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The effect of operational discharge capacity of pumps and sluices on flood hazards - A case study on discharging the Rhine and Meuse under sea level rise

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Abstract. Future sea level rise will likely hamper the discharge of excess water from low-lying water systems all around the world. One example of such a water system is the Rhine-Meuse delta in the Netherlands, which discharges to the North Sea. A possible mitigation strategy involves closing off the delta from the North Sea with large dams and to discharge the incoming river discharge with large pumping stations. In this study, we determine the required amount of pump capacity by including the new concept of operational discharge capacity. This way we can account for the variations in the available pump and sluice discharge capacity due to variations in the head difference between the sea and water system and possible technical malfunctions. The effect of variations in the operational discharge capacity on return periods of extreme water levels in the water system is assessed within a probabilistic and hydraulic model framework.

We find that variations in operational discharge capacity substantially increase maximum water levels in the water system and increase flood frequencies compared to simulations with the assumption of a constant and fully available discharge capacity. In one scenario of our case study, including the effect of operational discharge capacity leads to an increase in flood frequency from 1/10,000 years to 1/75 years. In our case study, most of the increase can be attributed to including sluice reliability. Including pump reliability increases the frequency of higher water levels in the reservoir, until a water level is reached at which the sluices are available. However, available sluices can prevent a further increase of reservoir water levels. The precise effect of operational discharge capacity will vary per water system and design set-up. Yet, the examples in this paper show a clear effect for most design scenarios. Therefore, the operational discharge capacity is a crucial parameter that should be taken into account in the design of pumping stations.

1 Introduction

In 2023, the Dutch Delta Program (Knowledge Programme Sea Level Rise) published four possible solutions to cope with the rising sea levels (2024a,2024b). Two of the proposed solutions involved closing off the Rhine-Meuse Delta from the sea

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and discharging the rivers by enormous pumping stations. In a first estimate the required pumping capacity would range from 3,800 m^3/s (in case of seaward expansion of the buffer capacity) to 11,900 m^3/s (for an unchanged buffer capacity) to maintain a 1:10,000 flood probability in the water system in case of 2 meter sea level rise. In this study, we further explore the design requirements for such a pump-sluice station. What would be the required operational reliability, what is the effect of the operational range of the pumps, and what is the effect of the pump curve on inland water levels?

Pumping stations play an important role in flood risk mitigation all over the world. Examples vary from large scale pumping stations at New Orleans (USA) and plans for Galveston (USA) and Marina Bay (Singapore) for storm water discharge, at IJmuiden (Netherlands) for daily water level regulation to numerous small scale pumps managing polder water levels worldwide. Climate change is expected to increase the flood risk in deltaic areas (Muis et al., 2015; Arnell and Gosling, 2014; Vousdoukas et al., 2018). While extreme river discharges in wet areas will increase, sea level rise will hamper the natural outflow of river discharge. This will elevate the mean water level of deltaic rivers and impede the discharge of extreme river flood waves (Waddington et al., 2022). Furthermore, climate change is exptected to challenge existing water systems regulated by pumps or sluices, such as in Italy (Cioffi et al., 2018), Germany (Bormann et al., 2024) and the Netherlands (Vermeulen and Honingh, 2021). Their overall discharge capacity will reduce as the average differential head between the sea water level and the water level in the water system will increase, leading to a shorter discharge window for discharge sluices or less efficient operation for pumps. Together with an increased drainage demand due to increased (peak) river discharges, intenser rainfall events and an increased frequency of compound events (Bevacqua et al., 2019), existing pump-sluice stations might need to expand.

The Rhine-Meuse Delta is one representative delta which will face these issues (Figure 1). The rivers Rhine and Meuse discharge into the North Sea where sea levels are expected to rise. It discharges to the sea via two branches. It is in open connection to sea via the Nieuwe Waterweg, with a storm surge barrier the Maeslant Barrier which can close in case of extreme sea water levels. The connection of the second branch at the Haringvliet is semi-closed. The Haringvlietsluices manage the inlet and discharge of water, thereby exerting more control on the inland water level. The functionality of the Haringvlietsluices will diminish as sea level rises and the inner water level is kept constant, as it will shorten the tidal window for discharging. Simultaneously, (extreme) river discharges of both rivers are also expected to increase (Deltares, 2020). Furthermore, sea level rise will increase the inland river water levels (De Bruijn et al., 2022). These developments will lead to an increase in flood probability in the Rhine-Meuse Delta.

If the discharge capacity of discharge sluices becomes insufficient, pumps could be installed. This was done in the Netherlands at IJmuiden (160 m^3/s in 1975, additional 100 m^3/s in 2004) and Lake IJssel (275 m^3/s in 2024). The addition of pumps to a water system will extend the discharge window, because sluices are only functional when the sea water level is below the basin water level. In contrast, pumps are operational when the sea water level is higher than the reservoir water level (Figure 2). By extending the discharge window beyond low tide a larger volume of water can be discharged during a tidal cycle, making the system more accommodating for extreme river discharges. As sea level rises the system will be more frequently within the operational window of the pumps than of the sluices.

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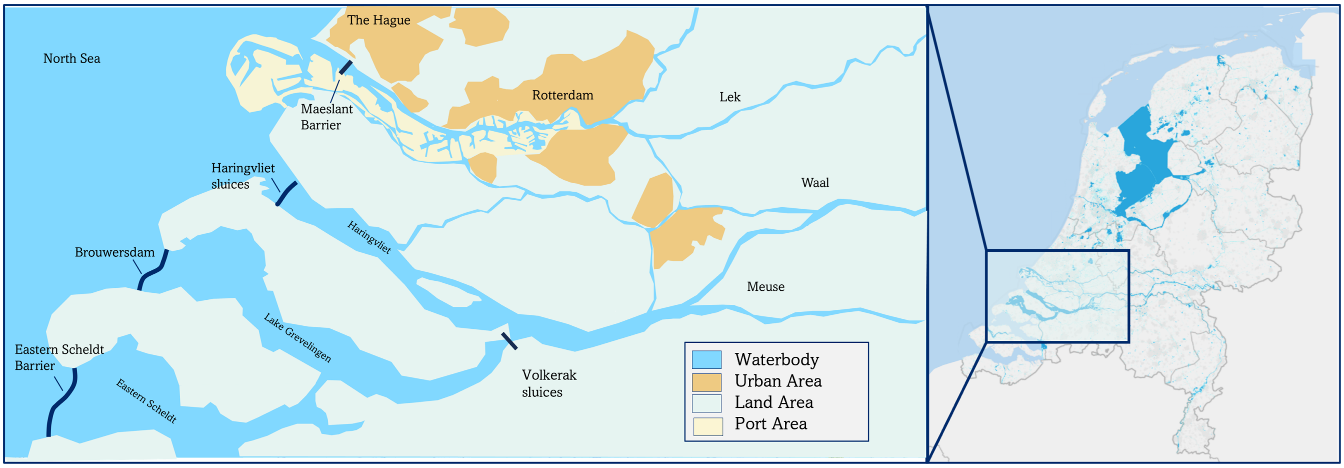

Figure 1. Overview of the Rhine-Meuse Delta.

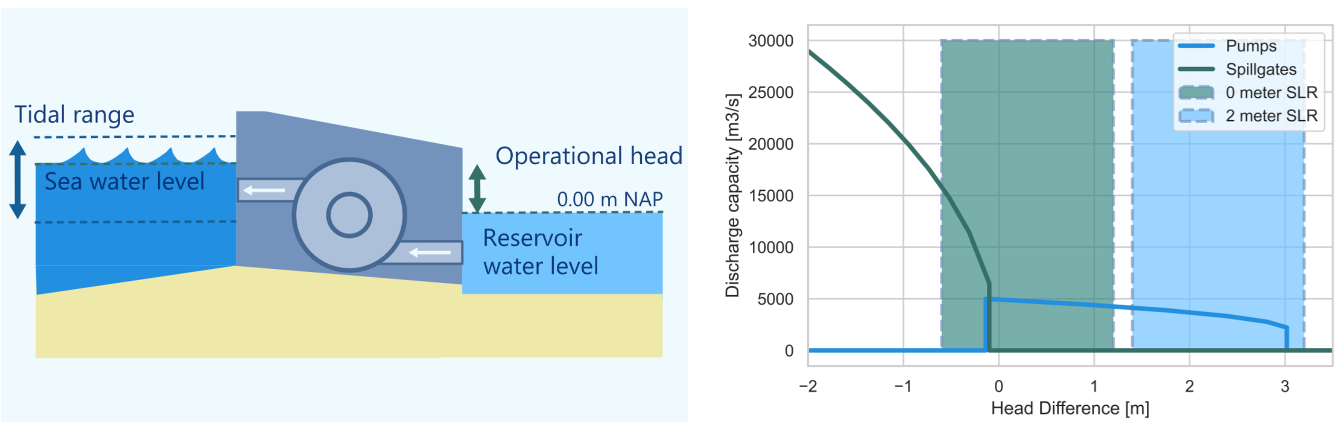

Figure 2. a) Cross-sectional view of a pumping station. It illustrates the operational pump head in green, which is the difference between the reservoir water level and the sea water level. The tidal range is indicated in dark blue. The white arrows indicate the flow direction. b) An example of the operational window of sluices and pumps with the operational pump head on the x-axis and the discharge capacity on the y-axis. Examples of a tidal range for 0 meter SLR and 2 meter SLR are indicated in by the green and blue areas respectively.

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In the design study of Knowledge Program Sea Level Rise[1] (2024a) and of Rijkswaterstaat (2015), the required future pump capacity was determined by finding an optimum between the available buffer capacity and the pumping capacity for a 10,000 year return period of a maximum inland water level of 3.16 m +NAP[2] in the reservoir. Both pumps and sluices were assumed to always be available at full capacity. In reality, the sluices and pumps will not always function at full capacity due to failures or standard maintenance. Furthermore, high sea water levels may reduce the pump performance (Vermeulen and Honingh, 2021) or even exceed the maximum operational pump head, leading to a full shut-down of the pumps. In this study, we define the available pump and sluice capacity at a point in time, influenced by the two above mentioned effects as the operational discharge capacity. Variations in the operational discharge capacity may have a significant effect on the flood risk in a water system (ten Veldhuis et al., 2011; Vermeulen et al., 2017) and thereby influence the optimal investment strategy.

The flood probability in a regulated water system is a balance between the installed pump and sluice discharge capacity, buffer capacity, buffer area, pump curve, pump reliability and incoming river discharge and sea level statistics. Individual combinations of elements have been studied in various disciplines (see below), but to our knowledge there are no studies that combine the effects of all these elements.

One method to determine the required pump capacity in Dutch polders which takes the buffer capacity into account is the rainfall duration curves (Holleman, 2008). Another applied method is to determine the design on model simulations with long precipitation records (Bosch et al., 2006). Pump failures are usually not taken into account in the design of polder pumping stations (Oostrum, 2019). Furthermore, the pumps employed in polders mostly operate at an approximately constant differential head, instead of the high differential head variations that can occur when discharging to tidal waters.

