence one of the failure mechanisms of the ball joint (the negative head difference larger than 1.5 m). To find the probability of this failure mechanism, a simple hydraulic model is needed. The system analysis is part of methodological step 1. The development of the simple hydraulic model is part of methodological step 2. The failure probability analysis will give insight in what the order the magnitude of the claimed flood risk reduction is.

2. **Develop a simple hydraulic model to address how the hydraulic effects (tide, seiches, river discharge) influence the simplified Maeslant barrier**
   A simple hydraulic model will be developed to find out what the influence of Delta21 on the water levels at the location of the Maeslant barrier is. Also, the probability of a negative head difference larger than 1.5 m can be calculated. To estimate the necessary probability which is needed to complete the failure probability analysis (methodological step 3), the components of which model consists of need to become probabilistic. The development of the model will be done according to the following steps:
   1. **Schematization of the real world into model elements**
      The Rhine-Meuse delta consists of many channels and storage basins, but for the modeling, many assumptions must be made as not all real-world phenomena can be schematized in this model.
   2. **Use the equations to represent the model elements**
      In this step, it is described which equations are being used to represent the elements and hydraulic conditions in the model.
   3. **Program the model based on the model schematization and equations**
      In this step, the model has been programmed in Python according to the schematization defined in step 1, and by using the equations in step 2.

4. **Verification of the programmed model**
   This will be done by checking if the equations in the programmed model are dimensionally correct and equivalent to the analytical equations. If this is true, they should produce the same results.
5. **Validation of the programmed model**
   The outcome of the model needs to be validated to check if it produces realistic output results (given the assumptions made in step 1).
   If the model works properly, the model can be adapted to make the variables probabilistic.
6. **Adapt the model to become fully probabilistic (level III) & run the Monte Carlo Simulation:** to obtain a probability based on a Monte Carlo approach, certain variables need to become stochastic variables. The incoming and outgoing discharge, the storm surge, the storage area and seiche amplitude will be stochastic variables. With the fully adapted level III model, the Monte Carlo simulation can be done to achieve the required probability.

3. **Analysis of the model results**
   The model will be run in a deterministic mode to highlight the sensitivity of the results to the different parameters. Then, the model will be run in the probabilistic mode and the Monte Carlo simulation will be done. The results will also be analysed.

4. **Perform a failure probability analysis to investigate how Delta21 affects the failure probability of closure**
   To verify the order of magnitude of the claimed flood risk reduction, a new risk analysis is needed. However, the risk analysis in this thesis is actually a failure probability analysis, as the cost of damages/failures is not being evaluated. The failure probability analysis will be done on a high level of abstraction, on system level. The approach that will be used to perform this failure probability analysis is based on the procedure described in Figure 16. Note that the last step in Figure 16 will not be done as the cost of the damages/consequences will not be analysed. This can be done using the following steps (Jonkman, Steenbergen, Morales-Nápoles, Vrouwenvelder, & Vrijling, 2017):
   1. **System definition**
      The goal of this step is to provide a definition and description of the system, the scope and the objectives of the analysis.
   2. **Qualitative analysis**
      In this step, the undesired events, failure mechanisms and scenarios are identified and described. The goal of this step is to have an overview of all possible undesired events and their consequences.
   3. **Quantitative analysis**
      In this step, the probabilities and consequences of the undesired events defined in the previous step are determined.

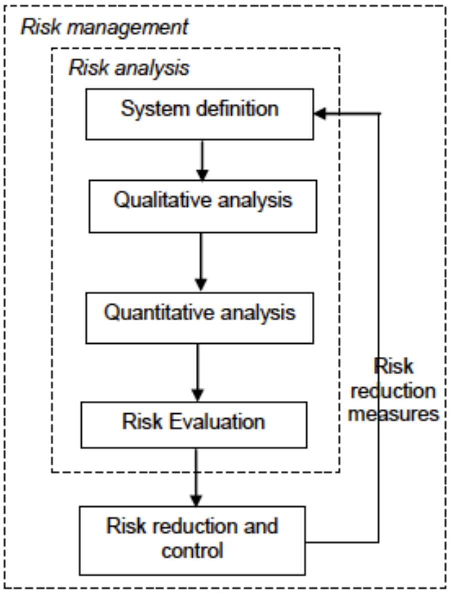


Figure 16: schematic view of steps in risk management (Jonkman, Steenbergen, Morales-Nápoles, Vrouwenvelder, & Vrijling, 2017)

1.5 Reading guide

The report is structured as follows:

- **Chapter 2** adresses **methodological step 1** (the hydraulic system analysis).
- **Chapter 3** adresses **methodological step 2** (the development of the simple hydraulic model).
- **Chapter 4** adresses **methodological step 3** (the analysis of the model results).
- **Chapter 5** adresses **methodological step 4** (the failure probability analysis to investigate how Delta21 effects the failure probability of closure).
- **Chapter 6** is the **discussion** in which the validity of the results is being discussed.
- **Chapter 7** gives the **conclusions and recommendations**. The answer to the main research objective and additional objectives mentioned in section 1.3 is given and based on this, some recommendations are given.

2 Hydraulic system analysis

This chapter explains how much the water rises at the back side of the barrier when the Maeslant barrier is closed with and without Delta21. It also addresses the discharge distribution in the current situation and in the Delta21 configuration. Seiches will be covered in a separate section in this chapter. This chapter is addresses methodological step 1 as defined in Section 1.4. This chapter also answers sub question 1 as defined in Section 1.3.

2.1 Current configuration

This section describes the current discharge distribution between the rivers in the Rhine-Meuse delta and what happens to the water levels behind the closed Maeslant (and Haringvliet) barrier. The reason why this needs to be known was provided in Section 1.4, but will be repeated here. It needs to be known how the current system works before the new system can be compared to the current system.

Discharge distribution

As mentioned earlier in the previous chapter, the water level at the inland side is gradually rising when the barrier is closed. The rate with which the water level rises depends on several parameters: the discharge, storage capacity of the area behind the barrier and the time the barrier is closed. Apart from knowing how long and where the water needs to be stored, the key parameter is the discharge.

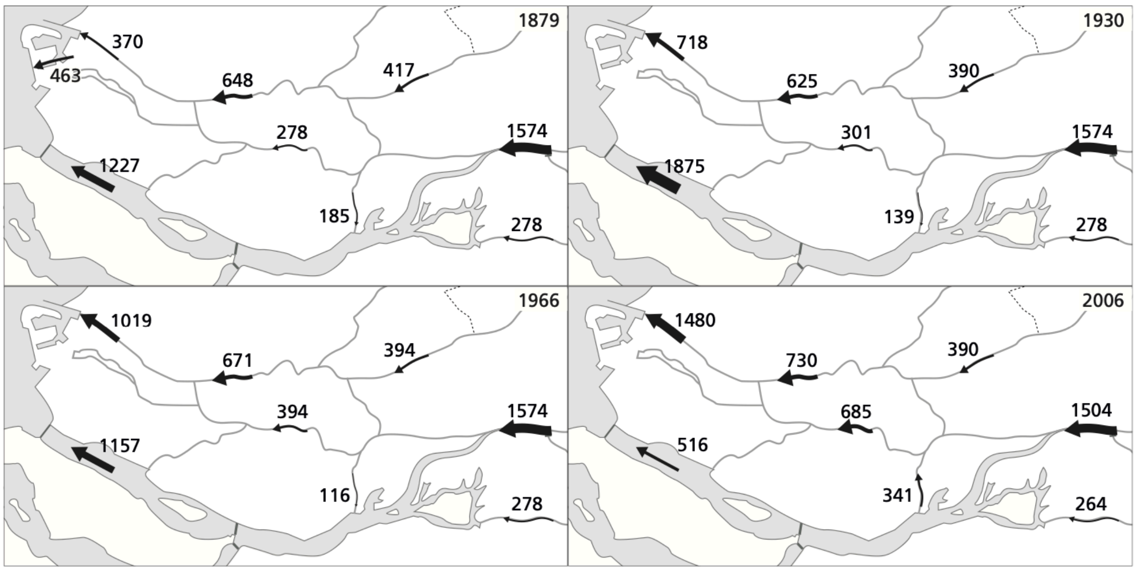


Figure 17: discharge distrubutions in various years (Huismans & Hoitink, 2017). Year indicated in the top left of each figure.

Huismans & Hoitink (2017) provided an overview of how the discharge distribution changed over time. This overview is provided in Figure 17. It can be observed that there are quite some differences occur between the years. The reason for these differences is human interference. The Dutch government decided that catastrophic floods as in 1953 should not happen anymore. This was the reason to establish the Delta Commission to develop plans to prevent future disasters like this from happening. The response was the construction of the Deltaworks. The construction of the Deltaworks influenced the discharge distributions in the Rhine-Meuse Delta (Huismans & Hoitink, 2017).

From Figure 17, it can be concluded that the New Waterway discharges most of the water. This is also quite logical since the Haringvliet sluices have a limited discharge capacity, while the Maeslant barrier is open most of the time which allows water to discharge freely. Considering sea level rise, my expectation is that less water will be able to flow through the Haringvliet sluices, because the outer water level slowly rises over time. Since no pumps are present there, the sluicing capacity will decrease, and more water will flow out through the New Waterway.

Water levels

This section addresses what happens to the water levels in the Rhine-Meuse Delta when the barriers (Haringvliet sluices, Maeslant- and Hartel barrier) are open and when they are closed. The increasing water level behind the barrier does not only have influence on the barrier itself, but also on the dikes in the hinterland. The hydraulic load these dikes depend on the water levels at the location of the dikes.
In the Rhine-Meuse Delta, the water levels within the delta are influenced by different processes. The Rhine-Meuse Delta can be divided in four main areas (Oerlemans, 2020):

- **Storm surge dominant area:**
  The water level in this area is mostly influenced by the storm surge coming from the sea.
- **Storage dominant area:**
  The water level in this area is mostly influenced by the storage capacity of the area.
- **Transition area:**
  This water level in this area is influenced partly by the storm surge and partly by the river discharge.
- **Discharge dominant area:**
  The water level in this area is mostly influenced by the river discharge.

The division which Oerlemans (2020) provided is also confirmed by Rijkswaterstaat (2014), as they divided the area in the same manner.

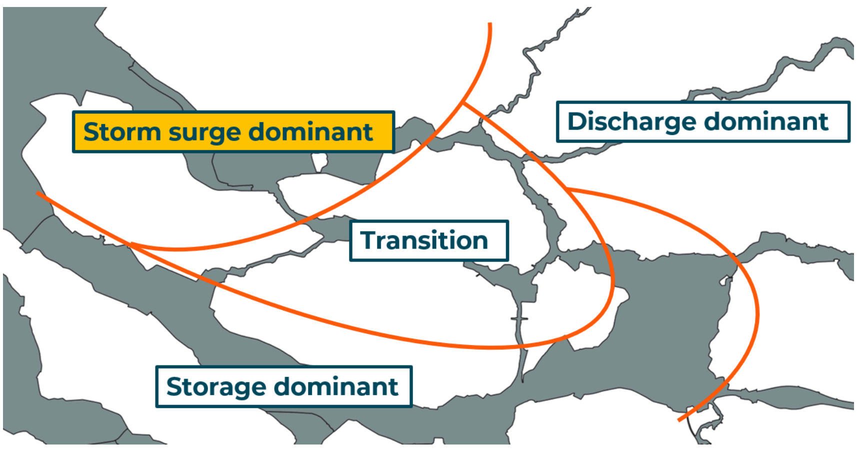


Figure 18: division of the Rhine-Meuse delta (Oerlemans, 2020). The Maeslant barrier is located in the storm surge dominant area which is highlighted in yellow.

Hydraulic structures located in the storm surge dominant area (the Maeslant- and Hartel barrier, dikes) are protecting the hinterland from the sea during storms. The area behind the Maeslant barrier is storm surge dominant because the water levels in this area are influenced highly by the sea. Dikes in the storage dominant area protect the land from the accumulating water in the basin during storms. The transition area is influenced mostly by the river discharge and storage dominant area during storms, as the storm surge is halted by the barriers in the storm surge dominant area. The discharge dominant area is influenced mostly by rivers. During storms, this area hardly experiences consequences of the slowly raising water level in the area behind the closed barriers.

Effect of the reduced reliability of the Maeslant barrier on the dikes in the hinterland
Since the Maeslant barrier is less reliable than initially was thought (design failure probability to close 1/1,000, actual failure probability to close 1/100), the dikes in the are also hinterland effected by this. This results in an increase in assessment level for the dikes in the hinterland. The increase in magnitude for this assessment level is provided in Figure 19. This, combined with sea level rise as described in Section 1.1.2, puts the dikes at even more risk.

The dikes which are most influenced are the dikes in the storm surge dominant area and transition area. Oerlemans (2020) also stated that areas outside of the storm surge dominant area are less sensitive to the failure probability of the Europoort barrier[1]. This is confirmed by Rijkswaterstaat (2006) in the "Achterlandstudie Maeslantkering", as depicted in Figure 19.

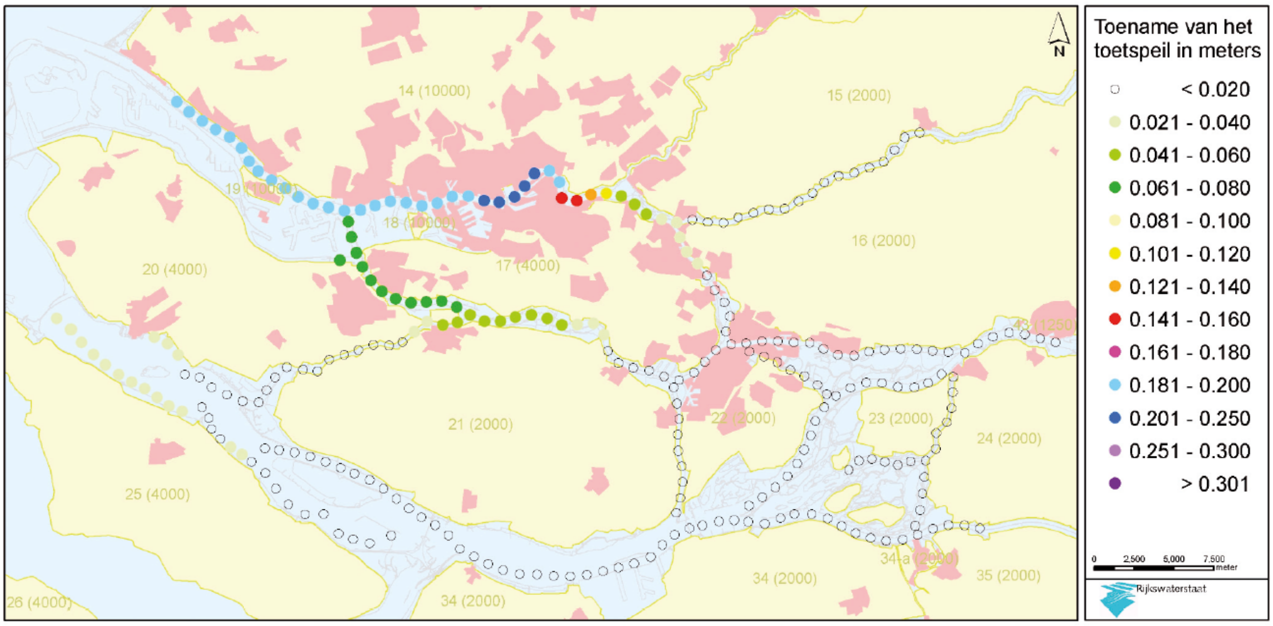


Figure 19: Increasing effect of the 1/100 failure probability of the Maeslant- and Hartel barrier on assessment levels in the Rhine-Meuse mouth (Rijkswaterstaat, 2006).

Water levels during storm conditions with high river discharges
If the Maeslant barrier is closed, the river water is building up gradually in the area behind the barrier. The water can be temporarily stored in a storage area, which extends

[1] The Europoort barrier consists of the Maeslant barrier, Hartel barrier and the dike segment between the two barriers which is depicted in Figure 20.

from the Biesbosch until the barriers. It can be seen as a big storage area of 540 km^2 with Dordrecht as the centre of this area (Tonneijck, de Boer, Vrijling, Berke, & Lavooij, 2018). The storage area is filled up with water from the Rhine and Meuse. The speed with which this area fills up depends on the height of the storm surge, the storm duration, and the magnitude of the river discharge. Seiches can influence the water level in front of the barrier. This will be covered in Section 2.3.

To summarize, the water level close to the barrier can be influenced by:

- The magnitude of the river discharge
- The storm surge height
- The storm duration
- The storage area
- Seiches

As mentioned in Section 1.2, the Maeslant barrier must be closed when a water level of NAP +3.0 m occurs at Rotterdam or NAP +2.9 m occurs at Dordrecht. When these water levels are reached depends on the river discharge, storm surge height from the seaside and storm duration.

According to RIZA, 35 storms with an average duration of 32 to 35 hours were present in the last century. (Tonneijck, de Boer, Vrijling, Berke, & Lavooij, 2018). Each storm has a certain storm surge height or storm set-up (in Dutch “opstuwing”) which is caused by the wind. According to Delta21 (2018), the storms can be divided in three categories based on storm surge height:

Storm surge height [m]	Storms per century [-]	Exceedance probability [years]	State of the Maeslant barrier
< 1.5	20	1/5	Open
1.5 – 2.5	6	1/16	Closed during HW, floating during LW
> 2.5	8	1/12	Closed and submerged

Table 2: classification of storms. Adapted from Delta21 (2018).

Delta21 (2018) states that storms with a storm surge of 1.5 – 2.5 m, the barrier is closed and submerged during HW but floating during LW to discharge the river water which has been building up behind the barrier. This is not the case for storms with a storm set-up higher than 2.5 m. This is to reduce the probability of failure, as it is risky to let the barrier float up in situations when the storm surge height is higher than 2.5 m. In case storm surges larger than 2.5 m, the storm duration becomes important. Delta21 (2018) provided Table 3 which describes the relation between the storm duration and the frequency of occurrence.

Storm duration [hours]	Storms surge height [m]	Storms per century [-]	Average storm duration [-]	Closing frequency [years]
> 12	> 2.5	6	18	1/16
> 24	> 2.5	2	30	1/50
> 36	> 2.5	0,1	42	1/500

Table 3: closing frequency of the Maeslant barrier for long storms in combination with high storm surges. Adapted from Delta21 (2018).

The next step to determine the increase in water level (at Dordrecht) behind a closed Maeslant barrier is to address the river discharge, as this is the main source of the incoming water. Delta21 (2018) provided a table which provides insight in how often certain discharges occur, and what the resulting increase in water level is in the storage area of 540 km^2 behind the barrier. This overview is provided in Table 4.

Discharge [m^3/s]	Probability of occurrence [years]	Water level increase [cm/hour]
5,000	1/1	3.3
7,000	1/2	4.7
9,000	1/10	6.0

Table 4: river discharge with water level increase in case of a closed Maeslant barrier. Adapted from Delta21 (2018)

The next step is to couple these river discharges with the storm durations to address the water level increase, which Delta21 (2018) also provided. This is given in Table 5.

Closing duration	With 5,000 m^3/s	With 7,000 m^3/s	With 9,000 m^3/s
18 hours	0.6 m	0.8 m	1.1 m
30 hours	1.0 m	1.4 m	1.8 m
42 hours	1.4 m	2.0 m	2.5 m

Table 5: increase in water level as a result of river discharge and duration. Adapted from Delta21 (2018).

Figure 20: the Europoort barrier (Rijkswaterstaat, 2012). Dike segment part of the Europoort barrier highlighted in black.

Figure 21 and Figure 22 depict the water level in front- and back of the barrier during the storm closures in 2007 and 2018. During these closures, the closure level was lowered from NAP + 3.0 m to NAP + 2.6 m.

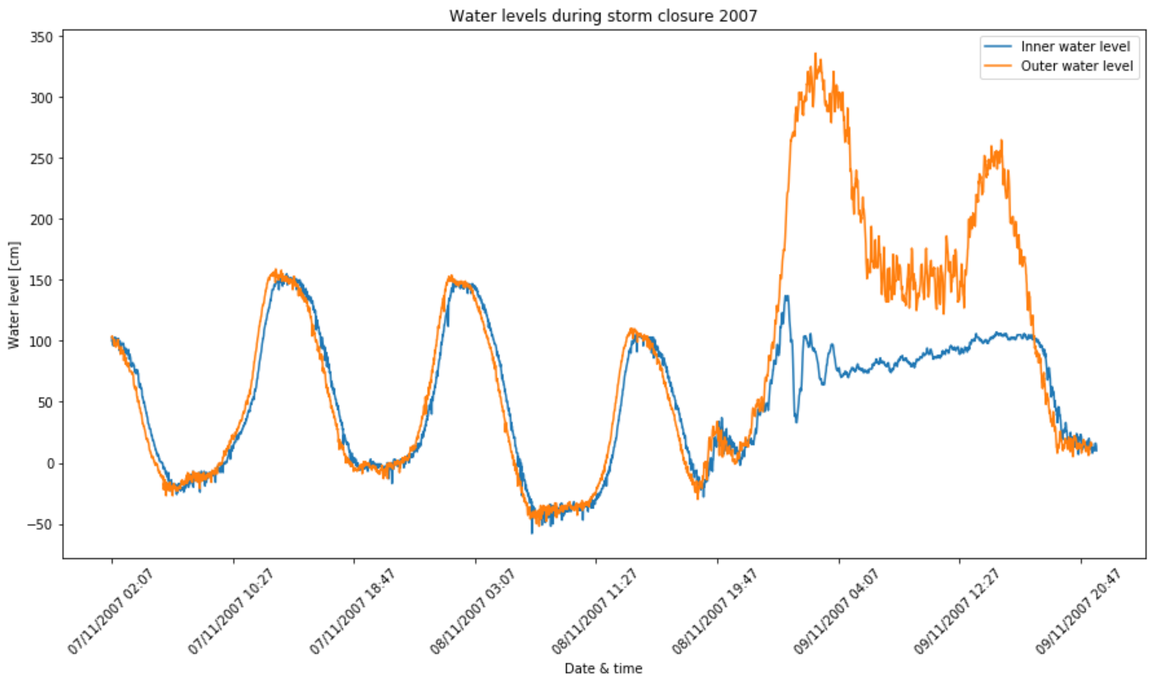


Figure 21: water levels in front and behind the Maeslant barrier during the storm closure in 2007

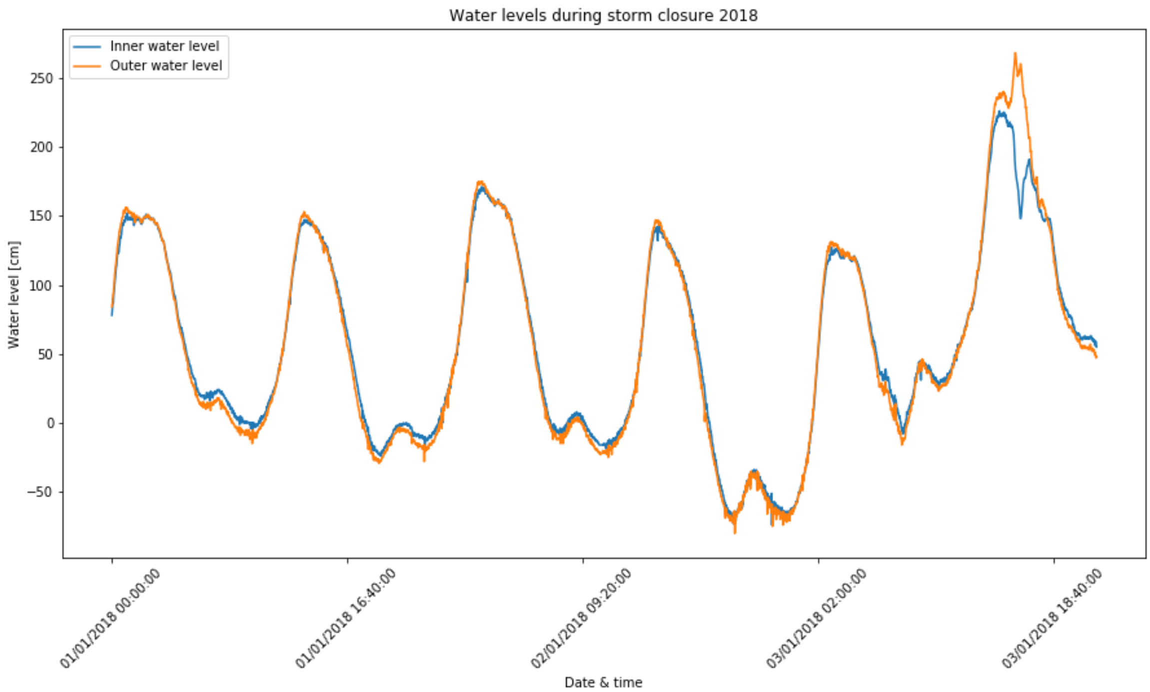


Figure 22: water levels in front and behind the barrier during the storm closure in 2018

2.2 Delta21 configuration

This section describes how the discharge is distributed in the Delta21 configuration, and how the water levels are behaving in case of closed storm surge barriers.

Discharge distribution

Buijs (2021) implemented Delta21 in the existing Rhine-Meuse Delta model and did simulations for two cases: one for a sea level rise of 0.2 m, depicted in Figure 23 and one for a sea level rise of 1.1 m, which is depicted in Figure 24.

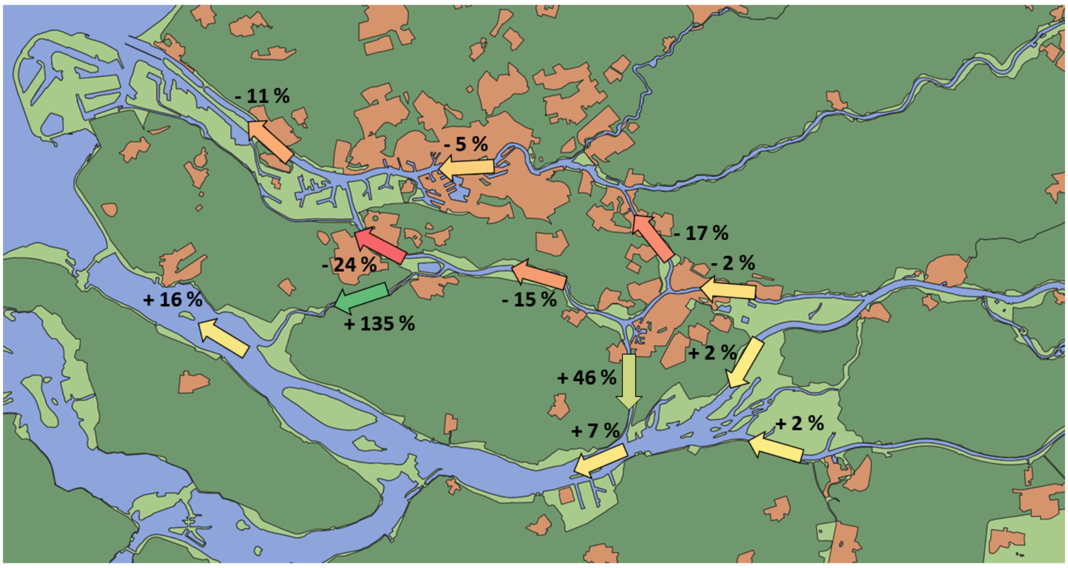


Figure 23: Change in average discharge in the Rhine-Meuse delta compared to the current system for a sea level rise of 0.2 m (Buijs, 2021).

After comparing the current system to the system with Delta21 included, more water is drawn to the Haringvliet area, and less is going toward the New Waterway. These figures do not describe the average discharge in a normal situation, but it describes a trend of changes when extreme discharges occur (Buijs, 2021).

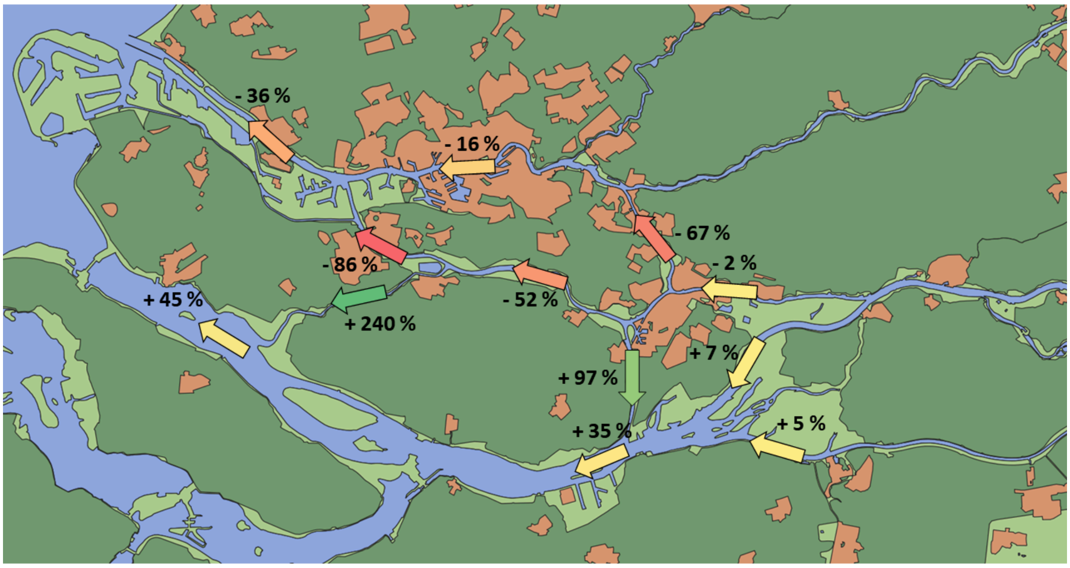


Figure 24: Change in discharge distribution in the Rhine-Meuse delta compared to the current system for a sea level rise of 1.1 m (Buijs, 2021).

Water levels

Oerlemans (2020) also implemented Delta21 in an existing Rhine-Meuse Delta model, including correct functioning barriers (Haringvliet sluices, Hartel- and Maeslant barrier). The change in water levels can then be estimated using simulation with the model. The water levels in normal conditions can be found, but also in storm conditions when the barriers (Europoort barrier and Haringvliet sluices) are closed.

High-water levels in normal conditions

It was shown in Figure 25 that Delta21 succeeds in reducing the water level at the transition- and flood storage areas under normal conditions. The water level reductions in the storm surge area dominated remains limited under normal conditions (when the Europoort barrier is open). This is also the cause why the water level reduction is very limited.

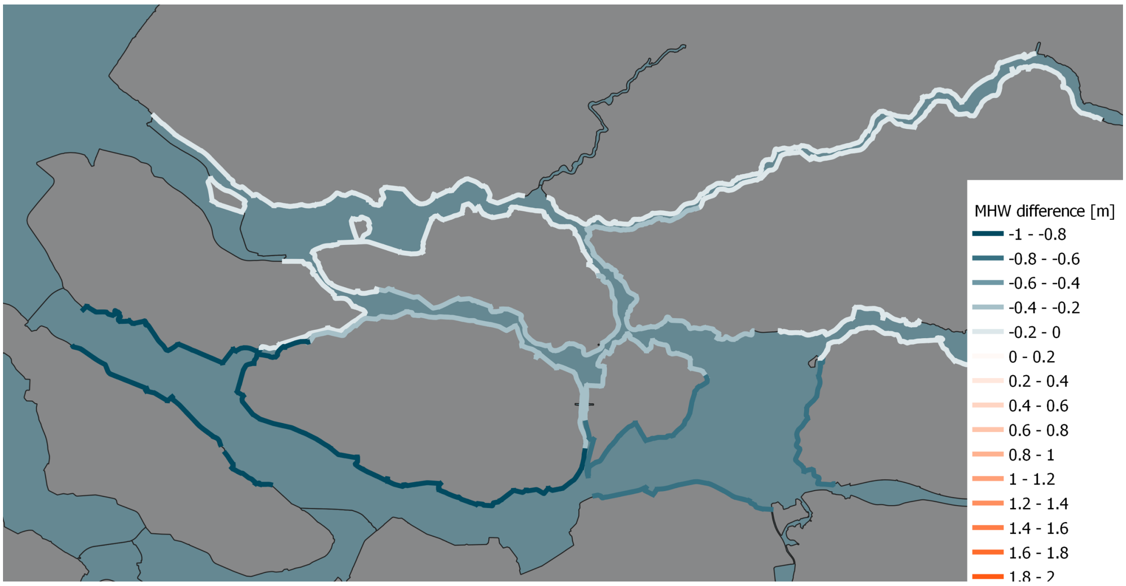


Figure 25: water level difference with Delta21 compared to MHW in the current situation, and a correct functioning Europoort barrier with failure probability of 1/100 (Oerlemans, 2020).

Water levels during storm conditions with high river discharges

The water levels close to the barrier in the Delta21 configuration are influenced by:

- The magnitude of the river discharge
- The Delta21 pump capacity
- The storm surge height
- The storage area
- The storm duration
- Seiches

Seiches will be covered in the next section. Oerlemans (2020) simulated a storm with a duration of five days, a storm surge height of 3.54 m and a river discharge of 10,000 m^3/s. An overview of the water level at Rotterdam was also given. The situation was also simulated with Delta21 implemented. The water levels at Rotterdam and Maassluis were reported by Oerlemans (2020) and is depicted in Figure 26. The Maeslant barrier closes at the black dashed line depicted in Figure 26. The fluctuations after the barrier has closed are translation waves which reflect on the closed barrier gates.

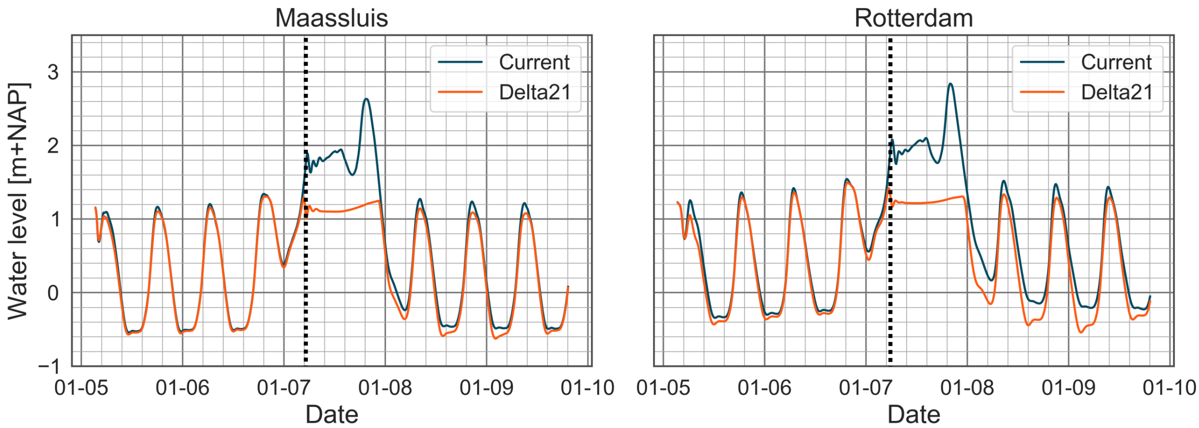


Figure 26: water levels during the storm with a storm surge height of 3.54 m and a river discharge of 10,000 m³/s (Oerlemans, 2020).

It can be clearly seen that the water level at Rotterdam is much lower during the storm when compared to the current situation. This is because the accumulating water behind the closed barriers is being pumped out to the sea by the pumps from Delta21. The discharge distribution differs very little in the current situation when compared to the Delta21 configuration. This is depicted in Figure 27. A positive discharge means flow to the sea, and a negative discharge means flow into the river from sea. The discharge is out of phase with the surface elevation (depicted on the left figure in Figure 26). The altercations in the beginning of the simulation represent the tidal cycle.

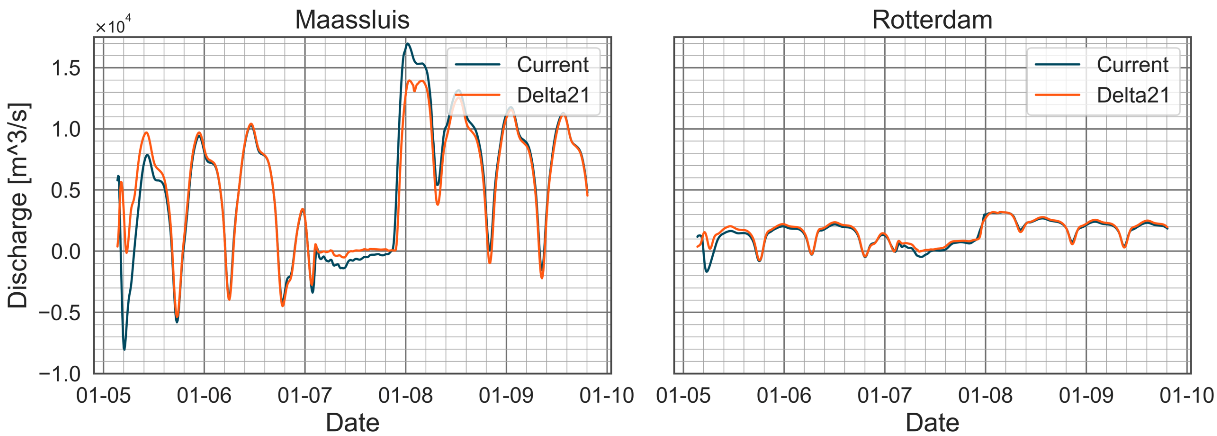


Figure 27: discharges in the New Waterway during the storm with a storm surge height of 3.54 m and a river discharge of 10,000 m³/s (Oerlemans, 2020).

