Thesis – Bachelor CE

Storage improvement of urban water catchments in Ede using the SOBEK model

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University of Twente – June 15, 2023

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Preface

This thesis is executed as a final assignment to achieve my bachelor's degree in Civil Engineering at the University of Twente. The research is conducted in collaboration with the Waterboard 'Vallei en Veluwe'. The thesis was proposed as an older problem of the waterboard but ended in an amazing adventure with interesting results. Being at the waterboard gave me great insights into what I want to be as a civil engineer, it is way more than making models or doing calculations, it is rather using your knowledge to help people. Seeing how all the people at the waterboard work each day to solve short and long-term issues motivates me to do more with my passion for water management.

A special thanks to Rosalie Arendt, my UT supervisor. The academic guidance was really helpful, it did not matter in which step of the process. For the research questions, proposal and the final report, all feedback and tips improved the outcome and I will take these new skills to my next study adventures.

For the waterboard, I could make a thank list as long as this page. All hydrologists were really supportive and interested in how it was going with the thesis and how it was going personally. Great talks gave me more insight into the context of the project and working at the waterboard in general. Being able to sit at the 'Hydrodesk' each week and at the hydrologist meeting each month gave me a really welcoming feeling. As well, a special thanks to my supervisors within the waterboard, Dimitri and Christian. Dimitri, thank you for giving so much insight into the actual water system. It is inspiring to see how you collaborate with so many parties, it makes you understand problems from a much broader view. You took the time to go with me on the site visit, without you, I would not have known what to look at. Christian, thank you for all the general and modeling help. The model improvements took a long time, but this time would have been doubled if Christian did not help me. I will miss the fun talks and tips for new music bands to listen to. I am grateful for all the new people I met, I cannot imagine getting this result without you.

I hope you enjoy reading my thesis.

Sincerely,

Jonne Hanning

Abstract

In the context of climate change and ongoing development in flood-prone areas, Europe faces the potential for an unprecedented increase in flood risk. The Netherlands, with its low-lying geography and high population density, is particularly vulnerable to this threat (Koopman, Kuik, Tol, & Brouwer, 2015). Implementing storage mechanisms to reduce regional flood peaks has been identified as a cost-effective adaptation strategy (Dottori, Mentaschi, Bianchi, Alfieri, & Feyen, 2023).

The Waterboard 'Vallei en Veluwe', responsible for managing the urban surface water system, has identified that the peak discharges in neighborhoods constructed in the 1970s and 1980s exceed the design norm of 3 l/s/ha. Although storage facilities are available, their utilization remains suboptimal. The objective of this thesis was to investigate the causes of underutilization and propose potential solutions for improving storage capacity in these urban catchments. The research focused on a case study of two catchments in Ede, Gelderland.

To begin, an enhanced SOBEK model was constructed by building upon an existing model. Data from the waterboard, municipality, and national authorities were integrated into the model. Remarkable deviations between the new and old data were observed. Measured discharge data from automatic weirs were used to calibrate the groundwater model. The model is verified through a site visit and validated using measured data. The performance of each model update is evaluated against the measured data using the Nash-Sutcliffe efficiency (NSE). Significant improvements in model performance (NSE increasing from -1.8 to 0.4) are observed, with the most substantial enhancement resulting from updating the paved area and overflow locations within the model.

Subsequently, the constructed model is utilized to identify the causes of underutilization. Hypotheses are tested by simulating peak events and assessing water levels. The simulations reveal that water levels can increase by up to 2 meters before flooding occurs. Additionally, the steady water levels exhibit minimal changes, supporting the hypothesis that the storage facilities are consistently full. Consequently, a full storage system leads to the direct discharge of peak events. It is concluded that the water systems were not originally designed to store peak discharges but rather to drain groundwater from surrounding neighborhoods. Implementing additional drainage methods can reduce the need for lower water levels, thus allowing for increased utilization of storage capacity. To pinpoint the specific problem areas, a map illustrating the relative discharge in l/s/ha per weir is produced. This map exposes the bottlenecks in the water system and can be used to identify the starting point for implementing interventions. The relative discharges downstream of both catchments exceed the norms, with values of 7.6 and 7.4 l/s/ha.

Finally, the potential solutions are modeled to assess their impact on peak discharge. Considering budgetary, temporal, and technical constraints, the selected intervention is the improvement of weirs, specifically by incorporating V-notch structures. V-notch weirs are chosen for their ability to modify discharge capacity based on water heights. Implementing V-notch weirs with heights of 0.3 and 0.5 meters results in a reduction of peak discharge by 20% and 40% respectively. The water levels upstream of the weirs temporarily rise but are lowered within 48 hours to limit changes in groundwater levels. Incremental implementation of V-notch weirs enables data collection between interventions, which facilitates the validation of changes in discharge, surface water levels, and groundwater levels.