The effect of variations in the operational discharge capacity within pumping systems caused by pump failure has been analysed in the disciplines of water supply (Ning et al., 1990; Ning and Mays, 1990) and storm water discharge (ten Veldhuis et al., 2011). The effect of the reliability of sluices is explored in Lewin et al. (2003). These studies indicate that pump or sluice failure can have a significant effect in the design of pumping systems. Kuijper et al. (2017) performed a model study which balanced pump capacity, sluice capacity, buffer capacity, while including operational pump head and pump failures. The effect of the operational pump head was simplified in this study as it only set a maximum operational head (Kuijper et al., 2017). It was not yet included that a pump operating far from the optimal operating point can experience a significant reduction the operational pump capacity.

The buffer capacity is a combination of the buffer area and the margins between the target water level and maximum water level. Exceedance of the buffer capacity will result in flooding. When expanding the buffer capacity, one could include a larger buffer area or increase the maximum water level. Both have an effect on the head difference between the reservoir and the sea, as the water level in the reservoir will rise faster or slower based on this choice, thereby influencing the functioning of the pumps and sluices. The target water level can also be increased, to longer enable discharging via the discharge sluices. The target water level was raised at Lake IJssel (2006) and the anticipated effects were determined in Deltacommissie (2008). KPSLR (2024b) and De Bruijn et al. (2022) have explored the effect of expanding the buffer area for the Rhine-Meuse Delta

[1] From now on refferred to as KPSLR

[2] NAP = Dutch Ordnance Datum, which approximately corresponds to mean sea level

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on the required pump capacity. Pumps and sluices were assumed to be fully reliable in these studies. Extensive studies on the required buffer capacity of reservoirs behind dams are available (Cannon, 2024). In these studies a balance between the incoming discharge, discharge capacity of the sluices and the buffer capacity is sought. However, these sluices are always able to discharge water, in contrast to our coastal adjacent water system, where it is influenced by the head difference between sea and reservoir.

In this study, we explore how operational discharge capacity affects the flood frequency and the optimal design of a water system with the Rhine-Meuse Delta as case study. Flood return periods are computed for multiple scenarios varying in buffer capacity, which are based on the scenarios of KPSLR (2024b) considering a 2 meter sea level rise. The effect of operational discharge capacity is determined by comparing a base scenario with perfectly functioning pumps and sluices to scenarios which consider variations in the operational pump capacity and sluice failures. To explore the relation between pump and sluice capacity, some scenarios are included where the discharge capacity of the Haringvlietsluices are fully replaced by pumps.

The case study and design scenarios used in this paper are introduced in the following section. In the third section the functioning of a pump is explained and the dynamics of pumps in a water system is described by a conceptual model. The probablistic method, and case-study specific materials can be found in section 4, including a description of the water model, the performed monte carlo simulations and parameters used per design scenario. The results of all simulations can be found in section 5. Its implications are discussed in the final section 'Discussion and Conclusions'.

2 Rhine-Meuse watersystem

2.1 System overview

The Rhine-Meuse Delta discharges water from an area of 220 000 km^2, including regions in Switzerland, France, Germany, Belgium, Luxembourg and the Netherlands. After entering the Netherlands, the Rhine and Meuse rivers bifurcate multiple times, creating a fine mazed river-lake network (see Figure 1). The Rhine enters the Netherlands at the German border at Lobith, after which it soon bifurcates into the Waal, IJssel and Nederrijn-Lek. The IJssel flows northwards and discharges into the IJssel Lake (area of 1100 km^2). The Meuse river enters the Netherlands at the south border with Belgium. It discharges to sea at the Rijnmond Delta, together with the Nederrijn-Lek and the Waal. The Rijnmond Delta consists of various canals and lakes with a total area of approximately 450 km^2. The Meuse has a mean discharge of 180 m^3/s and a 1:10,000 peak discharge of 3200 m^3/s. The Rhine has a mean discharge of 2000 m^3/s and a 1:10,000 peak discharge of 13000 m^3/s. After the bifurcation the discharge is generally divided in 2/3 (Waal), 2/9 (Nederrijn-Lek) and 1/9 (IJssel) (De Bruijn et al., 2022).

The Rhine-Meuse Delta is characterised by intertwined river branches of the Lek, Meuse and Waal entering from the east and two main outlets discharging into the North sea. The northern outlet is the Nieuwe Waterweg is in open connection to the North Sea, and is protected in case of high sea water levels by the Maeslant Barrier. The southern outlet the Haringvliet is closed off from the North Sea by the Haringvlietdam to prevent high water levels. Excess river discharge is discharged during low tide via de Haringvlietsluices located within the dam.

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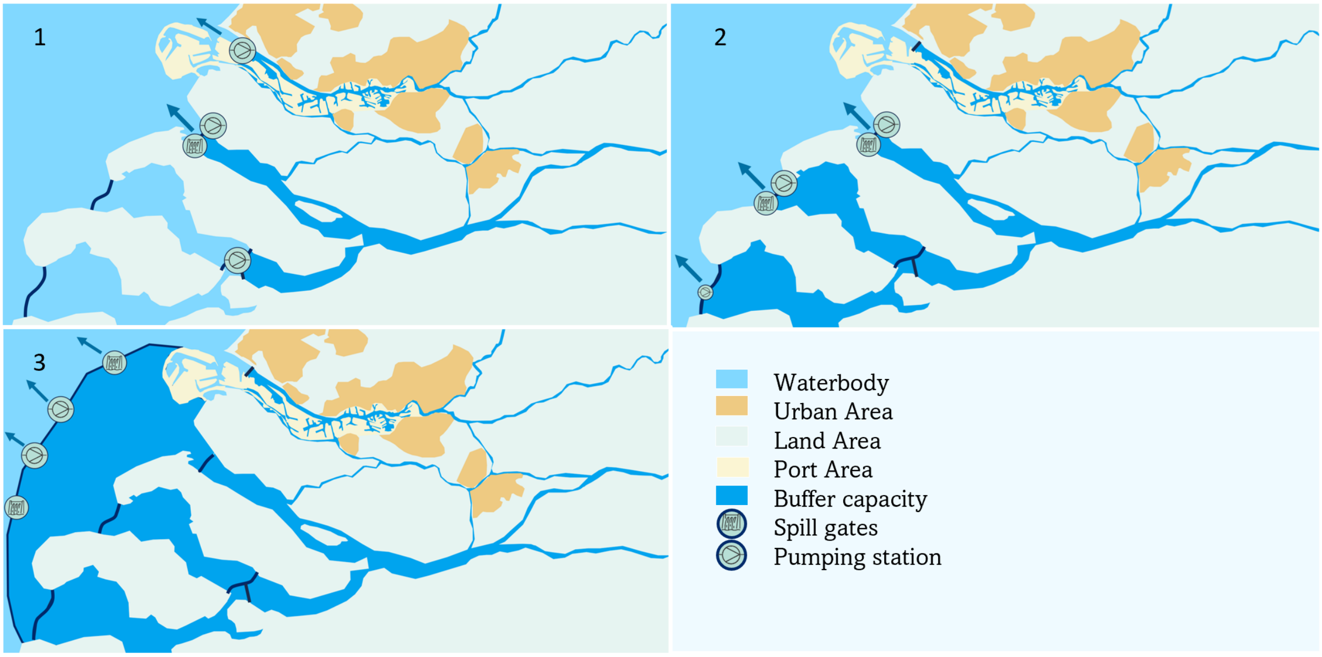

Figure 3. 3 schematic overviews of possible adaptation designs for the Rhine-Meuse Delta. The areas included in the buffer capacity are in dark blue. 1) First strategy with a buffer capacity of 450 km^2, 2) second strategy with a buffer capacity of 1000 km^2 and 3) the third strategy with a varying buffer capacity depending on the dam design.

2.2 Possible adaptation strategies

Future sea level rise scenarios will likely result in elevated water levels in the Rhine-Meuse Delta, leading to more frequent flood events. Four main possible adaptation strategies were proposed in recent years by Deltares (2020): ‘Protect open’, ‘Protect Closed’, ‘Proceed’, ‘Retract’. In this study, we focus on the strategies ’Protect Closed’ and ’Proceed’, as pumping stations will play an essential role in discharging excess water to sea in these paths. Preliminary design solutions were developed by KPSLR (2024a, b) for 2 meter and 5.4 sea level rise. In this study we only consider 2 meter sea level rise.

In the design scenarios the Rhine-Meuse Delta is assumed to be closed off from the North Sea, removing the tidal effects in the inland water bodies. Pumps and sluices can be constructed adjacent to the sea. The water bodies which are in a current open connection will function as a buffer (Figure 3.1), if the pump and sluice capacity is insufficient to match the incoming river discharge from the Rhine and Meuse. One important design parameter is the available buffer capacity. The Rhine-Meuse Delta is separated from Lake Grevelingen and the Eastern Scheldt water bodies at the Volkerak sluices. Extra buffer capacity can be created up to 1000 km^2 by including Lake Grevelingen and the Eastern Scheldt (Figure 3.2). As a third option a seaward dam could be constructed, further increasing the available buffer capacity (Figure 3.3). The precise buffer capacity is determined by the dam alignment. Full technical details on the scenarios used in the simulations can be found in Table 2 in Section 4.

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3 Operational discharge capacity and the water system dynamics

The water level in our design water system is controlled by pumps and discharge sluices. The water system itself can be generalized to a reservoir, which functions as a buffer (V_{buffer}) with a an incoming river discharge (Q_{in}) and an outgoing discharge to sea (Q_{out}). The most important processes and dependencies in a water system with a pumping-sluice complex are captured in our conceptual model. An important variable is the operational discharge capacity, which consists of the operational pump capacity and the operational sluice capacity. The operational discharge capacity is defined as the momentary available pump capacity during operation, which will likely vary from the theoretical maximum capacity for both pumps and discharge sluices. The main causes for this reduction in operational discharge capacity are the variation in head difference between the reservoir and the sea and the technical state of the pumps and sluices. Section 3.1 will further explain the concept of operational discharge capacity, before the water system dynamics are explained in section 3.2.

3.1 Operational discharge capacity

3.1.1 Q-h relation - Pumps

In this study we focus on the impeller pump type because of their high discharge capability (McAllister, 2014). An impeller pump transfers energy to the fluid through the rotational motion of the impeller. This increases the total head of the pump, which can manifest in either pressure, velocity or elevation. Each pump has a specific impeller geometry and design that affects the flow versus head characteristic, depicted in the pump curve (Figure 4). The pump curve of impeller pumps typically has a downward curved shape (Figure 4.a). As the fluid flows through the pump both eddy losses and friction losses occur (Wijdieks and Bos, 1994). The amount of energy loss varies with the flow velocity. Higher velocities cause higher friction losses, while eddies are more prominent at lower velocities. This results in a practical pump curve (Figure 4.a). To prevent damage to the pump, a minimum and maximum discharge is set, making up the operational window (Figure 4.c).