The outcome of the simulation which Oerlemans (2020) did proved that Delta21 has potential to greatly reduce the water level behind the (closed) Europoort barrier. This is beneficial, but it also has a consequence on the closing procedure of the Maeslant- and Hartel barrier. Since the water level is much lower at the rear side of the barriers, it will also take longer for the water level on the inner side to match with the water levels on the outer side for the gates to be opened. These water levels need to be almost equal to be able to open the gates of the barriers. The increased time required to reach the moment of opening could be an issue for the shipping industry. Oerlemans (2020) analysed the effect of different storm surges with various discharges on the duration of the closure. This is based on the simulation results, depicted in Figure 28.

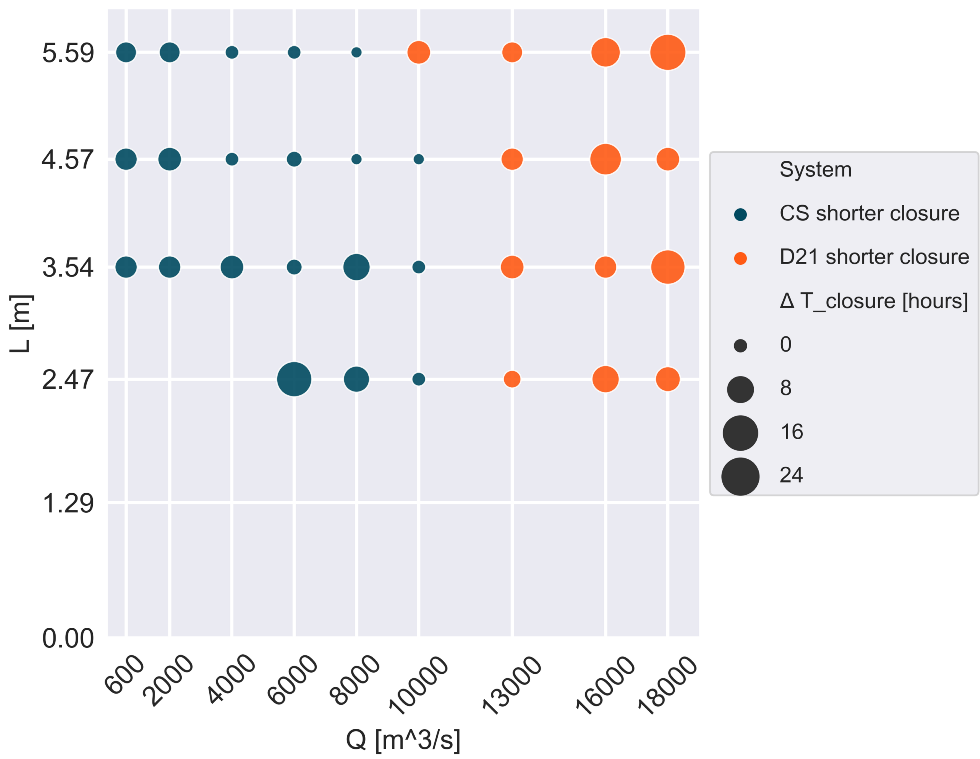


Figure 28: Relation between storm surge height (y-axis), discharge (x-axis) and closure duration (Oerlemans, 2020). Explanation of this figure is below the figure.

Figure 28 needs an explanation. On the y-axis, the simulated storm surge heights (2.47 m, 3.54 m, 4.57 m and 5.59 m) are plotted against the discharges on the x-axis. A blue circle means a shorter closure in the current configuration (without Delta21), and an orange circle means a shorter closure in the Delta21 configuration. The size of a circle represents the duration of the closure in hours. A small circle means a short closure duration, while a big circle represents a long closure duration (Oerlemans, 2020).

An interesting observation is that for discharges up to 10,000 m^3/s with all the storm surges the duration of a closure is shorter in the current situation. For discharges larger than 13,000 m^3/s, the opposite holds.

2.3 The impact of seiches on the functioning of the Maeslant barrier

In this section, seiches will be elaborated in depth. First it needs to be explained what they are exactly, how often occur and what their magnitude could be. Then, seiches will be examined at the location of the Maeslant barrier in the current configuration and in the Delta21 configuration.

To understand the impact of seiches is essential since it will be the only cause for a negative head difference in the Delta21 configuration. Potential structural measures (to improve the reliability of the Maeslant barrier even further in the new situation) cannot be developed if it is not fully understood what the effect of seiches is on the water level in front of the barrier.

2.3.1 Definition, frequency of occurrence & magnitude

As mentioned in Section 1.2.3, seiches play in important role in locally raising or lowering the water level at the location of the barrier.

Definition

"A seiche is a standing wave in a partially or fully enclosed water basin."

Seiches can occur in small harbours or larger sea regions. Seiches in small harbours can be generated by two mechanisms (de Jong M. P., 2004):

- By periodic changes: these are generally ascribed to atmospheric gravity waves which result relatively large amplitude fluctuations.
- By sharp jumps in atmospheric pressure: these are sometimes caused by cold fronts.

According to de Jong (2004), the larger sea regions were influenced by complete low-pressure areas. Due to these low-pressure areas, the water level locally increases. In such a case, wind induced set up can increase the amplitude of the seiche even further. This wind induced set up can initiate an oscillation in case the wind suddenly drops in magnitude or changes in direction (de Jong M. P., 2004).

Generating mechanism

Since the Maeslant barrier is located near the Rotterdam Harbour, it needs to be known in which situation (small harbours or larger sea regions) the harbour is classified. De Jong (2004) states that the spatial scales of the harbour are between these two situations. The main generating mechanism for seiches in the Rotterdam Harbour are wind-induced low-frequency waves which are generated offshore. These waves excite oscillations in coastal harbours which can be amplified in case of resonance, giving rise to seiches (de Jong M. P., 2004). These wind-induced low-frequency waves are caused by moving meso-scale atmospheric convection cells following a cold front passing over relatively warm sea water. The sea surface response is amplified when the system of atmospheric convection cells moves at a speed near that of free (long) gravity waves (de Jong M. P., 2004). This is the case in the southern North Sea, which is close to the Rotterdam Harbour. These waves have a period in the order of 1 hour.

The Rotterdam harbour is a semi-enclosed basin in which the seiches can occur. When the Maeslant barrier is open, seiches which occur in the Rotterdam harbour have no negative effect on the barrier. However, when the Maeslant barrier closes, a smaller, semi-enclosed basin is created on the seaside of the barrier, in which a seiche can also occur. These seiches have an average period of 30 minutes (de Jong M. P., 2004).

Frequency of occurrence

The occurrence seiches can be predicted, but to predict the magnitude of a seiche is still difficult (Rijkswaterstaat, 2012). The magnitude of a seiche at the location of the barrier is a function of the layout of the harbour basin (Rijkswaterstaat, 2012). De Jong (2004) analysed surface elevation records at Rozenburg in the period 1995-2001, and 49 seiches with a crest level higher than 0.25 m were observed in this period. 90% of these observed seiches occurred in the storm season. Additionally, all of these observed seiche events occurred following a cold-front passage over the harbour area, approaching from the sea (de Jong M. P., 2004). De Jong (2004) also reported that seiches in the western harbor area in Rotterdam always occur during stormy weather, but not all storms necessarily induce a (high) seiche.

Magnitude
The highest observed amplitude of seiche in the Rotterdam Harbour is 0.9 m. The typical amplitude range for seiches in the Rotterdam Harbour ranges from 0.5 m to 1.0 m (de Jong M. P., 2004).

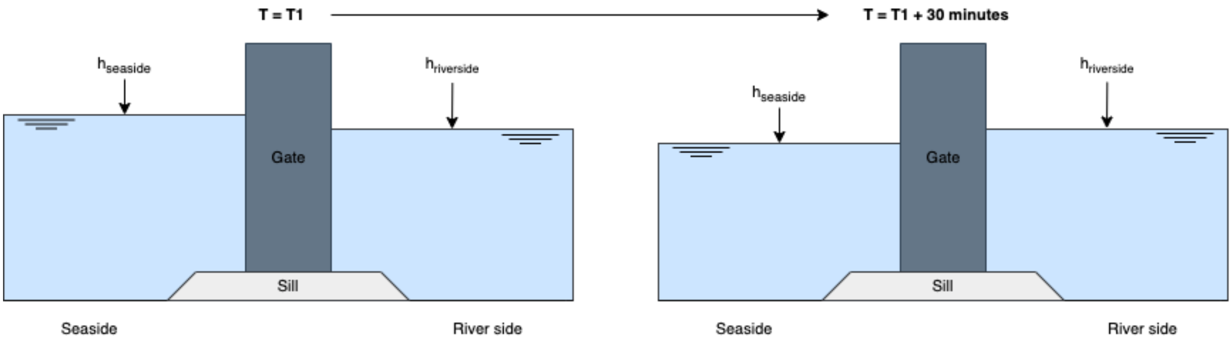


Figure 29: cross section of the gate resting on the riverbed sill, while in closed position

Figure 29 shows how a seiche can cause a negative head difference during a storm. A cross section of the sector gate resting on the sill during a storm is depicted. On the left side of the gate resides the seaside, on the right side of the gate, the riverside. On T = T1, $h_{seaside} > h_{riverside}$ which means that the head difference is positive. On T = T1 + 30 minutes, $h_{seaside} < h_{riverside}$, which means that a negative head difference has occurred. This is because a seiche with a period of 30 minutes has quickly decreased the water level on the seaside.

2.3.2 Effect of seiches in the current configuration

The main conclusion which needs to be drawn about seiches is that a seiche can increase or decrease the water level in front of the closed barrier. In case of an increase in water level, the water level in front of the barrier (seaside) is further raised which has a far lesser impact than when the water level decreases in front of the barrier (seaside). This is because of the design of the ball joint, as mentioned in Section 1.2.3. In case of a decrease in water level, the negative head difference could become too large. In this scenario, the barrier will start to float up and discharge water to level the negative head difference out. This is currently handled by the dynamic pre-stress management method (Rijkswaterstaat, 2012), as described in Section 1.2.3.

The probabilities of a seiche occurring with an open and closed Maeslant barrier from the original design (of the Maeslant barrier) are presented in Table 6. This data does not change in the situation with Delta21, as this is data for the outer side of the barrier, which remain unchanged.

Probability of exceedance [per year]	Seiche amplitude [m] (barrier closed)	Seiche amplitude [m] (barrier open)
10^{-2}	0.20	0.10
10^{-3}	0.45	0.30
10^{-4}	0.75	0.50
10^{-5}	1.10	0.75
10^{-6}	1.40	1.00
10^{-7}	1.80	1.30
10^{-8}	2.20	1.60

Table 6: probability of exceedance for various seiche amplitudes (Rijkswaterstaat, 1995)

2.3.3 Seiches in the Delta21 configuration

Since Delta21 is supposed to simplify the operation of the Maeslant barrier, the discharging function during storms would be omitted. Currently, the effects of seiches are mitigated by dynamic pre-stress management as described in 1.2.3. This cannot be the case anymore in the Delta21 configuration. A measure to mitigate the effect of seiches would be to implement passage openings in the sector gates. These were also present in the original design of the Maeslant barrier but were not incorporated because the cost was too high at the time.

To be able to explain the reasoning behind this, the following scenario could serve as an example for this: if a two-headed storm occurs and the LW level between the peaks of the two-headed storm is lower than the water level behind the barrier, a negative head difference will occur. In the current situation, the water level behind the barrier would be high. The chance that the negative head difference would occur is present, so discharging is needed if this condition is true. A graph which shows this will be presented later on in this chapter.
In the Delta21 configuration with the same two-headed storm, a negative head difference could still occur during LW, but the water level behind the barrier will be much lower than in the current situation (see Figure 26). This means that the chance of a negative head difference larger than 1.5 m would be smaller. Including the effect on seiches on the water level in front of the barrier in both cases would increase the chance of the negative head difference being larger than 1.5 m. To prove this, a hydraulic model is needed.

2.4 Concluding remarks

To conclude this chapter, the following things are known:

- It is now known how the Maeslant barrier operates in the current configuration and how it changes in the Delta21 configuration.
- It is now understood why seiches are so important if the discharging function of the Maeslant barrier is omitted.

In the next chapter, the hydraulic model will be developed to find out what the influence of Delta21 on the water levels at the location of the Maeslant barrier is. Seiches will also be included in the model.

3 The development of the simple hydraulic model for the Maeslant barrier

This chapter elaborates the development of the hydraulic model, which was defined as methodological step 2 in section 1.4. Each section in this chapter is one of the steps needed to develop the model, as described in section 1.4.

3.1 Schematization of the real world into model elements

To investigate the probability of a negative head difference larger than 1.5 m at the Maeslant barrier (including seiches) in the new situation with Delta21, the simple hydraulic model is necessary. This is also necessary to investigate if pumping in the Haringvliet area can reduce the probability of a negative head occurring. A schematization of the model that will be used in the simulation is depicted in Figure 30. The model simulates the following scenario: the situation during a storm in which the Maeslant barrier is still closed, and water has accumulated behind the barrier. On the seaside, a sinusoidal tide elevation with storm surge height will be simulated. Additionally, seiches with their exceedance probability for a closed barrier (as described in Section 2.3.2) will be added to the simulation. Figure 29 depicted how a seiche could lead to a negative head difference during a storm. Figure 31 depicts the cross section along the dashed line A-A in Figure 30, and it illustrates how the channel between the two basins is modelled. In reality, the water level between the basins does is not a straight line but curved due to backwater effects.

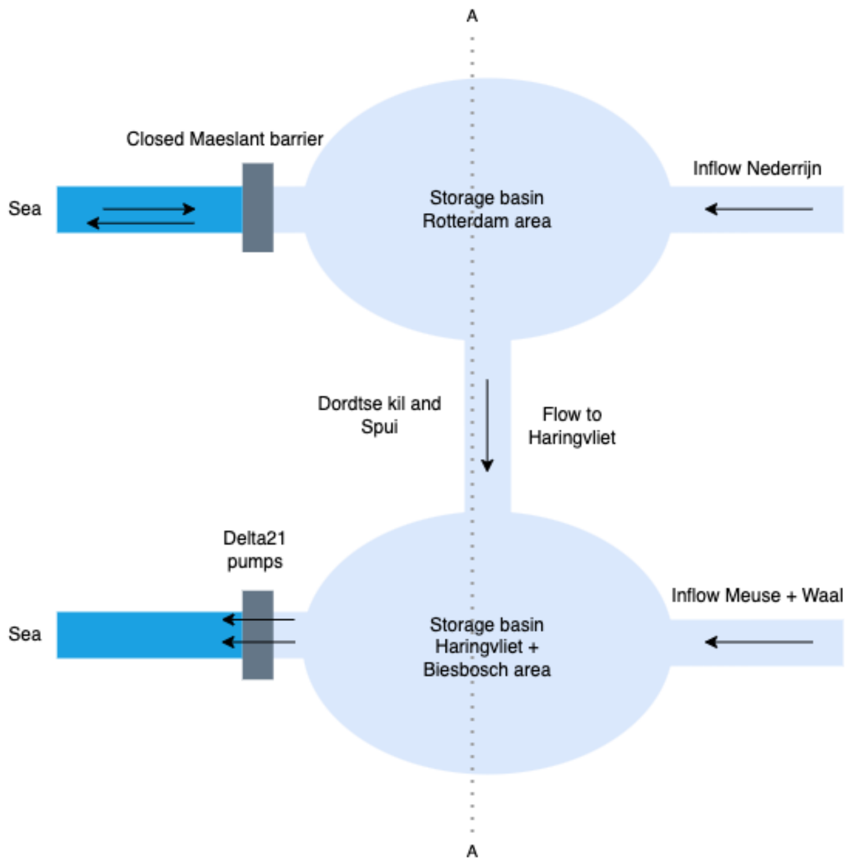


Figure 30: model schematization with Delta21

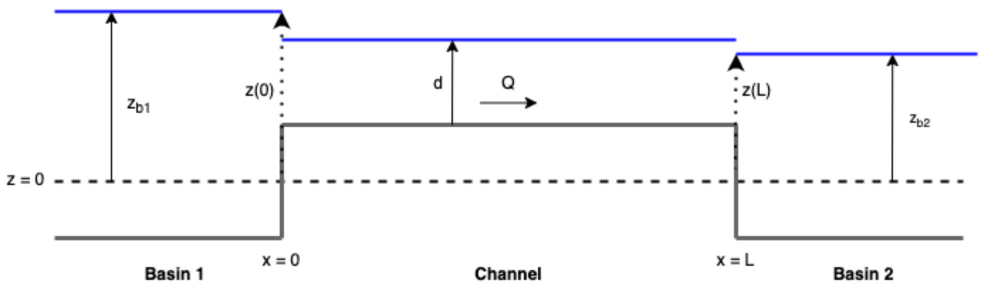


Figure 31: cross section A-A, the channel, with basin 1 (Rotterdam) on the left side and basin 2 (Haringvliet) on the right side.

Model components

The model consists of the following components:

- Two storage basins:
  - One storage basin behind the Maeslant barrier
  - One storage basin which consists of the Haringvliet, Biesbosch area and the Delta21 energy storage lake and tidal lake.
- Channel which connects the two basins
  The total cross-sectional area of this channel is equal to the cross-sectional area of the Dordtsche Kil and the Spui.
- Inflow:
  The water which is supplied from the rivers.
- Outflow:
  This can be either pumps from Delta21, discharging trough the Haringvliet sluices or pumps at the Maeslant barrier itself.
- The tidal signal from the sea.
- The storm surge height.
- Seiches at the seaside.
- The Maeslant barrier.

Assumptions

- In reality, the sector gates of the Maeslant barrier have a small gap between them when the gates are closed. This means that a small residual flow is present (can be inflow or outflow, depending on the head difference). This residual flow will be neglected because it is small compared to the in- and outflow of the basin. Thus, it is assumed that the gates close completely.
- The storage area is assumed to be constant in depth. Since this is a very basic model, the assumption is made that the storage area stays constant for all water depths. In reality, the bathymetry determines the storage area. Changes in bathymetry are not modelled here. As the water depth in the basin decreases, the instantaneous storage area decreases as well in reality.
- The basins are 0-dimensionally modelled. This means that inertia is not being accounted for.
- Friction is taken into account for the channel, but not for the other parts of the system (tide, basins)
- Seiches only occur when the barrier is closed, this is because they can increase or decrease the water level at the seaside of the barrier. In reality, seiches can occur when the barrier is closed and when the barrier is open.

- The whole closing procedure as described in Section 1.2.2 is not taken into account into the simulations. If the predicted water level does not exceed the predefined closure level during the storm anymore, the barrier opens (without modelling the opening procedure)
- It is assumed that there is no lag between the tide at the location of basin 1 and basin 2.
- The discharging procedure as described in Section 1.2.3 is not built into the model. In the model, the Maeslant barrier operates using the assumption that Delta21 discharges water when needed. The barrier closes at the beginning of a storm (that meets the closure criteria) and opens when the storm is over.

Goal of the simulation
The goal of the simulation is to find out what the influence of Delta21 on the water levels at the location of the Maeslant barrier is. Also, the probability of a negative head difference larger than 1.5 m can be calculated. This contributes to the failure probability of the ball joint, which will be explained in the next chapter. To obtain this probability, a Monte Carlo simulation will be done.

Method to calculate the required probability
To obtain the probability of a head difference larger than 1.5 m, a structured approach is needed. The approach which will be used has been described in Section 1.4.

3.2 Equations used to represent the model elements

Surface elevation basin
The model will be programmed in python. Since storage basins are going to be used, a balance for the volume in these storage basins is needed (Battjes J. A., 2002):

$$A_i \frac{dz_i}{dt} = Q_{in} - Q_{out} \qquad \text{Eq. 3.1}$$

In which:
A_i [m^2] = the instantaneous storage area in basin i
z_i [m] = height of the water above reference level z = 0 m in basin i
Q_{in} [m^3/s] = inflowing discharge into basin i
Q_{out} [m^3/s] = outflowing discharge out of basin i

The water level variation in the basin and the net discharge ($Q_{in} - Q_{out}$) are related by the continuity equation (equation 3.1). With the water level variation, the water levels in the basins can be updated. For each basin, equation 3.1 can be filled in:

$$A_{b1} \frac{dz_{b1}}{dt} = Q_{Nederrijn} - Q_{Channel} \qquad \text{Eq. 3.2}$$

In which:
A_{b1} [m^2] = the instantaneous storage area in basin 1
z_{b1} [m] = height of the water above reference level z = 0 m in basin 1
Q_{Rijn} [m^3/s] = inflowing discharge of the Rijn into basin 1

$Q_{Channel}$ [m^3/s] = outflowing discharge through the channel out of basin 1

$$A_{b2}\frac{dz_{b2}}{dt} = Q_{Channel} + Q_{Maas} + Q_{Waal} - Q_{out} \quad \text{Eq. 3.3}$$

In which:
A_{b2} [m^2] = the instantaneous storage area in basin 2
z_{b2} [m] = height of the water above reference level z = 0 m in basin 2
$Q_{Channel}$ [m^3/s] = inflowing discharge from the channel into basin 2
Q_{Maas} [m^3/s] = inflowing discharge of the Maas into basin 2
Q_{Waal} [m^3/s] = inflowing discharge of the Waal into basin 2
Q_{out} [m^3/s] = outflowing discharge out of basin 2 into the sea, via Delta21 pumps

The change water levels in basin 1 and basin 2 are linked through the discharge in the channel ($Q_{channel}$). By rewriting equation 3.2 and 3.3, an expression for the water level in the reservoir can be found by integrating both sides of the equation:

$$z_{b1} = \int_{t=t_1}^{t=t_2} \frac{Q_{Nederrijn}(t) - Q_{channel}(t)}{A_{b1}} dt + C \quad \text{Eq. 3.4}$$

In which:
A_{b1} [m^2] = the instantaneous storage area in basin 1
z_{b1} [m] = height of the water above reference level z = 0 m in basin 1
Q_{Rijn} [m^3/s] = inflowing discharge of the Rijn into basin 1
$Q_{Channel}$ [m^3/s] = outflowing discharge through the channel out of basin 1
C [m] = integration constant

$$z_{b2} = \int_{t=t_1}^{t=t_2} \frac{Q_{Channel}(t) + Q_{Maas}(t) + Q_{Waal}(t) - Q_{out}(t)}{A_{b2}} dt + C \quad \text{Eq. 3.5}$$

In which:
A_{b2} [m^2] = the instantaneous storage area in basin 2
z_{b2} [m] = height of the water above reference level z = 0 m in basin 2
$Q_{Channel}$ [m^3/s] = inflowing discharge from the channel into basin 2
Q_{Maas} [m^3/s] = inflowing discharge of the Maas into basin 2
Q_{Waal} [m^3/s] = inflowing discharge of the Waal into basin 2
Q_{out} [m^3/s] = outflowing discharge out of basin 2 into the sea, via Delta21 pumps
C [m] = integration constant

Friction in the channel
The discharge flowing through the channel depends on various things. The dimensions of the channel, the friction of the bottom, the energy losses due to inflow and outflow and the water levels at the ends of the channel. The channel will be modelled including energy losses due to inflow- and outflow losses and bottom friction. The channel will be assumed to be prismatic (no change in conveyance area over the length of the channel). Storage is also neglected in the channel. Thus, the water mass moves back and forth as a solid block in the channel. This is also called the rigid column approximation.

The width of the channel is equal to the average of the Dordtse kil and Spui. The same holds for the depth. The properties of the channel have been summarized in Table 7.

Equations used to represent the model elements

d [m]	B [m]	c_f [-]	L [km]
10	453	0.004	18

Table 7: properties of the channel which connects the two basins

The bottom friction coefficient c_f has a large influence on the discharge capacity of the channel. Since information on this parameter for the Spui and Dordtsche Kil is hard to find, assumptions will be made. Battjes & Labeur (2017) reported that c_f typically varies between 0.002 to 0.006. Therefore, a value of 0.004 will be used. This value is also used in the current 1D SOBEK model of the Rhine-Meuse Delta. However, the boundary values of 0.002 and 0.006 will also be used to check how large its influence on the discharge in the channel is. However, the chosen friction coefficient will also have an uncertainty of 1% (normally distributed) and the model uncertainty for the Monte Carlo simulation.

To obtain the discharge in the channel, the momentum equation needs to be used (Battjes & Labeur, 2017):

$$\frac{\partial Q}{\partial t} + \frac{\partial}{\partial x}\left(\frac{Q^2}{A_c}\right) + gA_c\left(\frac{\partial h}{\partial x}\right) + c_f\frac{|Q|Q}{A_cR} = 0 \qquad \text{Eq. 3.6}$$

Since the assumption was made that the channel is prismatic, the second term in the momentum equation can be set to zero. The remaining equation is then integrated over the channel length and rewritten, which results in the following equation (Battjes & Labeur, 2017):

$$l\frac{dQ_{Channel}}{dt} = gA_c(z_{b1} - z_{b2}) - \chi\frac{|Q_{channel}|Q_{channel}}{A_c} \qquad \text{Eq. 3.7}$$

with

$$\chi = \frac{1}{2} + c_f\frac{l}{R} \qquad \text{Eq. 3.8}$$

In which:

A_c [m^2] = conveyance area of the channel
l [m] = length of the channel
$Q_{Channel}$ [m^3/s] = discharge in the channel
z_{b1} [m] = surface elevation in basin 1
z_{b2} [m] = surface elevation in basin 2
c_f [-] = bottom friction in the channel
R [m] = hydraulic radius of the channel
g [m/s^2] = gravity constant

The parameter χ in equation 3.7 is the resistance coefficient. It consists of losses due to expansion loss and boundary resistance. The first term (the constant ½) in equation 3.8 is due to expansion losses when the water enters the basin from the channel, and the second term represents the boundary resistance (bed friction). Equation 3.7 can be solved numerically in Python to approximate the discharge in the channel. The semi-implicit method is used to approximate the discharge (this will be shown in the verification stage). Since z_{b1} and z_{b2} are linked through the channel, initial conditions are needed to be able to solve the discharge in the channel. Initially, it will be assumed that there is no flow through the channel. Using this assumption, the discharge in the next time step can be computed. With the computed discharge, the water levels can be updated for every time step (as the discharge depends on the water levels in the basins). This happens for all time steps which ultimately gives the discharge in the channel. Now, the water level on the outer side of the basin needs to be known to be able know the head difference over the Maeslant barrier. The water level on the outer side depends on:

- Seiches
- The tide
- The storm surge height

Seiches
As defined in Section 2.3.1, a seiches has the form of a standing wave. This standing wave needs to be modelled as well. The equation for the surface elevation for a standing wave is given by:

$$\eta_{seiche} = a\, cos\,(\omega t - kx) \qquad \text{Eq. 3.9}$$

In which:

η_{seiche}	[m]	= surface elevation
a	[m]	= wave amplitude
ω	[rad/s]	= angular velocity
k	[rad/m]	= wave number
t	[hr]	= time
x	[m]	= space

As mentioned in section 2.3.1, the average period of a seiche when the Maeslant barrier is closed was 30 minutes. With this information, the angular velocity can be calculated. For the amplitude of the seiche, various simulations will be used for various probabilities of exceedance, as reported in 2.3.2.

Tide
The tide consists of many constituents. The constituents have an influence on the surface elevation. The influence of the constituents on the tide differs per location. The equation for the surface elevation due to the tide is given by (Bosboom & Stive, 2015):

$$\eta_{tide} = a_0 + \sum_{n=1}^{N} a_n \cos\,(\omega_n t - \alpha_n) \qquad \text{Eq. 3.10}$$

In which:

η_{tide}	[m]	= surface elevation
a_0	[m]	= mean level
a_n	[m]	= amplitude of component number n
ω_n	[rad/s]	= angular velocity of component number n
α_n	[-]	= phase angle of component number n
N	[-]	= number of harmonic components

Since the Maeslant barrier is located close to Hoek van Holland, the dominating tidal constituents at that location will be used to determine the surface elevation due to the tide. To be able to predict the tide as accurate as possible, the three highest contributing constituents will be used. These can be found in Table 8.

Constituent	Amplitude a_n [m]	Phase angle α_n [deg]	Angular velocity ω_n [deg/hr]
M_2	0.79	86	28.984
S_2	0.19	147	30.000
M_4	0.17	165	57.968

Table 8: dominating constituents at Hoek van Holland (Bosboom & Stive, 2015)

Storm surge

The last contributing component of the surface elevation outside the barrier is the storm surge height. This is basically a sine wave of half a period with the amplitude equal to the storm surge height. The magnitude of the storm surge height can be set manually in case of a deterministic approach or picked out of an extreme value distribution in case of a probabilistic approach.

$$\eta_{ss} = A_{max} \sin\left(\frac{\omega t}{2}\right) \qquad \text{Eq. 3.11}$$

In which:

η_{ss} [m] = surface elevation
A_{max} [m] = maximum amplitude of the storm surge (excluding the tidal amplitude)
ω [rad/s] = phase velocity of the storm

To summarize:

The surface elevation on the **inner** side of the Maeslant barrier is determined by equation 3.4, 3.5 and 3.7:

$$z_{b1} = \int_{t=t_1}^{t=t_2} \frac{Q_{Nederrijn}(t) - Q_{channel}(t)}{A_{b1}} dt + C \qquad \text{Eq. 3.4}$$

$$z_{b2} = \int_{t=t_1}^{t=t_2} \frac{Q_{Channel}(t) + Q_{Maas}(t) + Q_{Waal}(t) - Q_{out}(t)}{A_{b2}} dt + C \qquad \text{Eq. 3.5}$$

$$l\frac{dQ_{Channel}}{dt} = gA_c(z_{b1} - z_{b2}) - \chi\frac{|Q_{channel}|Q_{channel}}{A_c} \qquad \text{Eq. 3.7}$$

The surface elevation on the **outer** side of the Maeslant barrier is determined by the sum of three components: seiches (equation 3.9), the tide (equation 3.10) and the storm surge height (ssh):

$$\eta_{outer} = \eta_{seiche} + \eta_{tide} + \eta_{ss} \qquad \text{Eq. 3.12}$$

These equations (equation 3.1 to 3.11) have been programmed in Python to find out how the system responds.

3.3 Verification of the programmed model

As mentioned in the previous section, equation 3.2 to 3.11 have all been implemented into the python model. For the resulting equations (3.2, 3.3, 3.7 and 3.11), it will be checked if the equations in the programmed model are dimensionally correct and equivalent to the analytical equations. After validation, it turned out that these equations have been correctly implemented into the programmed model. The programmed equations are equivalent to the analytical equations along with the correct units. The proof can be found in Appendix A.

3.4 Validation of the programmed model

In this section, the outcome of the created hydraulic model will be validated to check if it works properly given the assumptions which were made.

The validation will be done for individual model components (for the tide, seiches, storm set up) by plotting the water levels and analysing them to see if it reproduces the required results. After validating the individual elements, it could be concluded that the model produces the required and expected results. The validation for the individual elements can be found in Appendix B.

Validation of a model run which includes all the components will be done in two ways:

- By comparing the simulation results to the data of the actual storm closure in 2007. In this comparison, the simulated tide and storm surge can be verified.
- By comparing the simulation results to simulation results of other models which have been (partly) verified already. In this comparison, the behaviour of the modelled channel will be verified.

Validation of a model run (with all elements included)

Figure 32 depicts the water levels at the seaside (at Hoek van Holland) and behind the Maeslant barrier (at Maassluis) during the storm closure in 2007. The green line represents the water level on the seaside at Hoek van Holland, while the blue line represents the water level behind the Maeslant barrier at Maassluis. The orange line represents the water level at Hellevoetssluis, which is located in the Haringvliet area.

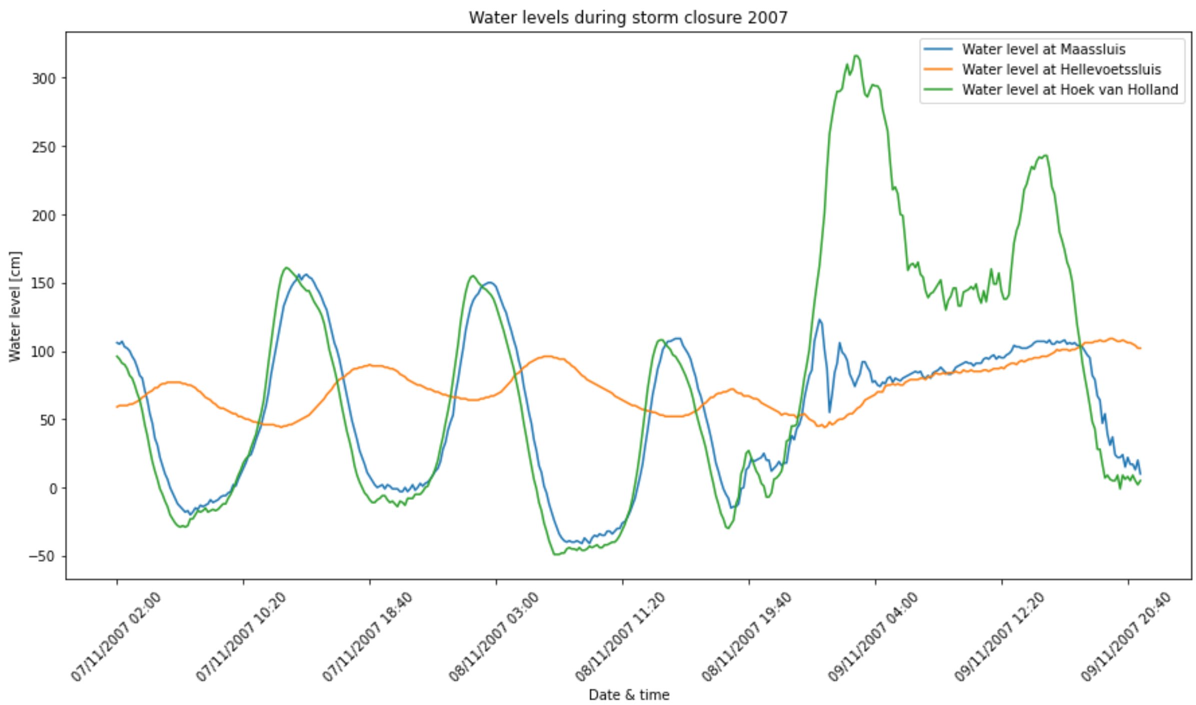


Figure 32: storm closure 2007

The base run from Section 0 depicted in Figure 33. When comparing the base run against the storm closure of 2007 in Figure 32, given the assumptions which were made, the tide and storm surge are reproduced quite good in the model. The peaks which can be seen in red are seiches with a period of 30 minutes. These occur only when the barrier is closed (in the model).

```
Qout : 10000.0 m3/s
Qin : 6350.0 m3/s
Qchannel : 1673.13 m3/s
Seiche amplitude : 0.2 m
Storm set-up amplitude: 3.0 m
Failure: 0
Time closed: 16 hours
Head difference when opening 0.093 m
```

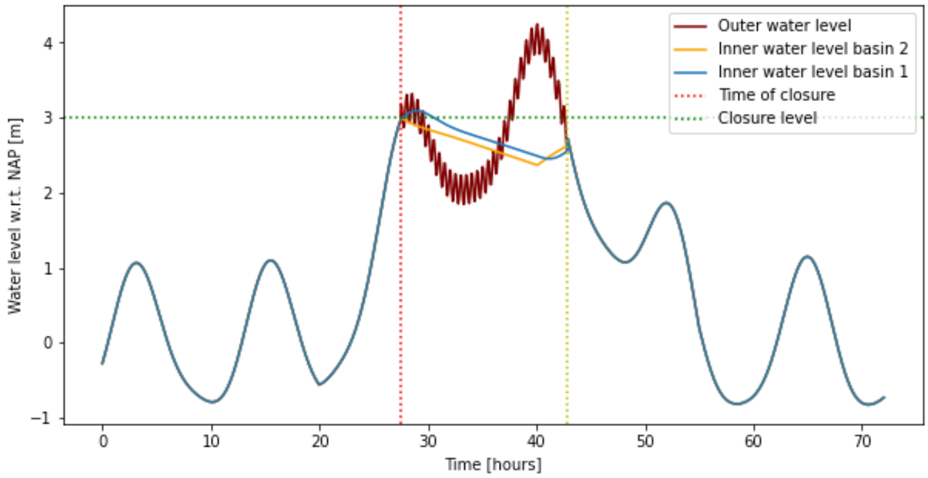


Figure 33: base run for pumping at full capacity

Validation for basin interaction through the channel

To validate the interaction between basin 1 and basin 2, the inner water level in the simple hydraulic model (Figure 33) needs to be compared to the inner water levels during the storm closure in 2007 (Figure 32). While the river discharge in the model run is not the same as during the river discharge in 2007, it can be observed that the lag between the water levels in basin 1 and basin 2 is present which can be explained by friction in the channel. Also, the water level is slightly higher behind the Maeslant barrier (basin 1) than in the Haringvliet (basin 2).