To implement the proposed interventions across the entire management area of the Waterboard, improvements in urban water models are necessary. Undertaking a comprehensive update of the urban model for the entire management area in a single project is more cost-effective, as the process of improving the model is time-consuming and independent of the amount of data. However, achieving this requires enhancing the integration of data between municipalities and the waterboard. Benefit linkage can be utilized to foster comprehensive and long-term collaboration. The most important municipal data for urban models are overflow locations and sewage capacity. The collaboration requires clear storage definitions, peak storage should be redefined and based on the amount of water that is not directly discharged, rather than the absolute volume of the storage facility.

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List of Abbreviations

Abbreviation	Definition
AHN	Current Dutch Elevation
BAG	Register of adresses and buildings
BGT	Register of large-scale topography
FEWS	Waterboard information system
GCM	Global Climate Models
H2gO	Municipal information system
HKV	Consultancy firm
IPCC	Intergovernmental Panel on Climate Change
KNMI	The Royal Netherlands Meteorological Institute
LGN4	Dutch land-use grid
MCDA	Multi-Criteria Decision Analysis
NAP	Dutch reference water height
NBW	National agreement on water
NSE	Nash–Sutcliffe Efficiency
PID controller	Proportional–Integral–Derivative controller
PVC	Polyvinylchloride, Plastics
RCM	Regional Climate Models
RCP	Representative Concentration Pathway
Register data	Waterboard register data
RMSE	Root Mean Square Error
RR model	Rainfall-Runoff model
T=100	Pecipitation event with occurance frequency of 1 per 100 years
TMX	Control system for automatic weirs
WUR	Wageningen University and Research
WWTP	Waste Water Treatment Plant

Introduction

Climate change is a significant issue that is causing several problems, including flooding. Warmer oceans, rising sea levels, and extreme weather events are all contributing to increased pressure on water bodies, leading to more frequent and severe floods (Ebi et al., 2021; UNFCCC, 2021). In Europe, river flood risk could rise to unprecedented levels due to global warming and continued development in flood-prone areas (Dottori et al., 2023). The Netherlands is a country that is particularly vulnerable to the impact of climate change due to its low-lying geography and high population density. The increase in paved areas, together with the increase in precipitation due to climate change, is leading to flood issues throughout the Netherlands. (Koopman et al., 2015; McDonald et al., 2014; Rijksoverheid, 2023)

The management of water resources in urban areas is complex and requires the integration of various disciplines, including hydrology, engineering, and social sciences. The challenges of urban water management in the Netherlands are multifaceted and require innovative solutions to ensure sustainable water management (Heidari, Arabi, Warziniack, & Sharvelle, 2021; Haasnoot et al., 2012). To address this issue, reducing regional flood peaks with storage is a cost-effective adaptation strategy (Dottori et al., 2023). The storage of water provides a means of managing stormwater runoff, which can help to reduce the risk of flooding (Kalbus, Reinstorf, & Schirmer, 2006). Water storage also ensures the availability of water during periods of high demand (Santos, Galvão, & Cardoso, 2019; Kneese, 2011).

The impacts of climate change on urban water management in the Netherlands are significant and require adaptation measures to ensure the resilience of water systems (IPCC, 2022). Climate change is leading to changes in precipitation patterns, which are affecting the availability of water resources in urban areas. The increasing frequency and intensity of extreme weather events, such as floods and droughts, are also affecting the management of water resources in urban areas (Rijksoverheid, 2023; European Environment Agency, 2014). Currently, a national approach is used to allocate water. More storage is established in the case of underperformance, and critical thinking about the current system is neglected (Rijksoverheid, 2011; Rijkswaterstaat, 2021).

To assess the capacity of regional water storage in the Netherlands, discharges and water heights are measured and evaluated. Water authorities conclude that currently, the storage capacity is insufficient (waterschap vallei& veluwe, 2013; waterschap vallei en veluwe, 2023). Nationally funded measures are taken to increase flood resilience (Delta commission, 2023, 2015). The waterboard is partially responsible for redesigning the regional water storage. The waterboard assessed surface water systems using discharge and water height data. This assessment exposed the weaknesses in the water system. Catchments in the 1970s and 1980s neighborhoods underutilize their storage capacity (waterschap vallei& veluwe, 2013).

The causes of underutilization are diagnosed with a model-based approach. Interventions are modeled to assess their impact on the capacity (Pandi, Kothandaraman, & Kuppusamy, 2021; Wheater et al., 1993; Dickinson, 1991). The SOBEK model by Deltares gives insight into detailed hydrological processes to identify a wide range of potential causes of underutilization (Deltares, 2013). The research areas are specific but the results are converted into a general policy for other neighborhoods. The results of the research can help a significant amount of catchments. It contributes to the adaptations to climate change and helps the guidance towards a resilient and sustainable water system (Unie van Waterschappen, 2022; waterschap vallei& veluwe, 2013; Delta commission, 2023).