The total head to be delivered by the pump consists of the static head (H_{st} [m]) and the dynamic head (H_{dyn} [m]) (Figure 4.c). The static head is equal to the head difference between the inflow and outflow level of the pump, independent of the flow velocity in the pump and system losses. The dynamic head incorporates the resistance of all surroundings (valves, pipes, waste grids etc.) except for the pump itself, steepening as the flow rate increases (Figure 4.b). The total head can be expressed in terms of the discharge:

$$H_{tot} = H_{st} + H_{dyn} \tag{1}$$

$$H_{tot} = H_{st} + CQ^2 \tag{2}$$

in which:

C = constant dependent on the hydraulic losses in the system [s/m^5]

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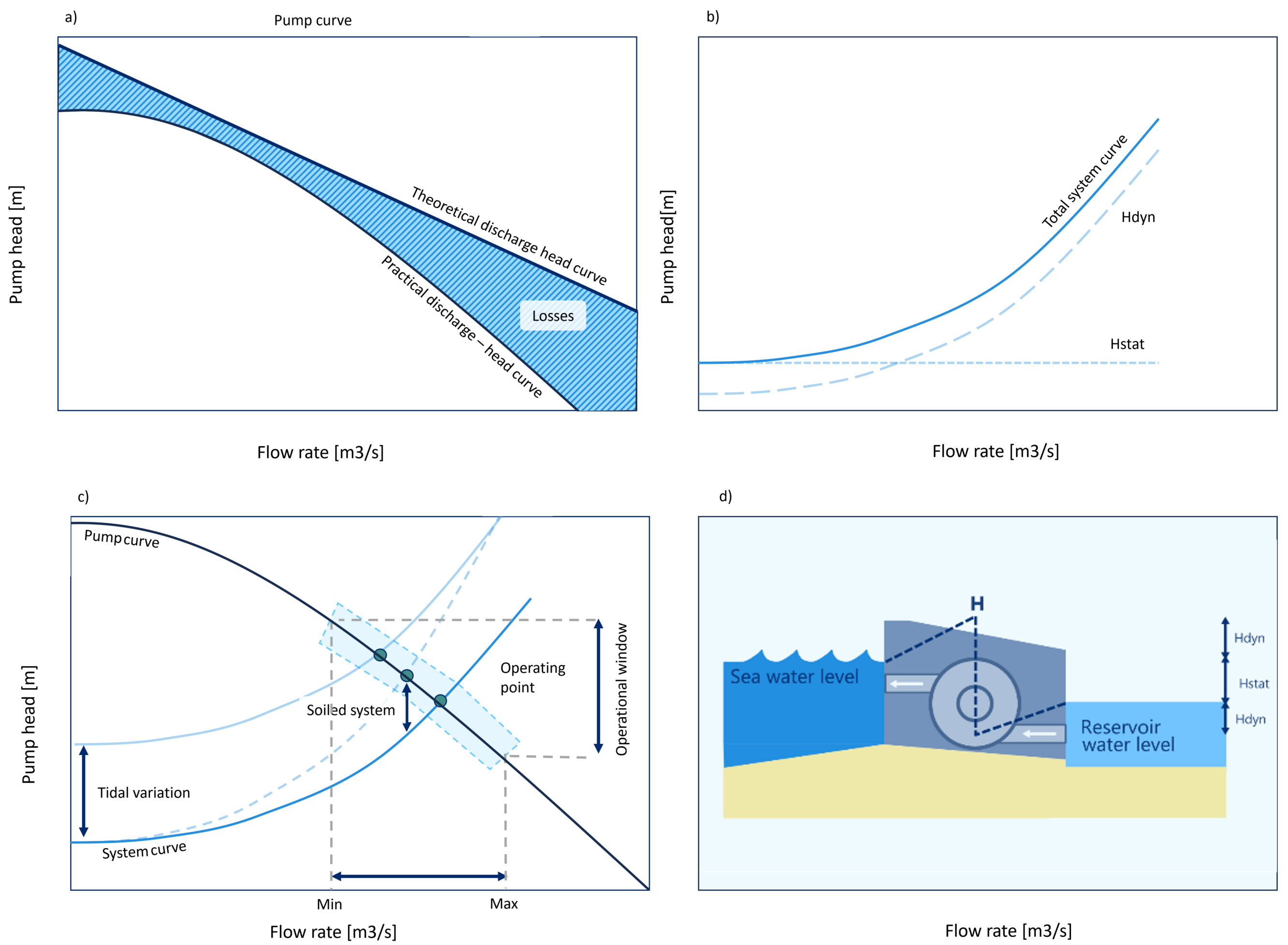

Figure 4. Schematized examples of a pump curve and system curve. a) Theoretical pump depicted minus the losses results in the practical pump curve. b) Total system curve consisting of the dynamic head and the static head. c) Combination of pump curve and system curve. Possible variations of a system curve due to tidal variation (continuous blue line) or a soiled system,leading to more friction losses (dashed, light blue line), which will cause different operating points. d) Cross-sectional view of a pumping station with schematic depiction of the change of the head along the direction of flow.

Q = discharge [m^3/s] This curve is referred to as the system curve. It can change as the surroundings of the pump change, due to tidal variations or for instance a clogged waste grid (Figure 4.c).

The operation of a pump and the realized pump capacity is the result of the interplay of the pump and the surrounding system. The pump curve and the system curve seek a balance, which determines the operating point. The pump curve and system curve are plotted together in Figure 4.c and the point of intersection will be the operating point. If the static head changes, this will change the balance in the pump-system and alter the operating point. Tides and storm surges cause a daily variation of the

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operating point and the available pump capacity (see Figure 4.c). Extremely high water levels could even make the required total head exceed the maximum operational threshold, leading to a full shut-down of the pumps. The operational range of the pump is determined by the minimum operational pump head and the maximum operational pump head, implemented to avoid pump damage. In reality more complicated features of pumps such as rotational speed and inlet velocity influence the operating point. These features are ignored in this paper, as the aim is to explore the possible effect of a varying pump capacity on the water system.

3.1.2 Q-h relation - Sluices

The operational sluice capacity is directly determined by the head difference between the reservoir and the sea. Discharging via sluices is only possible if the sea water level is lower than the reservoir. The discharge capacity will increase as the head difference increases (see Figure 2). The discharge capacity can be controlled by the position of the sluice door. This way discharge capacity can be limited to a maximum threshold, to prevent damage to the structure and scour protection at the toe.

3.1.3 Failure events

If a failure event occurs, one or more pumps or sluices will be temporarily unavailable, leading to a temporary loss in discharge capacity. Each possible failure event can be described by the the number of failed pumps or sluices, the repair time and the failure frequency. The number of failed pumps or sluices can vary from 1 to all in the system. The repair time can vary from mere hours to multiple months, depending on the failure cause and damage. Failure modes can be caused due to multiple failure mechanisms such as a mechanical malfunction, human interference, software issues, or external events. Determining a set of relevant failure modes and their corresponding characteristics for a specific pump-sluice station is often a study of its own. The temporary loss in capacity due to standard maintenance will not be included in this study, as it is well schedulable.

3.2 Water system dynamics including pumps and discharge sluices

The dynamics of a water system with pumps and discharge sluices are described in our conceptual model by design input variables, stochastic input variables, connecting variables and output variables (Figure 5).

One can influence the functioning of the water system by altering the design variables. One important design variable is the design of the pumping station and the associated pump curve. The maximum theoretical capacity of a pumping station is determined by the amount of pumps and the theoretical maximum capacity of each pump. The pump reliability is affected by design choices such as the amount of pumps many smaller or fewer larger pumps and the number of independent stations in the system. By opting for more lower capacity pumps the effect of a failure event can be limited. Similarly a large failure event might eliminate all available discharge capacity if all pumps are part of the same station. If discharge capacity is divided over independent stations with independent power supply and control, a large failure event at a station will only eliminate part of the total capacity. The pump curve will determine how much pump capacity will be lost during the tidal cycle or storm surges.

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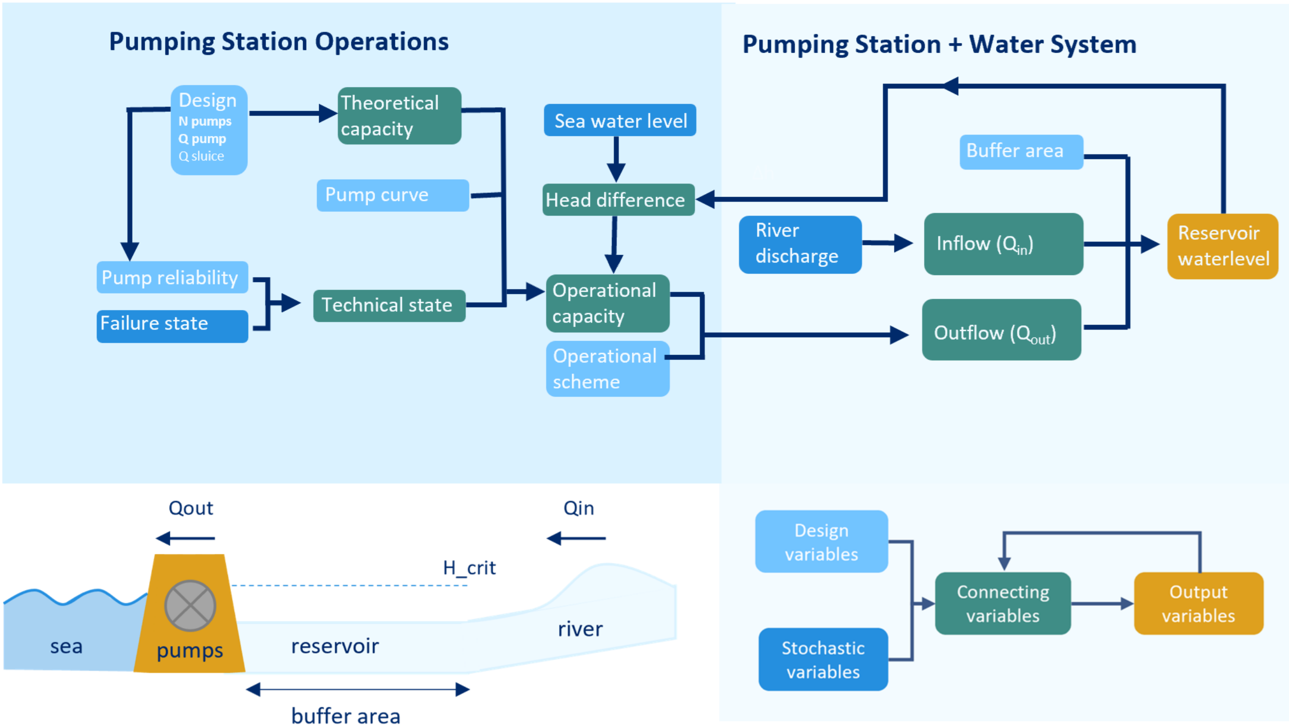

Figure 5. Conceptual model illustrating the relationship between operations at pump-level, the water system, and system failure. Arrows represent the flow of influence. Colours of the boxes represent the type of variable, indicated in the top figure.