3.5 Adapting the deterministic variables to probabilistic variables

As mentioned in section 1.4, step 6 of the methodology for the hydraulic model is to make the model probabilistic. This section provides an overview of all the probabilistic variables with their (possible) dependencies and distributions. The following variables will be made probabilistic:

- The high water levels
- The incoming river discharges
- The seiches
- The basin area
- The outflowing river discharges
- The friction coefficient

Each subsection will cover one of the parameters.

3.5.1 Adapting the high-water levels (including storm surge)

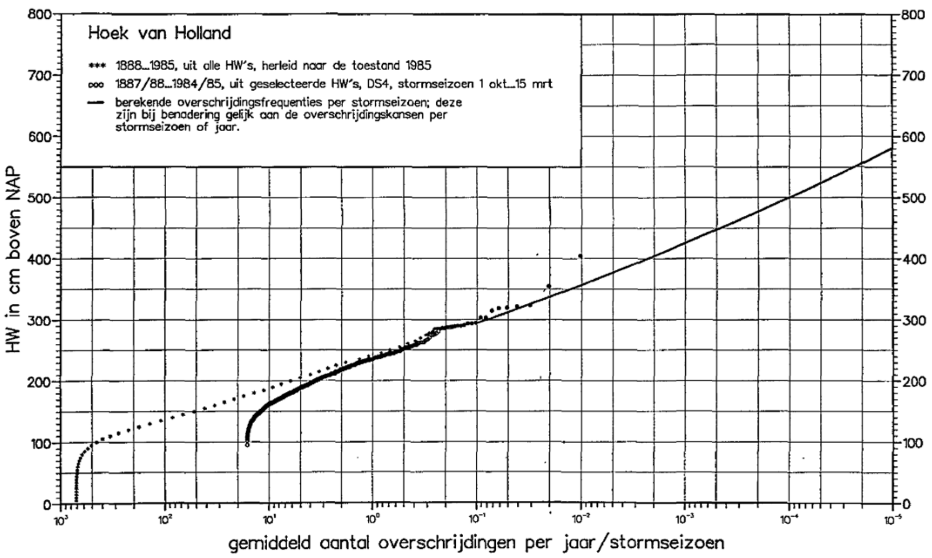


Figure 34: average number of exceedances for high water at Hoek van Holland per year per storm season w.r.t. NAP (Philippart, Dillingh, & Pwa, 1995)

Philippart, Dillingh & Pwa (1995) provided an exceedance probability curve for high water (including storm surges) at Hoek van Holland. This is based on data up to 1985. The values are distributed according to a Generalized Pareto Distribution (GPD) with certain parameters. The distribution is given by the following formula (Philippart, Dillingh, & Pwa, 1995) :

$$q_x = 0.5 * \left\{1 + \gamma \frac{x - \mu}{\sigma_u}\right\}^{\frac{-1}{\gamma}} \; for \; x - \mu \geq 0 \qquad \text{Eq. 3.13}$$

In which:

x	[cm]	= high water level
q_x	[-]	= average times per storm season in which a high-water level reaches or overtops level "x"
u	[cm]	= threshold with exceedance frequency of 0.5 times per storm season

σ_u [cm] = scale parameter for the GPD distribution for values above the threshold "u" for high water
γ [-] = scale parameter for the GPD distribution

The value for the scale parameters in 1985 is given in Table 9. Dillingh (2013) provided updated parameters for the situation in 2017. The GPD-distribution with the updated parameters will be used in the model because sea level rise needs to be accounted for. The updated parameters were obtained by increasing the threshold "u" in eq. 3.13 with the increase in high water levels in the period 1985-2017 (Dillingh, 2013).

Year	u [m]	σ_u [m]	γ [-]
1985	2.53	0.2474	0.03641
2017	2.58	0.2474	0.03641

Table 9: scale parameters for the GPD-distribution

The distribution has been plotted according to eq. 3.13 with the parameters from 2017. This is depicted in Figure 35. When the Monte Carlo simulation will be done, the amplitude of the storm surge will randomly (but weighted by the probabilities of the values) be picked out of this distribution. This draw is then subjected to a normal distribution with a mean which is equal to the value drawn out of the extreme value distribution and a standard deviation of 10 % to account for the model uncertainty.

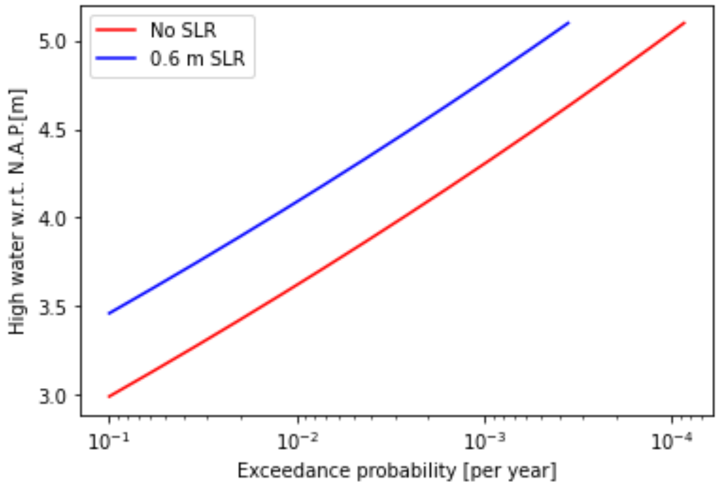


Figure 35: exceedance probability for various high-water levels with the 2017 GPD parameters

Figure 6 depicted how the closing frequency of the Maeslant barrier changed with sea level rise. For no sea level rise, the closure level (NAP + 3.00 m) of barrier occurs 0.1 times per year (once in 10 years). For 0.6 m sea level rise, the barrier closes 0.6 times per year (once in 1.67 years). This means that the closure level (NAP + 3.00 m) occurs once in 1.67 years.

To take 0.6 m sea level rise into account in the model, the high-water distribution according to equation 3.13 needs to be adjusted. This can be done by adjusting the parameter "u" in equation 3.13 to 3.05 m. With the updated parameter "u", the water level NAP + 3.00 m occurs once in 1.67 years in the model. The new exceedance probabilities for the situation with sea level rise are plotted in blue in Figure 35. It can be observed that the high-water levels have a higher exceedance probability with sea level rise.

3.5.2 Adapting the incoming river discharge

The incoming river discharge has a large impact on what happens to the water level in the basin when the Maeslant barrier is closed. The inflowing river discharge in the basin is the sum of the following components:

- The discharge of the Meuse
- The discharge of the Waal
- The discharge of the Nederrijn

Discharge of the Waal & Nederrijn

Each of these branches have their own exceedance probability for certain discharge values. However, if the discharge of the Rhine, measured at Lobith increases, the discharge in of the Nederrijn and Waal increase as well. This means that the discharge in the Waal and Nederrijn are dependent on the discharge of the Rhine, measured at Lobith. Table 10 summarises the dependencies of the river discharges. The dependencies of the discharges in the first column are checked against the discharges in the first row of the table. For example, the discharge of the Waal depends on the discharge of the Rhine. But the discharge of the Rhine is independent of the discharge of the Waal. Table 10 also indicates that high discharges in the Meuse do not occur simultaneously with high discharges of the Rhine. In reality, this can be the case if a large depression influences the flow area of both rivers. To model this is very complex and out of scope for this model, as several other stochastic parameters are required. Therefore, it is assumed that the discharge of the Meuse is independent of the discharge of the Rhine.

Discharge	Q_{meuse}	Q_{waal}	$Q_{Nederrijn}$	Q_{Rhine}
Q_{meuse}	-	Independent	Independent	Independent
Q_{waal}	Independent	-	Independent	Dependent
$Q_{Nederrijn}$	Independent	Independent	-	Dependent
Q_{Rhine}	Independent	Independent	Independent	-

Table 10: dependency table for the incoming river discharges

Figure 36 visually illustrates the dependence. The Boven-Rijn (Upper-Rhine) enters the Netherlands at Lobith. At the Pannerdense Kop, the Upper-Rhine splits into the Waal and the Pannerdenschcanal. At IJsselkop, the Pannerdenschcanal splits into the Ijssel and the Nederrijn. The IJssel flows into the IJsselmeer.

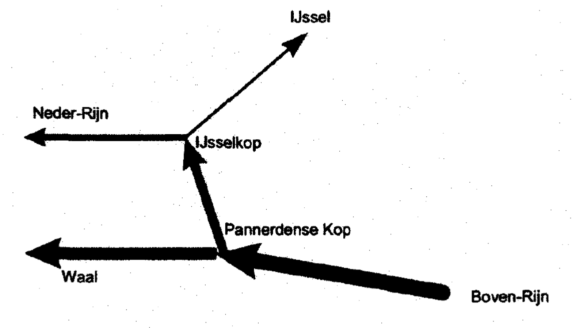


Figure 36: schematic overview of the Rhine branches (van der Most, 2000).

When high discharges occur, the discharge distribution across the branches depends on the geometry and bottom friction at the splitting points, on the summer- and winter riverbed profiles and the bottom friction of the river branches downstream of these splitting points. The discharge distribution across the branches is different for each discharge which is occurring because this depends on the contribution of the winter bed (van der Most, 2000).

For a discharge of 15,000 m^3/s in the period 1971-1980, the discharge was distributed as described in Table 11.

Rhine branch	**Discharge [m^3/s]**	**Percentage**
Upper rhine	***15,000***	***100 %***
To the Waal	9,550	64 %
To the Pannerdensch canal	5,450	36 %
Pannerdensch canal	***5,450***	***36 %***
To the Nederrijn	3,150	21 %
To the IJssel	2,300	15 %

Table 11: discharge distribution for the period 1971-1980 (van der Most, 2000)

However, Hermeling (2004) reported that due to autonomous bed level development, the discharge distribution would change. The changes are reported in Table 12.

Year	**Waal [m^3/s]**	**Nederrijn [m^3/s]**	**Ijssel [m^3/s]**
2020	+ 79.00	+ 15.00	- 94.00
2050	+ 128.00	+ 2.00	- 130.00

Table 12: changes in discharge distribution during governing high-water levels due to autonomous bed development (Hermeling, 2004)

During high discharge events (paired with high-water levels), bed level changes induce changes in the discharge distribution. This makes predicting the discharge distribution rather difficult during these high discharge events. Table 11 and Table 12 are being used to determine how much water each branch (the Waal and Nederrijn) gets from the Rhine. The values for the discharge itself are provided by Rijkswaterstaat (2018). Discharges up to 4,000 m^3/s have been measured, and discharges above 4,000 m^3/s have been calculated and extrapolated by the WAQUA model (Rijkswaterstaat, 2018).

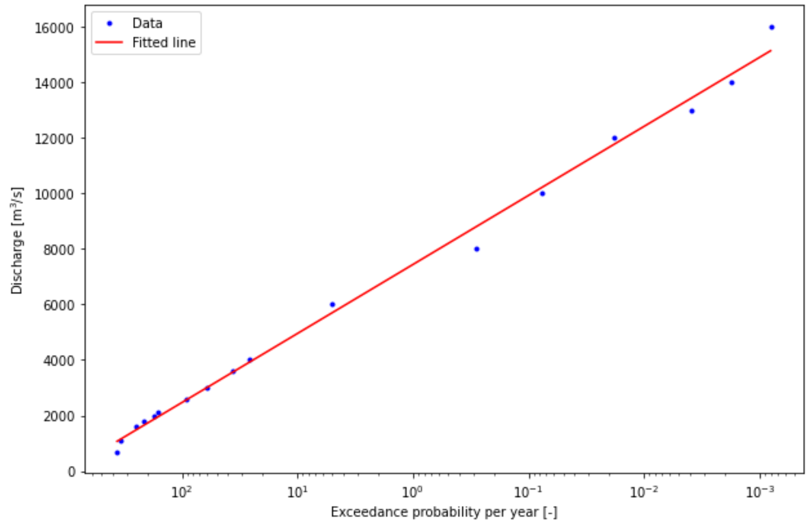


Figure 37: Exceedance probabilities for various discharges for the Rhine at Lobith

Rijkswaterstaat (2018) provided data (depicted as the blue dots in Figure 37) for the Rhine which includes the exceedance probabilities for a range of various discharges. Since the discharge distribution across the branches is known (the values in Table 11 and updating those with the values for 2020 mentioned in Table 12), an estimation can be made for the individual branches. The sum of the discharges of the individual branches will serve as inflow for the basin in the model. The exceedance probabilities for the Waal and Nederrijn are provided in Figure 38 and Figure 39 respectively.

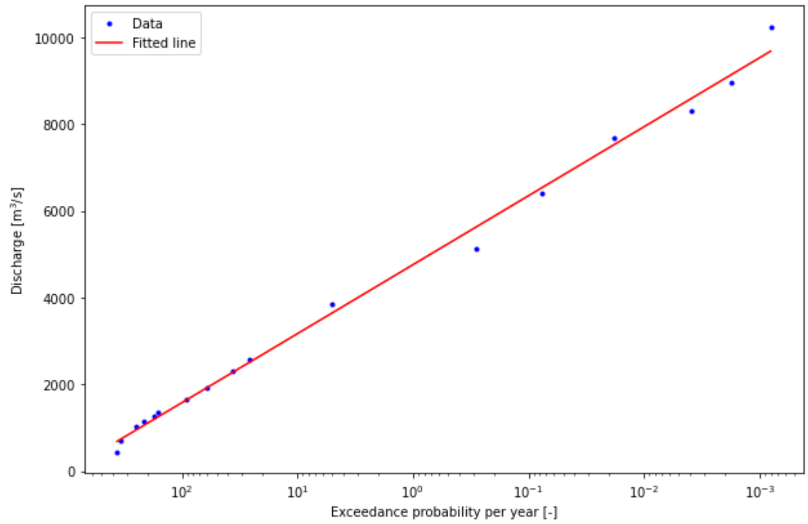


Figure 38: Exceedance probabilities for the calculated discharges for the Waal

The discharge of the Waal and Nederrijn both originate from the rhine and join in the basin before the Maeslant barrier. This also means that during the Monte Carlo simulation, a random (but weighted by their exceedance probabilities) discharge from the Rhine at Lobith will be picked (as they are the same for the branches). Then, the selected return period will be matched to the corresponding discharge per branche (as depicted in Figure 38 and Figure 39) to obtain the discharge values. This discharge value is then subjected to a normal distribution with a mean which is equal to the value drawn out of the extreme value distribution and a standard deviation of 10 % to account for the model uncertainty.

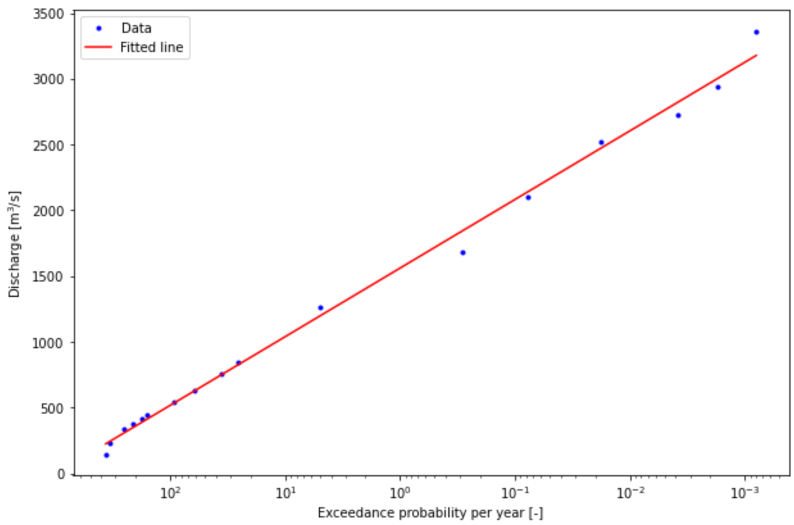


Figure 39: Exceedance probabilities for the calculated discharges for the Nederrijn

Discharge of the Meuse

Rijkswaterstaat (2019) also provided data (depicted in Figure 40) for the Meuse which includes the exceedance probabilities for a range of various discharges. Discharges up to 1,500 m^3/s have been measured and discharges larger than 1,500 m^3/s have been calculated and extrapolated using the WAQUA model.

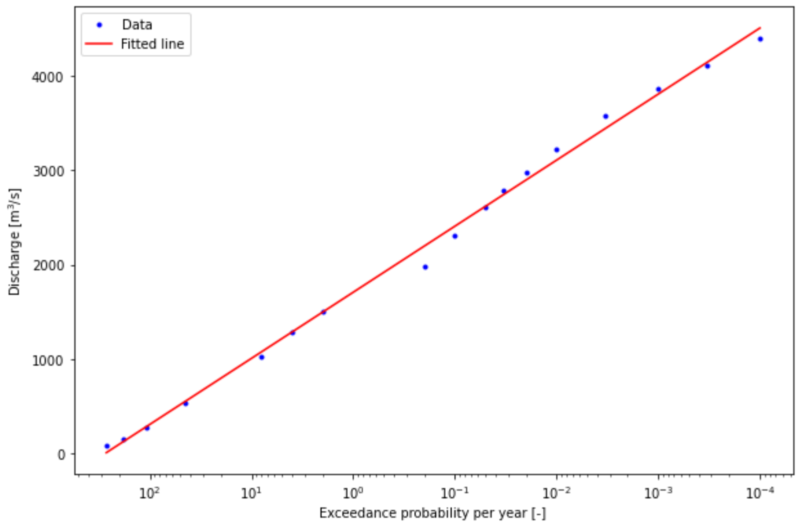


Figure 40: Exceedance probabilities for various discharges for the Meuse at St. Pieter

Since the Meuse is independent from the Rhine, a random discharge (but weighted by the probabilities) will be picked from the fitted line in Figure 40 for the Monte Carlo simulation (meaning it will not necessarily be a discharge with the same exceedance probability of the Waal and Nederrijn). The drawn discharge is then subjected to a normal distribution with a mean which is equal to the value drawn out of the extreme value distribution and a standard deviation of 10 % to account for the model uncertainty.

3.5.3 Adapting seiches

A seiche is also a probabilistic variable, as described in Section 2.3.2. Table 6 provided an overview for various seiche amplitudes with their corresponding probability of exceedance for a closed Maeslant barrier.

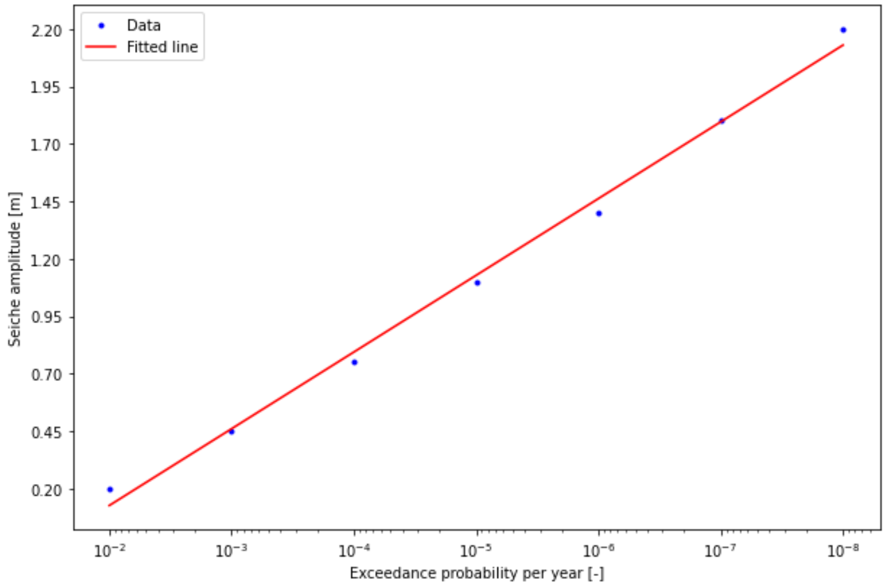


Figure 41: Exceedance probabilities for various seiche amplitudes (Rijkswaterstaat, 1995)

The values from Table 6 have been plotted in Figure 41, and a line has been fitted through these values. As reported in Section 2.3.2, seiches always occur during stormy weather, but not all storms necessarily induce a (high) seiche (de Jong, 2015). This means that the occurrence of seiches is (partially) dependent on the storm occurrence. If a storm occurs, high water at sea occurs. In reality, the weather type determines if a seiche will occur during a storm or not (de Jong, 2015). To model weather types and storm types is out of scope for this thesis, so for the model (and simplicity), it is assumed that during high water events (above a certain threshold, 2.5 m), seiches will occur.

In the simulation, a random (but weighted by the probabilities) seiche amplitude will be drawn from the extreme value distribution (but only if high water above the threshold is drawn from the distribution). This draw is then subjected to a normal distribution with a mean which is equal to the value drawn out of the extreme value distribution and a standard deviation of 10 % to account for the model uncertainty.

3.5.4 Adapting the basin area

The basin area was assumed to be constant. For the probabilistic analysis, an uncertainty for this area will be assumed. Initially, an uncertainty of 1% (= 2.7 km^2) will be assumed. It will also be assumed that this parameter is normally distributed. During the Monte Carlo simulation, a random area will be picked from the described distribution

3.5.5 Adapting the outflowing discharge

The outflowing discharge can be determined by which mode is desirable. Two modes can be desirable:

- **To maintain a target water level in the basin**
  The target water level in the basin is the closure level of the barrier. The idea is that the water level in the basin is being kept constant at this level. To accomplish this, the outflowing discharge should be approximately equal to the total incoming discharge. This means that the pumps of Delta21 do not pump at full capacity, but at the required capacity based on the inflowing discharge.

When the storm is over, the barrier can open again as soon as the outer water level is smaller or equal to the water level inside the basin (which will be approximately equal to the closure level of the barrier). Note: the barrier can only be opened if the predicted water level will not be higher than the closure level.

- **To discharge at maximum capacity**
  In this mode, when the barrier closes, the pumps of Delta21 operate at full capacity. According to Delta21 (2018), up to approximately 10,000 m^3/s can be pumped out of the basin if it is necessary. This will be done by 93 turbines which can be used as pumps when water needs to be pumped out. The 93 turbines/pumps operate in parallel, each with a capacity of approximately 107.52 m^3/s. So, if two or three pumps fail, it has a minor effect on the total pump capacity, as the remaining pumps can continue pumping. Therefore, an uncertainty of 2.5% (= 250 m^3/s) will be assumed for the outflowing discharge. It will also be assumed that the discharge is normally distributed as well, just like the basin area. During the Monte Carlo simulation, a random outflowing discharge will be selected from this distribution.

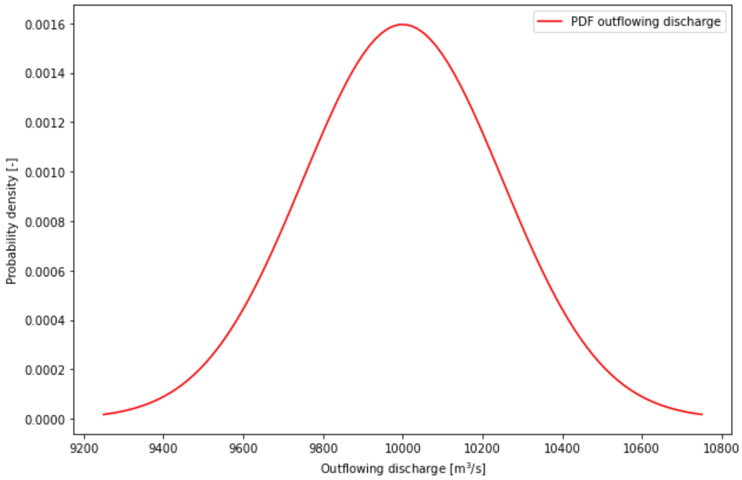


*Figure 42: PDF for the outflowing discharge with μ = 10,000 m^3/s and σ = 0.025*μ*

It could be better to use a lognormal distribution for the outflowing discharge and basin area, such that there is no possibility that a value larger than mean can be drawn (as this is physically not possible, the maximum capacity of the pumps or area of the basins cannot be exceeded). However, for this simple hydraulic model the normal distribution will be used.

3.5.6 Adapting the friction coefficient in the channel

The friction coefficient for the channel will have a normal distribution with a mean of 0.004 and a standard deviation of 1%. It should be said that the friction coefficient can vary along the channel in reality (depending on the depth, coarse or fine bed material, etc.)

3.5.7 The tidal signal

The tide will be assumed to be constant as described in Section 3.2. It is difficult to make this a stochastic variable. One could say that it is possible to make this a stochastic variable by taking an arbitrary value of the tide series and applying a normal distribution around this variable and picking a final value out of this distribution. This has been executed, but this increased the running time of the model considerably and result of the simulation (probability) was not sensitive to this. Therefore, it has been chosen to keep the tide as a deterministic variable.

3.5.8 Summary of the probabilistic variables

Table 13 summarizes the variables which were described above with their distributions and other parameters (if applicable). Additionally, a model uncertainty of 10 % will be assumed (normally distributed).

Variable	Distribution	Mean	Standard deviation
High water	Generalized Pareto distribution (GPD)	-	-
Seiches	Exponential	-	-
Incoming river discharge	Exponential	-	-
Outflowing discharge	Normal	Varies	2.5 %
Basin area	Normal	270	1 %
Tidal signal	Deterministic	-	-
Friction coefficient channel	Normal	0.004	1 %
Model uncertainty (Applied to all probabilistic variables)	Normal	-	10 %

Table 13: summary of the variables and their corresponding distributions

Table 14 provides an overview of the dependencies between the variables. The variables in the vertical column are checked for dependence against the variables in the first row. For example, the outflowing discharge depends on the incoming river discharge (up to 10,000 m^3/s) for mode 1, when a target water level needs to be maintained in the basins. But the incoming river discharge does not depend on the outflowing discharge. Another example is that seiches depend on high water (= occurrence of storms), but high water (= occurrence of storms) does not depend on seiches. As mentioned earlier, to account for the dependency between seiches and storms, it is linked to the high-water variable in the model. Since storm behavior and characteristics are not built into the model, the only way to link seiches and storms is through high-water events.

Variable	**Seiches**	**Incoming river discharge**	**Outflowing discharge**	**Basin area**	**High water**	**Tide**
Seiches	-	Independent	Independent	Independent	Dependent	Independent
Incoming river discharge	Independent	-	Independent	Independent	Independent	Independent
Outflowing discharge (Delta21)	Independent	Dependent (scenario 1) Independent (scenario 2)	-	Independent	Independent	Independent
Basin area	Independent	Independent	Independent	-	Independent	Independent
High water	Independent	Independent	Independent	Independent	-	Independent
Tide	Independent	Independent	Independent	Independent	Independent	-

Table 14: overview of the dependencies between variables

The implementation of the probabilistic variables in the programmed hydraulic model can be found in Appendix E.

3.6 Concluding remarks

The main goal for this chapter was to develop the hydraulic model and adapt the variables to make the model probabilistic. This has been accomplished by executing the defined steps mentioned in methodological step 2 in section 1.4:

1. Schematization of the real world into model elements
2. Use the equations to represent the model elements
3. Program the model based on the model schematization and equations
4. Verification of the programmed model
5. Validation of the programmed model
6. Adapt the model to become fully probabilistic (level III) & run the Monte Carlo simulation

In the next chapter, methodological step 3 will be addressed. The model will be run deterministically and probabilistically and for both cases the results will be analysed. The sensitivity of the model results to various parameters will also be identified.

4 Analysis of the model results

This chapter addresses methodological step 3. This chapter also answers sub question 2 as defined in Section 1.3. The model will be run in a deterministic and probabilistic mode, and the model results will be analysed. A sensitivity analysis for various parameters will also be done to highlight the importance of various parameters.

The model can be run in two modes:

- Mode 1: by maintaining a **target water level** in the basin
- Mode 2: by **discharging at maximum capacity** of the Delta21 pumps

Both modes have some advantages and disadvantages. The advantage of using mode 1 is that the duration of a closure event is most likely slightly shorter than if mode 2 is being used. For mode 2 (pumping at full capacity), pumping is stopped once the peak of the storm has passed to minimize the duration of a closure.

4.1 Deterministic model results

The verification and validation stages of the model showed that the model accurately reproduces a normal storm with the tide and seiches when the Maeslant barrier is closed, given the assumptions which were made when developing this model. A remark to be made is that the current operating rules of the Maeslant barrier have not been programmed into the model, so the results are only valid for the operating rules which were defined in this model.

The following input variables were used for the deterministic simulation:

- A total simulation time of 72 hours
- A storm set-up with a maximum amplitude of 3.00 m
- A storm duration of 35 hours
- Seiches with a variable amplitude and a period of 30 minutes
- The tide, implemented as described in Section 3.2.
- The storage area of 270 km^2 (including the additional area produced by Delta21)
- A friction coefficient c_f of 0.004.
- The inflowing discharge:
    - A discharge of 6,000 m^3/s (exceedance probability of 0.01 per year) for the Rhine
    - A discharge of 1,250 m^3/s (exceedance probability of 0.01 per year) for the Meuse
- An outflowing discharge of:
    - 6,350 m^3/s (= Q_{in}) for mode 1
    - 10,000 m^3/s for mode 2
- A closure level of NAP + 3.0 m (at Hoek van Holland). This means that when the barrier is finished closing, the water level is equal to approximately NAP + 3.0 m. The closing procedure itself is not included in the model. In reality, the barrier starts the closing procedure before the water level actually reaches NAP + 3.0 m, in such a way that the barrier is closed when the water level reaches NAP + 3.0 m.
- The barrier can open as soon as the outer water level drops below the closure level. However, it can be useful to open the barrier as soon as the water level

drops below the design value of the river dikes (assumed to be NAP + 3.0 m) behind the Maeslant barrier and if it does exceed the closure level. In this way, the barrier can open sooner, while minimising failure rates of the dikes in the hinterland. However, the inner and outer water levels need to be approximately equal to be able to open the sector gates.

Remark: the barrier currently needs to be closed if the expected water level at Rotterdam and Dordrecht will exceed NAP + 3.0 m and NAP + 2.9 m respectively. This is also the reason for setting the closure level at NAP + 3.0 m to represent the situation as accurate as possible (while considering the assumptions for the model).

Base run

To explain how the model results, a base run for mode 1 and mode 2 will be done and explained. The model output for mode 1 is depicted in Figure 43. In this run, no negative head difference larger than 1.5 m is observed.

```
Qin : 6350.0 m3/s
Qchannel : 1221.69 m3/s
Seiche amplitude : 0.2 m
Storm set-up amplitude: 3.0 m
Failure: 0
Time closed: 16 hours
```

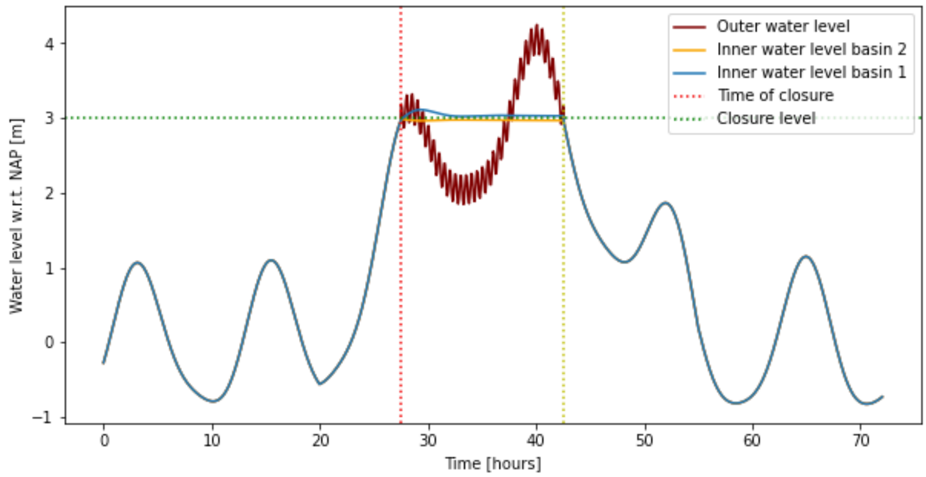


Figure 43: Model output for mode 1 with seiches with an amplitude of 0.20 m.

In Figure 43, the storm starts at T = 20 hours, which lasts until T = 44 hours. The blue line depicts the water level inside basin 1. The yellow line represents the water level in basin 2. The outer water level is represented by the red line, and the sharp peaks which can be seen on this red line represent seiches with a period 30 minutes. As this is a simulation done for mode 1, the water level inside the basin stays approximately constant.

Above each graph, the values of the relevant parameters are reported (the total incoming discharge, the discharge in the channel the amplitude of a seiche, the amplitude of the storm set-up, if failure occurs (1) or not (0) and the duration of the closure).

Figure 44 represents the model output for mode 2. The water levels in the basins 1 and 2 drop because the total incoming discharge is smaller than the outgoing discharge (the

Delta21 pumps). When pumping is stopped at T = 40 hours in basin 2, the water level in basin 1 lags a bit behind. This is caused by the friction and limitations of the channel.

```
Qout : 10000.0 m3/s
Qin : 6350.0 m3/s
Qchannel : 1673.13 m3/s
Seiche amplitude : 0.2 m
Storm set-up amplitude: 3.0 m
Failure: 0
Time closed: 16 hours
Head difference when opening 0.093 m
```

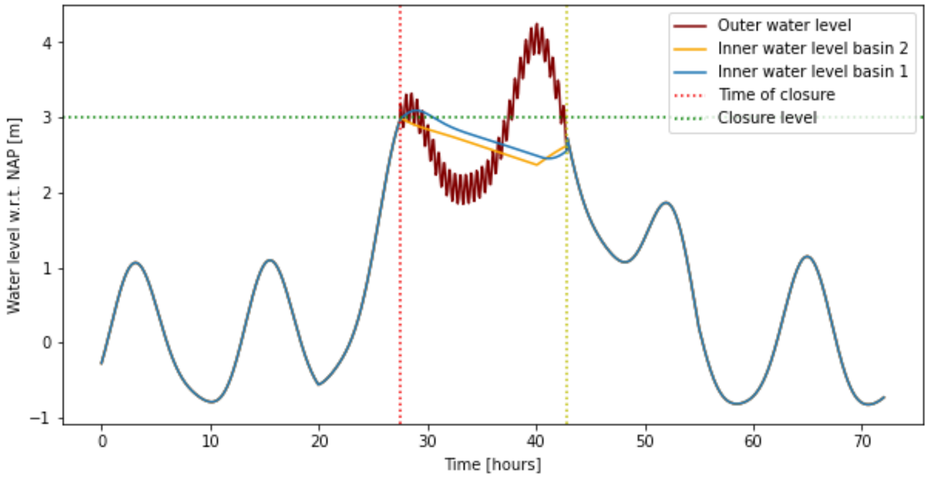


Figure 44: model output for mode 2 with seiches with an amplitude of 0.20 m

In both runs, the water level inside the basin shortly increases first after the barrier is closed, before dropping again. This is because the outflowing discharge in basin 1 depends on the discharge in the channel (which connects basin 1 to basin 2). In the beginning, the head difference between the basins is small. This means that the channel cannot discharge water out of basin 1 until the water level in basin 2 drops sufficiently enough.

4.2 Sensitivity of the model results due to various parameters

In the deterministic runs, the value for each parameter can be adjusted manually for each parameter. By adjusting a parameter in each run (while keeping the other parameters unchanged), the sensitivity of the model results to the adjusted parameter could be identified. Several deterministic runs have been done to highlight the influence of key parameters. In those runs, the following parameters were varied:

- **The friction coefficient**
  As mentioned in section 3.2, the value for the friction coefficient is in the range between 0.002 and 0.006. A mean value of 0.004 would be used in the simulations. Changing the value to 0.002 or 0.006 did not produce significant changes in the model output.
- **The seiche amplitude**
  If the seiche amplitude is increased, failure is more likely as the water level on the seaside drops even more. When using a target water level (mode 1), failure occurs earlier than when pumping at full capacity (mode 2) with the increased seiche amplitude.
- **The outer water level by adding sea level rise:**

If no pump capacity is present, failure occurs for a sea level rise of 0.5 m and 1m with an incoming river discharge of 6,350 m^3/s as described in section 4.1. With pump capacity of Delta21, failure does not occur anymore with the same incoming discharge. This illustrates the importance of the pump capacity.

- **The incoming river discharge & closure level:**
  If extremely high incoming river discharges occurs (>10,000 m^3/s), failure can be avoided by choosing a lower closure level.

A full explanation for the sensitivity analysis for each parameter mentioned above along with water level graphs can be found in Appendix C.