The report will begin with a literature review to obtain currently available knowledge. Literature about the areas, hydrological processes, water system interventions, changing climate, and current policies are included. These topics form the context and background for the research. The methodology elaborates on how the research is conducted and the results section displays the outcomes. The discussion evaluates the results and the reliability of the results. Conclusions are drawn from the results and discussion and recommendations are proposed for the waterboard.

Literature review

2.1 Project area description

The research area under consideration is located in Ede, Gelderland, Enschede, comprising two catchments, namely Veldhuizen and Rietkampen and Frankeneng. These catchments are situated on the east side of Ede, whereas the west side is constructed on an elevated push moraine, leading to the discharge of water from the higher regions into a series of basins positioned between the east and west sides of Ede (Dino loket, 2023). To manage this situation, a water system has been designed and constructed by the water board, specifically for the west side of Ede, to drain the water to the national surface water system (waterschap vallei en veluwe, 2023). The figures 1, 2a, and 9b provide the geographical locations of the neighborhoods in Ede and the water system details as recorded in the register data provided by the water board for Veldhuizen and Rietkampen and Frankeneng, respectively.

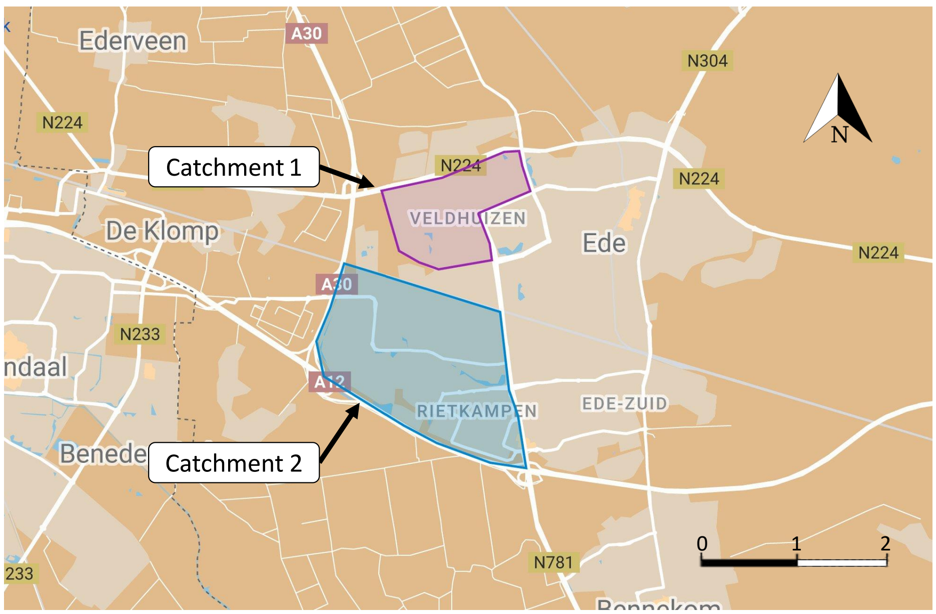
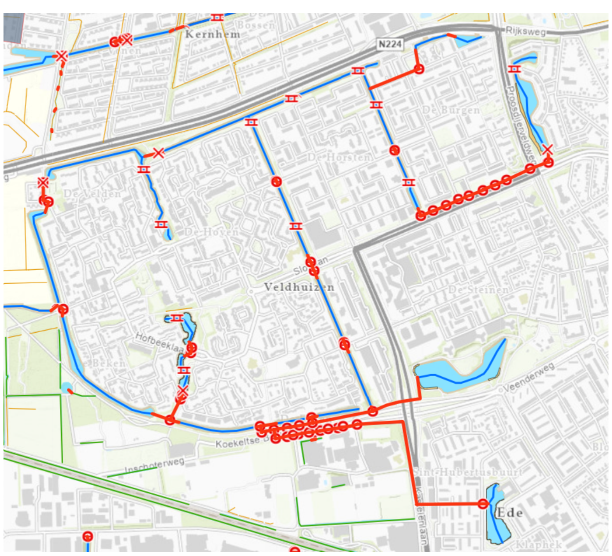
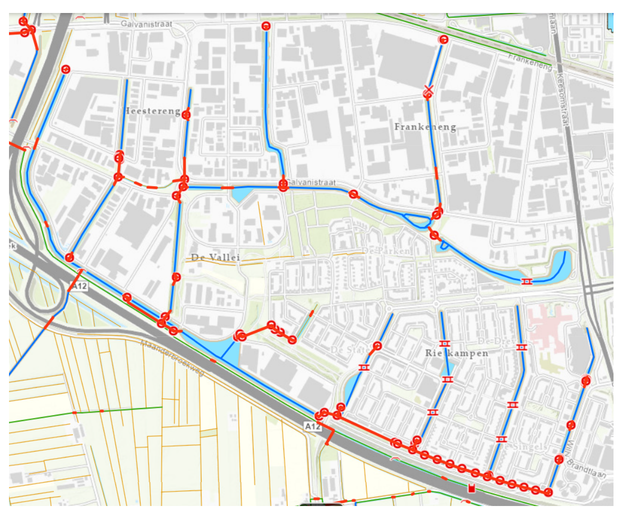