Another design choice is the buffer capacity, which is a combination of the buffer area and the difference between the target water level and critical water level ($\Delta h = h_{crit} - h_{target}$). A larger buffer area and lower Δh will lead to slower variations in the reservoir water level, which will also influence the head difference, in turn influencing the outgoing discharge.

External influences on the water system are captured in the stochastic variables. In our case study, the incoming discharge consists of river discharge, but this could also be precipitation, or discharge from sluices and pumps in other case-studies. Another stochastic variable is the sea water level, which includes the tidal cycle and storm surges. As described in the previous paragraph, high sea water levels can temporarily reduce the operational pump capacity. If sea water levels are lower than the reservoir water level, discharge sluices can be enabled. A larger head difference will increase the operational sluice capacity (Figure 2). One more stochastic variable is the failure state of the pumps and sluices, which will temporarily reduce the operational discharge capacity.

Ultimately, we are interested in the exceedance frequencies of the maximum water level in the reservoir. The water level in the reservoir feeds back to the water system through its influence on the head difference. The evolution of the water level in the reservoir is determined by the interplay of the incoming discharge, outgoing discharge and the buffer area size. Due to the presence of a buffer volume the incoming discharge has to surpass the outgoing discharge for a period of time (Δt) in order to

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reach the critical water level ($\Delta t * Q_{in} > \Delta t * Q_{out} + V_{buffer}$). Therefore a temporary decrease in the operational discharge capacity due to storm surge or pump or sluice failure does not lead to a system failure by itself. The incoming discharge should exceed the outgoing discharge for a sufficiently long period, which makes the temporal aspect of both flood waves and storms or failure events important to the water system.

4 Methodology: Monte Carlo Simulations

This study aims to obtain the extreme value statistics of the reservoir water levels and especially the exceedance frequencies of the maximum water level of the reservoir (3.16 m +NAP). These exceedance frequencies are obtained by performing two monte carlo experiments (Figure 6), which are described in Section 4.1. The method to generate events for the monte carlo experiments can be found in Section 4.2. In the experiments 30-day simulations with a reservoir model are conducted (Section 4.3), after which the extreme statistics and the exceedance frequencies are calculated (Section 4.4). This process is conducted for the scenarios depicted in Figure 3, and for different reliability set-ups in the water system. Further details can be found in Section 4.5.

4.1 Set-up of the monte carlo experiments

Failure of the water system can occur during three possible circumstances if operational discharge capacity is included: 1) extreme river discharges, without a pump or sluice failure event, 2) extreme river discharges, with a pump or sluice failure event and 3) non-extreme river discharges, with a pump or sluice failure event. Therefore, our domain of interest includes both extreme and non-extreme river statistics, which are described by different types of probability distributions. Therefore, our study consists of two separate Monte Carlo experiments (Figure 6). One with non-extreme river discharge events to account for system failure due to pump failure and one with river flood waves corresponding to extreme conditions, where pump and sluice failures are possible. Non-extreme river discharges conditions without a pump or sluice failure are excluded from the analysis as it will not lead to high water levels in the reservoir.

A peak discharge with a 2 year frequency is used as a threshold to identify a flood wave as extreme. Therefore, the frequency of occurrence of an extreme river discharge $f(Q_{ex})$ is ≈ 0.5 times per year and the frequency of occurrence of a pump failure $f(Fail)$ is ≈ 7.2 times per year. The monte carlo experiments result in conditional exceedance probabilities (one provided an extreme river discharge occurs and one provided a pump failure occurs). Both experiments correspond to 30,000 years (our return period of interest being 10,000 years x 3). The first experiment is conducted 15,000 times (0.5 x 30,000) and the second experiment is conducted 210,000 times (7.2 * 30,000).

4.2 Reservoir Model

The water system is schematized to a reservoir model with a buffer area, incoming river discharge and an outgoing discharge via the pumps and sluices (Figure 7). The incoming discharge consists of a combination of river discharge from the River Rhine

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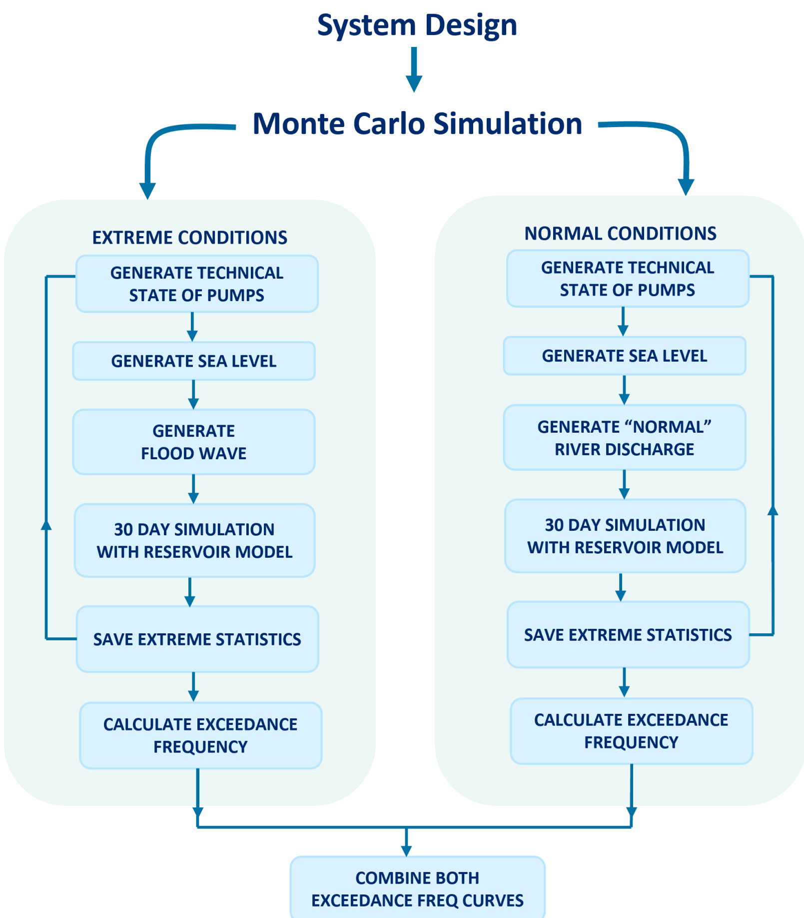

Figure 6. Flowchart of the conducted monte carlo simulations for each design scenario. Events under non-extreme and extreme river discharges are simulated separately and the results are combined afterwards. The left side describes the steps for events with 1) extreme river discharges, without pump or sluice failure event, 2) extreme river discharges, with a pump or sluice failure event and the right side describes events with 3) non-extreme river discharges, with a pump or sluice failure event.

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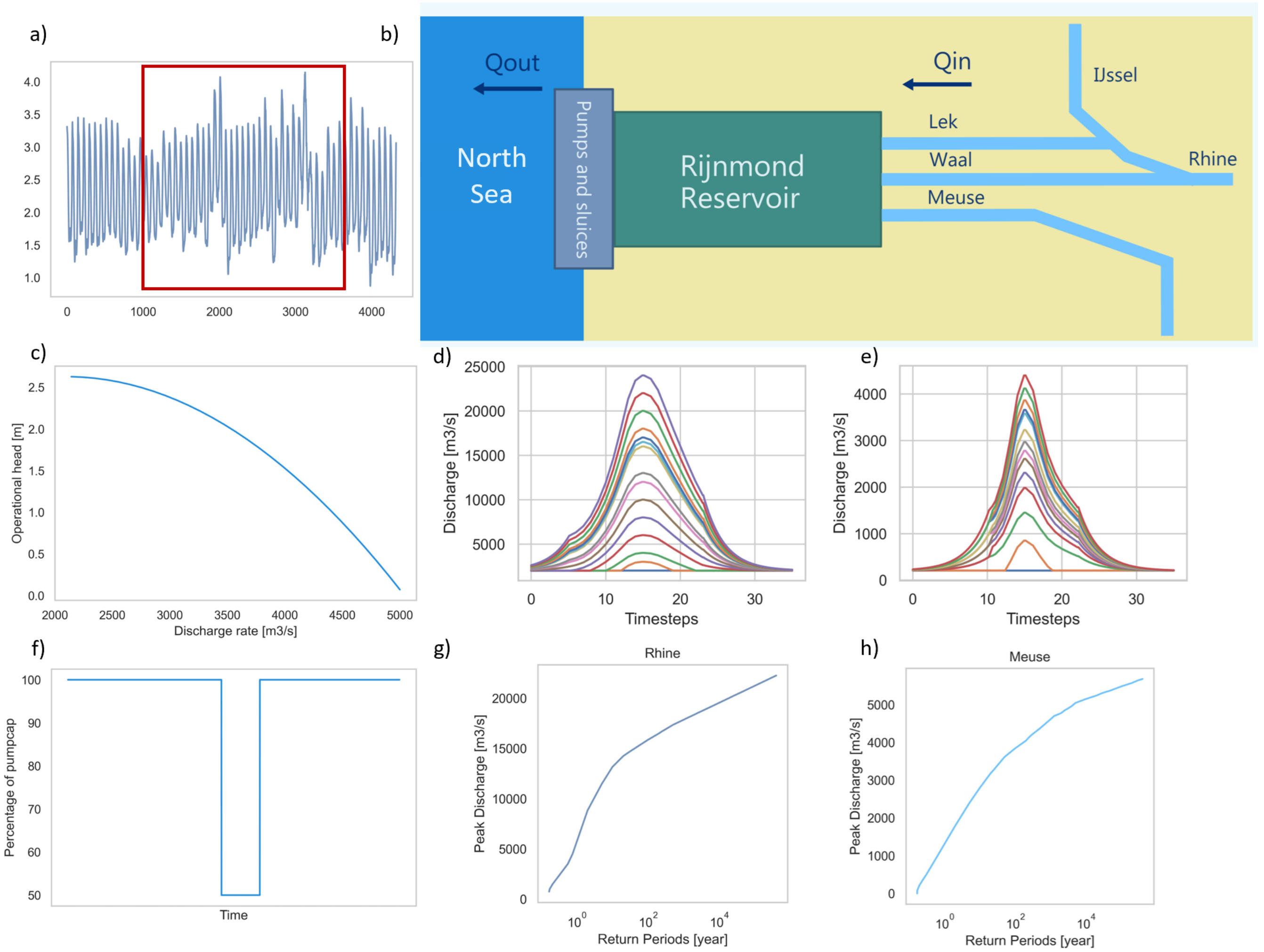

Figure 7. Schematic overview of our model area with incoming discharge from the Lek, Waal and Meuse and the outgoing discharge to the North Sea at the pumping-sluice station and sluices. The buffer area in the model equals the reservoir area. Below examples of as sample from the water level at Hoek van Holland are shown (left), the pump curve and an example of a pump failure (middle) and the extreme river discharge statistics and standard flood waves for the Rhine and Meuse.