Conclusions after the sensitivity analysis

Three main conclusions can be drawn after the sensitivity analysis:

1. **Higher seiche amplitudes will cause more failures**
2. **The pump capacity has no negative effects on the Maeslant barrier**
   The Maeslant barrier only benefits from the pump capacity.
3. **The higher the expected river discharges are, the lower the closure level needs to be**
   The significance of choosing the closure level has been demonstrated extensively in the runs which were done. The higher the expected river discharges are, the lower the closure level needs to be to avoid failure. This conclusion is confirmed in Figure 45. Even extremely high river discharges can be handled with the pump capacity of Delta21 if the right closure level (lower is better) is chosen. The only downside of choosing a lower closure level is the duration of a closure, but safety goes above all.

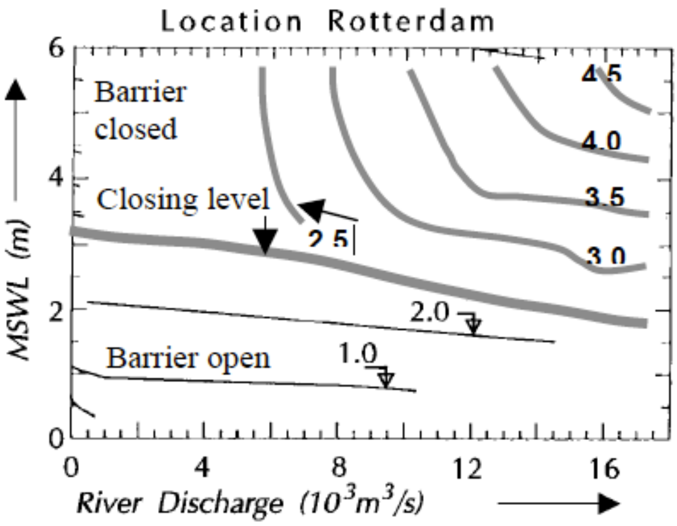


Figure 45: water levels at Rotterdam as a function of the river discharge (Janssen).

4.3 Probabilistic model results

The difference between the probabilistic and deterministic results is that for every probabilistic run, different values for the parameters are automatically chosen, while in the deterministic runs the values of the parameters stay the same unless they are changed by hand.

Figure 46 and Figure 47 illustrate **one** random (out of the 10^5 – 10^6 runs) probabilistic run (for mode 1 and 2) in which the values for the variables are picked out of their corresponding distributions, as defined in Section 0.The picked parameters used in this corresponding run is also reported above the graph. A closure level of NAP + 3.0 m was used for the probabilistic calculations. If runs with other closure levels are desired, this can be done quite easily by adjusting the closure level and running the model again. The main reason for including Figure 46 and Figure 47 is to illustrate that the model works when values for the parameters is chosen randomly out of the corresponding distributions per parameter.

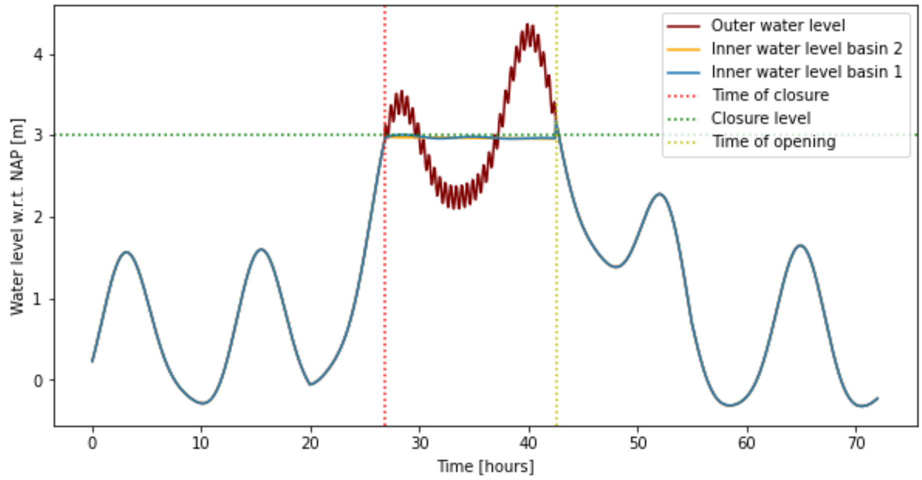


Figure 46: probabilistic model output for mode 1 with the parameters used

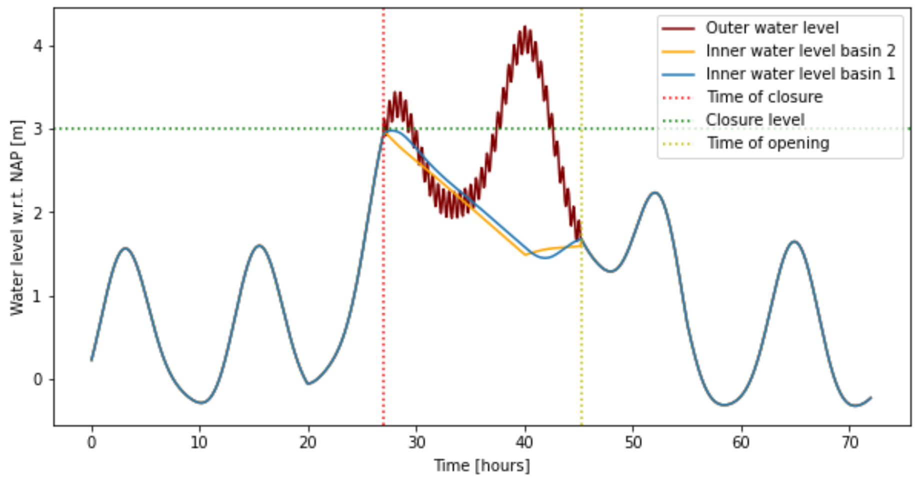


Figure 47: probabilistic model output for mode 2 with the parameters used

To obtain the desired probability (probability of a negative head difference (N.H.D.) larger than 1.5 m), a Monte Carlo simulation has been done. This has been done by running the model in mode 1, and after this in mode 2.

Six Monte Carlo simulations have been done:

1. For the current situation (no pumping) with no sea level rise (without D21).
2. For the current situation (no pumping) with 0.6 m sea level rise (without D21).
3. While maintaining a target water level with no sea level rise (with D21).
4. While maintaining a target water level with 0.6 m sea level rise (with D21).
5. While pumping at full capacity with no sea level rise (with D21).
6. While pumping at full capacity with 0.6 m sea level rise (with D21).

Simulation nr. [-]	**Probability of a N.H.D. > 1.5 m** [per year]	**Closure frequency** [per year]	**Probability of a N.H.D. > 1.5 m** [per closure]
1	$6.40*10^{-3}$	$1*10^{-1}$	$6.40*10^{-2}$
2	$1.82*10^{-2}$	$6*10^{-1}$	$3.00*10^{-2}$
3	$6.70*10^{-4}$	$1*10^{-1}$	$6.70*10^{-3}$
4	$1.78*10^{-3}$	$6*10^{-1}$	$2.96*10^{-3}$
5	$5.50*10^{-4}$	$1*10^{-1}$	$5.50*10^{-3}$
6	$1.05*10^{-3}$	$6*10^{-1}$	$1.75*10^{-3}$

Table 15: resulting probabilities from the Monte Carlo simulations

The results have been summarized in Table 15. The probabilities for the negative head difference larger than 1.5 m per year have been obtained by running 10^5 runs in the model and counting in how many runs the negative head difference was larger than 1.5 m. The probability is then obtained by dividing the number of fails by the total number of runs. The probabilities (with unit per year) for the runs with sea level rise are higher since the barrier must close more often. To convert the probabilities to the unit [per closure], the probabilities per year are divided by the closure frequency. These probabilities are reported in the last column in Table 15

The main conclusions which can be drawn from Table 15 are:

1. In the runs with Delta21, the probability of a negative head difference larger than 1.5 m is almost a factor 10 lower than in the current situation without Delta21.
2. By pumping at full capacity, the probability that a negative head difference larger than 1.5 m occurs is slightly lower.
3. The simulated sea level rise of 0.6 m **increased** the probability that a negative head difference larger than 1.5 m occurs per year.

Thus, goal of the model has been accomplished and the calculated probabilities can be used in the failure probability analysis which will be elaborated in the next chapter. Some additional conclusions can be drawn from the analysis of the runs in which a negative head difference larger than 1.5 m occurred.

Analysis of the runs in which the barrier structurally failed

For both modes, the runs in which a negative head difference larger than 1.5 occurred will be analysed to see under which conditions a negative head difference larger than 1.5 m occurs. The discharge, storm amplitude and seiche amplitudes were analysed, as these have the biggest impact on the outcome of the simulation. These have a very low probability, because in all the failed runs one of the parameters has a low exceedance frequency (per year). This means that it is very unlikely that such an event can happen.

The analysis of the failed runs can be found in Appendix A. The conclusions from the analysis in appendix A:

- If low storm amplitudes (low high-water levels) and high seiche amplitudes occur at the same time, more negative head differences > 1.5 m are observed.
- If low seiche amplitudes and high river discharges occur at the same time, more negative head differences > 1.5 m are observed.
- In the current situation, seiches with an amplitude in the range [0.20 m, …,0.70 m] cause negative head differences > 1.5 m, while in the Delta21 situation only seiches with an amplitude in the range [0.35 m, …, 0.70 m] cause failure. It can be concluded that seiches with an amplitude in the range [0.20 m, …, <0.35m] do not cause failure anymore in the situation with Delta21.

4.4 Concluding remarks

The goal of this chapter was to analyse the model results of the deterministic runs and the probabilistic runs. The sensitivity of the model results to various parameters have also been identified. The following conclusions can be drawn after running the model **deterministically** and doing the sensitivity analysis:

1. Higher seiche amplitudes will likely cause more failures
2. The pump capacity of Delta21 has no negative effects on the Maeslant barrier
3. The higher the expected river discharges are, the lower the closure level of the Maeslant barrier needs to be.

After running the model **probabilistically**, the following conclusions have been drawn:

1. In the runs with Delta21, the probability of a negative head difference larger than 1.5 m is almost a factor 10 lower than in the current situation (without Delta21).
2. If Delta21 is pumping at full capacity, the probability that a negative head difference larger than 1.5 m occurs is slightly (22%) lower than when a target water level is being maintained.
3. The simulated sea level rise of 0.6 m **increased** the probability that a negative head difference larger than 1.5 m occurs per year.
4. If low storm amplitudes (low high-water levels) and high seiche amplitudes occur at the same time, more negative head differences > 1.5 m are observed.
5. If low seiche amplitudes and high river discharges occur at the same time, more negative head differences > 1.5 m are observed
6. In the current situation, seiches with an amplitude in the range [0.20 m, …,0.70 m] cause negative head differences > 1.5 m, while in the Delta21 situation only seiches with an amplitude in the range [0.35 m, …, 0.70 m] cause failure. It can be concluded that seiches with an amplitude in the range [0.20 m, …, <0.35m] do not cause failure anymore in the situation with Delta21.

5 Failure probability analysis of the Maeslant barrier in the Delta21 configuration

This chapter addresses methodological step 4 in Section 1.4, which verifies if Delta21 indeed reduces failure probability of closure of the Maeslant barrier. Additionally, this chapter provides answers to the main research question, sub question 3 and 4 as defined in Section 1.3. The results of this failure probability analysis are very optimistic, and the reduction of the failure probabilities cannot be any higher (the reductions of the new probabilities have been rounded up to the upper bound).

The failure probability analysis will be done according to the methodology which was described in Section 1.4, and will be done on a high level of abstraction. This means that the analysis will be performed on top level considering the global systems of the Maeslant barrier and Delta21, without considering the "small" components (for example electrical components on motherboards, or mechanical components in the driving mechanism of the sector gates). Therefore, a top-down approach is preferred instead of a bottom-up approach.

5.1 System definition

The failure probability analysis concerns the Maeslant barrier. As described in Section 1.2, it is a complex structure with a lot of components and processes. The components and processes of the Maeslant barrier will be decomposed into three main categories:

- Systems: contains the operating mechanisms.
- Structures: contains the structural elements.
- Operations: contains the decision-making processes and human actions.

Systems

The Maeslant barrier contains a lot of operating mechanisms, but since the risk analysis is done on a high level of abstraction, only the main operating mechanisms will be named:

- The ball joints
- The ballast system
- The locomobile
- The dock doors

Structures

The following structural elements have been categorized:

- The foundation of the ball joint
- The gates (the retaining wall and truss arms)
- The bed protection

Operations

The following operational elements have been categorized:

- The BOS system
- The BESW system
- Human recovery actions

- Maintenance carried out by humans

The elements in the three categories are all related/coupled with each other through the control system BESW. BESW sends out commands to initiate and control actions when this is required. To complete the required action, elements from all three categories work together.

5.2 Qualitative analysis

In this section, the failure mechanisms and failure scenarios will be identified and described.

As mentioned in Section 2.2, not only the failure probability of closure is important, but also the failure probability of opening for very large river discharges. The barrier could also fail to open if the pumps of Delta21 fail, as the water levels behind the barrier would rise considerably if no water would be pumped out of the storage area. Failure of the Delta21 pumps could not only fatal for the Maeslant barrier, but also for the dikes in the hinterland (behind a closed Maeslant barrier). If the water level keeps rising because no water can be discharged, eventually the water level will be higher than the design water level of the dikes. This can lead to failure of the dikes which results in flooding of the areas behind the dikes.

In the qualitative analysis, the ultimate limit state (ULS) of the Maeslant barrier will be analysed. In this state, the barrier can no longer fulfil its main design critera which is to prevent the hinterland from flooding during storms.

An important definition for this section is to define when the barrier is closed: the barrier is closed if the sector gates are closed horizontally **and** if the barrier fully submerged.

The Maeslant barrier consists of many components and processes, as mentioned in section 5.1. There are many scenarios that failure of one of these components or processes could lead to flooding of Rotterdam. These scenarios will be covered in the next two sub-sections. First, failure scenarios for the Maeslant barrier in the current situation will be covered, followed by the failure scenarios for the Maeslant barrier in the Delta21 configuration.

5.2.1 Failure mechanisms

Since the barrier consists of many systems, structures, and operations (as defined in section 5.1), failure of one of these systems, structures or operations could lead to failure to close (or open) of the barrier. The event “failure to close” happens when there is a closure request, but the barrier is still parked in the parking dock and cannot close because one of the systems, structures or operations has failed. It will be explained how failure of one the following **systems** could lead to the event “failure to close”:

- Ballast system failure
- Locomobile failure
- Dock door failure

It will also be explained how failure of the following **operations** could lead to the event "failure to close":

- The BOS system
- The BESW system

When one of the **structures** (the foundation of the ball joint or the trusses of the sector gates for example) fails, this is classified as structural failure. Next, the failure of the mentioned systems and operations will be explained.

Ballast system failure
The ballast system makes sure that the sector gates can be immersed when the barrier closes, and to let the sector gates lift up when the barrier needs to open again. Thus, it consists of a submerge- and uplift system. If one of these systems fail, the event "failure to close" can occur.

Locomobile failure
Lastly, the locomobile can directly cause "failure to close" because the sector gates cannot be closed horizontally. If the locomobile fails, it is not likely that it can be fixed quickly. Since it is an old machine, it is assumed that not all the parts have spare parts directly available for replacement.

Dock door failure
The dock doors could also fail to open when they are required to open. In a situation where this could occur and the prediction is that it cannot be repaired before the storm arrives, the dock door can be demolished in the worst-case scenario. Then, the barrier could still proceed to close and protect the hinterland. After the storm, the dock doors can be replaced as soon as possible such that maintenance can be done on the retaining wall of the sector gates for example.

BOS and BESW failure
BESW controls all the systems of the Maeslant barrier. If BESW fails, the systems cannot not perform operation, as they have not been instructed to do so by BESW. Human interference is needed in this case. If BOS fails to give out a closure command to BESW, the barrier will not close without human interference. This causes a "failure to close". BOS failing to give a closure command when it is needed can be because of incorrect predictions for example.

5.2.2 Failure scenarios

A failure scenario occurs when the failure mechanisms (which have been described in the previous section) in combination with some additional events cause the event "flooding of Rotterdam". These additional events are:

- A second peak arrives in the same storm with a water level $h_{sea} > 3.6$ m.
- A second storm with water level $h_{sea} > 3.6$ m arrives before the repair of the damaged barrier is complete.

Figure 48 depicts a cross section of the flood protection system for Rotterdam. It makes it clear why the water level NAP + 3.6 m is important. If the Maeslant barrier fails, flooding of Rotterdam will occur if $h_{sea} >$ NAP + 3.6 m (height of the river dike).

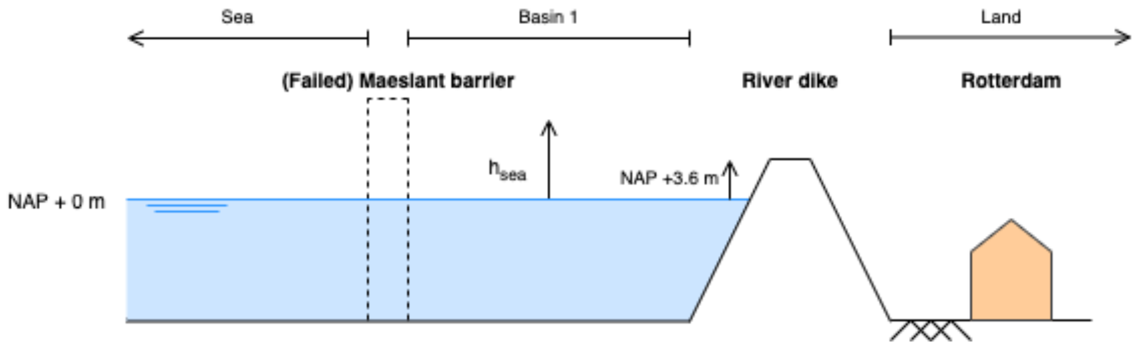


Figure 48: cross section of the flood protection system for Rotterdam, adapted and modified from Mooyaart (2022)

The failure scenarios will be called flooding scenarios. Now that all the failure mechanisms and additional events are known, the flooding scenarios can be determined. Some of the flooding scenarios (for Rotterdam) for the Maeslant barrier in the current situation and the new situation with Delta21 are identical, and some will change.

Flooding scenarios current situation

First, the flooding scenarios for the Maeslant barrier in the current situation will be described. An important reminder is that the Maeslant barrier in the current situation **can discharge** water during a storm when it is needed (see section 1.2.3 for further explanation). **Flooding of Rotterdam** for the Maeslant barrier in the current situation can happen because of 5 scenarios:

- **Flooding scenario 1**
  Flooding of Rotterdam happens because the barrier fails to close due to one of the failure mechanisms mentioned in the previous section, and a storm arrives with a water level h_{sea} > NAP + 3.6 m
- **Flooding scenario 2**
  Flooding of Rotterdam happens because of structural failure
- **Flooding scenario 3**
  The barrier is closed during a storm and there is an uplift request. The uplift system of the ballast system fails: the sector gates cannot be lifted to allow water to flow beneath it towards the sea. This causes water to build up behind the barrier. Eventually, the negative head difference will be larger than 1.5 m, which causes the ball joint to fail structurally. During the same storm, a second peak is expected with a water level h_{sea} > NAP + 3.6 m, which causes flooding of Rotterdam. It is assumed that the ball joint cannot be repaired during the storm before the second peak arrives
- **Flooding scenario 4**
  This scenario is almost identical to flooding scenario 3. The barrier is closed during a storm and there is an uplift request. The uplift system of the ballast system fails: the sector gates cannot be lifted to allow water to flow beneath it towards the sea. This causes water to build up behind the barrier. Eventually, the negative head difference will be larger than 1.5 m, which causes the ball joint to fail structurally. During the same storm, a second peak is expected, but with a water level h_{sea} < NAP + 3.6 m. Thus, there will be no flooding of Rotterdam during the storm. The barrier can be repaired after the storm. However, if a new

storm arrives with a water level h_{sea} > NAP + 3.6 m within the repair time, flooding of Rotterdam will occur, since the barrier could not close.

- **Flooding scenario 5**
  This scenario is almost identical to scenario 4. The barrier is closed during a storm, and the storm is over and there is an uplift request to open the barrier. The uplift system of the ballast system fails: the sector gates cannot be lifted to allow water to flow beneath it towards the sea. This causes water to build up behind the barrier. Eventually, the negative head difference will be larger than 1.5 m, which causes the ball joint to fail structurally. After the storm, the water level h_{sea} < NAP + 3.6 m. Thus, there will be no flooding of Rotterdam after this storm. The barrier can be repaired after the storm. However, if a new storm arrives with a water level h_{sea} > NAP + 3.6 m within the repair time, flooding of Rotterdam will occur, since the barrier could not close. This scenario can also be seen as the event “failure to open”.

Flooding scenarios for the Delta21 configuration
Lastly, the flooding scenarios for the Maeslant barrier in the Delta21 configuration will be described. An important reminder is that the Maeslant barrier **cannot discharge** water during a storm in the Delta21 configuration. **Flooding of Rotterdam** for the Maeslant barrier in the Delta21 configuration can happen because of 5 scenarios:

- **Flooding scenario 1**
  Flooding of Rotterdam happens because the barrier fails to close due to one of the failure mechanisms mentioned in the previous section, and a storm arrives with a water level h_{sea} > NAP + 3.6 m
- **Flooding scenario 2**
  Flooding of Rotterdam happens because of structural failure
- **Flooding scenario 3**
  The barrier is closed during a storm and a negative head difference larger than 1.5 m occurs, which causes the ball joint to fail structurally. During the same storm, a second peak is expected with a water level h_{sea} > NAP + 3.6 m, which causes flooding of Rotterdam. It is assumed that the ball joint cannot be repaired during the storm before the second peak arrives.
- **Flooding scenario 4**
  This scenario is almost identical to flooding scenario 3. The barrier is closed during a storm and a negative head difference larger than 1.5 m occurs, which causes the ball joint to fail structurally. During the same storm, a second peak is expected, but with a water level h_{sea} < NAP + 3.6 m. Thus, there will be no flooding of Rotterdam during the storm. The barrier can be repaired after the storm. However, if a new storm arrives with a water level h_{sea} > NAP + 3.6 m within the repair time, flooding of Rotterdam will occur, since the barrier could not close.
- **Flooding scenario 5**
  This scenario is almost identical to scenario 4. The barrier is closed during a storm, and the storm is over and there is an uplift request to open the barrier. The uplift system of the ballast system fails: the sector gates cannot be lifted to allow water to flow beneath it towards the sea. This causes water to build up behind the barrier. Eventually, the negative head difference will be larger than

1.5 m, which causes the ball joint to fail structurally. After the storm, the water level h_{sea} < NAP + 3.6 m. Thus, there will be no flooding of Rotterdam after this storm. The barrier can be repaired after the storm. However, if a new storm arrives with a water level h_{sea} > NAP + 3.6 m within the repair time, flooding of Rotterdam will occur, since the barrier could not close. This scenario can also be seen as the event "failure to open".

Event trees

In the following event trees, it is assumed that the ball joint fails when the negative head difference is larger 1.5 m. Flooding scenario 3 and 4 can be represented in an event tree which makes it easier to understand. This is depicted in Figure 49**.** This event tree holds for the situation with Delta21, not for the current situation. The red and amber dashed lines indicate the paths for scenario 3 and 4 respectively.

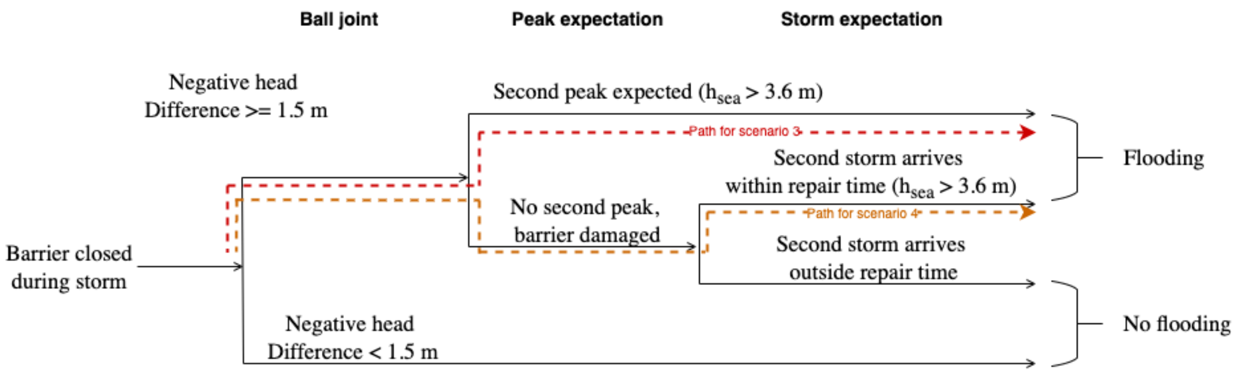


Figure 49: event tree for flooding scenario 3 and 4 due to failure of the ball joint in the Delta21 configuration

Flooding scenario 5 can also be represented in an event tree. This is depicted in Figure 50. This event tree holds for the configuration with Delta21, but also for the current configuration. The events required for flooding scenario 5 are the same for both configurations, but only their probabilities change. The dashed line marks the path for scenario 5 to occur.

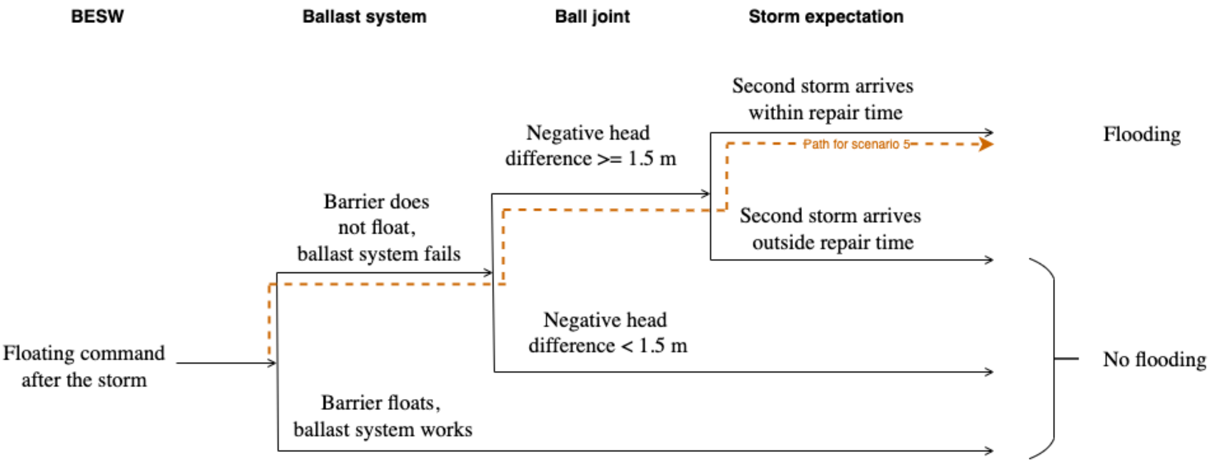


Figure 50: event tree for flooding scenario 5 due to failure of the ballast system in the Delta21 configuration

Based on the failure mechanisms, flooding scenarios and reasoning, a fault tree can be composed for the event "Flooding of Rotterdam" with the Maeslant barrier in the current situation (depicted in Figure 51), and one with Maeslant barrier in the Delta21 configuration (depicted in Figure 52).

Qualitative analysis

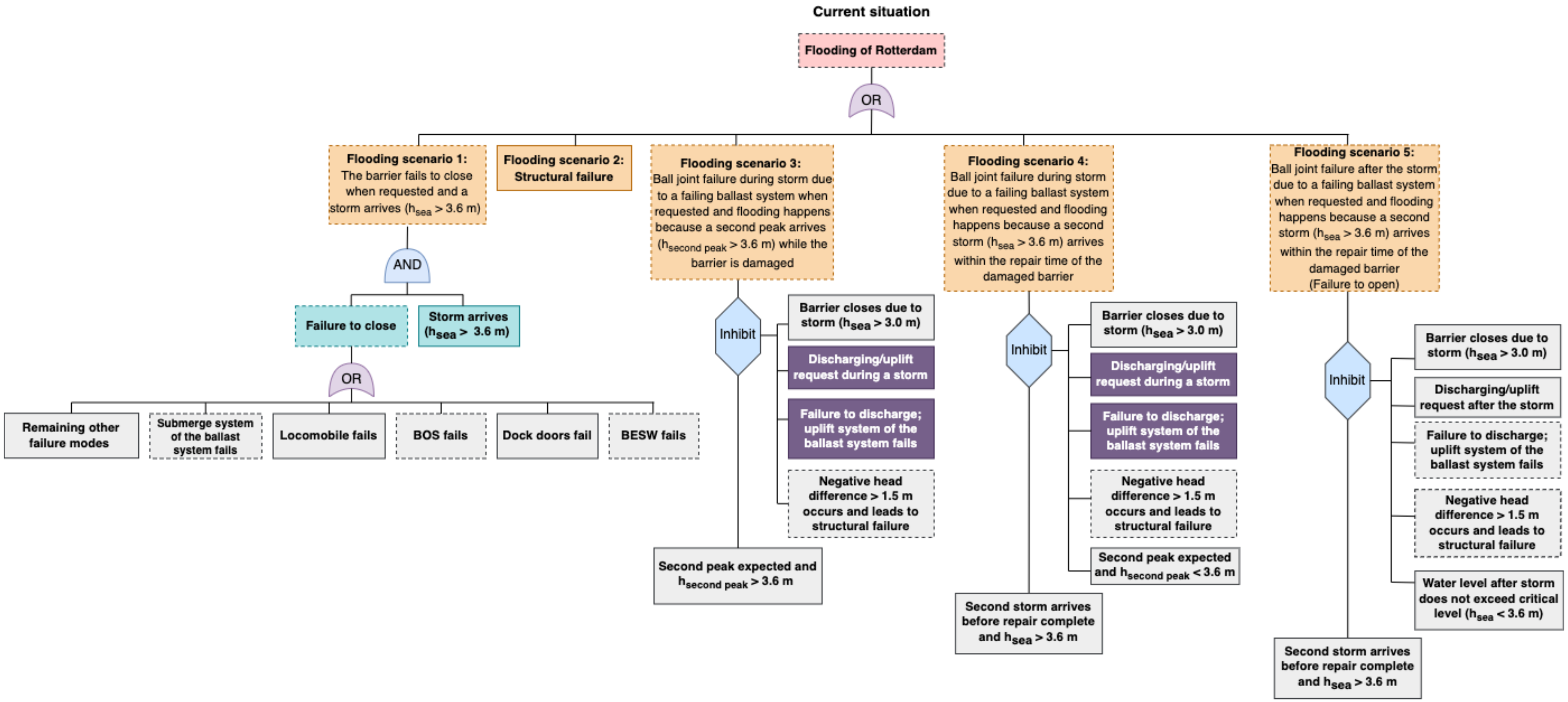


Figure 51: fault tree for flooding of Rotterdam in with the Maeslant barrier in the current configuration

The events highlighted in purple in flooding scenarios 3 and 4 are removed in the situation with Delta21. Furthermore, the probabilities of the events with a dashed border are changing in the situation with Delta21.

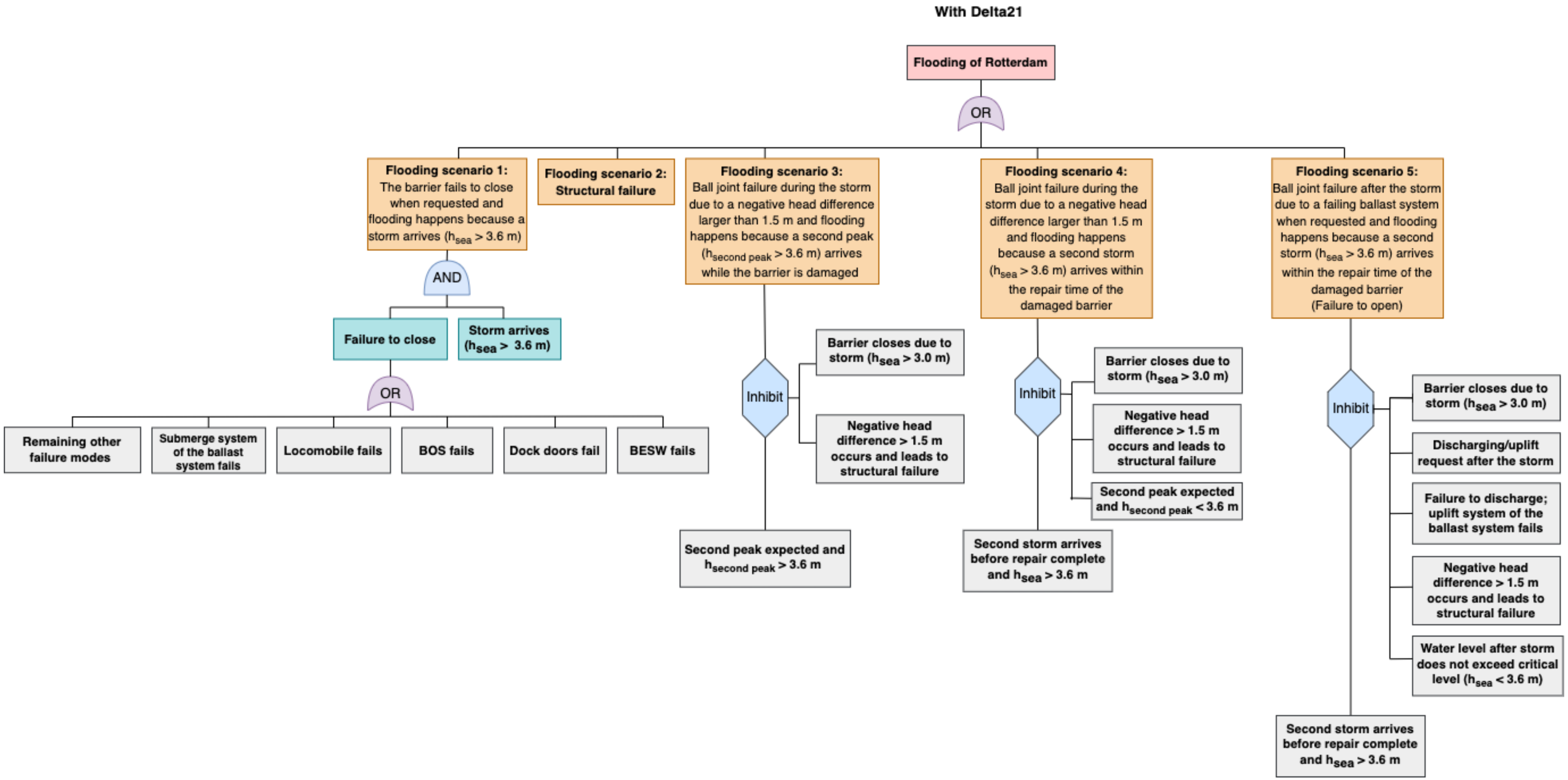


Figure 52: fault tree for flooding of Rotterdam in with the Maeslant barrier in the Delta21 configuration

Figure 52 is the resulting fault tree for the situation with Delta21. The events which were highlighted in purple in Figure 51 have been left out in the fault tree for the situation with Delta21, as reported.

5.3 Quantitative analysis

In this section, the probabilities of the defined failure mechanisms and failure scenarios will be determined. Rijneveld (2008) provided failure probabilities for the main components of the Maeslant barrier[2] in the current configuration. These will be used in to determine the failure probabilities for the main components in the Delta21 configuration.

Probabilities of the failure mechanisms

First, the probabilities for the failure mechanisms mentioned in the previous section in the Delta21 configuration will be determined.

Ballast system failure

In the situation with Delta21, the ballast system is going to be used only twice in one closure event (to immerse the sector gates when the barrier needs to close, and to let them float again when the barrier needs to be opened). In the current situation, the ballast system can be used during closure to let the sector gates float up in case of a (critical) negative head difference to discharge water in between storm peaks (at low water). The main conclusion is that the ballast system is going to be used less often. This is depicted in Figure 53. The number of operations by the ballast system is reduced with at most 50% for the case that there needs to be discharged only one time during a storm.