Figure 1: Catchment locations



(a) Catchment Velthuizen



(b) Catchment Rietkampen & Frankeneng

Figure 2: Water system of both catchments (waterschap vallei en veluwe, 2023)

2.1.1 Current storage systems

The current storage system consists of different tools to retain and store the water in the catchments. The first feature is upstream storage at the starting point of the system, all water from the elevated east side of Ede is stored in these basins (In de buurt Ede, 2021). Additional features contain other surface water basins integrated into the urban design, which are regulated by non-automatic weirs (waterschap vallei en veluwe, 2023). At the end of both catchments an automatic weir is used to regulate the outflow of water. This outflow provides the validation and calibration data for the model (Pannekoekgww, 2014).

2.2 Hydrological background urban catchments

Rainwater is a result of a complex interplay of macro and micro factors, operating on a global scale. The water cycle is influenced by a range of factors, including the ocean, atmosphere, sun, wind, land use, and other factors (Ito & Inatomi, 2012). These factors are continually subjected to changes, which, when combined, make it difficult to predict both average precipitation and extreme events with a high degree of accuracy (Wild & Liepert, 2010). In this section, the aim is to focus on the local water cycle, specifically considering precipitation at a local statistical scale instead of the complex global water system. A distinction between the theory of urban (sub)surface water and groundwater is made to gain a better understanding of the local water cycle (Batelaan & Smedt, 2007; Ito & Inatomi, 2012).

2.2.1 Storm water

Upon narrowing the focus to a smaller catchment, the system can be described as a single raster model, as depicted in Figure 3 (Batelaan & Smedt, 2001). In this model, the catchment comprises of water inflow, internal water processes, and water outflow. The outflow can be deduced when the inflow and internal processes are known. The catchment's surface water processes of utmost importance include: (Batelaan & Smedt, 2001, 2007)

- Precipitation, which is the inflow of rainwater that varies over time and space. It is caused by the transition of water from a gaseous state to a liquid state. (Batelaan & Smedt, 2001)
- Evaporation, which is the process of water transitioning from a liquid to a gaseous state back into the atmosphere. (Batelaan & Smedt, 2007)
- Transpiration, which is the release of water from plants, beginning at the roots and released through the leaves. (Batelaan & Smedt, 2001)
- Impervious runoff, which is the surface water runoff from impermeable surfaces. (Batelaan & Smedt, 2001)
- Soil recharge, which is the process of surface water flowing into the soil towards the groundwater. (Batelaan & Smedt, 2007)
- Surface water storage, which is utilized to delay runoff. (Batelaan & Smedt, 2001)

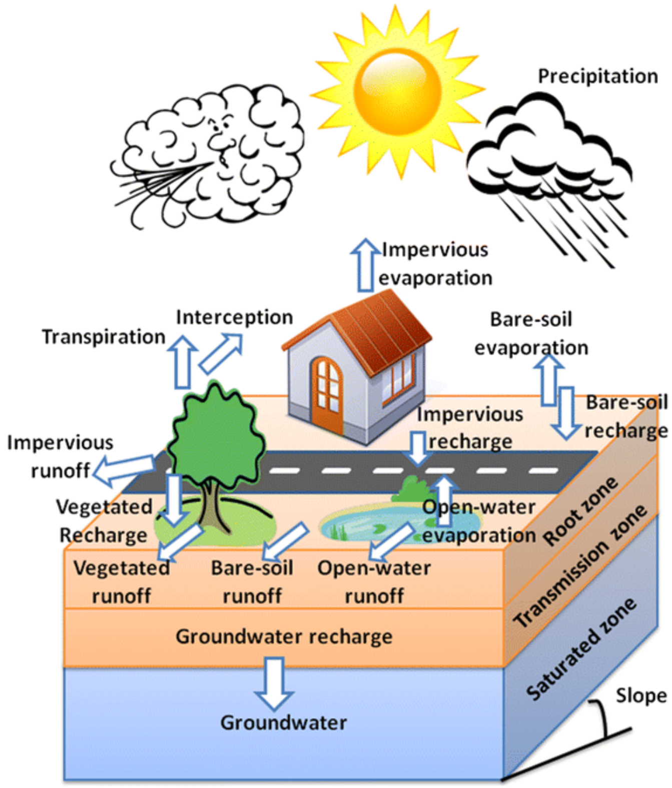


Figure 3: Schematic representation water balance (Batelaan & Smedt, 2001)