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and the River Meuse. Excess water can be discharged to sea via sluices or pumps, depending on the head difference between the reservoir and the North Sea at Hoek van Holland (Figure 2).

The reservoir model calculates the new water level ($h_c(t+1)$) for each new time step (Δt =10 min) in the simulation based on the difference (ΔV) between the incoming volume ($Q_{in}*\Delta t$) and outgoing volume ($Q_{out}*\Delta t$), divided by the buffer area (A_{res})(Eq.3). The model will always aim to drain the entire buffer and incoming discharge during the time step if the operational discharge capacity is sufficient. This way the buffer is fully available for potential future peak discharges. To determine $Q_{out}*\Delta t$, the reservoir model calculates the required discharge volume (V_{req}) and the the available discharge capacity (Q_{av}) (Figure 8). The required discharge volume is the water volume required to discharge all incoming river discharge in time step t and the water volume is already in the buffer (Eq. 3). However, the required discharge volume will not always match the available discharge capacity. If $V_{req} > Q_{av}*\Delta t$, the outgoing volume is limited by the available discharge capacity. If $V_{req} < V_{av}$ sufficient capacity is available to drain the reservoir (Figure 8).

The available discharge capacity is determined by the head difference between the sea and the reservoir. Discharge via the sluices is assumed to be possible if the sea water level is lower than reservoir water level. Otherwise, the pumps can be used to discharge excess water. The total operational pump capacity is determined by the individual operational capacity of a pump $Q_{pump,oc}$, which is determined based on the head difference and the pump curve of Figure 7.c. The formula for the pump curve can be found in the supplementary material. This is multiplied by the number of non-failed pumps (Eq. 4).

The Haringvlietsluices (see Figure 1) are assumed to be available for natural discharge in our future water system. The capacity of the sluices (Q_{sluice}) can be calculated with Equation 6, with a maximum discharge of 25,000 m^3/s. The sluice-complex consists of 17 sluices (n) of 56 meter width (W) each. A discharge coefficient of c=1 and a gravitational constant of g=9.81 m/s^2 is assumed. If a sluice failure occurs, the number of sluices (n) will be reduced (Figure 8). The possibility of non-closure of the sluice doors is not taken into account in this study.

$$h_c(t+1) = h_c(t) + (Q_{in}(t) - Q_{out}(t)) * \Delta t / A_{res} \tag{3}$$

$$V_{req} = Q_{riv} * \Delta t + h_c(t) A_{res} \tag{4}$$

$$Q_{pump,av} = Q_{pump,oc}(h_{res} - h_{sea}) * (N_{pump} - N_{fail}) \tag{5}$$

$$Q_{sluice} = n * c * W * h_{sea} * \sqrt{2 * g * (h_{res} - h_{sea})} \tag{6}$$

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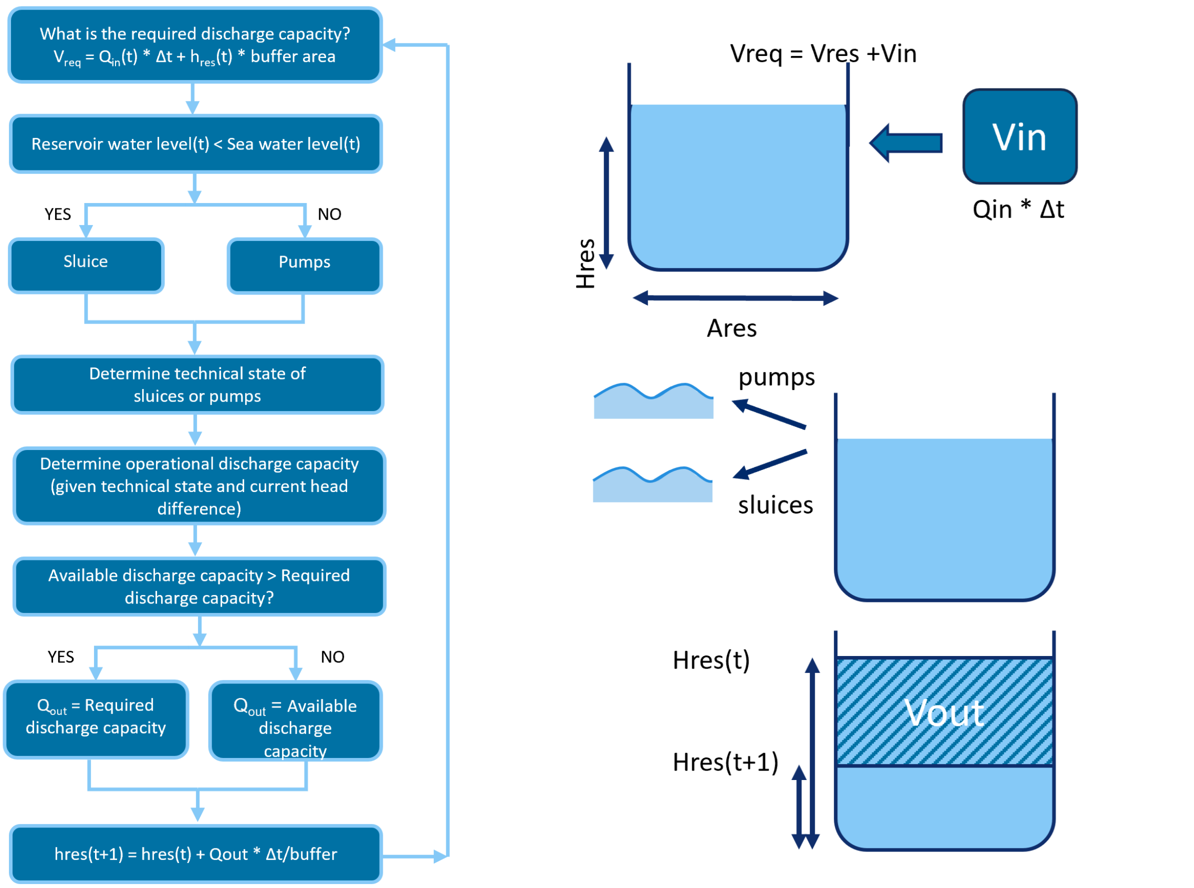

Figure 8. Flow chart of all steps taken in the reservoir model during each time step.

270 ## 4.3 Generation of synthetic events

Prior to the simulations all synthetic events are generated. Three types of input are generated per event: time sequences of river discharge, sea water levels and the technical state of the pumps. The method to obtain the river discharge and technical state of the pumps differs per experiment and is discussed below. All variables can be found in Table 2 and 3.

For the extreme conditions simulations, a new flood wave is determined for each simulation based on the peak discharge of
275 both the river Rhine and Meuse of the event which are scaled to standard flood wave curves from KPSLR (2024b) (see Figure 7.d and e). The extreme river discharges samples are taken from the extreme discharge statistics of climate scenario WH2100 of the KNMI, conform the study of KPSLR (2024b). The Rhine peak discharges are reduced to 84.6 % as some discharge at Lobith is lost at bifurcations. In case of non-extreme river statistics samples are taken from lognormal distributions of the Rhine (conform Zhong et al. (2012)) and Meuse (fit from data discharges at Borgharen obtained from Rijkswaterstaat (2024)).

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In case of river discharges below 2000 m^3/s (Rhine) and 200 m^3/s (Meuse) the river discharge is assumed constant during the simulation.

A time-series of winter sea water levels at Hoek van Holland from 1987-2007, also used in KPSLR (2024b), was transformed by uniformly adding 2 meter sea level rise. For each event a 30 day length sample is taken from the Hoek van Holland time-series. Peak river discharges from the Rhine and Meuse and storm surges at Hoek van Holland are assumed to be independent in this study. As only for a time lag of 6 days a significant dependence is found, and it is suggested that there is no need for considering dependence in flood protection and policy-making by to Klerk et al. (2015).

We use a synthetic set of failure modes inspired by but not similar to the failure frequencies estimated for pumping station IJmuiden and the Haringvlietsluices (Table 1). Sluice failures are included similarly for both types of monte carlo experiments. If the sea water level is below the reservoir level at time step t, the model checks if the doors are already open (reservoir water level (t-1) < sea water level (t-1)). If this is not the case, the doors should open at time step t and a failure event is possible. First, there is a check if an earlier failure event in the simulation may be still active. If not, an event is drawn from the set of failure modi of Table 1, including the possible event of a non-failure. If the event of a non-failure is drawn, all 17 sluices are used in the calculation of the available sluice discharge capacity, otherwise the amount of failed sluices is subtracted for the duration of the repair time.

Pump failures are included differently for the extreme and non-extreme monte carlo experiments. The non-extreme monte carlo experiment only simulates events where a pump failure event occurs. For each event a failure mode is sampled from a multinomial distribution consisting of the conditional probabilities of each failure mode, given a pump failure occurs (Table 1). The moment of failure in the simulation (t_f) is drawn from a uniform distribution U(0,t_{end}). An example time-series can be found in Figure 7.d.

Pump failure may occur in the simulations of the extreme river statistics monte carlo experiment, though not necessarily. Prior to the generation of events a synthetic time- sequence of pump failures is generated. This sequence is generated by determining the time between failures for each possible failure mode of Table 1. The time between failures (Δt_f) is determined by sampling from an exponential distribution where λ is the frequency of occurrence of the failure mode. If a failure mode occurs at time step t, the pump capacity is reduced with the capacity of the failed pumps for the length of the repair time $t_f = t + \Delta t_f$. The results for all failure modi are combined to a single time-series of failures. A 30 day sample is uniformly drawn for each event in the simulation from the generated time-series of failures.

4.4 Extreme water level statistics

The exceedance probability for both monte carlo experiments can be calculated based on the maximum water level during simulated event, which correspond to the two branches of Figure 6. To obtain the exceedance frequency of maximum water levels in the water system, the results of both monte carlo experiments should be multiplied by the frequency of occurrence of the conditional event, being an extreme river discharge and a pump failure. Afterwards the two conditional frequencies can be summed (Equation 7). The frequency of water levels above 0.00 meter +NAP under non-extreme river discharges without a pump failure is assumed to be zero.

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Table 1. Overview of the synthetic set of failure modes and the corresponding failure frequencies and repair time. The values of the pumps represent the probability for each failure mode given a pump event occurs. The values for the sluices represent the probability of each failure mode per demand of opening.