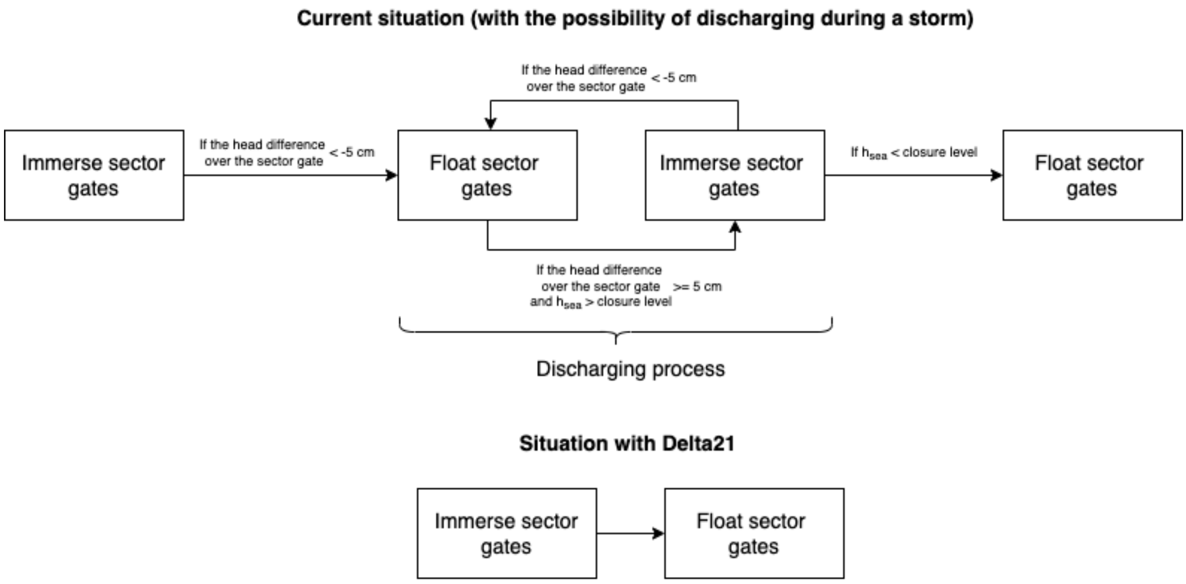


Figure 53: reduction in operations for the ballast system

This means that contribution of the ballast system to the failure probability of closure reduces. However, it needs to be noted that the probability of a discharging request during a storm is around 1% per closure. The probability of a discharging request during a storm for this analysis has been calculated with the developed hydraulic model and was estimated to be 26% per closure. When applying 0.6 m sea level rise, this increased to 57% per closure. These probabilities will be used in the calculations in the quantitative analysis. Rijneveld (2008) showed that the probability that the ballast

[2] The probabilities for non-availability due to not noticeable failure will be used in the quantitative analysis

system is not available due to not noticable failure was $1.25*10^{-3}$ per closure for the current situation. For the case without sea level rise, this is $4.8*10^{-3}$ per discharging request, and with sea level rise $2*10^{-3}$ per discharging request for the current situation. As said earlier, these probabilities are being reduced with at most 50% for the Delta21 configuration. So this equals $2.4*10^{-3}$ per discharging request without sea level rise, and $1*10^{-3}$ for the case with sea level rise for the Delta21 configuration. As mentioned in section 5.1, the ballast system consists of a submerge- and uplift system. It is assumed that each system contributes evenly to the total failure probability of the ballast system, so the failure probability of the submerge system is 50% of the total failure probability of the ballast system. The same holds for the uplift system.

BOS failure

As reported in Section 1.2.4, BOS uses less sources of information to decide whether to close the barrier or not. Each source of information has its own uncertainty which ultimately influences the failure probability of BOS. If less sources of information are being used, less uncertainty is present, which should lower the failure probability of BOS). This is depicted in Figure 54.

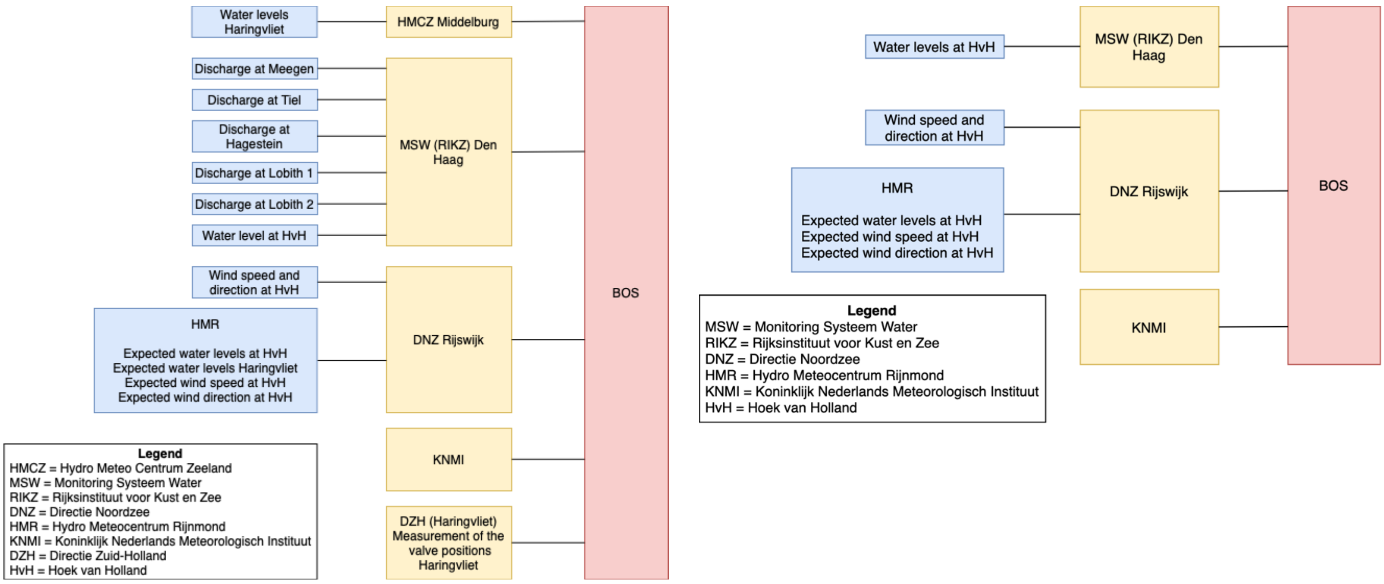


Figure 54: BOS without (left) and with (right) Delta21

Rijneveld (2008) reported that the failure probability of BOS is $1.90*10^{-4}$ per closure. This probability is the total failure probability for non-availability of BOS. This probability is not only due to the uncertainties of the information sources which are used to make predictions, but also of the reliability of the software and hardware which is used to operate BOS. Since the hardware doesn't change, it can be said that the change in the failure probability of non-availability of BOS is determined by the change in reliability of the software.

Looking at the current situation (Figure 54), a total of twelve information sources are being used to make the decision of whether to close the barrier or not. In the new situation, this is reduced to five information sources. Thus, that is a reduction of 58%. Assuming that every information source contributes evenly to the failure probability of BOS (and the previous assumption), it can be said that the failure probability in the situation with Delta21 is equal to $7.91*10^{-5}$ per closure. This is a reduction of 58% compared to the current failure probability of BOS. As said in Section 1.2.4, the sources of information that can be removed for BOS for the Maeslant barrier, need to be included for the control system for Delta21.

BESW failure

To see the effects of Delta21 on BESW, the process scheme needs to be analyzed. Figure 55 depicts the process scheme for the Maeslant barrier in the current situation. In almost every operation, BESW gives commands to the different elements of the barrier to make the required operation happen. Once the barrier is ready for closure (the orange box), it gets very complex. The procedure in which the sector gates are being floated out (blue boxes) is a very complex process which depends on a lot of conditions. With Delta21, this entire process can be simplified a lot. A possible new process scheme for the situation with Delta21 is depicted in Figure 56.

Since the hardware that powers BESW doesn't change, it can be said that the change in the failure probability of non-availability of BESW is determined by the change in reliability of the software. The number of operations in which BESW is needed between "barrier on standby" and "retain water" has been reduced to 4 (see Figure 56). In the current situation, the number of possible operations in which BESW is needed between "barrier on standby" and "retain water" is 9 (see Figure 55). Assuming that every operation contributes evenly to the failure probability of BESW, it can be said that the failure probability of BESW in the new situation with Delta21 is reduced with about 56%. Rijneveld (2008) reported that the probability of non-availability for the current situation is $5*10^{-4}$ per closure. Thus, for the new situation it becomes $2.22*10^{-4}$ per closure.

Locomobile & dock doors

Delta21 influenced the previous components in a positive way, by reducing their failure probability. However, Delta21 does not influence the failure probability of the dock doors or locomobile. However, Delta21 may affect certain components of the locomobile, but since the failure probability analysis is done on a high level of abstraction, it is neglected here. Since the failure probability of these components do not change, the values reported by Rijneveld (2008) will be used. For the locomobile, it is $1.30*10^{-3}$ per closure, and for the dock doors it is $5.30*10^{-4}$ per closure.

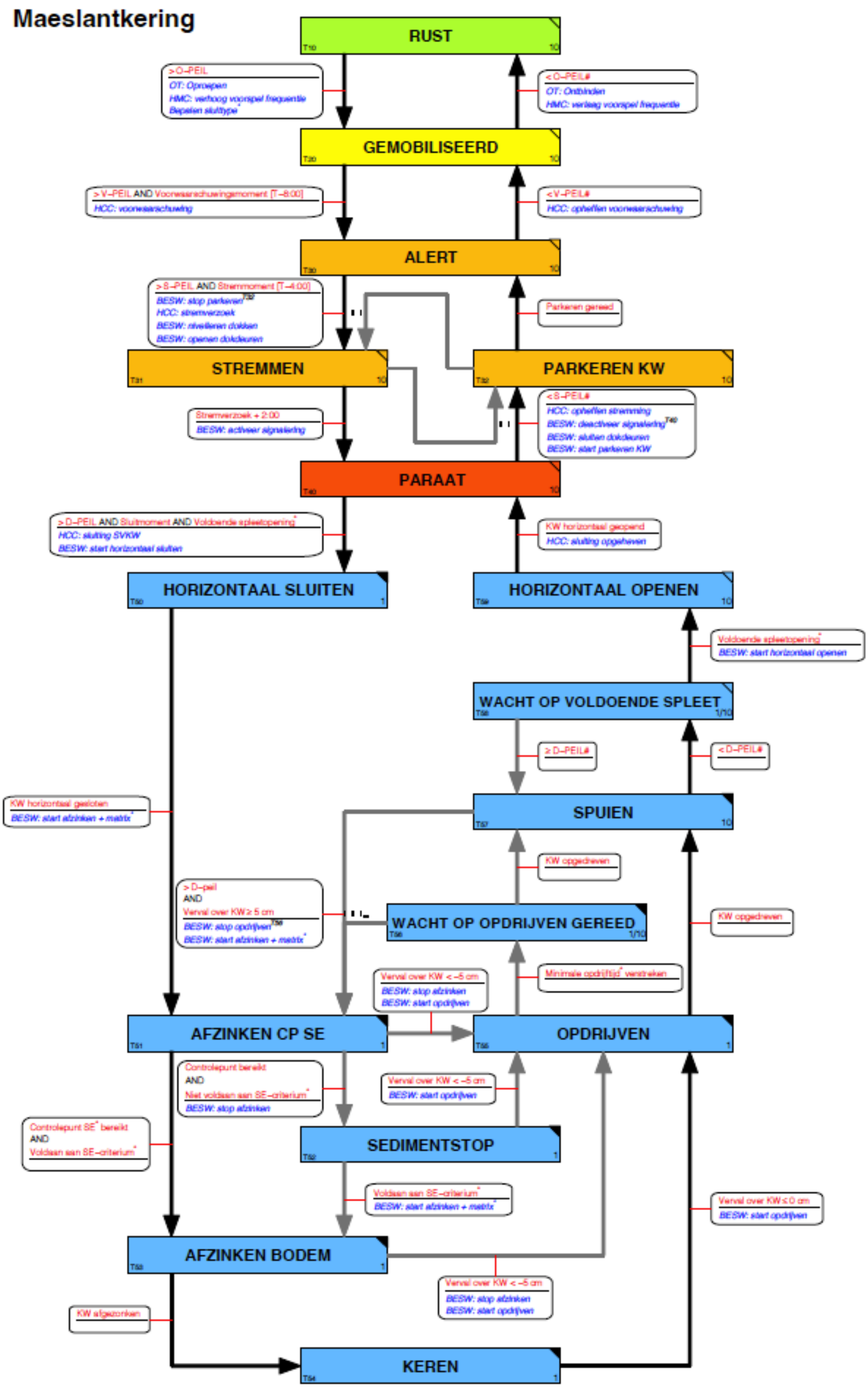


Figure 55: process scheme for the Maeslant barrier in the current situation (Rijkswaterstaat, 2015)

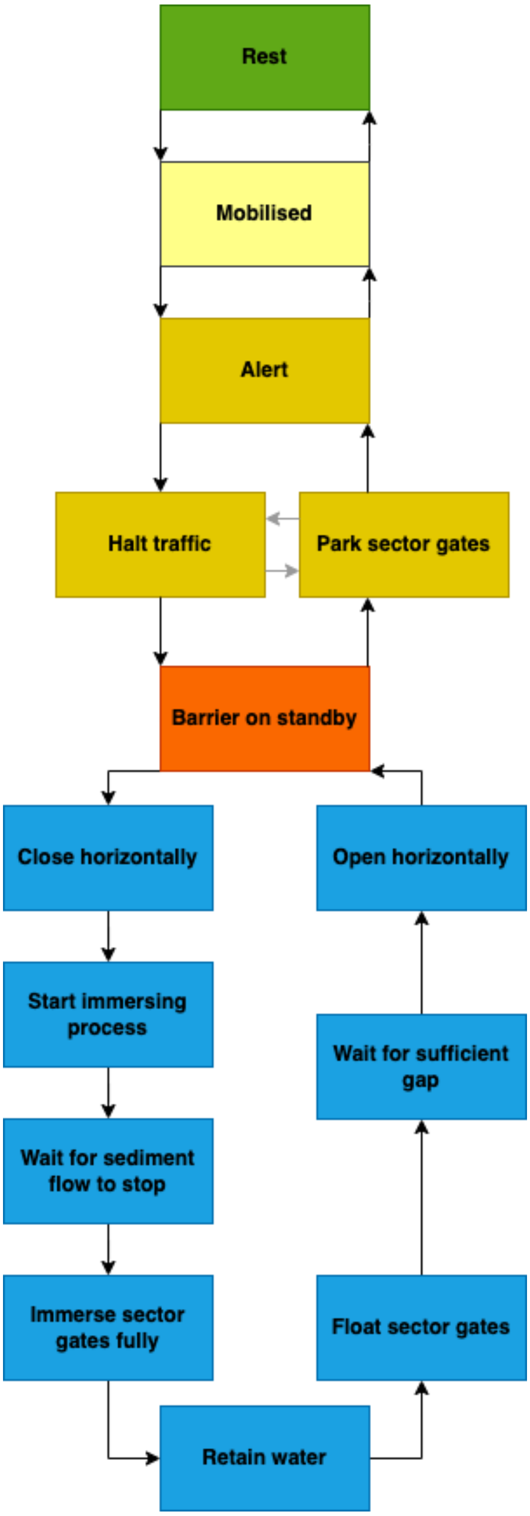


Figure 56: possible simplified process scheme for the Maeslant barrier with Delta21

Probabilities of the additional events

Lastly, the probabilities for the additional events need to be determined. Recalling the additional events from section 5.2.1:

- **A second peak arrives in the same storm with a water level h_{sea} > 3.6 m**
  This event can also be described as follows: "a second peak arrives, and the water level exceeds 3.6 m". Following the probability calculus rules and by assuming that these events are independent, the final probability for the additional event can be found by using the following equations:

$$P(Second\ Peak\ \cap\ h_{sea} > 3.6\ m) = P(Second\ Peak) * P(h_{sea} > 3.6\ m) \quad \text{Eq. 5.1}$$

  Since all the storms are two peak storms in the model, it can be said that the probability that a second peak arrives is equal to 1 per closure:

$$P(Second\ Peak) = 1\ per\ closure \quad \text{Eq. 5.2}$$

  Current situation
  The probability that the water level at sea exceeds 3.6 m can be derived from Figure 35 and is equal to $1*10^{-2}$ per year:

$$P(h_{sea} > 3.6\ m) = 10^{-2}\ per\ year \quad \text{Eq. 5.3}$$

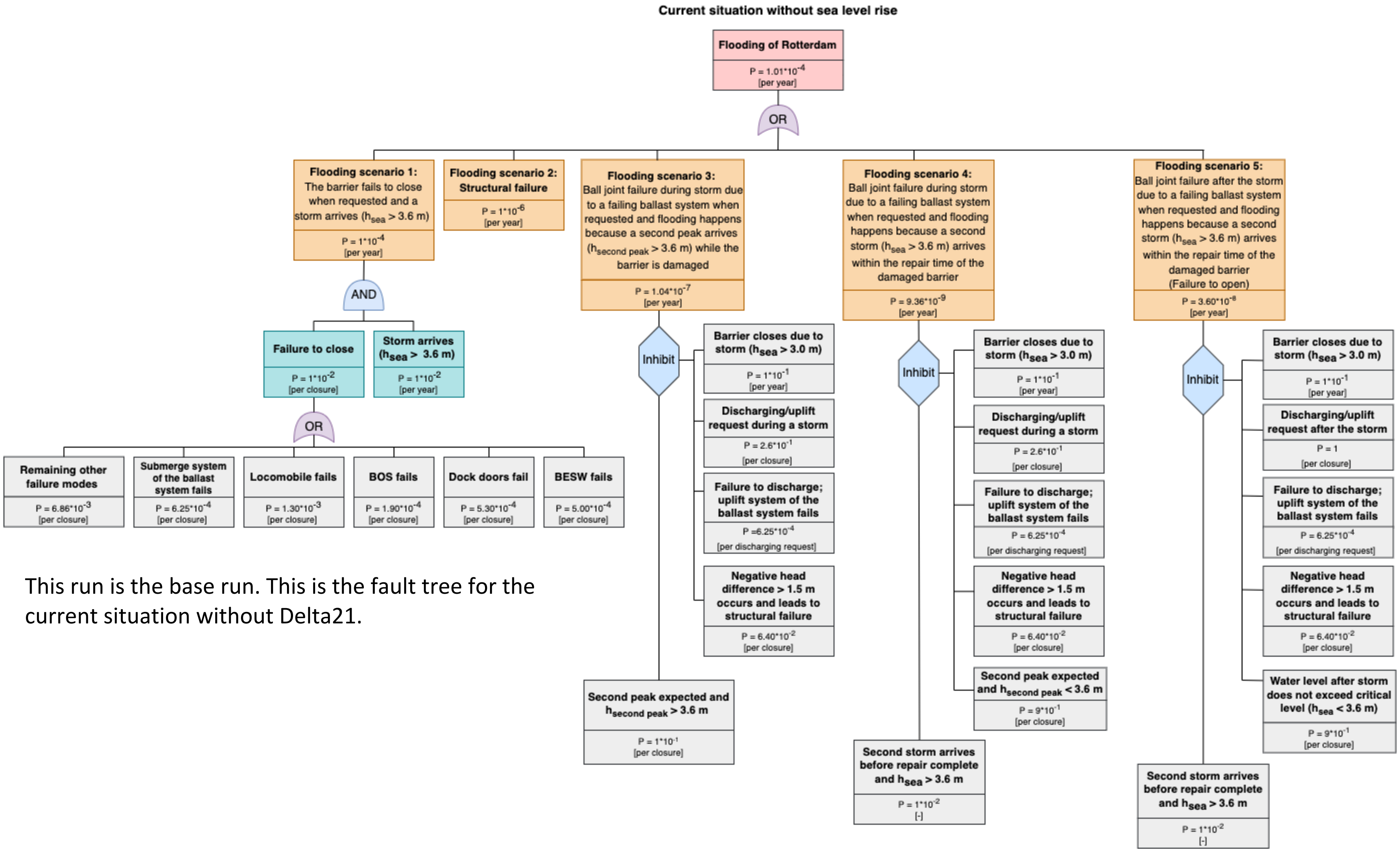
  By substituting eq. 5.2 and 5.3 into equation 5.1, a probability of $1*10^{-2}$ per year is found for the current situation for this additional event.
  Situation with 0.6 m sea level rise
  For the situation with sea level rise, the probability of the event "the water level exceeds 3.6 m" increases and can also be derived from Figure 35. This probability becomes $6*10^{-2}$ per year and is substituted into eq. 5.3. Then, eq. 5.3 and 5.2 are substituted into eq. 5.1 again to obtain the final probability of $6*10^{-2}$ per year for the situation with 0.6 m sea level rise.
- **A second peak arrives in the same storm with a water level h_{sea} < 3.6 m**
  This probability is 1 minus the probability calculated in the previous step: $9*10^{-2}$ per year for the current situation, and $4*10^{-2}$ per year for the case with 0.6 m sea level rise.
- **A second storm with water level h_{sea} > 3.6 m arrives before the repair of the damaged barrier is complete**
  This situation can also be described as a storm arrives with water level h_{sea} > 3.6 m and the repair is not completed within 1 year. The repair time is assumed to be 1 year. By assuming that these events are independent and following the probability calculus rules, P(h_{sea} > 3.6) can be multiplied with the repair time of 1 year. The probability that h_{sea} > 3.6 m is 10^{-2} per year for the current situation, and with 0.6 m sea level rise this changes to $6*10^{-2}$ per year. This gives a probability of 10^{-2} for the current situation and $6*10^{-2}$ for the situation with 0.6 m sea level rise. These probabilities are dimensionless.

For each of these runs done in section 4.3, the corresponding fault tree will be shown with the corresponding probabilities per failure mechanism or event. This is the most orderly manner to show the results, as there are a lot of numbers involved. The fault trees for each run:

1. For the current situation (without Delta21 and no pumping) with no sea level rise, depicted in Figure 57.
2. For the current situation (without Delta21 and no pumping) with 0.6 m sea level rise, depicted in Figure 58.
3. While maintaining a target water level with no sea level rise, depicted in Figure 59.
4. While maintaining a target water level with 0.6 m sea level rise, depicted in Figure 60.
5. While pumping at full capacity with no sea level rise, depicted in Figure 61.
6. While pumping at full capacity with 0.6 m sea level rise, depicted in Figure 62.

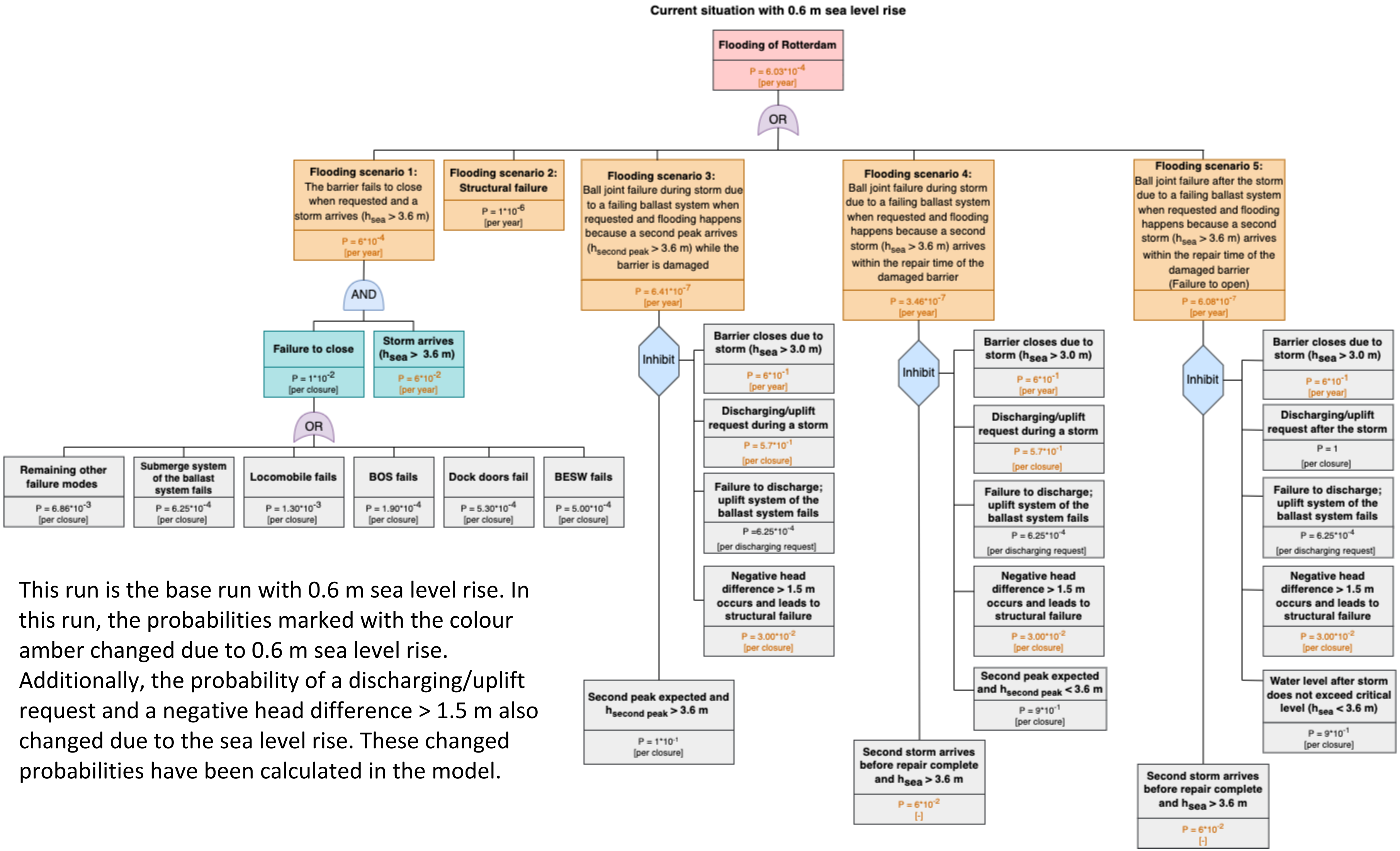
For the runs with Delta21 (run 1 – 4), the probabilities which change due to Delta21 in comparison with the current situation are highlighted in red. The probabilities which are highlighted in the color amber, are the probabilities which change due to sea level rise (see run 2,4,6). For the calculation of the probability of the top event it is assumed that all the failure mechanisms and events are independent. Furthermore, various gates are used in the fault tree:

- The OR gate: the probability for the top event found by summing the probabilities of the underlying events
- The AND gate: the probability for the top event is found by multiplying the probabilities of the underlying events.
- The INHIBIT gate: equivalent to the AND gate



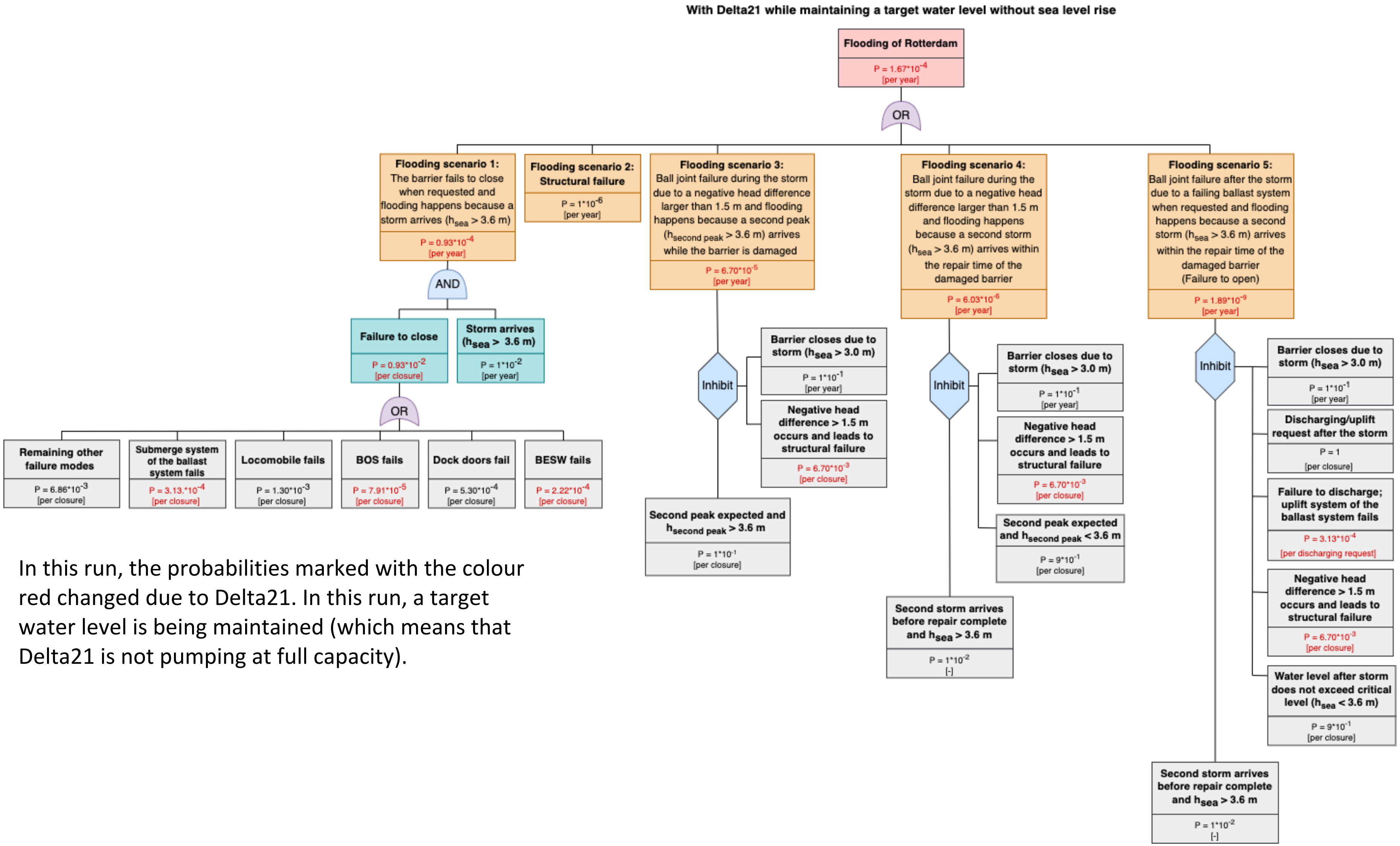
This run is the base run. This is the fault tree for the current situation without Delta21.

Figure 57: fault tree for run 1: current situation (without Delta21) without sea level rise



This run is the base run with 0.6 m sea level rise. In this run, the probabilities marked with the colour amber changed due to 0.6 m sea level rise. Additionally, the probability of a discharging/uplift request and a negative head difference > 1.5 m also changed due to the sea level rise. These changed probabilities have been calculated in the model.

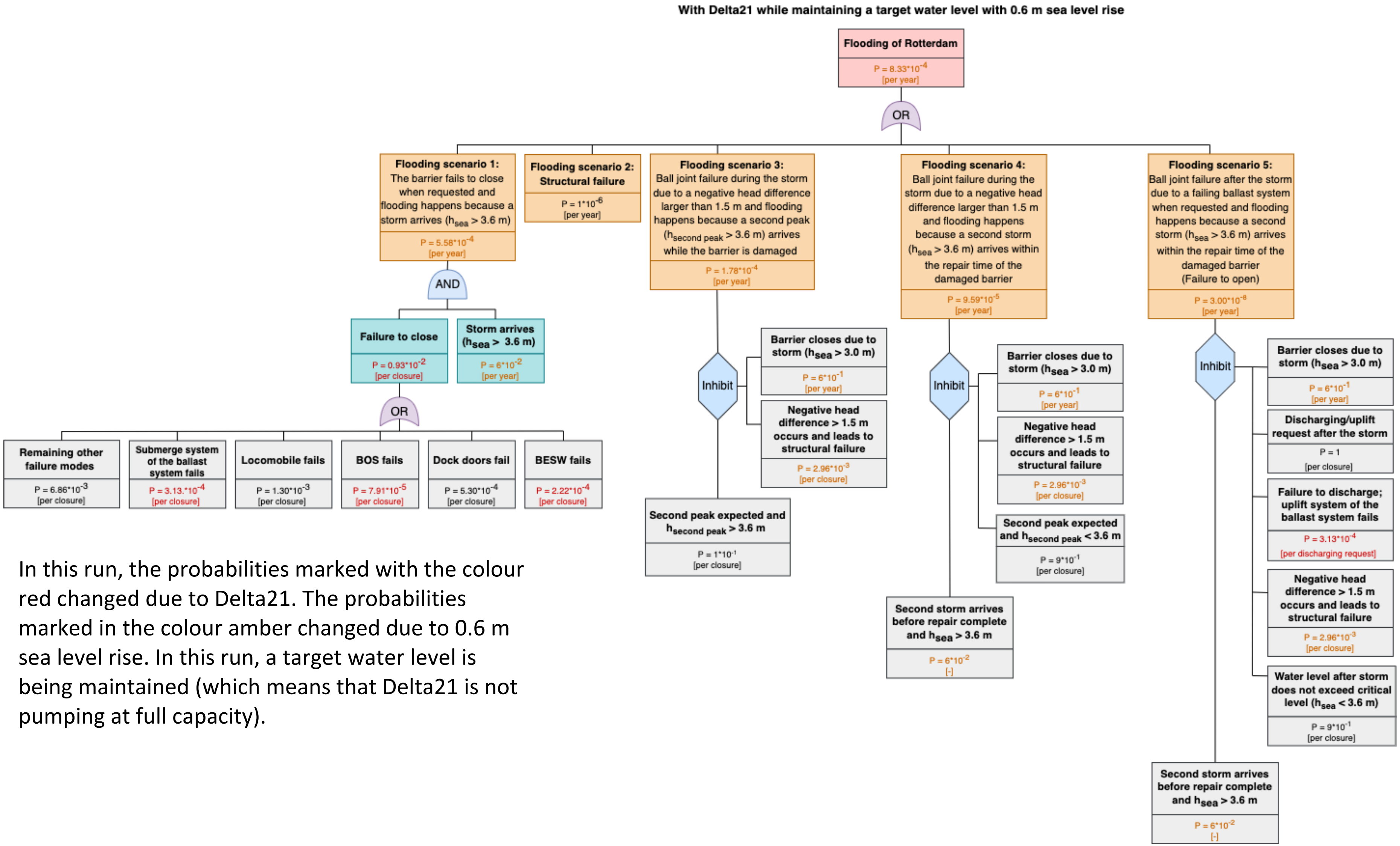
Figure 58: fault tree for run 2: current situation (without Delta21) with 0.6 m sea level rise



In this run, the probabilities marked with the colour red changed due to Delta21. In this run, a target water level is being maintained (which means that Delta21 is not pumping at full capacity).