2.2.2 Ground water

When water flows over impervious surfaces such as soil, it infiltrates into the soil and groundwater system, undergoing various processes. The principles of these processes can be described using the bucket model (Booij, 2020). The bucket model, depicted in Figure 4, is based on three storage types, namely surface water storage, soil moisture storage, and groundwater storage. These storage types represent the different reservoirs that water can be stored in within the soil and groundwater system. (Booij, 2020; Dickinson, 1991)

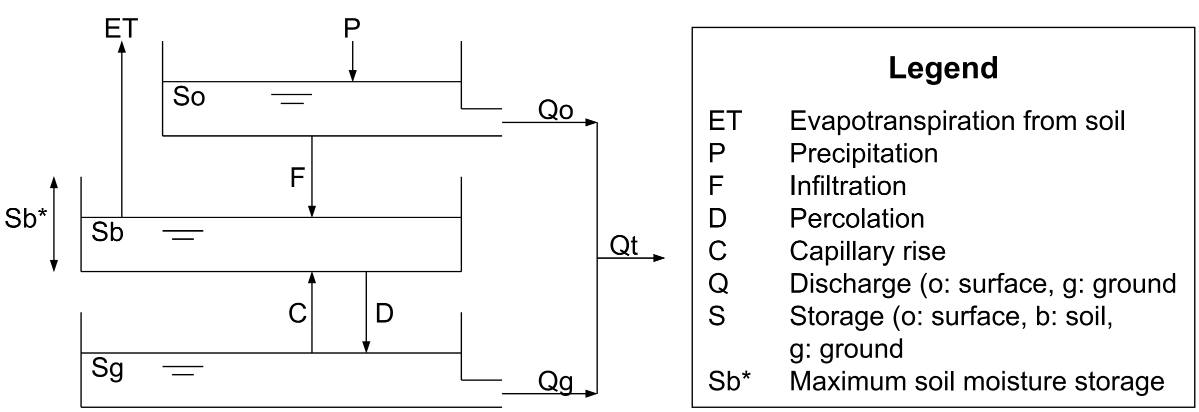


Figure 4: Schematic representation of ground water processes (Booij, 2020)

Effective water retention in soil is vital for the efficient functioning of water systems and encompasses both functional retention and storage. This retention process can be best understood through the principles of groundwater modeling which share similarities with surface water systems. With a clear understanding of water inflow and internal processes, the outflow can be deduced. The groundwater model is primarily driven by concepts based on the bucket model, as described in the relevant literature. (Booij, 2020; Dickinson, 1991)

The drivers of groundwater modeling include key factors such as evapotranspiration, infiltration, percolation, capillary rise, and discharge/runoff of groundwater into the surface water system (Sood & Smakhtin, 2015). The same article states that evapotranspiration, a combination of evaporation and transpiration, has a similar impact on the system, and plays a crucial role in modeling. Infiltration is a significant factor and is caused by gravity (Gregory, Dukes, Jones, & Miller, 2006). Percolation is also caused by gravity and involves the movement of soil moisture into the groundwater aquifer (Sammis, Evans, & Warrick, 1982). Capillary rise is driven by the adhesion and cohesion of water particles in permeable soils (Rukundo, Do, & Gan, 2019), while discharge/runoff of groundwater into the surface water system is an essential component of the groundwater model (Kalbus et al., 2006).

The equations for all processes and interactions involved in the groundwater model are based on various theories and depend on the specific model. While complex equations yield higher precision, they also increase the vulnerability to biased or erroneous information. Another issue related to modeling is the requirement for substantial computational power. (Kalbus et al., 2006)

2.2.3 Storm water management

The water cycle is a natural system that can be manipulated to improve livability (Batelaan & Smedt, 2007). A range of measures can be employed to reduce the likelihood of floods, ensure the availability of a reliable water supply, safeguard water quality and mitigate against the possibility of droughts. Several physical components are utilized in water management for the purpose of system interconnectivity, water level maintenance, storm water drainage and system accessibility. (Santos et al., 2019)

Rectangular weir

Weirs serve a crucial function in regulating water height in water systems. For small catchments, the rectangular weir has been widely adopted as a conventional weir. It comprises a nonadaptable sheet with a rectangular flow gap, as depicted in Figure 5 (Bengtson, 2022; Gharahjeh, Aydin, & Altan-Sakarya, 2015). The key dimensions of a rectangular weir include the water head (h), crest width (b), total width (B), and crest height (P) (Bos, 1975). The discharge (Q) through the weir is expressed as a function of water head, crest width, gravitational acceleration (g), and discharge coefficient (C_d), which represents all assumptions made in Equation 1 (Rehbock, 1929).