	1 pump	10 pumps	50 pumps	100 pumps
0.5 day	6.31e-1	1.23e-1	6.61e-3	1.04e-1
1 day	2.52e-2	4.93e-3	2.64e-4	4.16e-3
2 days	1.26e-2	2.46e-3	1.32e-4	2.08e-3
5 days	7.57e-3	1.47e-3	7.95e-5	1.24e-3
7 days	1.13e-2	2.21e-3	1.19e-4	1.87e-3
14 days	2.02e-2	3.45e-3	1.85e-4	2.91e-3
30 days	2.27e-2	3.94e-3	2.11e-4	3.33e-3

	1 sluice	5 sluice	9 sluice	17 sluice
0.5 day	5.00e-1	1.00e-1	1.00e-2	1.00e-4
1 day	2.00e-2	4.00e-3	4.00e-4	4.00e-6
2 days	1.00e-2	2.00e-3	2.00e-4	2.00e-6
5 days	6.00e-3	1.20e-3	1.20e-4	1.20e-6
7 days	9.00e-3	1.80e-3	1.80e-4	1.80e-6
14 days	1.60e-2	2.80e-3	2.80e-4	2.80e-6
30 days	1.80e-2	3.20e-3	3.20e-4	3.20e-6

Table 2. Overview of variables of the Monte Carlo experiments. Some variables are applicable to both monte carlo experiments, some only to the extreme river statistics and some only to the non-extreme statistics.

Overview of parameters

Variable	Symbol	Unit	Distr.	MC	Source
Sea level	H_{sea}	m	measured data	both	KPSLR (2024b)
Extreme river discharge	Q_{riv}	m3/s	extreme value	extreme	KPSLR (2024b)
Non-extreme River discharge Rhine	$Q_{riv,r}$	m3/s	log-normal (μ = 7.65, σ= 0.51)	non-extreme	Zhong et al. (2012)
Non-extreme River discharge Meuse	$Q_{riv,m}$	m3/s	log-normal = (μ = 5,20, σ= 0.91)	non-extreme	KPSLR (2024b)
Sluice failure modes	P(Fail)	-	multi-nominal	both	Bodelier (2023)
Pump failure modes	P(Fail)	-	multi-nominal	both	Vermeulen et al. (2017)
Time between pump failures	Δt_f	s	exponential	extreme	Table 1

$$f(h_r > H) = P(h_r > H|Fail) * f(Fail) + P(h_r > H|Q_{ex}) * f(Q_{ex}) \quad (7)$$

315 In which $f(h_r > H)$ is the exceedance frequency of the water level in the reservoir (H_r), $P(h_r > H|Fail)$ as the probability of exceedance of H_r given a pump failure, f(Fail)(= 7.2) is the frequency of pump failures per year, $P(h_r > H|Q_{ex})$ as the probability of exceedance H_r given an extreme river discharge and $f(Q_{ex})$(= 0.5) as the frequency of an extreme discharge per year.

4.5 Design Scenarios

320 The goal of this study is to explore the effect of operational pump capacity and sluice reliability on the flood frequency of a water system. Simulations with different settings of pump reliability and sluice reliability are conducted to compare the effects of different calculation settings and design choices. Four different types of simulations are conducted: a simulation with a

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theoretical constant pump and sluice capacity (similar to KPSLR (2024b)), a simulation with only operational pump capacity, a simulation including operational pump capacity and sluice reliability, and a scenario where only pumps are installed. If only pumps are installed, the sluice capacity is replaced with extra pump capacity. Additionally, the effect of design parameters such as buffer capacity and theoretical pump capacity on the operational discharge capacity is explored by including the different design scenarios of Figure 3. An overview of all scenarios with the used design and included processes can be found in Table 3. More detailed information on the parameters used in every simulation are summed up in Table 2.

Table 3. Overview of parameters for different design scenarios.

	buffer capacity $[km^2]$	pump capacity $[m^3/s]$	operational pump capacity $[m^3/s]$	sluice failure $[-]$
1.1 - baseline scenario (maximum discharge capacity)	450	6000	no	no
1.2 - operational pump capacity	450	6000	yes	no
1.3 - operational discharge capacity	450	6000	yes	yes
1.4 - no sluices + operational pump capacity	450	18700	yes	N/A
2.1 - baseline scenario (constant discharge capacity)	1000	5000	no	no
2.2 - operational pump capacity	1000	5000	yes	no
2.3 - operational discharge capacity	1000	5000	yes	yes
2.4 - no sluices + operational pump capacity	1000	15700	yes	N/A
3.1 - baseline scenario (constant discharge capacity)	1900	3800	no	no
3.2 - operational pump capacity	1900	3800	yes	no
3.3 - operational discharge capacity	1900	3800	yes	yes
3.4 - no sluices + operational pump capacity	1900	12200	yes	N/A

5 Results

5.1 Extreme water level statistics with both pumps and discharge sluices

First the effect of including variations in the operational discharge capacity on extreme water levels in the reservoir compared to the baseline scenario is shown. Figure 9 shows the exceedance frequency curves of the maximum water level in the reservoir for the three design scenarios of Figure 3. The upper panels show the return period of extreme water levels in the reservoir in case of only a variation in operational pump capacity for scenarios 1.2, 2.2 and 3.2 from Table 3. The lower panels depict the return periods extreme water levels in case of a varying operational capacity of both pumps and sluices for scenarios 1.3, 2.3 and 3.3.

Including operational pump capacity only leads to increased water levels under normal conditions, which would never lead to increased water levels under a constant operational discharge capacity. Also moderate river discharges can lead to increased water levels in the reservoir if a pump or sluice failure or a pump shut-down due to storm surge occurs. As soon as the reservoir

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water level equals the outside sea level, the discharge sluices can take over (see Figure 10). This limits the extreme water levels under normal conditions. Little effect can be observed under extreme conditions, the peak river discharge will be discharged by the sluices when the reservoir water level is above the average sea water level. Higher water levels in the reservoir are caused by insufficient sluice discharge capacity during extremely high river discharges even though all sluices are fully functional.

Including the possibility of sluice failure has a large impact on the reservoir water levels during extreme conditions as this can limit the discharge capacity even during high reservoir water levels (see lower panels). If the pump/buffer capacity ratio increases the effect of including the variation in operational capacity also increases (for instance scenario 1.3 has a higher effect compared to scenario 3.3). These water systems rely more heavily on the discharge capacity than buffer capacity to accommodate the incoming river discharge and are therefore more heavily impacted by including the variation of the operational discharge capacity. For design scenario 2, a water level of 3.22 m +NAP has a return period of 10,000 years if a constant operational capacity is applied (scenario 2.1). Yet, under scenario 2.3 this water level will occur every 75 years, and the water level with a 10,000 year return period has increased to 4.72 m +NAP.

Adding operational discharge capacity also affects the overall time period of increased water levels in a water system. Figure 11 shows the percentage of time that the water level is above 1.00 m +NAP and 2.00 m +NAP for design scenario 2 (buff.= 1000 km^2). Scenario 2.3 with both operational pump capacity and operational sluice capacity included leads to the highest time percentages above the water level thresholds. Scenario 2.2 which only included operational pump capacity shows an increase of time percentages as well, while Figure 9.b shows that this scenario had little effect on the return periods of the extreme water levels.

5.2 Extreme water level statistics with only pumps and no discharge sluices

The presence of the Haringvlietsluices in the water system greatly influences the dynamics in the system and the effect of including variations in the operational discharge capacity. To explore the effects of variations in operational pump capacity in a system with only pumps, scenarios 1.4, 2.4 and 3.4 were created where the sluice capacity was replaced by additional pump capacity. The corresponding exceedance frequency curves of Figure 12 have a very different shape than the curves in Figure 9. The effect of operational pump capacity is smaller during normal conditions, as the probability of insufficient discharge capacity due to failure is lower due to the higher amount of initially installed pump capacity. The exceedance curves show a very steep incline under extreme conditions. The effect of an occurrence of insufficient pump capacity compared to the incoming river discharge capacity is much higher as it can lead to a permanent higher reservoir water level than the sea water level. In that case, no water can be discharged as most pumps cannot discharge against a negative differential head. As the river discharge keeps entering the reservoir, this leads to extremely high water levels for higher return periods.

This effect is illustrated in Figure 13, where two illustrative events show the large effect a slight increase in the peak river discharge can have. In the first event (Figure 13.a and c) the peak river discharge remains just below the maximum pump capacity, leading to only an increase of 1 meter in the reservoir water level. The peak river discharge exceeds the maximum discharge capacity in the second event (Figure 13.b and d.). Around time step 2500 the reservoir water level permanently exceeds the sea water level leading to a full shut-down of the pumps for the rest of the simulation period. This example

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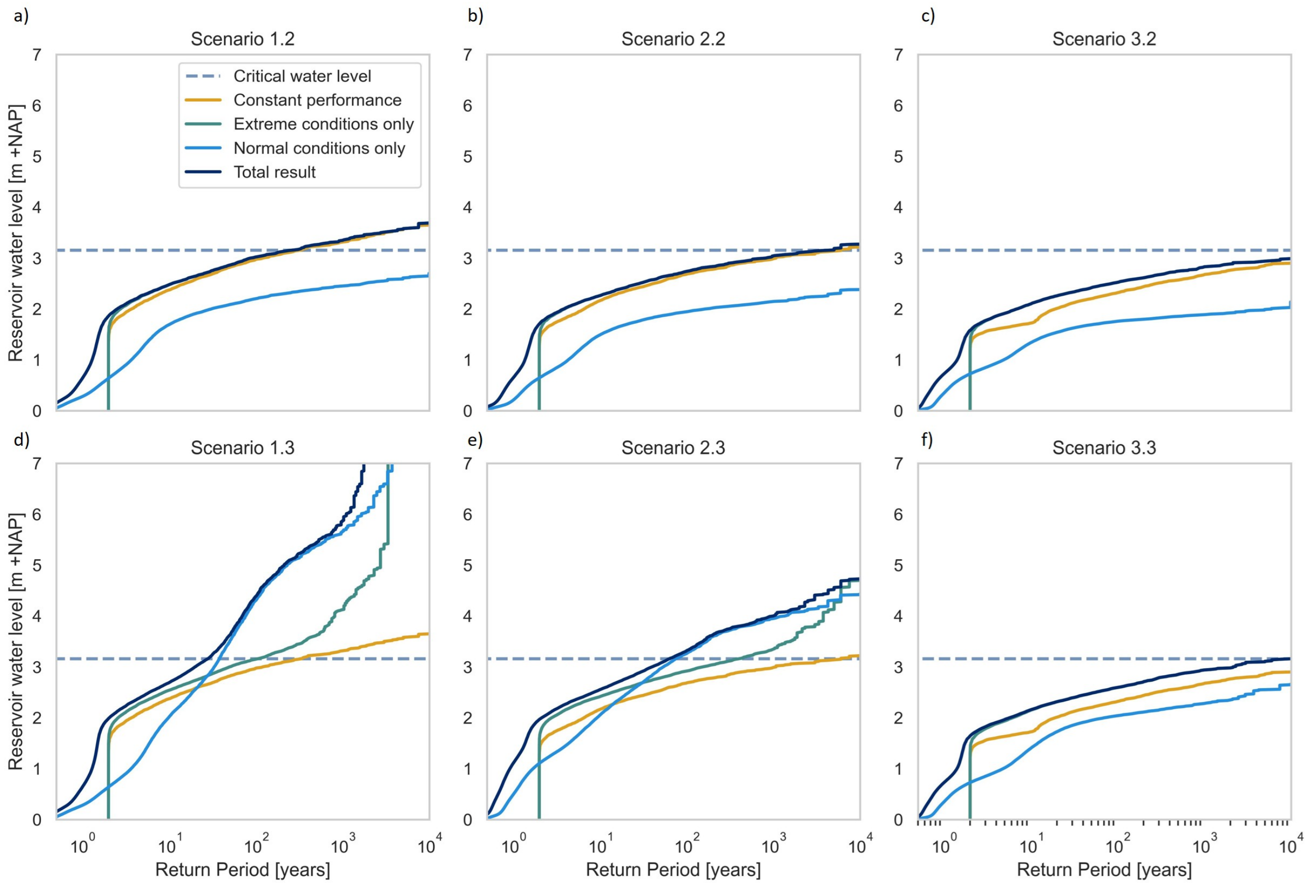