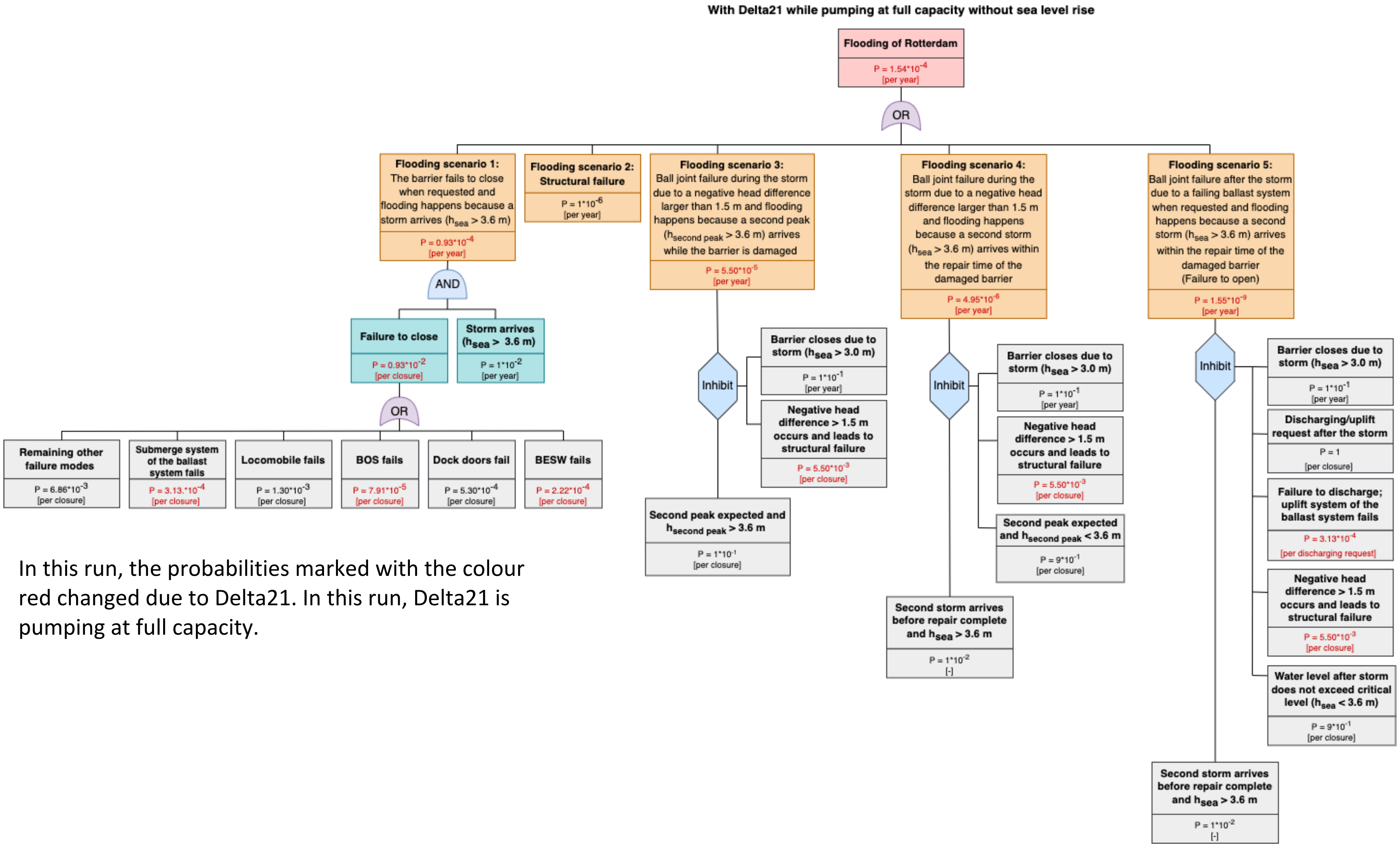
Figure 59: fault tree for run 3: Delta21 maintaining a target water level without sea level rise

Quantitative analysis



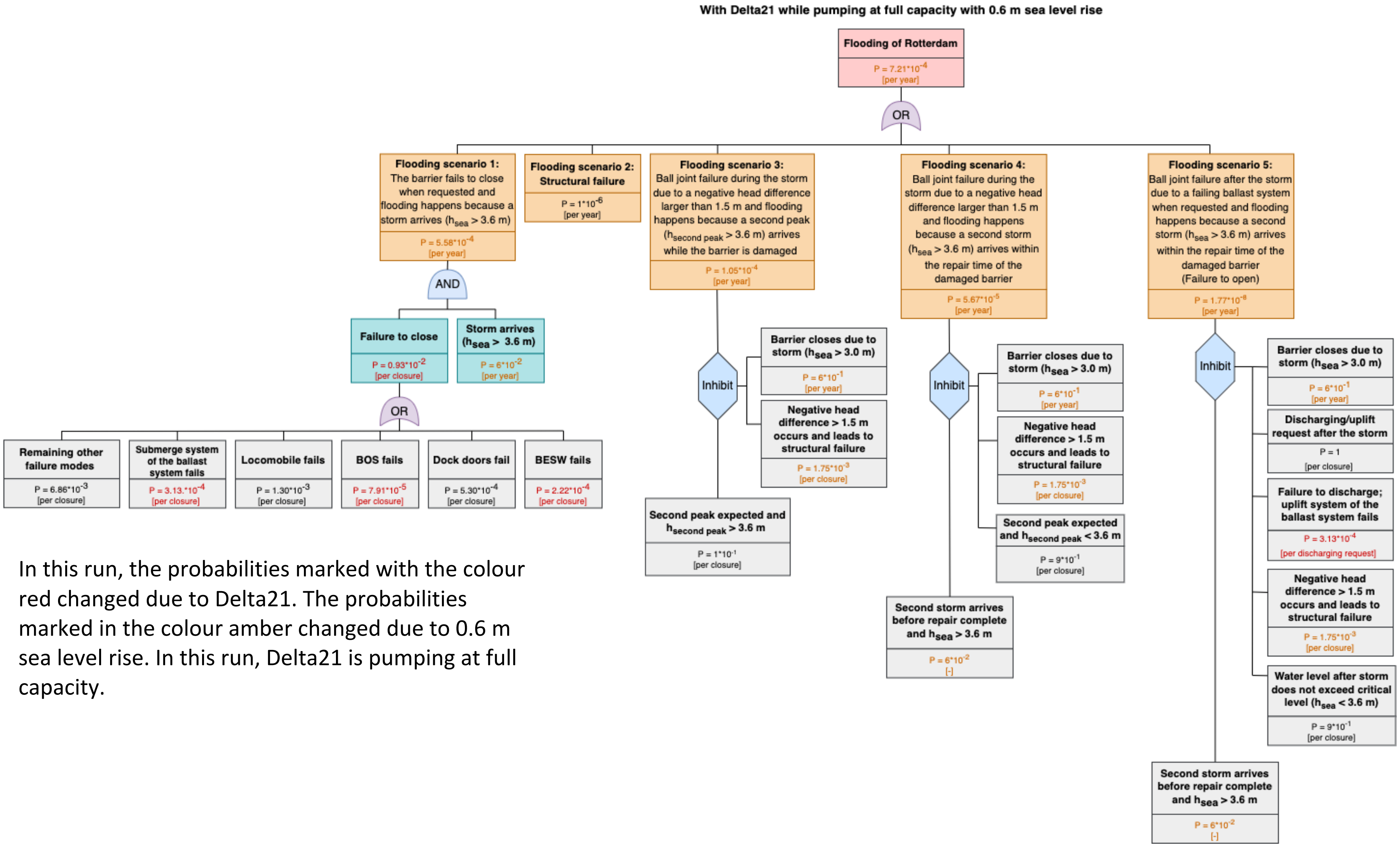
In this run, the probabilities marked with the colour red changed due to Delta21. The probabilities marked in the colour amber changed due to 0.6 m sea level rise. In this run, a target water level is being maintained (which means that Delta21 is not pumping at full capacity).

Figure 60: fault tree for run 4: Delta21 maintaining a target water level with 0.6 m sea level rise



In this run, the probabilities marked with the colour red changed due to Delta21. In this run, Delta21 is pumping at full capacity.

Figure 61: fault tree for run 5: Delta21 pumping at full capacity without sea level rise



In this run, the probabilities marked with the colour red changed due to Delta21. The probabilities marked in the colour amber changed due to 0.6 m sea level rise. In this run, Delta21 is pumping at full capacity.

Figure 62: fault tree for run 6: Delta21 pumping at full capacity with 0.6 m sea level rise

5.4 Results

In this section, the probabilities from the fault trees with Delta21 be compared to the probabilities from the fault trees for the current situation without Delta21

5.4.1 Failure probability of closure

When comparing the probability of the event "failure to close" in the fault trees with Delta21 (Figure 59 to Figure 62) to the fault trees for the current situation without Delta21 (Figure 57 and Figure 58), a reduction of 7% per closure can be observed. In the current situation without Delta21, the failure probability of closure is $1*10^{-2}$ per closure, and in the situation with Delta21 it became $0.93*10^{-2}$ per closure. This answers the main research objective which was defined in section 1.3.

5.4.2 Probability of flooding for Rotterdam

To keep the results orderly, the probabilities of the top event of all the fault trees (flooding of Rotterdam) have been presented in Table 16.

Run	**Probability of flooding for Rotterdam** [per year]
1. Current situation without sea level rise (without D21)	$1.01*10^{-4}$
2. Current situation with 0.6 m sea level rise (without D21)	$6.03*10^{-4}$
3. Maintaining a target water level without sea level rise (with D21)	$1.67*10^{-4}$
4. Maintaining a target water level with 0.6 m sea level rise (with D21)	$8.33*10^{-4}$
5. Pumping at full capacity without sea level rise (with D21)	$1.54*10^{-4}$
6. Pumping at full capacity with 0.6 m sea level rise (with D21)	$7.21*10^{-4}$

Table 16: summary of the probabilities of the top events from all the fault trees

Run 1 represents the current situation without Delta21. Run 2 represents the current situation without Delta21 with 0.6 m sea level rise. For the situation **without** sea level rise, run 3 and 5 (runs with Delta21) will be compared to run 1:

- When comparing run 3 to run 1, Delta21 increases the flood risk for Rotterdam with approximately 65% if a target water level is being maintained in the basins. When comparing run 5 to run 1, Delta21 increases the flood risk for Rotterdam with approximately 53% if the Delta21 pumps are pumping at full capacity.

For the situation **with** sea level rise run 4 and 6 (runs with Delta21) will be compared to run 2:

- When comparing run 4 to run 2, Delta21 increases the flood risk for Rotterdam with approximately 38% if a target water level is being maintained in the basins. When comparing run 6 to run 2, Delta21 increases the flood risk for Rotterdam with approximately 20% if the Delta21 pumps are pumping at full capacity.

5.4.3 Concluding remarks

Based on these comparisons it can be concluded that running the Delta21 pumps at full capacity is better than when maintaining a target water level in the basins as the probability of flooding for Rotterdam is lower for run 5 in comparison with run 3. This also holds for the runs with sea level rise. It can also be concluded that Delta21 slightly increases the flood risk of Rotterdam. This is against all the expectations.

Why is this the case? The probability of the top event ("Flooding of Rotterdam") is the sum of the probabilities of flooding scenarios 1 to 5. In the situation with Delta21, the probability for flooding scenarios 3 and 4 increase significantly in comparison to the situation without Delta21. The probability of flooding scenarios 3 and 4 is obtained by multiplying the probabilities of all the underlying events. In the current situation, the probabilities of flooding scenarios 3 and 4 are in the order of 10^{-7}. Because these probabilities are small, their contribution to the probability of flooding is almost negligible (as the order of the probability of flooding is 10^{-4}).

In the situation with Delta21, the probabilities of flooding scenarios 3 and 4 are in the order 10^{-5}, and the contribution to the probability of flooding becomes significant. This the consequence of removing the discharging function in which the ballast system played a role. The ballast system has a very small failure probability and lowered the probabilities of flooding scenarios 3 and 4 in the current situation. This whole explanation can be more easily understood by comparing Figure 63 with Figure 64. Thus, it can be said that Delta21 succeeds in reducing the probability of a negative head difference > 1.5 m and the failure probability of closure, but this reduction is not enough to reduce the overall flood risk as the probability for flooding scenario 3 and 4 increases too much.

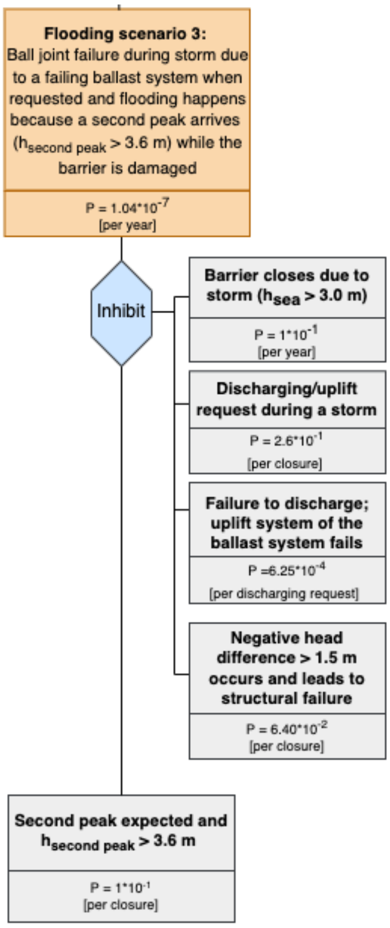


Figure 63: flooding scenario 3 from run 1 (without Delta21)

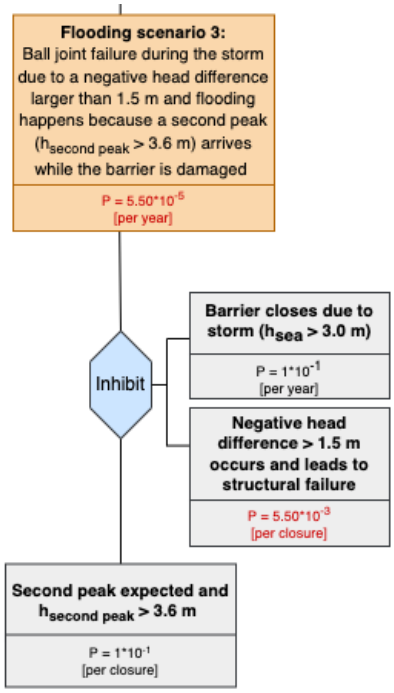


Figure 64: flooding scenario 3 from run 5 (with Delta21)

6 Discussion

To obtain the answer to the main research question, several assumptions and simplifications were made. The final result depends largely on these assumptions and simplifications. These assumptions and simplifications can be subject to a discussion. Most of the critical assumptions were made in Chapter 3 and Chapter 5. The most influential assumptions will be discussed in this section.

6.1 Critical assumptions for the hydraulic model

- The occurrence of seiches in the model is based on whether a storm with an amplitude larger than a specified threshold (can be varied in the model) occurs. In reality, the type of storm and weather conditions determine if the situation is favourable for seiches to occur.
- The storage area of the basins is assumed to be constant. This is not the case in reality. The storage area influences which volume of water that can be stored in the basins, and this varies with depth according to the bathymetry in the basins.
- It is assumed that storms and high river discharges occur independently of each other. However, it is quite possible that this can be dependent. If a large depression arrives it can cause a storm surge and a lot of precipitation in the stream area of the river. The precipitation which is not absorbed by the ground will ultimately end up in the river, which will increase the discharge. One of the conclusions is that the higher the expected river discharge, the lower the closure level of the barrier should be to minimize failure.
- It is assumed that Delta21 is **always** able to discharge water up to a quantity of 10,000 m^3/s. This means that Delta21 cannot fail in this model.
- The closing procedure of the Maeslant barrier is not modelled. In the model, it is assumed that the barrier is closed when the water level reaches the closure level. This should be implemented in more advanced models as it takes some time before the barrier is actually closed.
- The location at which the water level is measured which is used to determine whether the barrier needs to close or not in the Delta21 is Hoek van Holland. The consequence of this is that with sea level rise, the closure level is met more frequently (Delta21 cannot lower the water level at Hoek van Holland). If the location at which the closure level is determined would be Rotterdam, the barrier **might** need to close less often, as Delta21 has the capacity to lower the water level at Rotterdam. More research is needed to address this.

6.2 Critical assumptions in the failure probability analysis

- The risk analysis has been done on a high level of abstraction, in which only the main components have been looked at. However, the small components which power the main components (such as electrical systems, valves, hydraulic cylinders etc.) also contribute to the risk of flooding in reality.
- It has been assumed that the main components are independent of each other. This may be the case for the main components, but is it the case for the smaller components which have not been considered?

- Human failure is a failure mechanism which has not been elaborated, while it can have a significant (positive or negative) impact.
- BESW gives commands to the various components when needed. In the failure probability analysis is assumed that BESW succeeds in giving the command, but will the command actually be executed by the receiving component? This question has not been answered in this thesis but is important as well for the overall reliability of the barrier.
- The reduction of the failure probability for BOS is based on the reduction of the sources of uncertainty. It was assumed that all the sources of uncertainty contribute evenly to the total uncertainty. This is not the case in reality. Measurements will have a lower uncertainty than predictions.
- The failure probability of Delta21 has not been included in the risk analysis, because it was assumed that Delta21 does not fail. However, every system can fail and has a failure probability.

7 Conclusions and recommendations

7.1 Conclusions

The main research question and objective of this master thesis was defined as follows:

"How does the failure probability of closure of the Maeslant Barrier with the simplified closure operation change in the Delta21 configuration?"

To answer the main research question, the methodology which was defined in section 1.4 was used:

1. **A hydraulic system analysis was done**
   This analysis explained how the water levels and discharge distribution changed in the Delta21 configuration. It also provided insight in what the role of seiches in the Delta21 configuration was.
2. **A simple hydraulic model was developed**
   The hydraulic model provided insight in how Delta21 affected the water levels at the location of the Maeslant barrier.
3. **The results from the hydraulic model were analysed**
   The analysis of the results obtained using the hydraulic model highlighted the power of the Delta21 pump capacity. The sensitivity of the model results to various parameters were also identified.
4. **A failure probability analysis for the Maeslant barrier in the Delta21 configuration was done**
   The failure probability analysis provided insight in how the failure probability of closure changed in the situation with Delta21.

The failure probability of closure was reduced by a maximum of approximately 10% in the situation with Delta21 due to the simplified closure operation in which the current discharging function is omitted. The reduction is marginal, because failure mechanisms which are not affected by Delta21 contribute much more to the failure probability of closure than the failure mechanisms which are affected by Delta21. Therefore, the main research question and sub question 3 (as defined in section 1.3) have been answered.

Additionally, the failure probability analysis also provided insight in how the probability of flooding changed for Rotterdam with the Maeslant barrier in the Delta21 configuration. While the failure probability of closure was reduced by a maximum of 10% with Delta21, the probability of flooding for Rotterdam has increased by 53% when using the full pump capacity from Delta21. This increase is not significant since the probability of flooding remains very small. Due to the removal of the discharging function with Delta21, the probability of several flooding scenarios increased drastically which resulted in the increase in the probability of flooding for Rotterdam. Although Delta21 seems like a more robust system in comparison with the existing discharging function, it could not be proven that it also reduces the probability of flooding for Rotterdam in comparison with the current situation in this study.

Thus, it can be said that Delta21 succeeds in reducing the probability of a negative head difference > 1.5 m and the failure probability of closure, but this reduction is not enough to reduce the overall flood risk as the probability for flooding scenario 3 and 4 increases too much.

With a sea level rise of 0.6 m, the probability of flooding increased approximately 500% for the current situation (without Delta21). When comparing Delta21 with no sea level rise to Delta21 with 0.6 m sea level rise, the increase was also present with approximately 375%. This is less than 500%, which indicates that Delta21 is effective to mitigate the effects of sea level rise compared to the situation without Delta21.
Apart from the main conclusions, some additional important conclusions have been drawn in Chapter 4:

- If seiches with high amplitudes occur, a negative head difference > 1.5 m will occur more likely.
- The higher the expected river discharges are, the lower the closure level of the Maeslant barrier needs to be.
- In the runs with Delta21, the probability of a negative head difference larger than 1.5 m is almost a factor 10 lower than in the current situation (without Delta21).
- The simulated sea level rise of 0.6 m **increased** the probability that a negative head difference larger than 1.5 m occurs per year.
- Sea level rise increases the probability of a discharging request per closure.
- If low storm amplitudes (low high-water levels) and high seiche amplitudes occur at the same time, more negative head differences > 1.5 m are observed.
- If low seiche amplitudes and high river discharges occur at the same time, more negative head differences > 1.5 m are observed.
- In the current situation, seiches with an amplitude in the range [0.20 m, …,0.70 m] cause negative head differences > 1.5 m, while in the Delta21 situation only seiches with an amplitude in the range [0.35 m, …, 0.70 m] cause failure. It can be concluded that seiches with an amplitude in the range [0.20 m, …, <0.35m] do not cause failure anymore in the situation with Delta21.

7.2 Recommendations

Based on the conclusion and discussion, several recommendations can be given:

- The dependence of seiches on storm- and weather types should be implemented into the 1D-SOBEK model in which Delta21 has been built in. This model will give more accurate results when compared to the simple hydraulic model which was used in this master thesis.
- Translation waves which reflect on the sector gates can also increase the negative head difference. This has not been taken into account in this study, but it recommended to address the effects of these translation waves in a more detailed study.
- The control system which operates Delta21 in combination with the Maeslant barrier should be designed and the behaviour of this should also be implemented into the 1D-SOBEK model with Delta21 to get even more reliable results.
- Sea level rise **increases** the closing frequency of the barrier. Further research should be done on how this impacts the reliability of the components of the Maeslant barrier.
- More research should be done to find the optimum closure level with Delta21 in combination with high river discharges.
- In order to determine if a closure is needed or not, it should be investigated how the location at which the water level is measured affects the closure frequency.

As mentioned in the section 6.1, for the configuration with Delta21, the location at which the water level is measured to determine if a closure is needed is Hoek van Holland. Delta21 does not affect the water level at Hoek van Holland. Should this be Rotterdam, Delta21 can reduce the water level at this location using its pump capacity which could avoid closure of the Maeslant barrier, as the closure level is met less frequently at Rotterdam than at Hoek van Holland.

- The removed sources of information for BOS in the Delta21 configuration are needed in the new control system for Delta21. It should be investigated how this impacts the to be designed control system for Delta21.

When these recommendations have been realised, a more thorough and accurate failure probability analysis of the whole system (including failure of Delta21) should be done to get the more complete answer.

Appendices

A Verification of the programmed model

This appendix covers the verification of the programmed model. For the resulting equations (3.2, 3.3, 3.7 and 3.11), it will be checked if the equations in the programmed model are dimensionally correct and equivalent to the analytical equations.

Since equation 3.2, 3.3 and 3.7 are ordinary differential equations, they must be transformed to algebraic equations. The ordinary differential equations cannot be programmed into a computer model because they are continuous, while algebraic equations can be programmed because they are not continuous because of the discretization which will be applied. The time will be discretized from $t = t_0$ to $t = t_N$ in which N denotes the total amount of time steps which will be used. The numerical implementation was provided by Battjes & Labeur (2017), but it has been adopted for the model in consideration.

Verification of eq. 3.2, 3.3 and 3.7 (surface elevation basins and discharge channel)
Recalling equation 3.2 for the surface elevation in basin 1, but semi-discretized at time $t = t_n$:

$$A_{b1}\left.\frac{dz_{b1}}{dt}\right|_{t_n} = Q_{Nederrijn} - Q_{channel}^{n} \tag{A.1}$$

Note: the basin area for basin 1, A_{b1} and discharge from the Nederrijn are constants (they do not change in time). That is why they do not have to be approximated numerically. However, the discharge in the channel needs to be calculated for every time step. This will be shown later in this section. The time derivative of the water level will be approximated with the forward Euler method:

$$\left.\frac{dz_{b1}}{dt}\right|_{t_n} \approx \frac{z_{b1}^{n+1} - z_{b1}^{n}}{t_{n+1} - t_n} \tag{A.2}$$

By substituting eq. A.2 into A.1, the numerical approximation for the water level in basin 1 is found:

$$z_{b1}^{n+1} = z_{b1}^{n} + \Delta t_n \frac{Q_{Nederrijn} - Q_{channel}^{n}}{A_{b1}} \tag{A.3}$$

The code on line 2 in the code snippet below is equivalent to the second term in the right-hand side of equation A.3. By adding this term to the code in line 3, it can be said that the code in line 3 is equivalent to the numerical approximation of equation 3.2 which was given in equation A.3. The units for the variables are reported at the end of the line between the square brackets.

In the same way, equation 3.3 can be discretized and that results in the code in line 6 and 7. Line 7 is equivalent to the numerical approximation of equation 3.3.

```
1.  #update the water level in basin 1 (equation 3.4)
2.  dz1 = ((Qin1-Qch[final+i])/A1)*dt                                  #[m]
```

```
3.  zin_closed [final+i+1] = zin_closed[final+i] + dz1                          #[m]
4.
5.  #update the water level in basin 2 (equation 3.5)
6.  dz2 = (((Qch[final+i]+Qin2)-Qout)/A2)*dt                                    #[m]
7.  zb3 [final+i+1] = zb3[final+i]+dz2                                          #[m]
8.
9.  #calculate the discharge in the channel (equation 3.7)
10. kappa = chi*np.abs(Qch[final+i])/Ac                                         #[m/s]
11. dQ = g*Ac*(zin_closed[final+i+1]-zb3[final+i+1])-kappa*Qch[final+i]
12. Qch[final+i+1] = Qch[final+i] + dQ/(L/dt+2*kappa)                          #[m3/s]
```

Lastly, the discharge in the channel needs to be approximated numerically. By applying the backward Euler method for semi-discretization of the time derivative of the discharge, the following expression is found:

$$\left.\frac{dQ_{channel}}{dt}\right|_{t_n} \approx \frac{Q^n_{channel} - Q^{n-1}_{channel}}{t_n - t_{n-1}} \qquad \text{A.4}$$

By substituting equation A.4 into equation 3.7 at time level $t = t_{n+1}$, the numerical solution for the discharge in the channel can be found:

$$Q^{n+1}_{channel} = Q^n_{channel} + \frac{\Delta t_n}{l}(gA_c(z^{n+1}_{b1} - z^{n+1}_{b2}) - \chi\frac{|Q^{n+1}_{channel}|Q^{n+1}_{channel}}{A_c} \qquad \text{A.5}$$

The code in line 10 and 11 are intermediate steps to keep the python code readable. When combining them into line 12, it is equivalent to equation A.5 which is the numerical approximation of the discharge in the channel. The units are correct, as the final discharge should be in the unit [m^3/s].

The used numerical method is stable if the time step (Δt_n) is smaller than 7,638 seconds. The time step used in the model is 260 seconds, so the method will be stable.

Verification of equation 3.9 (seiches)

The python implementation of equation 3.9 (which defines a seiche) can be found in the code snippet below, along with the units per parameter. The code in line 9 is equivalent to equation 3.9. The amplitude "a" can be varied.

```
1.  T = 30*60                                     #[s]
2.  d = 15                                        #[m]
3.  c = np.sqrt(9.81*d)                           #[m/s]
4.  L = c*T                                       #[m]
5.  Lbasin = 5000                                 #[m]
6.  w = (2*np.pi)/T                               #[rad/s]
7.  k = (2*np.pi)/L                               #[rad/m]
8.  x = np.linspace(0,Lbasin,len(t))              #discretized space
9.  nseiche = a*np.cos(w*(t*3600)-k*x)            #[m]
```

Verification of equation 3.10 (tide)

The python implementation of equation 3.10 (which defines the tide) can be found in the code snippet below, along with the units per parameter. The variable "ntidetotal" in line 12 is the final surface elevation after summarizing the surface elevation due to the three different constituents.

```
1.  an = [0.79, 0.19, 0.17]                                                    #[m]
2.  wn = [0.50722,0.525,1.01]                                                  #[rad/s]
3.  alphan = [1.505, 2.5725, 2.8875]                                           #[rad]
4.
5.  #calculate the surface elevation changes due to the tide (with M2,S2 and M4 components)
6.  tide = np.zeros(len(t))
7.  for j in range(len(t)):
8.      ntide = 0
9.      for i in range(len(an)):
10.         ntide = ntide + an[i]*np.cos((wn[i]*t[j])-alphan[i])
11.     tide[j] = ntide
12. ntidetotal = tide                                                          #[m]
```

Verification of equation 3.11 (storm surge)

The last component which needs to be verified is the storm surge. The storm surge is calculated exactly as equation 3.11 described in line 1 in the code snippet below:

```
1.  nsurge[indexbegin+i] = Amax*np.sin(2*np.pi*(t[indexbegin+i]-t[indexbegin])/(2*Tstorm))    #[m]
```

B Validation of the programmed model

This appendix covers the validation for the individual model components (for the tide, seiches, storm set up) by plotting the water levels and analysing them to see if it reproduces the required and expected results.

Validation of seiches

In Figure 65, a seiche with an amplitude of 0.45 m has been plotted (while using the code above). The period is exactly 30 minutes, and the amplitude is also exactly 0.45 m.

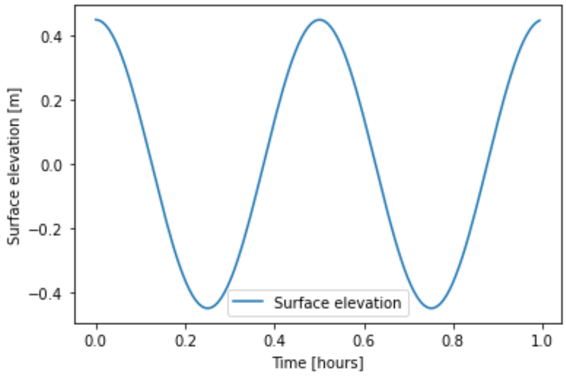


Figure 65: validation run for seiches with an amplitude of 0.45 m

It can be concluded that the model reproduces seiches correctly, as intended.

Validation of the tide

When comparing the tide calculated with the model (depicted in Figure 66) with the calculated expected tide from Rijkswaterstaat (2022) (depicted in Figure 67), it can be observed that the period is around 12 hours for both graphs, and the order of magnitude for the amplitude is also approximately equal.

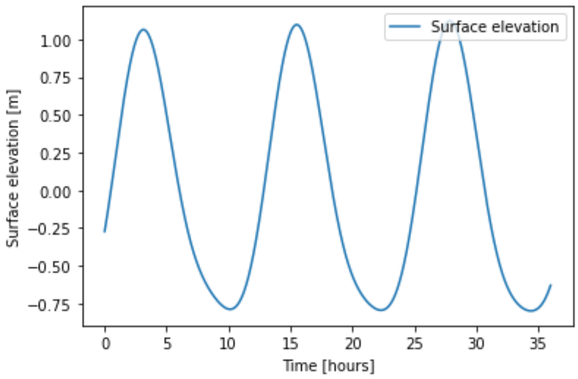


Figure 66: validation run for the tide

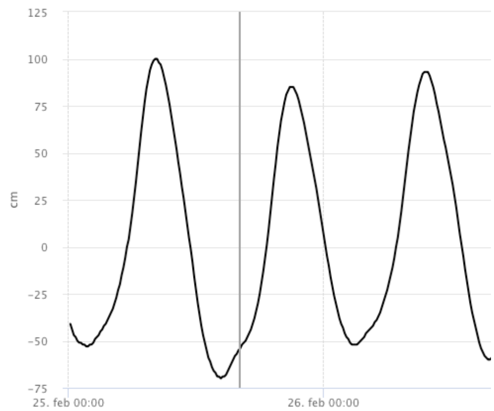


Figure 67: expected tide at Hoek van Holland (Rijkswaterstaat, 2022)

Thus, it can be concluded that the tide is approximated sufficiently accurate in the model (given the assumptions which were made in the model).

Validation of the storm surge

To validate if the storm surge is produced accurately in the model, a storm surge with an amplitude of 3.0 m and a duration of 35 hours will be plotted. The storm is supposed to start at T = 20 hours. Figure 68 depicts the results. It can be observed that the storm starts at T = 20 hours and lasts until T = 55 hours. The graph shows that the model reproduces the required storm surge accurately.

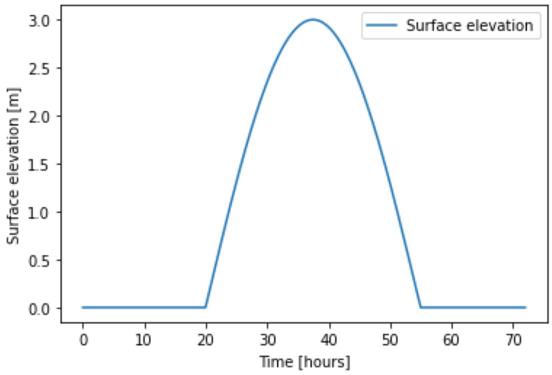


Figure 68: validation run for the storm surge

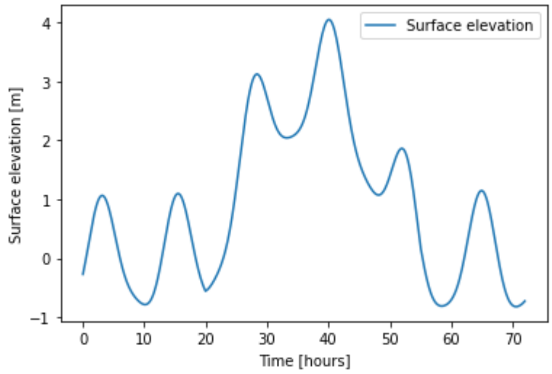


Figure 69: water levels for storm surge + tide

When adding the water levels of the storm surge and tide, the water levels for a storm can be reproduced in the model. This is depicted in Figure 69. The addition of seiches will be explained in the full model run, in which the barrier is also present (as they only occur in the model when the barrier is closed).

C Sensitivity of the model results to various parameters

In the deterministic runs, the value for each parameter can be adjusted manually for each parameter. By adjusting a parameter in each run (while keeping the other parameters unchanged), the sensitivity of the model results to the adjusted parameter could be identified. Several deterministic runs have been done to highlight the influence of key parameters. The sensitivity of the model results due to the adjusted parameters will be covered in this appendix.

The friction coefficient
Since the outflow in basin 1 fully depends on the discharge in the channel, the effect of this parameter is very important. In section 3.2 it was stated that the value of the friction coefficient c_f typically lies between 0.002 and 0.006 and that the mean value of 0.004 would be used for the simulations. It was also stated that it would be checked what the discharge for the boundary values would be.

Running the model as described above for $c_f = 0.002$ gives a mean discharge of 1,733 m^3/s in the channel. Using a $c_f = 0.006$ gives a mean discharge of 1,652 m^3/s in the channel. Taking $c_f = 0.004$ gives a mean discharge of 1,697 m^3/s. The conclusion is that the discharge does not vary drastically by choosing a friction coefficient which is very low or very high (in the given range of 0.002 – 0.006). Therefore, it is justified to use the mean value of 0.004 (which is subjected to a normally distributed 1% uncertainty and the model uncertainty of 10% as mentioned in section 3.2).

The seiche amplitude
The simulation is done with the parameters mentioned in the introduction of this section for mode 1 (maintaining a target water level) and mode 2 (pumping at maximum capacity) with seiche amplitudes of 0.20 m (results depicted in Figure 43 and Figure 44) and 0.75 m.

To highlight the effect of an increased seiche amplitude, the all the previously mentioned variables are kept constant and the seiche amplitude is increased to 0.75 m, the critical situation becomes clear. Figure 70 depicts a situation like this. A negative head difference larger than 1.5 m can be observed between at T = 32 hours. This means that the barrier structurally fails in this situation.

```
Qin : 6350.0 m3/s
Qchannel : 1222.14 m3/s
Seiche amplitude : 0.75 m
Storm set-up amplitude: 3.0 m
Failure: 1
Time closed: 17 hours
Time at failure: 32 hours
```

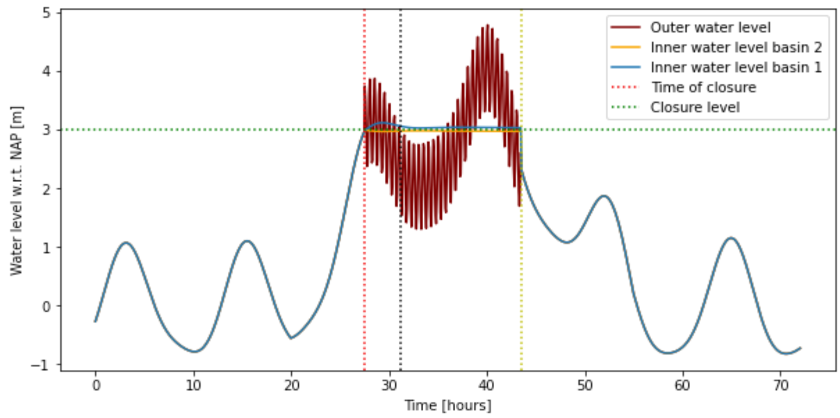


Figure 70: Model output for mode 1 with a seiche amplitude of 0.75 m

Figure 71 depicts mode 2 with a seiche with an amplitude of 0.75 m. In this situation, a negative head difference larger than 1.5 m has also been observed at T = 33 hours. The barrier also fails in this situation.

```
Qout : 10000.0 m3/s
Qin : 6350.0 m3/s
Qchannel : 1682.91 m3/s
Seiche amplitude : 0.75 m
Storm set-up amplitude: 3.0 m
Failure: 1
Time closed: 17 hours
Time at failure: 33 hours
```

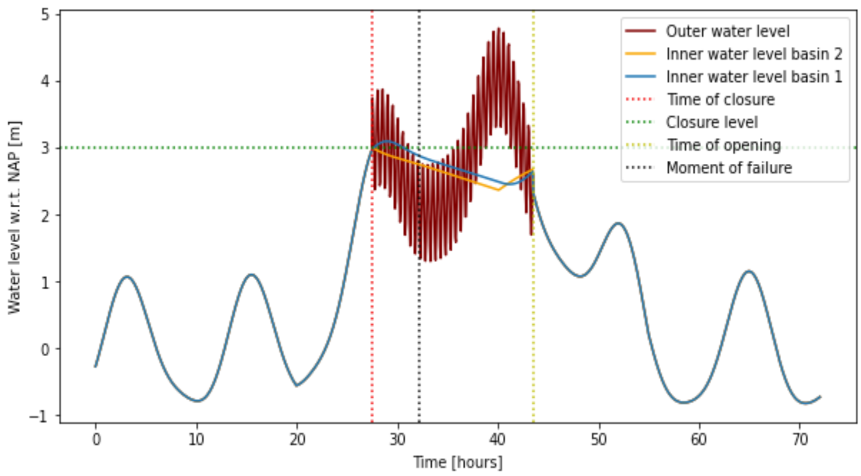


Figure 71: model output for mode 2 with a seiche amplitude of 0.75 m.