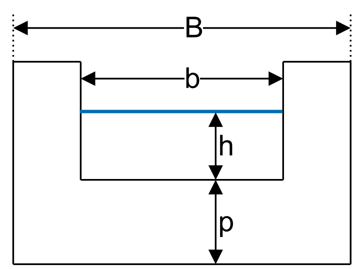


Figure 5: Cross-section rectangular weir

$$Q = \frac{2}{3} C_d \sqrt{2g} b h^{3/2} \quad (Bos, 1975) \tag{1}$$

V-notch weir

The V-notch weir depicted in Figure 6 features a triangular-shaped water gap, which facilitates a higher discharge rate at elevated water levels. As such, the V-notch weir exhibits superior performance to the rectangular weir in cases of extreme rainfall conditions (Ibrahim, 2015). The discharge rate of the V-notch weir may be expressed as equation 2, which is a function of the notch angle (α), h, g, and C_d (Chanson & Wang, 2013)

$$.Q = \frac{8}{15} C_d tan(\alpha) \sqrt(2gh^5) \quad (Chanson \& Wang, 2013) \tag{2}$$

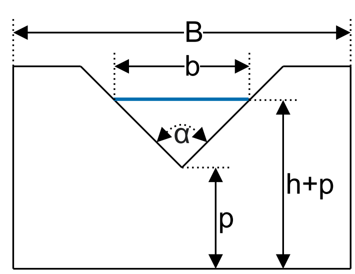


Figure 6: Cross-section V-notch weir

Automatic weir

In water engineering, conventional weirs are known to be rigid structures that remain constant over time. However, the changing conditions of water systems can lead to their underperformance. To overcome this limitation, automatic weirs have been developed that can alter their crest height based on the water level, thus providing more control and resilience in operation (Hoitink, Dommerholt, & Gerven, 2008). As shown in the side view in Figure 7, the crest height of an automatic weir comprises a static crest height (p1) and a dynamic crest height (p2). The frontal cross-section of an automatic weir is equivalent to that of a rectangular weir, as shown in Figure 5. (Lozano, Arranja, Rijo, & Mateos, 2010; KWT waterbeheersing, n.d.)
The discharge (Q) through an automatic weir can be calculated using Equations 1 and 3, Equation 1 requires the water head (h) of the rectangular weir shown in Figure 5. The water head (h) can be described in terms of the angle β, water height (H), tilting crest length (L), and the rigid crest height (p1) (Shafiei, Najarchi, & Shabanlou, 2020).

$$h = H - p1 - cos(\beta)L \quad (Shafiei\ etal.,2020) \tag{3}$$

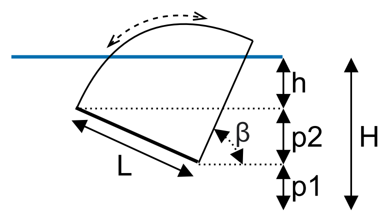


Figure 7: Side view automatic weir

PID controller

The water level in the automatic weir is regulated through the utilization of a Proportional-Integral-Derivative (PID) controller. This controller functions by assessing the disparity between the desired water level and the current water level. In order to maintain stable control over the water level, the PID controller incorporates a feedback loop that comprises three essential components: Proportional, Integral, and Derivative. Figure 8 visually represents the sequential operation of the PID controller. (Baratov, Bon, Chulliyev, Shoyimov, & Abdullayev, 2021)

- The proportional element indicates the difference between the actual and desired water level.
- The integral element integrates the error over time, it eliminates steady-state errors by cumulative error description.
- The derivative element takes into account the rate of change of the error, it measures how quickly the error is changing and can be used to provide a predictive element to the controller.

If oscillations occur, adjustments should be made to these values, ensuring a balance between counteracting oscillations and maintaining a realistic response from the controller (Baratov et al., 2021).

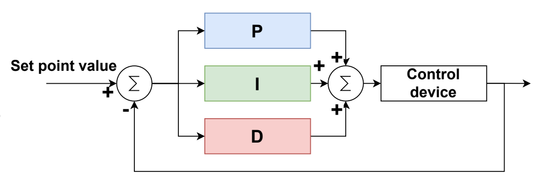


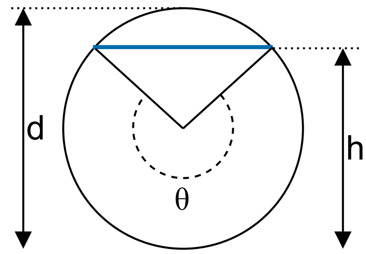
Figure 8: PID controller sequence

Culvert
In the Dutch water system, both surface and subterranean interventions are employed. Due to the intricate design of urban areas, the surface water system cannot be seamlessly integrated. Culverts, which are essentially pipes, serve as a means to connect surface water systems in these zones. The discharge (Q) through a culvert can be expressed as Equation 6 which is a function of the Manning's roughness coefficient (n), cross-sectional area of flow (A), hydraulic radius (H), and slope (S) (*Culvert Hydraulics: Basic Principles*, n.d.). Figure 9a depicts the cross-section of a culvert. The cross-sectional area (A) and hydraulic radius (H) can be represented as functions of the angle (θ) and culvert height (D), and are calculated with Equation 4 and 5, respectively (Normann, Houghtalen, & Johnston, 2001).