Figure 9. Exceedance frequencies for the design scenarios 1.2, 2.2 and 3.2 of Table 3 are depicted in the upper sub-plots and scenarios 1.3, 2.3 and 3.3 in the lower sub-plots. The upper sub-plots include a variation in the operational pump capacity. The lower sub-plots include a variation in operational discharge capacity of both pumps and sluices. The yellow curves show the result of baseline scenario for each design scenario (scenarios 1.1, 2.1 and 3.1). The results of the monte carlo simulation with non-extreme river statistics are shown in light blue, the results with extreme river statistics are shown in green. The final result is shown in dark blue, which is the addition of the light blue and green curve. If only operational pump capacity is included and the sluices are fully reliable, a lower effect is observed. The highest effect of operational discharge capacity can be observed in the lower left sub-plot, where the pump/buffer capacity ratio is highest.

illustrates that when planning to only employ pumps it is extremely important to take into account the operational window of 375 the pumps.

5.3 Required additional buffer area when taking into account variations in operational discharge capacity

As observed in Figure 9, the effect of operational discharge capacity depends on the system characteristics. The scenarios of Table 3 have different values for the installed pump capacity and the buffer capacity. The water level in water systems with

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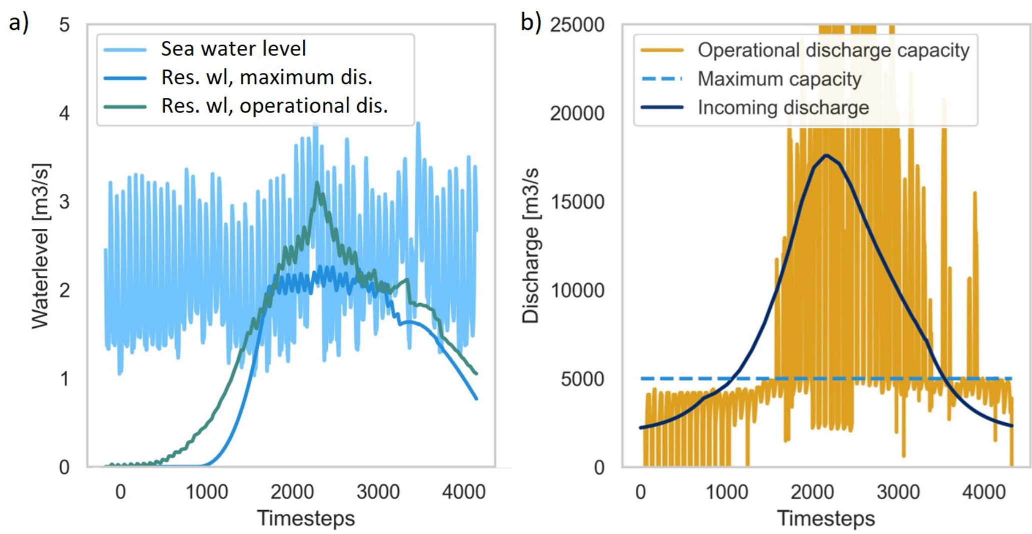

Figure 10. Results of simulations of an individual event in our monte carlo experiments. Left plot shows water level variations over time. The right plot shows the variation in the outgoing capacity during the event. The maximum pump capacity is set at 5000 m^3/s and the sluices are activated if the outgoing capacity surpasses this value.

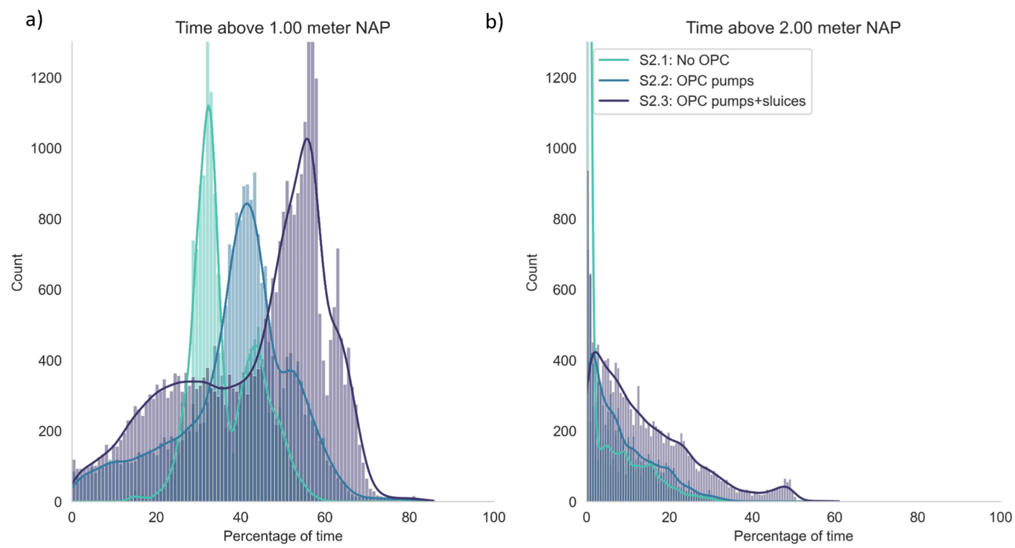

Figure 11. Depiction of percentage of the simulation time where the water level was higher than the threshold value. It is portrayed by histograms with the number of events on the y-axis and the percentage on the x-axis. 'No OPC' complies to scenario 2.1, 'OPC pumps' complies to scenario 2.2 and 'OPC pumps+sluices' complies to scenario 2.3.

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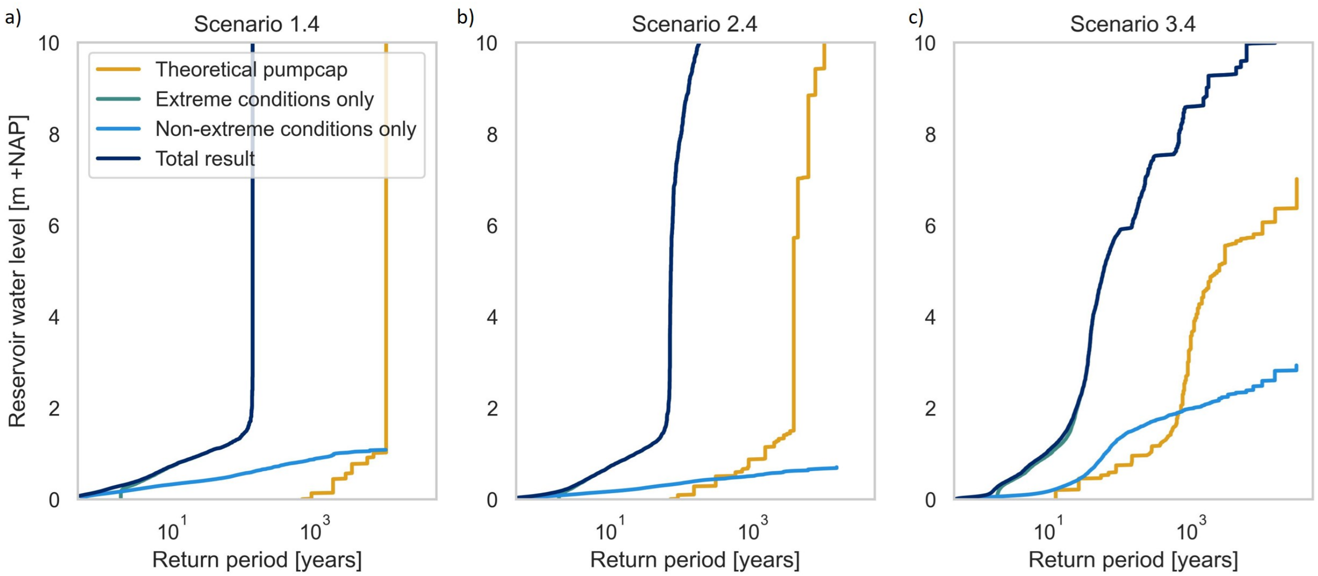

Figure 12. Exceedance frequencies for the design scenarios 1.4, 2.4 and 3.4 of Table 3 are depicted in the sub-plots. Only pumps are installed in these design scenarios. Operational discharge capacity is included in each scenario, but the ratio between pump capacity and buffer capacity varies per scenario. The yellow curves show the result of base scenario for each design scenario. The final result is shown in dark blue, which is the addition of the light blue and green curve. Note that the green curve mostly lies under the dark blue curve in all 3 sub-plots. The highest effect of operational discharge capacity can be observed in the left plot, where the pump/buffer capacity ratio is highest.

a relatively small buffer area will respond quickly in case of a excess of incoming water discharge. For a similar volume of excess river discharge, a water system with a larger buffer area will respond much slower, which can be observed in Figure 9. Ultimately, extra buffer area or installed pumps is required to account for the lower performance of the water system due to the inclusion of operational discharge capacity in the analysis. Figure 14 shows the additional required buffer area if operational discharge capacity is included, compared to a simulation under a fully available discharge capacity, corresponding to the baseline scenarios at a return period of 10,000 years. If both pumps and sluices are available for discharge (dark blue dots), the effect of operational discharge capacity decreases as the buffer area increases and the pump capacity decreases. Interestingly, if only pumps are available (light blue dots), there seems to be an optimum in the ratio of pumps capacity and buffer capacity. The effect of the operational discharge capacity is largest for the scenario with the highest amount of installed pump capacity and the lowest buffer area. The least additional buffer area is required for the middle scenario, while Figure 9 shows that water levels are higher for scenario 2 with 1000 km^2 than for scenario 3 with 1900 km^2. However, as the buffer area for scenario 3 almost doubles the buffer area of scenario 2, there can be more excess water volume in the reservoir for scenario 3 than scenario 2. The optimum can be explained by the difference of evolution of the head difference due to the different buffer areas. Another explanation could be the ratio between installed pump capacity and the discharge during extreme flood waves. If a failure event