When comparing Figure 70 and Figure 71, failure occurs one hour later. This can be explained by the lower water level when Delta21 pumps at full capacity. Failure in this run (Figure 71) can be avoided by choosing a lower closure level, for example at NAP + 2.5 m. This is depicted in Figure 72.

```
Qout : 10000.0 m3/s
Qin : 6350.0 m3/s
Qchannel : 1683.93 m3/s
Seiche amplitude : 0.75 m
Storm set-up amplitude: 3.0 m
Failure: 0
Time closed: 18 hours
```

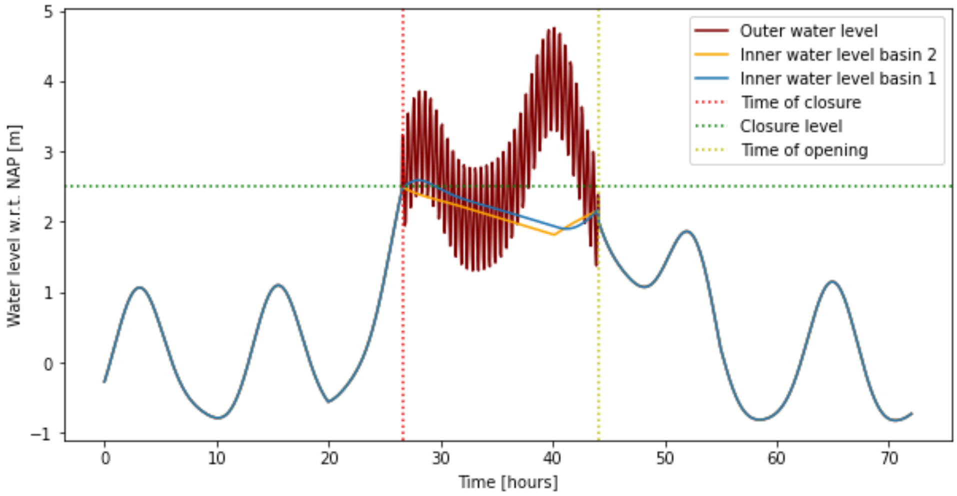


Figure 72: model output for mode 2 with a seiche amplitude of 0.75 m, but with a closure level of NAP + 2.5 m.

The outer water level by adding sea level rise

To prove the significance of pumping, the situation with and without pumping capacity with sea level rise need to be simulated. To simulate sea level rise in a deterministic run, all the water levels can be raised by the chosen amount of sea level rise (while keeping the closure level constant at NAP + 3.00 m).

By setting Q_{out} to zero, the situation without the pumping capacity of Delta21 becomes clear. Keep in mind that the Maeslant barrier cannot discharge river water between the two peaks. The Maeslant barrier in this model has been stripped of its capability to discharge river water in case of a negative head difference during a storm.

The water levels in the basins obviously increase as no water is being pumped out. This is depicted in Figure 48. A Rhine discharge of 6,000 m^3/s and a Meuse discharge of 1,250 m^3/s cannot be handled without failure without Delta21. This is the case for a closure level of NAP + 3.0 m. Figure 74 and Figure 75 show the situation without any pumps present for 0.5 m and 1.0 m sea level rise respectively. For both cases with sea level rise, failure occurs. The failures occur because the water level rises too quickly, and when the water level at sea is equal to the water level behind the barrier, the barrier cannot be opened because the water level is much higher than the closure level. Should there be chosen to open the barrier, the river dikes will fail, and the hinterland will be flooded.

Figure 76 - Figure 78 depict the same situations, but with the pump capacity of Delta21 available. In these runs no failures are observed.

```
Qout : 0.0 m3/s
Qin : 6350.0 m3/s
Qchannel : 351.3 m3/s
Seiche amplitude : 0.45 m
Storm set-up amplitude: 3.0 m
Failure: 1
```

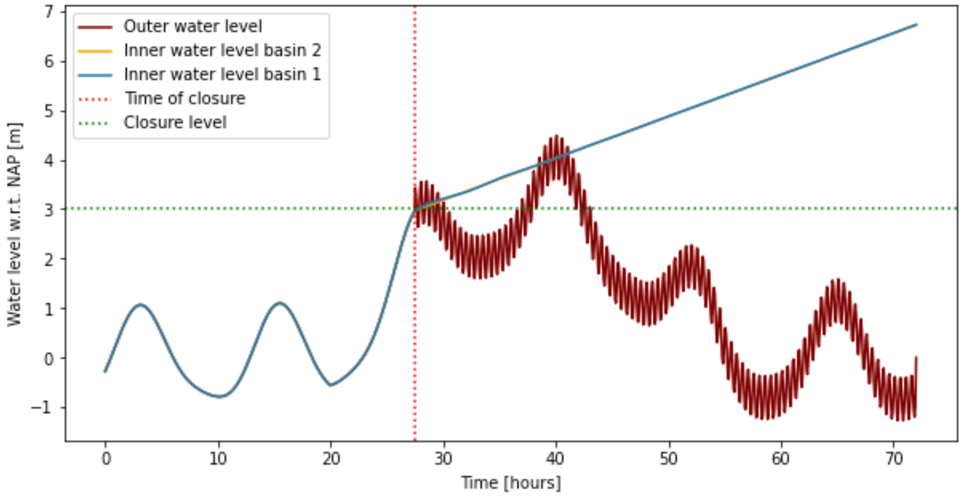


Figure 73: situation without Delta21 and sea level rise

```
Qout : 0.0 m3/s
Qin : 6350.0 m3/s
Qchannel : 364.58 m3/s
Seiche amplitude : 0.45 m
Storm set-up amplitude: 3.0 m
Failure: 0
```

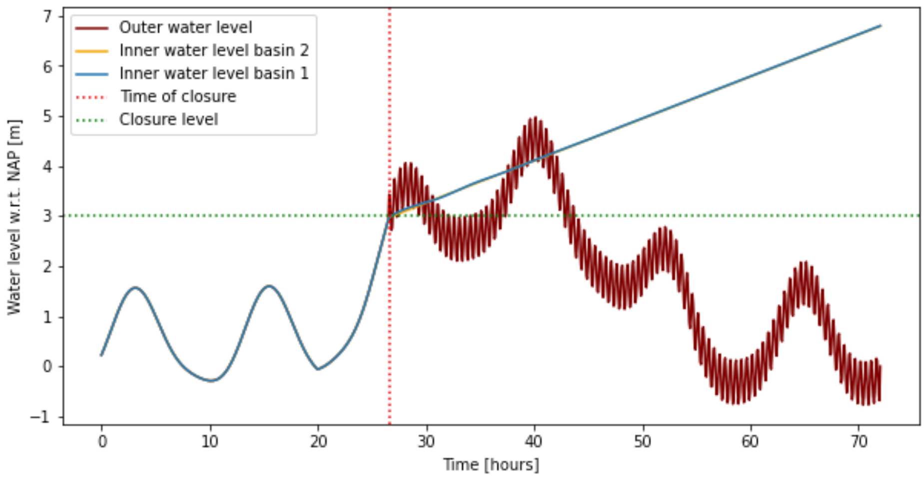


Figure 74: situation without Delta21 and 0.5m sea level rise

```
Qout : 0.0 m3/s
Qin : 6350.0 m3/s
Qchannel : 345.17 m3/s
Seiche amplitude : 0.45 m
Storm set-up amplitude: 3.0 m
Failure: 0
```

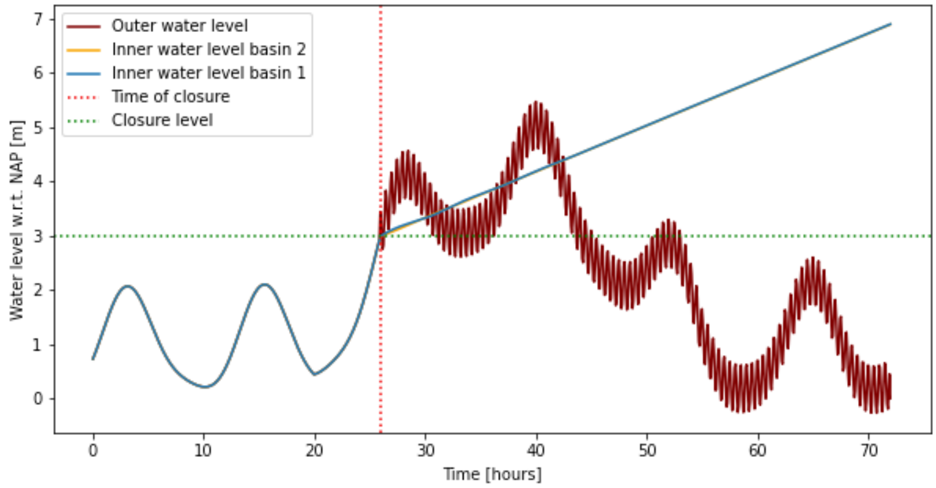


Figure 75: situation without Delta21 and 1 m sea level rise

```
Qout : 10000.0 m3/s
Qin : 6350.0 m3/s
Qchannel : 1684.03 m3/s
Seiche amplitude : 0.45 m
Storm set-up amplitude: 3.0 m
Failure: 0
Time closed: 16 hours
```

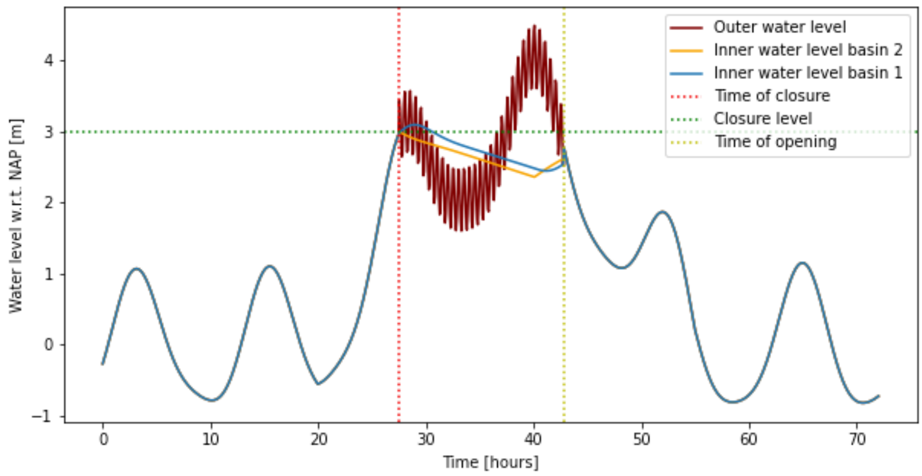


Figure 76: situation with Delta21 without no sea level rise

```
Qout : 10000.0 m3/s
Qin : 6350.0 m3/s
Qchannel : 1688.56 m3/s
Seiche amplitude : 0.45 m
Storm set-up amplitude: 3.0 m
Failure: 0
Time closed: 17 hours
```

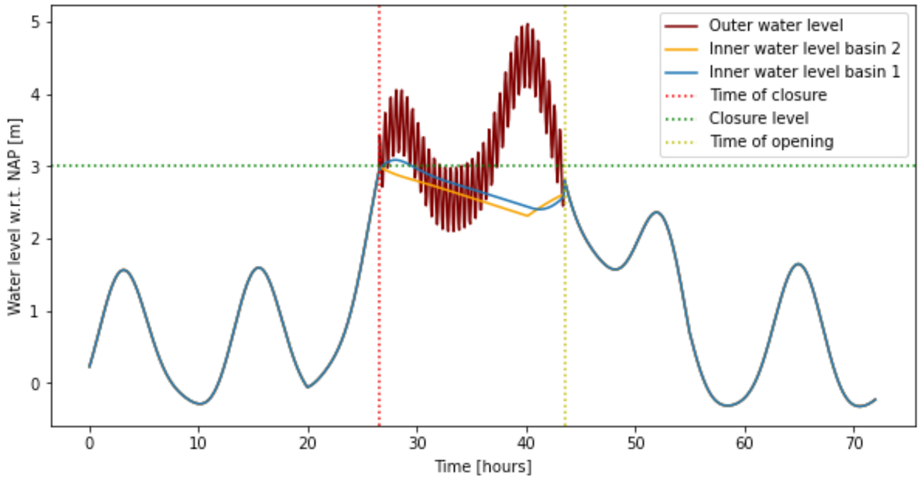


Figure 77: situation with Delta21 and 0.5 m sea level rise

```
Qout : 10000.0 m3/s
Qin : 6350.0 m3/s
Qchannel : 1685.07 m3/s
Seiche amplitude : 0.45 m
Storm set-up amplitude: 3.0 m
Failure: 0
Time closed: 19 hours
```

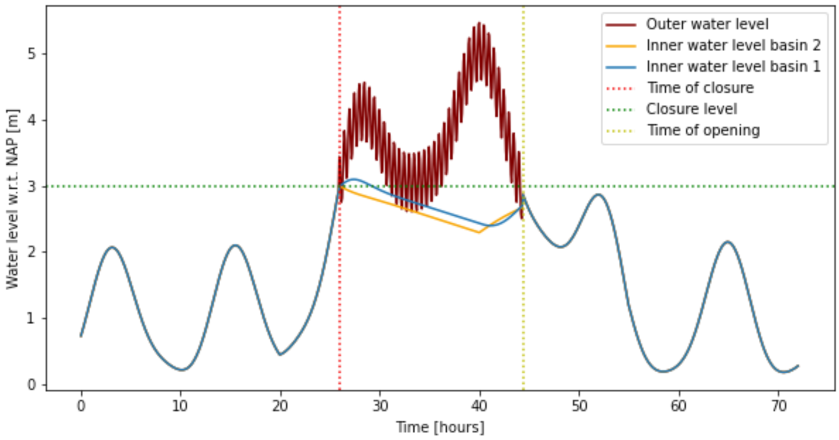


Figure 78: the same situation with Delta21 and 1 m sea level rise

The incoming river discharge & closure level

The runs which were done were with already high discharges for the Rhine and Meuse. However, the discharges can be increased further to see what happens with the water levels. This situation will be simulated for mode 2 (pumping at full capacity) only.

For the next runs, a Rhine discharge of 10,000 m^3/s and a Meuse discharge of 3,000 m^3/s will be chosen (both with an exceedance probability of 10^{-2} per year), with a seiche amplitude of 0.45m will be used.

```
Qout : 10000.0 m3/s
Qin : 11500.0 m3/s
Qchannel : 1794.53 m3/s
Seiche amplitude : 0.45 m
Storm set-up amplitude: 3.0 m
Failure: 1
Time closed: 15 hours
Time at failure: 33 hours
```

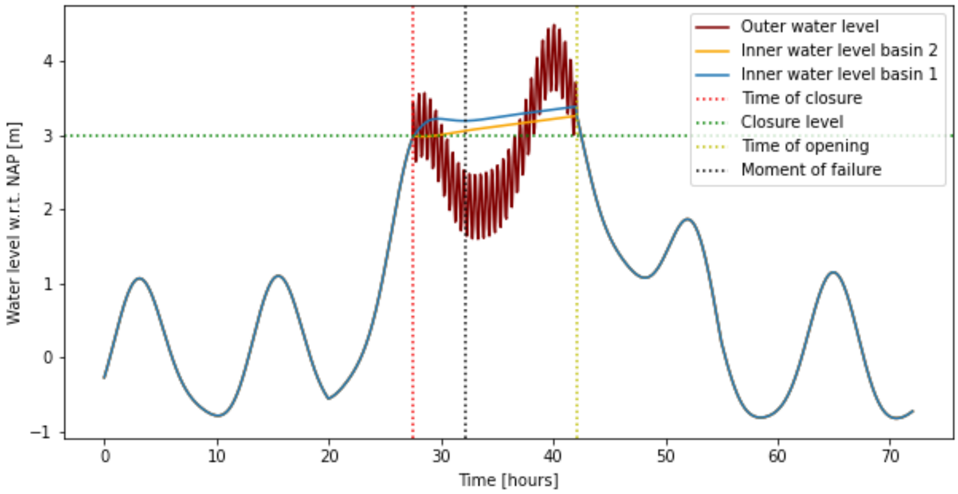


Figure 79: extremely high river discharges with seiches of 0.45m

Figure 79 shows that Delta21 fails to reduce the water level in combination with the extremely high discharges which were chosen. Failure occurs at T = 33 hours. However, by lowering the closure level to NAP + 2.5 m, failure does not occur. Figure 80 depicts this situation. The same conclusion can be drawn for even higher discharges.

```
Qout : 10000.0 m3/s
Qin : 11500.0 m3/s
Qchannel : 1778.8 m3/s
Seiche amplitude : 0.45 m
Storm set-up amplitude: 3.0 m
Failure: 0
Time closed: 17 hours
```

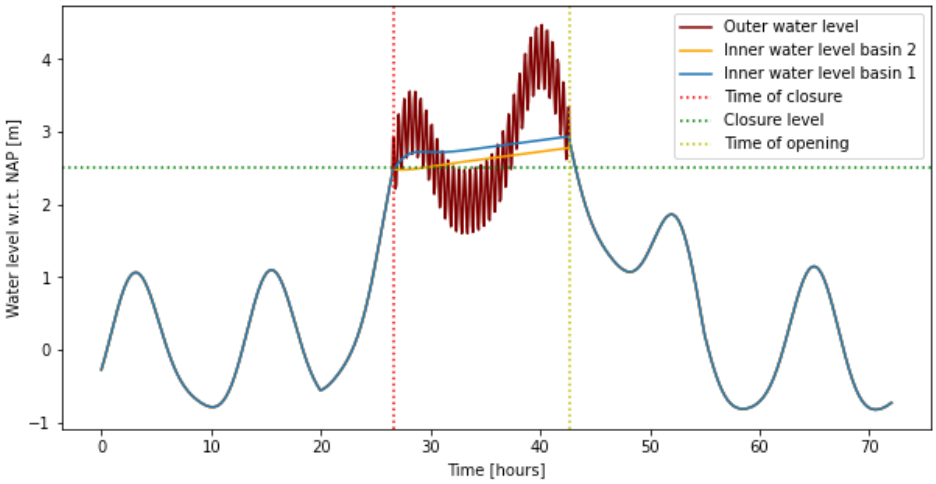


Figure 80: extremely high river discharges, but with a lower closure level

By choosing a discharge of 16,000 m^3/s for the Rhine, and 4,000 m^3/s for the Meuse (both with an exceedance probability of 10^{-3} per year), the impact of choosing the right closure level becomes very clear. This is depicted in Figure 81 and Figure 82.

```
Qout : 10000.0 m3/s
Qin : 17600.0 m3/s
Qchannel : 2148.42 m3/s
Seiche amplitude : 0.45 m
Storm set-up amplitude: 3.0 m
Failure: 1
```

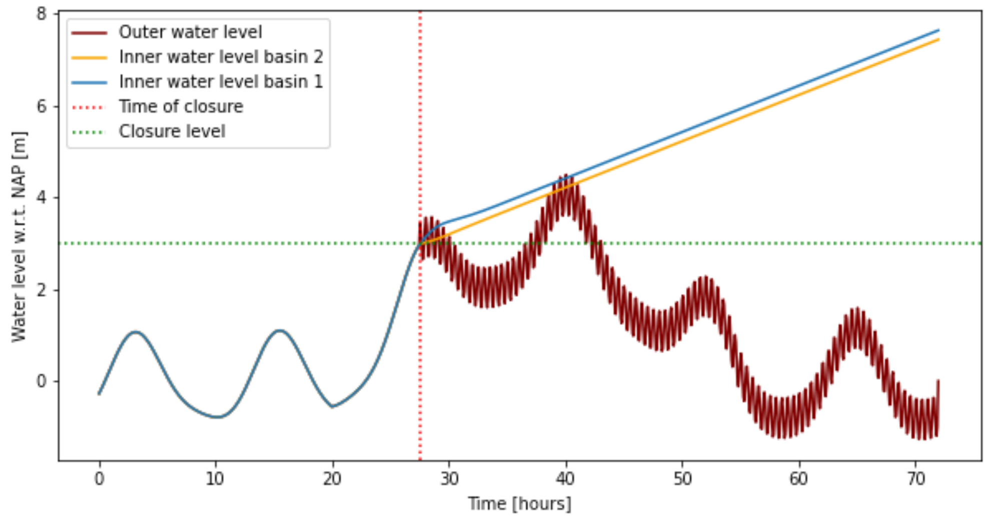


Figure 81: extremely high river discharges with a closure level of NAP + 3,0 m

```
Qout : 10000.0 m3/s
Qin : 17600.0 m3/s
Qchannel : 2189.39 m3/s
Seiche amplitude : 0.45 m
Storm set-up amplitude: 3.0 m
Failure: 0
Time closed: 18 hours
```

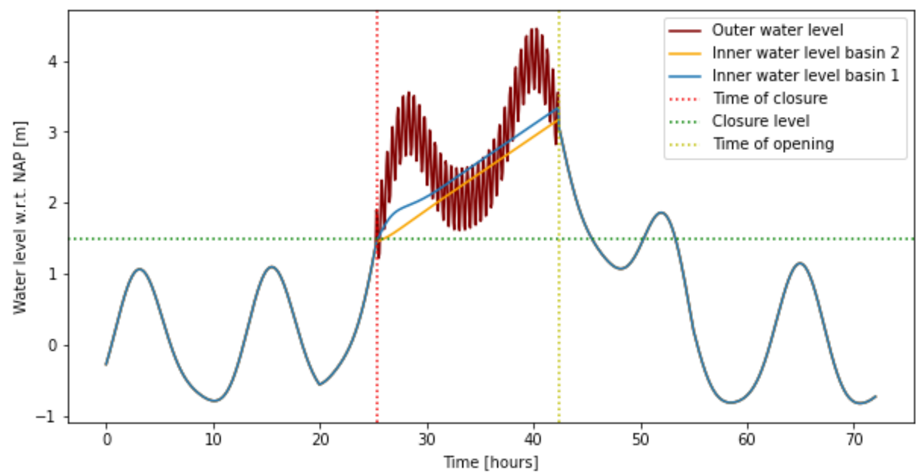


Figure 82: extremely high river discharges with a closure level of NAP + 1.5 m

Failure occurs with a closure level of NAP + 3.0 m, because the water level inside gets too high before the barrier can be opened. When the water levels outside and behind the barrier are equal, the barrier cannot be opened since the maximum water level of the storm occurs at that time (Figure 81). By lowering the closure level to NAP + 1.5 m, the situation can still be managed (Figure 82) and failure can be avoided. Note that the barrier opens at NAP + 3.0 m in this simulation, to minimize the closure time. This can be done because the expected water level will not rise above the closure level anymore.

D Analysis of the runs in which the barrier failed

In this appendix, the runs in which a negative head difference larger than 1.5 m occurred will be analysed for all the four Monte Carlo simulations which were done.

Run 1: maintaining a target water level without sea level rise

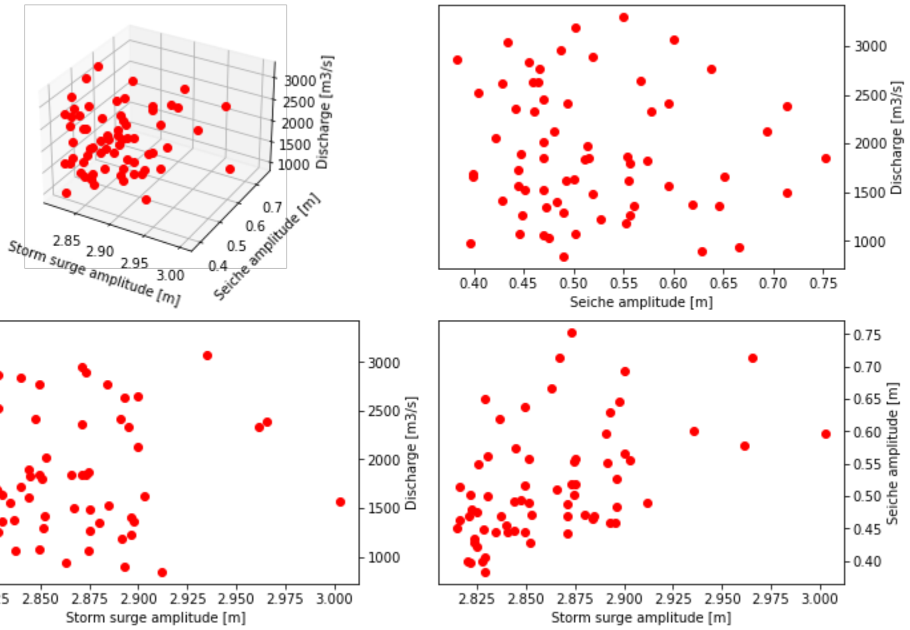


Figure 83: the values of the discharge, seiche- and storm amplitude for the runs in which a negative head difference larger than 1.5 m was observed when maintaining a target water level (without sea level rise)

The results have been plotted in Figure 83. In the top left plot, the values for the discharge, storm- and seiche amplitudes have been plotted in 3D. The remaining plots show the 2D plot between the variables. The conclusion is that for low storm amplitudes in combination with medium sized seiches and average discharge, failure occurs. Figure 84 and Figure 85 depict what the exceedance probability for the seiche- and storm amplitude (high water level) are in the failed runs. The exceedance probabilities of the seiche amplitudes in the failed runs are concentrated. The same holds for the storm amplitude.

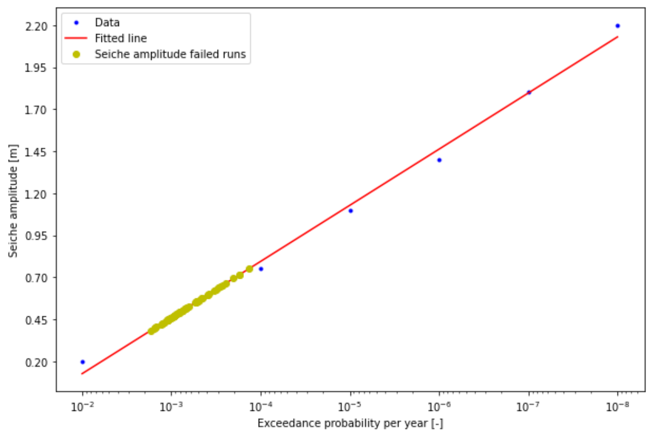


Figure 84: exceedance probabilities for the seiche amplitudes in the failed runs

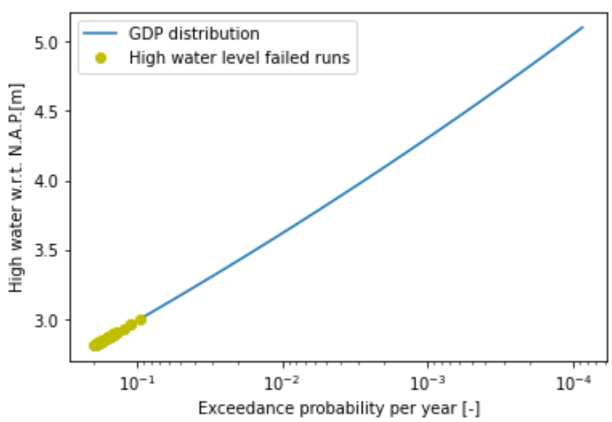


Figure 85: exceedance probabilities for the storm amplitude

Run 2: maintaining a target water level with 0.6 m sea level rise

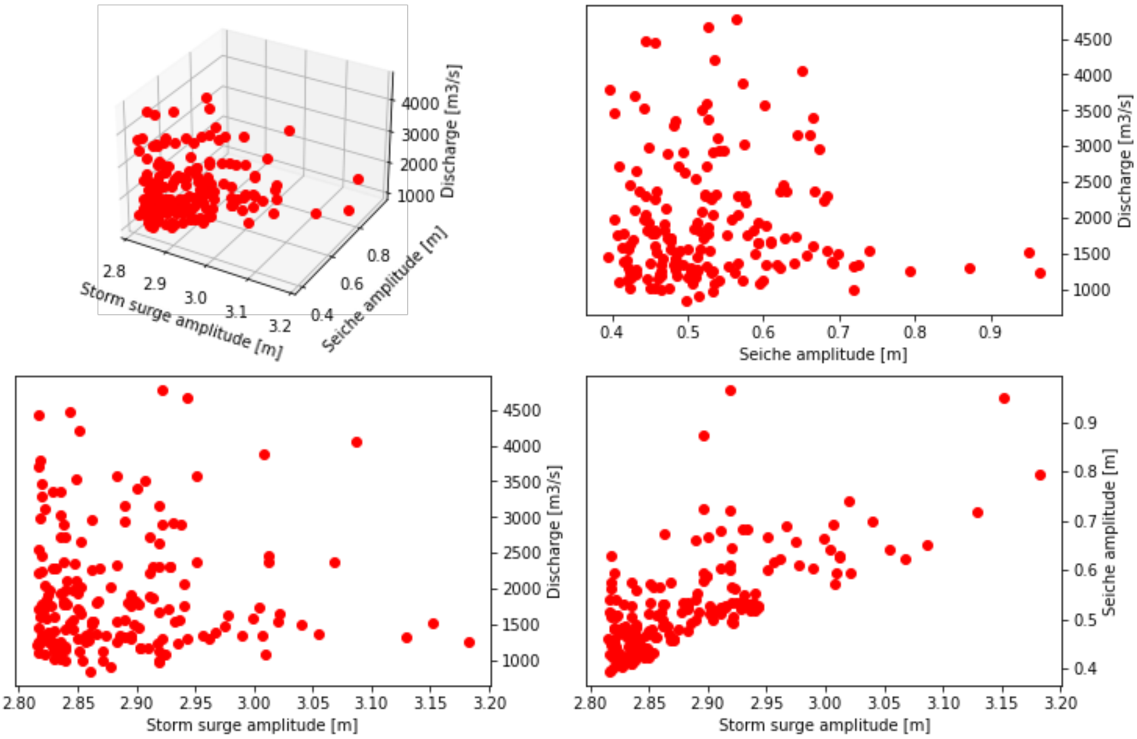


Figure 86: the values of the discharge, seiche- and storm amplitude for the runs in which a negative head difference larger than 1.5 m was observed when maintaining a target water level with 0.6 m level rise

The results have been plotted in Figure 86. In the top left plot, the values for the discharge, storm- and seiche amplitudes have been plotted in 3D. The remaining plots show the 2D plot between the variables. The conclusion is that for low seiche amplitudes, high discharges cause failure and for high seiche amplitudes, low discharges can cause failure (this can be concluded from the top right figure). Figure 87 and Figure 88 depict what the exceedance probability for the seiche- and storm amplitude (high water level) are in the failed runs. The exceedance probabilities of the seiche amplitudes in the failed runs are fairly concentrated. The same holds for the storm amplitude.

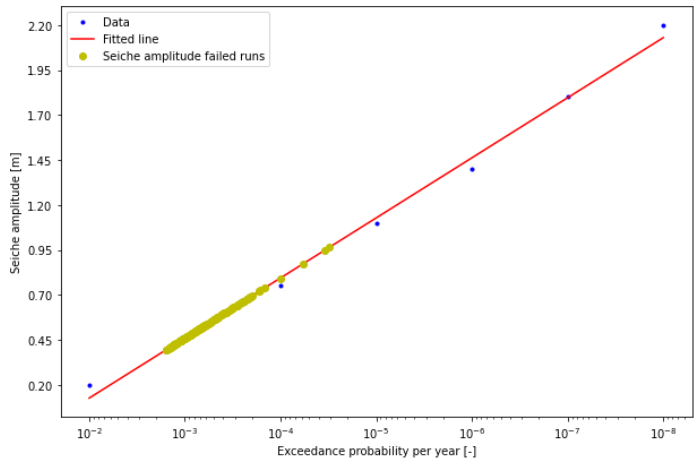


Figure 87: exceedance probabilities for the seiche amplitudes in the failed runs

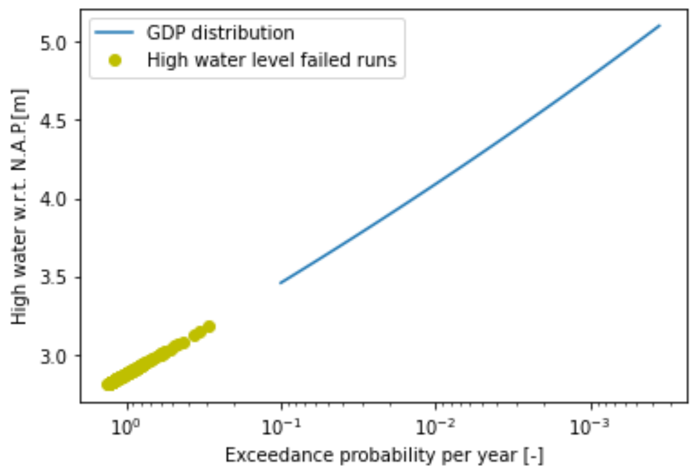


Figure 88: exceedance probabilities for the storm amplitude

Run 3: pumping at full capacity without sea level rise

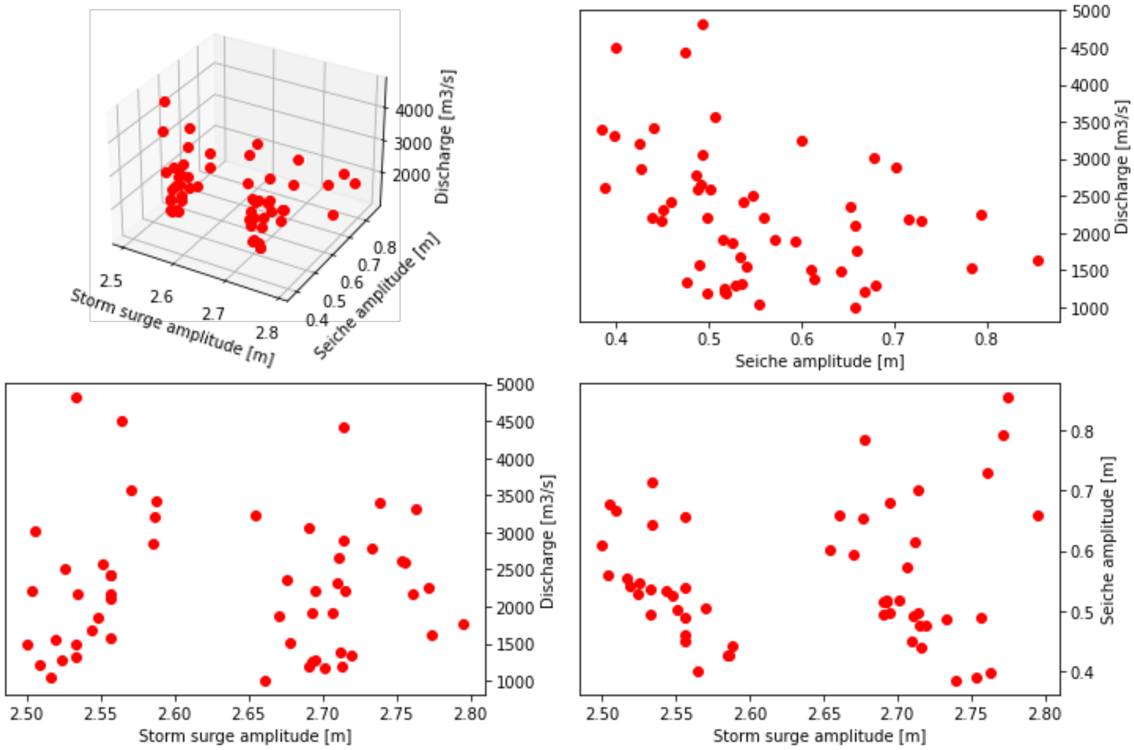


Figure 89: the values of the discharge, seiche- and storm amplitude for the runs in which a negative head difference larger than 1.5 m was observed when pumping at full capacity (without sea level rise)

The results have been plotted in Figure 89. In the top left plot, the values for the discharge, storm- and seiche amplitudes have been plotted in 3D. The remaining plots show the 2D plot between the variables. The conclusion is that low storm amplitudes cause failure (Figure 91). Figure 90 and Figure 91 depict what the exceedance probability for the seiche- and storm amplitude (high water level) are in the failed runs. The exceedance probabilities of the seiche amplitudes in the failed runs are concentrated. The same holds for the storm amplitude.