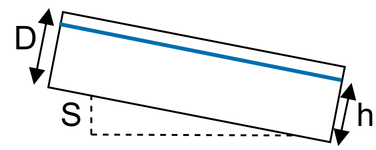
$$A = \frac{1}{8}(\theta - sin\theta)D^2 \quad (Normann\ etal.,2001) \tag{4}$$

$$R = \frac{D(\theta - sin\theta)}{8sin\theta/2)} \quad (Normann\ etal.,2001) \tag{5}$$

$$Q = \frac{1}{n}AR^{(}\frac{2}{3})\sqrt{S} \quad (Normann\ etal.,2001) \tag{6}$$



(a) Cross-section culvert



(b) Side view culvert

Figure 9: Culvert

Storm water overflow
A culvert has a limited capacity. During extreme events, an overflow can be used to guide the water elsewhere. The discharge of the overflow is based on the water heights. Models use a wide variety of calculation methods. (Mohandes et al., 2022)

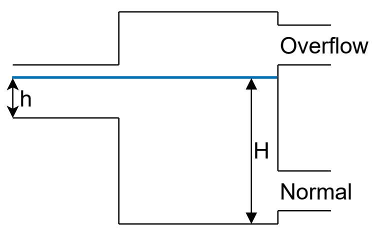


Figure 10: Side view overflow

Manholes Manholes are a crucial component of culvert systems, serving various functions such as maintenance, inspection, cleaning, and obstruction removal. They also provide an effective means of modifying culvert slope, changing alignment or direction, and joining culverts. In addition, manholes are commonly equipped with overflows to facilitate maintenance. (Abbas, Ruddock, Alkhaddar, Rothwell, & Andoh, 2018; Gebreegziabher & Demissie, 2020)

Figure 11: Manhole (Gelderlander, 2018)

2.3 Available simulation models

Hydrological catchment assessment is a complex process that does not have a universally accepted theorem or model. The selection of a model depends on factors such as budget, available data, research purpose, and prevailing paradigms (Sood & Smakhtin, 2015). Consequently, there exist numerous hydrological simulation models, each potentially implemented using different simulation software. To identify possible simulation models, different classifications are evaluated to ensure that a diverse collection of models is included (Pandi et al., 2021).

2.3.1 Classification

Models can be categorized using different classifications, a general classification is based on the structure of the model. The models are distinguished as metric models, conceptual models, physics-based models, and hybrid models. (Jackson et al., n.d.)

1. **Metric models** are mainly based on observations, evaluating the system response to the available data. (Wheater et al., 1993; Jackson et al., n.d.)

2. **Conceptual models** specify the structure of the model prior to any modeling. Not all parameters have a direct physical interpretation (Wheater et al., 1993; Jackson et al., n.d.)

3. **Physics-based models** are based on continuum mechanics, using equations of motion to represent hydrological processes such as evapotranspiration and infiltration. (Beven, 2010)

4. **Hybrid models** seek to combine the advantages of the other model classifications. Metric-conceptual models integrate the data-based approach with conceptual modeling. Physically-based-conceptual models simplify mathematical physics in a conceptual manner. (Wagener, Mcintyre, Lees, Wheater, & Gupta, 2003)

Inclusion of pertinent models within the four classifications is undertaken. An illustrative depiction of these potential models, categorized by classification, is furnished in Table 8 of the Appendix section referenced as 8.1. Additionally, each model is accompanied by a comprehensive exposition of its merits and demerits, serving to provide a holistic overview of the model. Section 8.2 elaborates on the Multi-Criteria Decision Analysis which assesses the models. The SOBEK model is used for its fitness to the criteria and familiarity within the waterboard.

2.3.2 Important parameters and assumptions

The SOBEK model can use the register data for the attributes, the precipitation data, and the land-use data of the municipality and can be validated with the measured data. There is a wide variety of parameters that can be altered. A physical examination can give better insight into the uncertainty of these parameters (Deltares, 2013). The input, output, and general process are shown in Figure 12. A few uncertain parameters and assumptions that can have a large effect on the final discharge of the model are: the vegetation type and height, the storage basin size and shape, the permeability of the surface, the inflow of ground water, the evapotranspiration potential, and the interaction with the downstream water system (Appsolutelydigital, 2013; Vanderkimpen, Melger, Peeters, et al., 2009).