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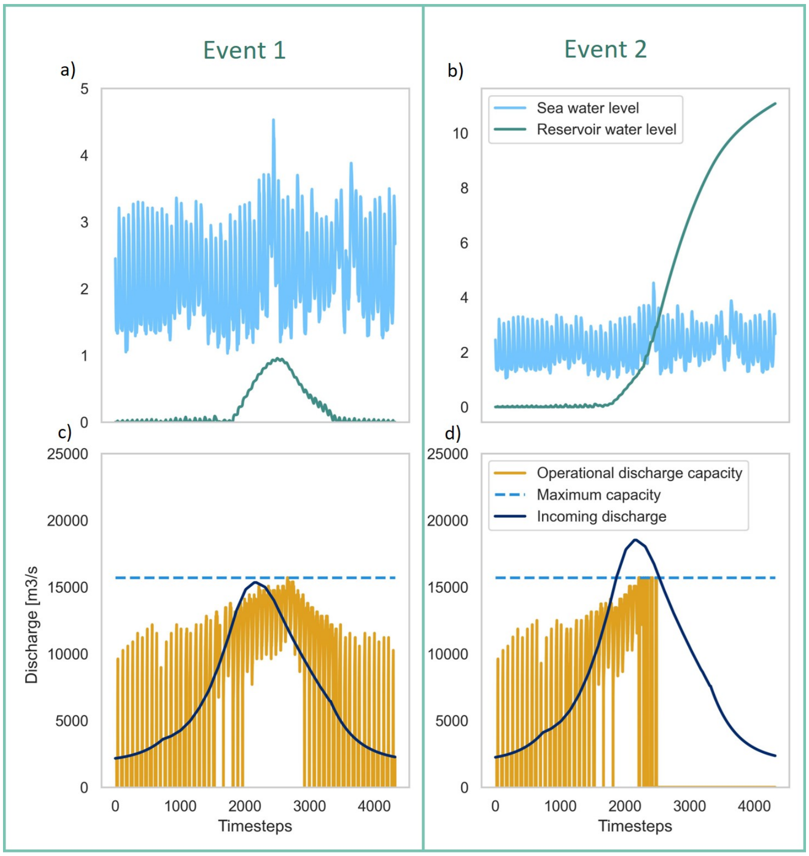

Figure 13. Results of simulations of two illustrative events in our experiments. Left plots shows the water dynamics for event 1, where the peak of the incoming river discharge remained below the maximum pump capacity. The right plots show an event where the peak river discharge exceeds the maximum pump capacity. At around time step 2500 the water level at sea remained below the reservoir water level, disabling the pumps. The water level in the reservoir will keep increasing, which explains the extremely high water levels in Figure 12.

occurs that shuts down half the pumps in the station, there would still be more pumps available to discharge the flood wave in scenario 2 than in scenario 3. The precise reason for the found optimum should be further researched.

6 Discussion and Conclusions

This study explored the importance of the operational discharge capacity when finding the required balance in sluice capacity, pump capacity and buffer capacity to safely discharge the rivers Rhine and Meuse under sea level rise. In this way, we extend on previous research (KPSLR, 2024a; Rijkswaterstaat, 2015) that considered a constant discharge capacity rather than the variations in availability included in the operational discharge capacity. We have included new system dynamics by introducing sluice reliability and operational pump capacity. A first conceptual overview of the most important interactions in a water

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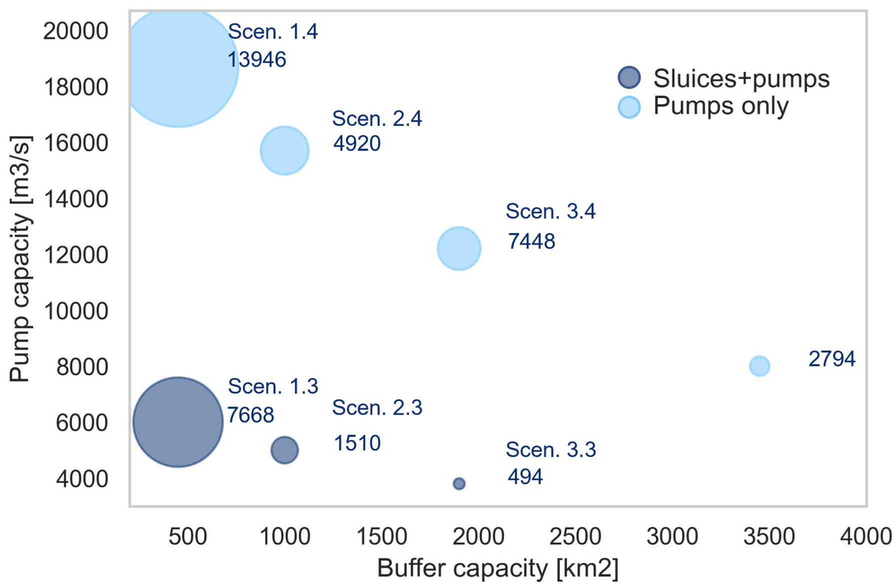

Figure 14. Comparison of the results of the different scenario runs of Table 3. The plot shows the additional required buffer area due to variations in operational discharge capacity for scenarios with pumps and sluices (dark blue) and scenarios with pumps only (light blue). The y-axis gives the installed pump capacity, the x-axis the available buffer capacity. The text next to the dots indicates the additional required buffer area in km^2 to store the additional excess water volume caused by higher water levels at a return period of 10,000 years if operational discharge capacity is included. The size of the dots indicates the ratio between the additionally required buffer capacity and the available buffer area.

system regulated by pumps and/or sluices was presented and the effects of introducing operational discharge capacity on the required pump capacity was calculated by use of monte carlo simulations.

When operational discharge capacity is included for both pumps and sluices, this leads to substantially higher water levels in the water system of our case-study. The frequency of the system failure increases from once every 10,000 years to once every 75 years in our calculations in Figure 9.b. The percentage simulated periods with risen water levels in the reservoir will increase as well.

The Haringvlietsluices play a dominant role in the Rhine-Meuse Delta. For scenario 2, a buffer capacity of 1000 km^2 there is a 1/5 ratio between the design capacity of the pumps and the sluices. This explains why when only operational pump capacity was included in Figure 9.b, it did not lead to higher maximum water levels in the reservoir. The discharge sluices can compensate for the lost pump capacity. Yet, introducing operational pump capacity did raise the reservoir water levels on average and more frequently, as the discharge sluices are only functional if the reservoir water levels are above sea water levels. To explore the effect of operational pump capacity if only pumps are installed, we also included scenarios where the sluice capacity was replaced by pumps as a means to discharge to sea. These results show that potential pump failure events,

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the pump curve and maybe most of all the operational window of the pumps are very important to consider in the pump design. These design considerations may be more of influence than the precise amount of pumps that are installed. As the pumps will mostly have to function at a relatively high head difference under a potentially relative low efficiency. Designing pumps with a high operational threshold in combination with high discharge capacity will be a challenge for pump manufacturers.

The impact of operational discharge capacity on a water system depends on the ratio of buffer capacity in the system and the maximum pump and sluice capacity. This relationship is not linear. The effect of operational discharge capacity decreases as the buffer capacity increases if both discharge sluices and pumps are active and if only pumps are active. Yet, if only pumps are active this decrease in the effect is smaller than if both pumps and discharge sluices are active (Figure 14). This can be explained by the smaller dependency on pumps if discharge sluices are present, which can compensate for the potentially lost pump capacity.

The results of this paper show that the Haringvlietsluices will potentially have an important role in this water system to prevent extreme water levels in the future, though they will not be in use as often as today. The sluices can only be opened if the water levels in the reservoir are already high, meaning there is a high river discharge or a large pump failure event. At these conditions, their discharge capacity is essential to prevent further increase of the water levels that could lead to system failure. Simultaneously, as the sea level rises it will become more difficult to test the reliability of the discharge sluices, as they cannot be tested during normal conditions. If the target water level in the reservoir remains unchanged, the reservoir water level under normal conditions will remain well below the sea level, preventing the sluice doors to be opened. Therefore, it is well possible that the future reliability of the discharge sluices will reduce compared to the settings used in this study. Furthermore, the consideration of maintenance costs should be taken into account, which will be relatively high compared to the frequency of use.

A main limitation in this study is the sensitivity of the results to the stochastic input. The frequency of high water levels in the reservoir are highly dependent on the extreme discharge statistics of the river Rhine. River discharge statistics used in this study are more extreme than projected by the KNMI'14 climate change scenarios (KNMI and Deltares, 2015), where peak discharges above 18,000 m^3/s were reduced due to physical limitations of the river system. Furthermore, a standard flood wave curve has been applied when generating the 30-day events , which was fitted to a peak discharge. In reality, this waveform is more variable, which could be crucial for the $Q_{in}* \Delta t$ that must be accommodated by the reservoir system and, consequently, how quickly the maximum water level is reached. Another limiting factor is the simulation time of 30 days for each event. At the end of a simulation period with a flood wave, the buffer is often not yet empty, though each new simulation will start with an empty buffer. During one winter a combination of flood waves, pump or sluice failures or a prolonged slight increase in river discharge may occur. These situations might not lead to an extreme water level during a 30 day period, but will slowly fill the buffer during the winter. Additionally, the extremely high water levels in the reservoir will not occur in reality, as a flooding would have occurred before these can be reached. In this study, the water levels in the reservoir are treated as an indication of the excess volume of water and the extra buffer capacity that would be required to accommodate this excess water volume.

It is difficult to determine which failure modes attribute most to the flood frequency of a water system. It could be the large failure events with long repair time but with a very low frequency of occurrence, or the single pump failure with a relatively high frequency of occurrence. Furthermore, the effect of multiple design choices is not easy to determine without re-executing the entire monte carlo experiment. All variables in Figure 5 are of influence on the water system dynamics, yet the importance of each design choice will likely vary per water system. It would require a lot of monte carlo experiments to find an optimum for all these design variables. A method to quickly assess the effect of these design choices is missing, which will advance the design cycle for pumping stations in open water systems a step further. To enable policy-makers to decide on the various design options investment costs, maintenance costs and functional costs and the flood damages should also be included in the design cycle.

This first exploration of the effect of operational discharge capacity on water systems with large pump-sluice stations has shown that it should be considered in design studies. It can lead to higher frequencies of critical water levels and longer periods of increased water levels. The precise effect of the operational discharge capacity on the frequencies of high water levels largely depends on the water system set-up. Though the used models were applied to a Dutch case-study, this approach could be applied to other deltaic water systems around the world. The dependence between peak river discharges and storm surge was not included in this study of the Rhine-Meuse delta following Klerk et al. (2015), but it might be necessary to include for other water systems. As operational discharge capacity is a new concept, more research will be required to be able to set design requirements for specific pump-sluice stations. These first explorations in this paper do show that potential effects of operational discharge capacity should not be omitted in pump design studies.

Code and data availability. The supporting Python code and used data can be downloaded at https://github.com/gijzenl/Paper-Operational-Discharge-Capacity/.

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