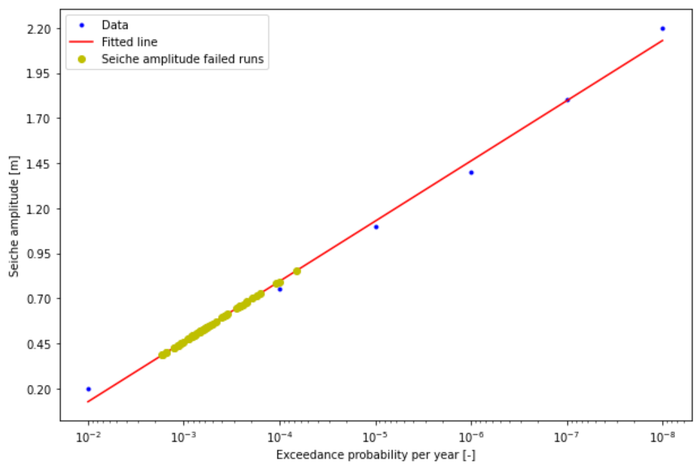


Figure 90: exceedance probabilities for the seiche amplitudes in the failed runs

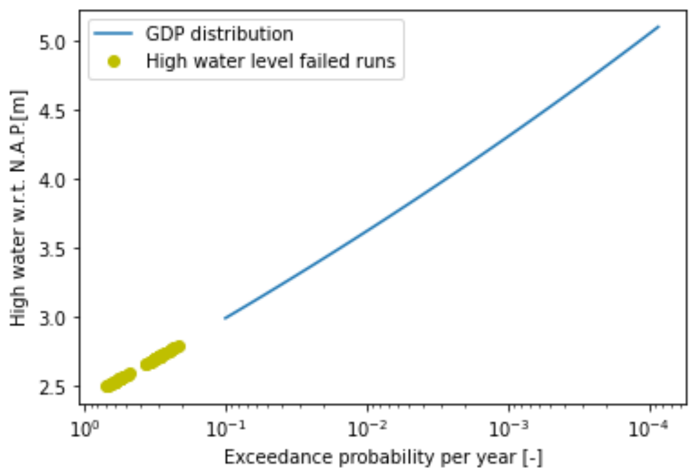


Figure 91: exceedance probabilities for the storm amplitude

Run 4: pumping at full capacity with 0.6 m sea level rise

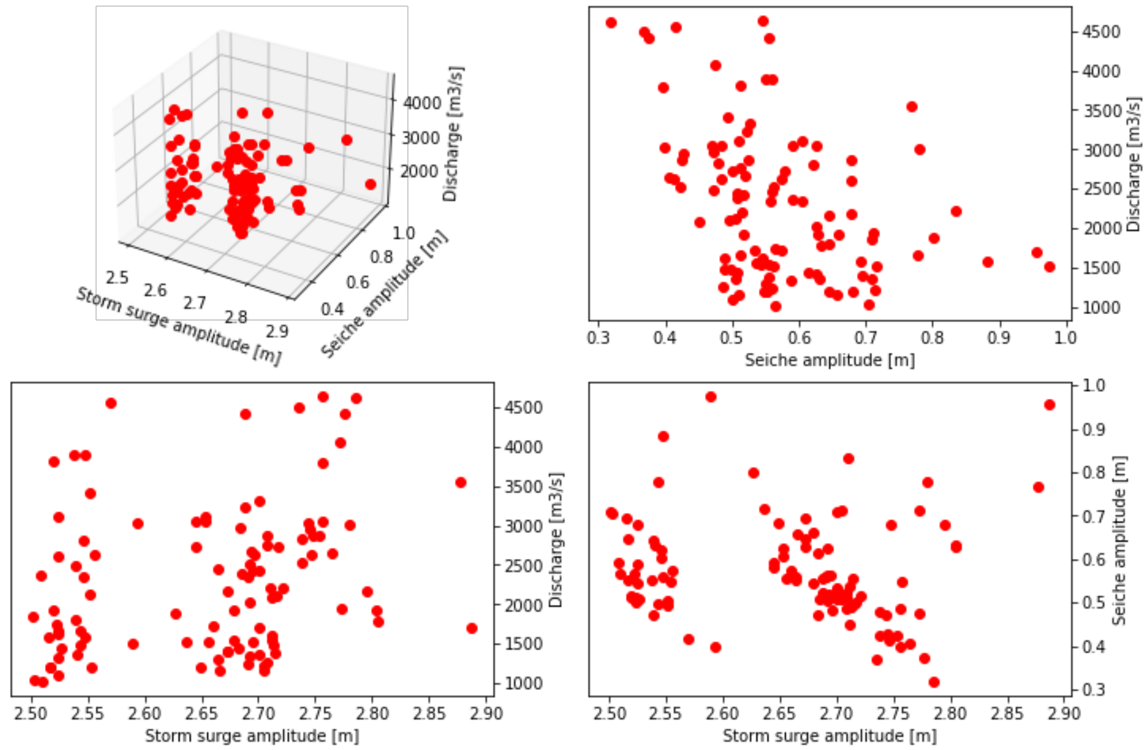


Figure 92: the values of the discharge, seiche- and storm amplitude for the runs in which a negative head difference larger than 1.5 m was observed when pumping at full capacity with 0.6 m sea level rise.

The results have been plotted in Figure 92. In the top left plot, the values for the discharge, storm- and seiche amplitudes have been plotted in 3D. The remaining plots show the 2D plot between the variables. For lower storm amplitudes paired with higher seiche amplitudes, more failures occur. Also, for lower storm amplitudes with high discharges more failures occur as well. Figure 93 and Figure 94 depict what the exceedance probability for the seiche- and storm amplitude (high water level) are in the failed runs. The exceedance probabilities of the seiche amplitudes in the failed runs are less concentrated than for mode 1, while it is quite scattered for the storm amplitude.

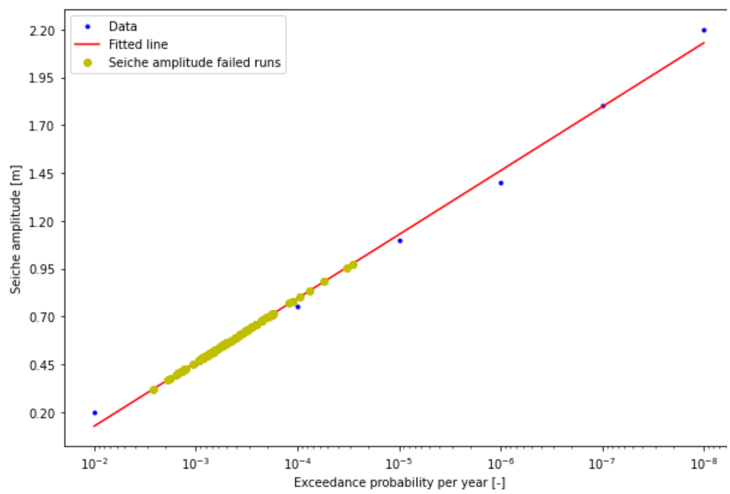


Figure 93: exceedance probabilities for the seiche amplitudes in the failed runs

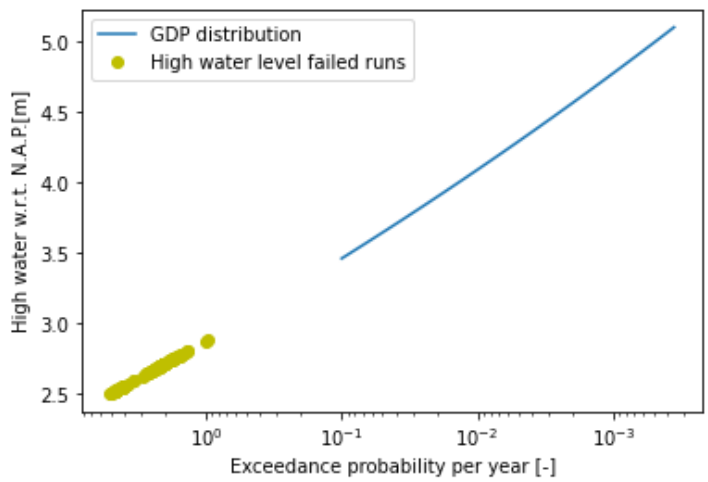


Figure 94: exceedance probabilities for the storm amplitudes in the failed runs

Run 5: current situation without sea level rise

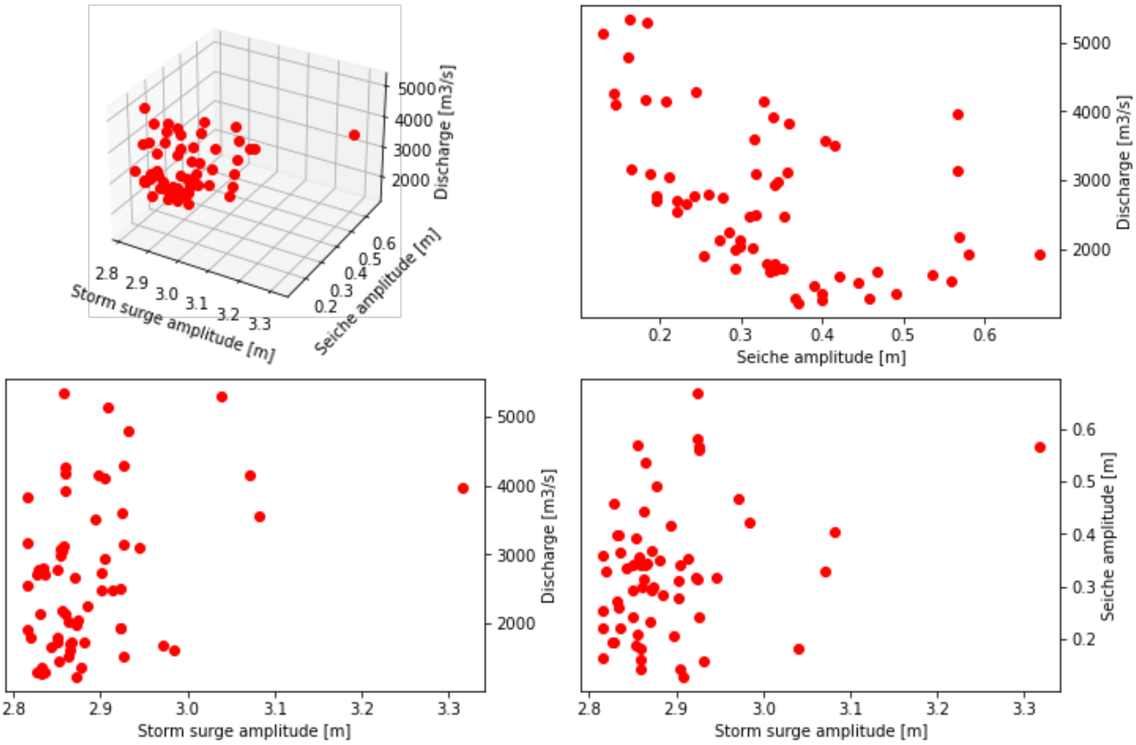


Figure 95: the values of the discharge, seiche- and storm amplitude for the runs in which a negative head difference larger than 1.5 m was observed for the current situation.

The results have been plotted in Figure 96. In the top left plot, the values for the discharge, storm- and seiche amplitudes have been plotted in 3D. The remaining plots show the 2D plot between the variables. For lower storm amplitudes paired with various seiche amplitudes, more failures occur. Also, for lower storm amplitudes with high discharges more failures occur as well. Figure 96 and Figure 97 depict what the exceedance probability for the seiche- and storm amplitude (high water level) are in the failed runs. The exceedance probabilities of the seiche amplitudes in the failed runs are less concentrated at the lower amplitudes. The same holds for the storm amplitudes.

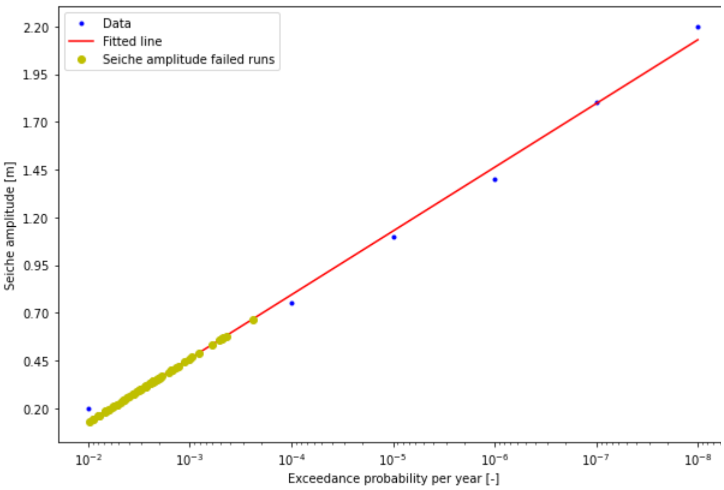


Figure 96: exceedance probabilities for the seiche amplitudes in the failed runs

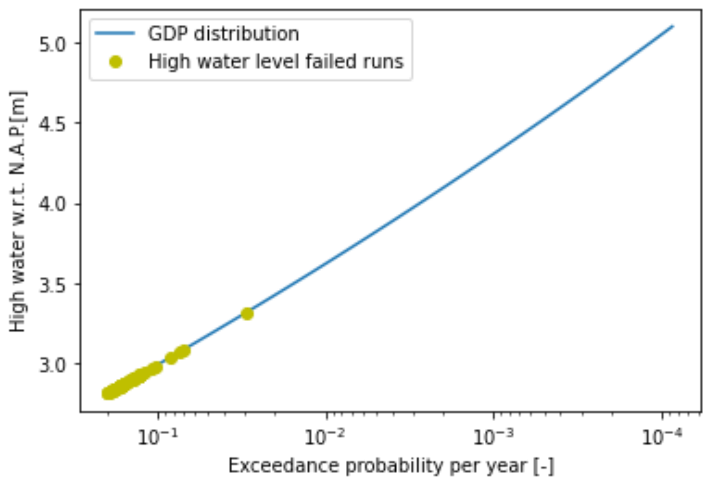


Figure 97: exceedance probabilities for the storm amplitudes in the failed runs

Run 6: current situation with 0.6 m sea level rise

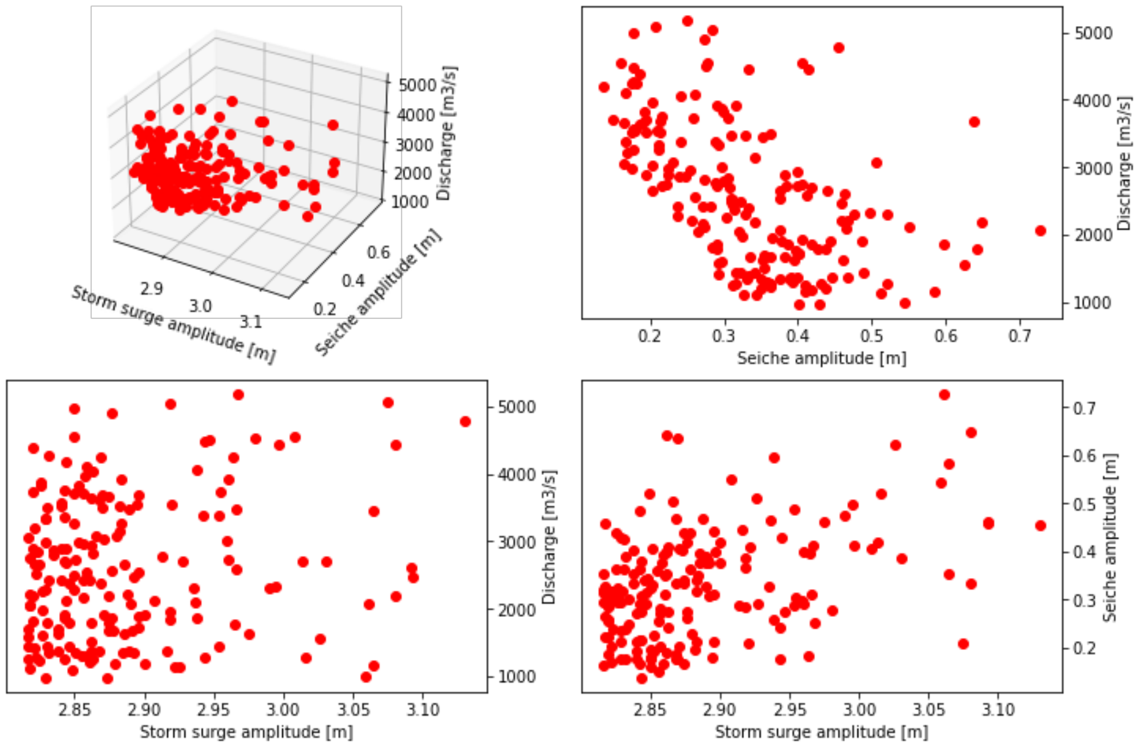


Figure 98: the values of the discharge, seiche- and storm amplitude for the runs in which a negative head difference larger than 1.5 m was observed for the current situation with 0.6 m sea level rise.

The results have been plotted in Figure 98. In the top left plot, the values for the discharge, storm- and seiche amplitudes have been plotted in 3D. The remaining plots show the 2D plot between the variables. For lower storm amplitudes paired with higher seiche amplitudes, more failures occur. Also, for lower storm amplitudes with high discharges more failures occur as well. Figure 99 and Figure 100 depict what the exceedance probability for the seiche- and storm amplitude (high water level) are in the failed runs. The exceedance probabilities of the seiche amplitudes in the failed runs are concentrated at the lower values again. The same holds for the storm amplitudes.

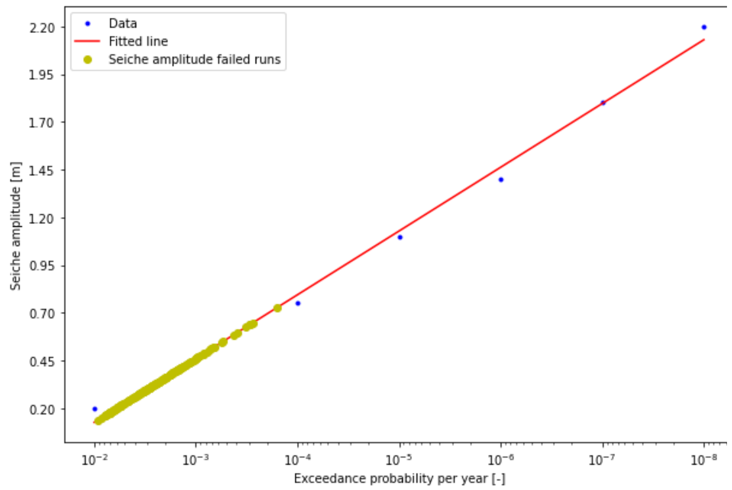


Figure 99: exceedance probabilities for the seiche amplitudes in the failed runs

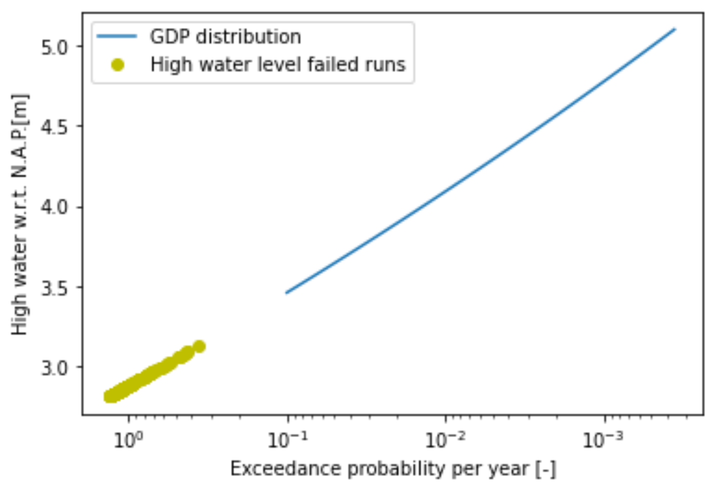


Figure 100: exceedance probabilities for the storm amplitudes in the failed runs

E Python script for the hydraulic model

```
#importing the libraries
import numpy as np
import matplotlib.pyplot as plt
from scipy.stats import norm
from scipy.stats import linregress
from peakdetect import peakdetect
import random
import time
import math
from random import choices
from mpl_toolkits.mplot3d import Axes3D
%matplotlib inline

#simulation time
hours = 72
t = np.linspace(0,hours,1000)

#function for returning a draw from a normal distribution
def normaldist(m,s):
    mu = m
    sigma = s*mu
    x = np.linspace(mu-3*sigma, mu+3*sigma,100)
    y = norm.pdf(x,mu,sigma)
    draw = choices(x,weights=y)
    return draw[0]

#probabilistic basin size
def basinsize():
    mu = 255*100**3
    sigma = 0.01*mu
    areas = np.linspace(mu-3*sigma, mu+3*sigma,100)
    y = norm.pdf(areas,mu,sigma)
    draw = choices(areas,weights=y)
    #plt.figure()
    #fig = plt.figure(figsize=(10,6.67))
    #plt.plot(areas,y,color='red')
```

```
    #plt.xlabel('Basin size [$km^2$]')
    #plt.ylabel('Probability density [-]')
    return draw[0]

#probabilistic outflow discharge
def outflow(m):
    mu = m
    sigma = 0.025*mu
    out = np.linspace(mu-3*sigma, mu+3*sigma,100)
    y = norm.pdf(out,mu,sigma)
    draw = choices(out,weights=y)
    return draw[0]

#extreme value distribution rivers
def inflowrijn():
    Qrijn = np.array([16000,14000,13000,12000,10000,8000,6000,4000,3600,3000,2600,2100,2000,1800,1590,1100,700])
    Tw = np.array([1/1250,1/570,1/260,1/55,1/13,1/3.5,5,26,36,61,92,159,176,212,252,337,364])
    xss = np.linspace(min(Tw),max(Tw),500)
    xexp = np.log(Tw)
    fittedr = linregress(xexp,Qrijn)
    yexpr = fittedr[1] + fittedr[0]*np.log(xss)
    rdrijn = choices(yexpr,weights=np.log(xss))
    rdwaal = 0.64*rdrijn[0]
    rdnederrijn = 0.21*rdrijn[0]
    waal = normaldist(rdwaal,0.1)
    nederrijn = normaldist(rdnederrijn,0.1)
    #fig = plt.figure(figsize=(10,6.67))
    #plt.semilogx(Tw,0.21*Qrijn,'b.',label='Data')
    #plt.semilogx(xss,0.21*yexpr,'r',label='Fitted line')
    #plt.xlabel('Exceedance probability per year [-]')
    #plt.ylabel('Discharge [m$^3$/s]')
    #plt.gca().invert_xaxis()
    #plt.legend(loc='best')
    return waal, nederrijn

#maas
def inflowmaas():
    Qm = [4396,4113,3862,3578,3226,2969,2781,2609,2308,1982,1500,1280,1030,530,280,155,80]
    Tm = np.array([1/10000,1/3000,1/1000,1/300,1/100,1/50,1/30,1/20,1/10,1/5,2,4,8,45,107,184,268])
    xss = np.linspace(min(Tm),max(Tm),500)
    xexp = np.log(Tm)
```

```
    fitted = linregress(xexp,Qm)
    yexp = fitted[1] + fitted[0]*np.log(xss)
    rdm = choices(yexp,weights=np.log(xss))
    maas = normaldist(rdm[0],0.1)
    #fig = plt.figure(figsize=(10,6.67))
    #plt.semilogx(Tm,Qm,'b.',label='Data')
    #plt.semilogx(xss,yexp,'r',label='Fitted line')
    #plt.xlabel('Exceedance probability per year [-]')
    #plt.ylabel('Discharge [m$^3$/s]')
    #plt.gca().invert_xaxis()
    #plt.legend(loc='best')
    return maas
#
#
#
#
#
#
#
#
#

def seiches(mode):
    probexc = [10**(-2),10**(-3),10**(-4),10**(-5),10**(-6),10**(-7),10**(-8)]
    amplitudes = [0.20,0.45,0.75,1.10,1.40,1.80,2.20]
    xse = np.linspace(min(probexc),max(probexc),500)
    xsexp = np.log(probexc)
    fitted = linregress(xsexp,amplitudes)
    ysexp = fitted[1] + fitted[0]*np.log(xse)
    ri = choices(ysexp,weights=xse)
    amp = normaldist(ri[0],0.1)
    if mode == 1:
        a = amp
    if mode == 0:
        a = 0.45
    T = 30*60
    d = 15
    c = np.sqrt(9.81*d)
    L = c*T
    Lbasin = 5000
    w = (2*np.pi)/T
```

```
        k = (2*np.pi)/L
        x = np.linspace(0,Lbasin,len(t))
        nseiche = a*np.cos(w*(t*3600)-k*x)
        return nseiche, a

    #function for the friction coefficient
    def friction(mean):
        draw = normaldist(mean,0.01)
        return normaldist(draw,0.1)

    #tide parameters
    #amplitude params M2, S2, M4
    def tide():
        an = [0.79, 0.19, 0.17]
        wn = [0.50722,0.525,1.01]
        alphan = [1.505, 2.5725, 2.8875]

        #calculate the surface elevation changes due to the tide (with M2,S2 and M4 components)
        tide = np.zeros(len(t))
        for j in range(len(t)):
            ntide = 0
            for i in range(len(an)):
                ntide = ntide + an[i]*np.cos((wn[i]*t[j])-alphan[i])
            tide[j] = ntide
        ntidetotal = tide
        return ntidetotal

    #used to check various things, source:
    #https://www.codegrepper.com/code-examples/python/find+closest+number+to+zero+in+array+in+python
    def find_nearest(array, value):
        array = np.asarray(array)
        idx = (np.abs(array - value)).argmin()
        return array[idx], idx

    #ssh parameters
    #sstorm start moment, duration
    def storm(mode, SLR):
        Tstormstart = 20               #timestamp in hours when the storm starts
        Tstorm = 35                    #storm duration
        Tstormend = Tstormstart + Tstorm

```

```
        #create timespace for the storm for which the storm only acts on the specified start time and duration
        tstorm = np.zeros(len(t))
        indexbegin = 0
        indexend = 0
        for i in range(len(t)):
            if Tstormstart < t[i]:
                break
            indexbegin = i

        for j in range(len(t)):
            if Tstormend < t[j]:
                break
            indexend = j
        #print(indexend)
        #set the indexes in which the storm should act, based on the duration and start time
        for i in range(indexend-indexbegin):
            tstorm[indexbegin + i] = 1


        #create storm shape, and (random) maximum amplitude
        #data
        hwl = [3,3.15,3.40,3.60,3.80,4.10,4.30,4.50,4.70,4.80,5.10]
        exhwl = [0.1,0.05,0.02,0.01,0.005,0.002,0.001,0.0005,0.00025,0.0002,0.0001]

        #if SLR is desired, the threshold value of 3.05m should be used instead of 2.58m
        #fitting the data
        if SLR == 0:
            x1 = np.linspace(2.99007265375027 66,5.10,100)
            q1 = np.zeros(len(x1))
            for i in range(len(x1)):
                q1[i] = 0.5*(1+0.03641*((x1[i]-2.58)/0.2474))**(-1/0.03641)
            draw = choices(x1,weights=q1,k=1)

        if SLR == 1:
            x2 = np.linspace(3.4600726537502764,5.10,100)
            q2 = np.zeros(len(x2))
            for i in range(len(x2)):
                q2[i] = 0.5*(1+0.03641*((x2[i]-3.05)/0.2474))**(-1/0.03641)
            draw = choices(x2,weights=q2,k=1)

        ampl = normaldist(draw[0],0.1)
```

```

        if mode == 1:
            #since the tide is included in the HWL's,
            #it needs to be subtracted from the picked HWL because it will be added again to the tide
            tideamplitude = (tide().max()-tide().min())/2
            Amax = ampl - tideamplitude
        if mode == 0:
            Amax = 3
        nsurge = tstorm
        for i in range(indexend-indexbegin):
            nsurge[indexbegin+i] = Amax*np.sin(2*np.pi*(t[indexbegin+i]-t[indexbegin])/(2*Tstorm))

        #total outer elevation before closure: tide and storm surge contributing
        zout_open = tide() + nsurge
        return zout_open,indexend, Amax

    #Set the closure level for when the barrier closes, at HvH
    def simulation(plot,mode,probabilistic,SLR):

        #define closure level and target water level for the basin when the barrier is closed.
        closurelevel = 3

        #define failure parameter (initially 0)
        fail = 0

        closure = 0
        #initialize parameters
        zout_open, indexend, Amax = storm(probabilistic,SLR)

        #only run the simulation if the barrier actually has to close
        if zout_open.max() >= 1.05*closurelevel:
            #state that the barrier closes
            closure = 1

            #initialize the remaining parameters
            nseiche, aseiche = seiches(probabilistic)
            ntidetotal = tide()
            waal, rijn = inflowrijn()
            maas = inflowmaas()
            A1 = 0.15*basinsize()
            A2 = 0.85*basinsize() + 15*100**3
```

```python

        #run the model in probabilistic mode
        if probabilistic == 1:
            sensitivity = 0.1
            Qin1 = rijn
            Qin2 = maas + waal
            if mode == 'SP':
                uit = math.ceil((Qin1+Qin2)/100)*100
                Qout = outflow(m=uit)
            if mode == 'D21':
                Qout = outflow(m=10000)
            if mode == 'CS':
                Qout = 0
            if plot == 1:
                print('Qout :',math.ceil(Qout*100)/100,'m3/s')

        #run the model in deterministic mode
        if probabilistic == 0:
            Qr = 6000
            Qm = 1250
            Qin1 = 0.21*Qr
            Qin2 = 0.64*Qr+Qm
            sensitivity = 0.2
            if mode == 'SP':
                Qout = math.ceil((Qin1+Qin2)/100)*100
            if mode == 'D21':
                Qout = 10000
            if mode == 'CS':
                Qout = 0
            if plot ==1:
                print('Qout :',math.ceil(Qout*100)/100,'m3/s')

        #get the time index where the closure level is met in time
        final = 0
        for i in range(len(zout_open)):
            if zout_open[i] >= closurelevel:
                break
            final = i

        #inner and outer level equal until closure:
        zin_closed = np.zeros(len(t))
```

```
        zout_closed = np.zeros(len(t))
        for i in range(final+1):
            zin_closed[i] = zout_open[i]
            zout_closed[i]= zout_open[i]

        #when the barrier is closed, seiches can occur on the outer side if the water level exceeds 2.5m, so add seiches from the moment of closure
        #until the end of the simulation

        for i in range(len(zout_open)-final-1):
            if Amax >= 2.5 :
                zout_closed[final + i] = zout_open[final+i] + nseiche[i]
            else:
                zout_closed[final + i] = zout_open[final+i]

        #channel properties and initial conditions
        B = 453
        d = 10
        Ac = B*d
        L = 18000
        cf = friction(0.004)
        chi = 0.5 + cf*L/d
        g = 9.81

        #create a timeseries over which the inner water level changes (dz) due to storage, in- and outflow
        Qch = np.zeros(len(t))
        dt = (t[1]-t[0])*3600

        #create array for the water levels in basin 2
        zb3 = np.zeros(len(zin_closed))
        zb3[0:final] = zin_closed[0:final]
        zb3[final] = zin_closed[final] -((Qin2-Qout)/A2)*dt

        #find the water level at closure, and update the water level inside the basin using the previously calculated dz
        #water level in basin 1 (behind the Maeslant barrier) is zin_closed
        #water level in basin 2 (haringvliet) is zb2
        maximum = zout_closed.argmax()

        for i in range(len(t)-final-1):
            #optimise discharging procedure
            if (((final+i) >= maximum) and (mode == 'D21') and (zin_closed[final+i] <= closurelevel)):
```

```
                Qout = 0
            #update the water level in basin 1
            dz1 = ((Qin1-Qch[final+i])/A1)*dt
            zin_closed [final+i+1] = zin_closed[final+i] + dz1

            #update the water level in basin 2
            dz2 = (((Qch[final+i]+Qin2)-Qout)/A2)*dt
            zb3 [final+i+1] = zb3[final+i]+dz2

            #calculate the discharge in the channel
            kappa = chi*np.abs(Qch[final+i])/Ac
            dQ = g*Ac*(zin_closed[final+i+1]-zb3[final+i+1])-kappa*Qch[final+i]
            Qch[final+i+1] = Qch[final+i] + dQ/(L/dt+2*kappa)

        #check when the water level is below closure level, and check for all modes when the barrier can be opened
        if mode == 'SP':
            indexopen = 0
            index = 0
            for i in range(len(t)-maximum):
                if (zout_closed[maximum+i] > 1.05*closurelevel):
                    index = maximum + i
                indexopen = index
                indexopen2 = indexopen

        if (mode =='D21') or (mode =='CS'):
            indexopen = 0
            indexopen2 = 0
            index = 0
            for i in range(len(t)-maximum):
                if (zout_closed[maximum+i] > 1.05*closurelevel):
                    index = maximum + i
                indexopen = index
                indexopen2 = index
            diff = np.zeros(len(t))
            diff2 = diff
            delta = 0
            delta2 = 0
            for i in range(len(t)-indexopen):
                diff[indexopen+i] = np.abs(zout_closed[indexopen+i]-zin_closed[indexopen+i])
                diff2[indexopen+i] = np.abs(zout_open[indexopen+i]-zb3[indexopen+i])

```

```python



            for i in range(len(diff)-indexopen):
                if np.abs(diff[indexopen+i]) <= sensitivity:
                    break
                delta = i+1
            for i in range(len(diff2)-indexopen):
                if np.abs(diff2[indexopen+i]) <= sensitivity:
                    break
                delta2 = i+1
            indexopen = indexopen + delta
            indexopen2 = indexopen2+delta2

        #define array in which the water level differences are measured when the barrier is closed
        #continously calculate head difference while the barrier is closed
        differences = np.zeros(len(t))
        for i in range(indexopen-final):
            differences[final+i] = zout_closed[final+i]-zin_closed[final+i]

        #check if a negative head difference larger than -1.5m occurs, if so, return 1 for failure
        failindex = 0
        spui = 0
        if (mode =='SP') or (mode =='D21'):
            for i in range(indexopen-final):
                if (differences[final+i] < -1.5):
                    fail = 1
                    break
                failindex = final + i
            for i in range(maximum-final):
                if (differences[final+i] < -0.05):
                    spui = 1
                    break

            if indexopen >= len(t):
                fail = 1

        #used to calculate if there is a negative head difference without D21 (Qout=0)
        if mode =='CS':
            for i in range(maximum-final):
                if (differences[final+i] < -1.5):
```

```
                    fail = 1
                    break
                for i in range(maximum-final):
                    if (differences[final+i] < -0.05):
                        spui = 1
                        break

        #remove seiches when this moment is reached, because seiches are being looked at when the barrier is closed
        #set inner water level to equal the outer one, because the barrier is open
        for i in range(len(t)-indexopen):
            zout_closed[indexopen+i] = zout_open[indexopen+i]
            zin_closed[indexopen+i] = zout_closed[indexopen+i]
        for i in range(len(t)-indexopen2):
            zb3[indexopen2+i] = zout_open[indexopen2+i]

        #plot z_in and z_out against time. various output modes.
        if plot == 0:
            return fail, Qin1+Qin2, Amax, aseiche

        if plot == 1:
            fig = plt.figure(figsize=(10,5))
            plt.plot(t,zout_closed,label='Outer water level',color='maroon')
            plt.plot(t,zb3,label='Inner water level basin 2',color='orange')
            plt.plot(t,zin_closed,label='Inner water level basin 1')
            plt.axvline(x=t[final],label='Time of closure',color='r',ls=':')
            plt.axhline(y=closurelevel,label='Closure level',color='g',ls=':')
            plt.xlabel('Time [hours]')
            plt.ylabel('Water level w.r.t. NAP [m]')

            print('Qin :',math.ceil((Qin1+Qin2)*100)/100,'m3/s',
                  '\nQchannel :',math.ceil(Qch[final:maximum].mean()*100)/100,'m3/s',
                  '\nSeiche amplitude :',math.ceil(aseiche*1000)/1000,'m',
                  '\nStorm set-up amplitude:',math.ceil(100*Amax)/100,'m','\nFailure:',fail)
            if indexopen < len(t):
                plt.axvline(x=t[indexopen],label='Time of opening',color='y',ls=':')
                print('Time closed:',math.ceil(t[indexopen]-t[final]),'hours')

            if ((fail == 1) and (indexopen < len(t))):
                plt.axvline(x=t[failindex],label='Moment of failure',color='black',ls=':')
                print('Time at failure:',math.ceil(t[failindex]),'hours')
            plt.legend(loc='best');
```

```

            if plot == 2:
                fig2 = plt.figure(figsize=(10,5))
                plt.plot(t,Qch,label='Discharge in the channel')
                plt.axvline(x=t[final],label='Time of closure',color='r',ls=':')

                if mode == 'D21':
                    plt.axvline(x=t[maximum],label='Time when Qout = 0',color='y',ls=':')

                if indexopen < len(t):
                    plt.axvline(x=t[indexopen],label='Time of opening',color='black',ls=':')
                plt.xlabel('Time [hours]')
                plt.ylabel('Discharge [m3/s]')
                plt.legend(loc='upper right');

            if plot == 3:
                return closure, spui
        if closure == 0:
            #print('Barrier does not close, maximum expected water level < closure level')
            return closure, fail, 0, 0, 0

#Running the Monte Carlo simulation
%timeit
t1 = time.time()
counter = 0
N = 100000
data = []
for i in range(N):
    closure, fail, Q, A, aseiche = simulation(plot=0,mode='SP',probabilistic=1,SLR=1)
    if closure == 1:
        if fail == 1:
            counter = counter + 1
            test = (Q,A,aseiche)
            data.append(test)
    else:
        continue
t2 = time.time()
print('Probability of a N.H.D > 1.5 m =',counter/N,'per year',
      '\nTime taken:',(t2-t1)/60,'minutes')
np.savetxt('SPSLR.txt',data)

```

```
    #calculate the probability that discharging ("spuien") is necessarry in this model, should be the same for all modes
    %timeit
    t1 = time.time()
    counter = 0
    closures = 0
    N = 10000
    for i in range(N):
        sample = simulation(plot=3,mode='CS',probabilistic=1,SLR=0)
        if sample[0] == 1:
            closures = closures + 1
            if sample[1] == 1:
                counter = counter + 1
        else:
            continue
    t2 = time.time()
    print('\nProbability of discharging = ',counter/closures,'per closure',
          '\nTime taken:',(t2-t1)/60,'minutes')
```

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