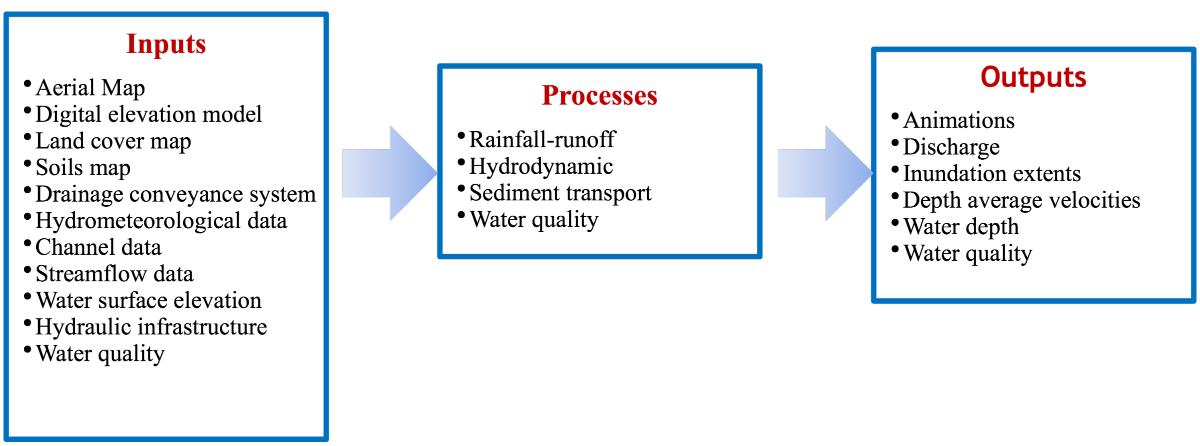


Figure 12: SOBEK input, output and general processes (Appsolutelydigital, 2013)

The SOBEK model is a versatile tool that allows for the modeling of both two-dimensional (2D) and one-dimensional (1D) systems. The 2D module simulates overland flow, which is useful for flood assessments, while the 1D module incorporates water systems and interventions as described in Section 2.2.3 (Deltares, 2013). Real-life data can be included in the Rainfall-Runoff (RR) module, which results in more realistic modeling. The SOBEK model involves several steps as displayed in Figure 13. Initially, the network, settings, and meteorological data are provided. Subsequently, the physical schematisation of the model is designed, and the simulation is conducted. The output of the simulation model is diverse and includes water heights, discharges, and flow velocities at all points and times, as well as maps, charts, and tables. Furthermore, side views provide more insight into the distribution within the system. (Betrie et al., 2011)

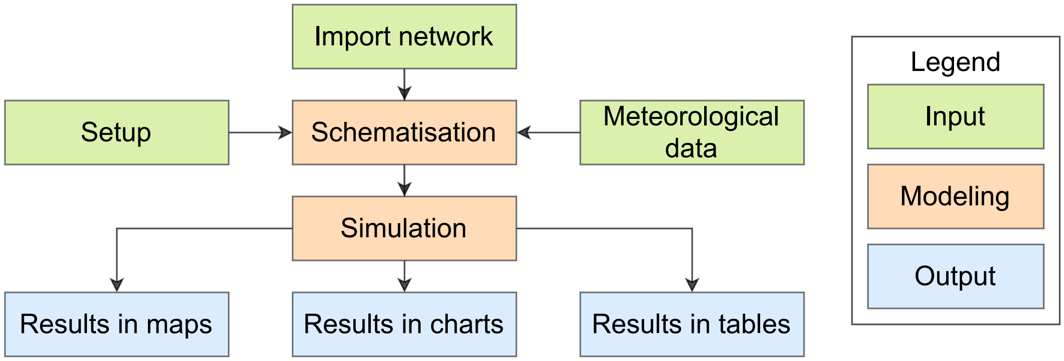


Figure 13: Schematic representation of processes in SOBEK (Deltares, 2013)

2.4 Change in precipitation

Throughout the world, a major issue that has emerged is the fluctuation of precipitation resulting from climate change. To address this issue, proactive measures are recommended instead of reactive adaptation. This is particularly important given that the impacts of increasing precipitation can already be observed in water systems (Bador et al., 2020). To estimate the magnitude of this increase, the European Environmental Agency (EEA) employs a combination of Regional and Global Climate Models (RCM& GCM). The RCM& GCM simulations project a percentage increase in precipitation across Europe, as shown in Figure 14. These estimates are based on a Representative Concentration Pathway (RCP) of 8.5 (Pierce, Barnett, Santer, & Gleckler, 2009). Specifically, for the Netherlands, the RCM& GCM simulations suggest an average increase in precipitation of 15%-25% during winter and 5%-15% during summer. (Bador et al., 2020; European Environment Agency, 2014; IPCC, 2022)

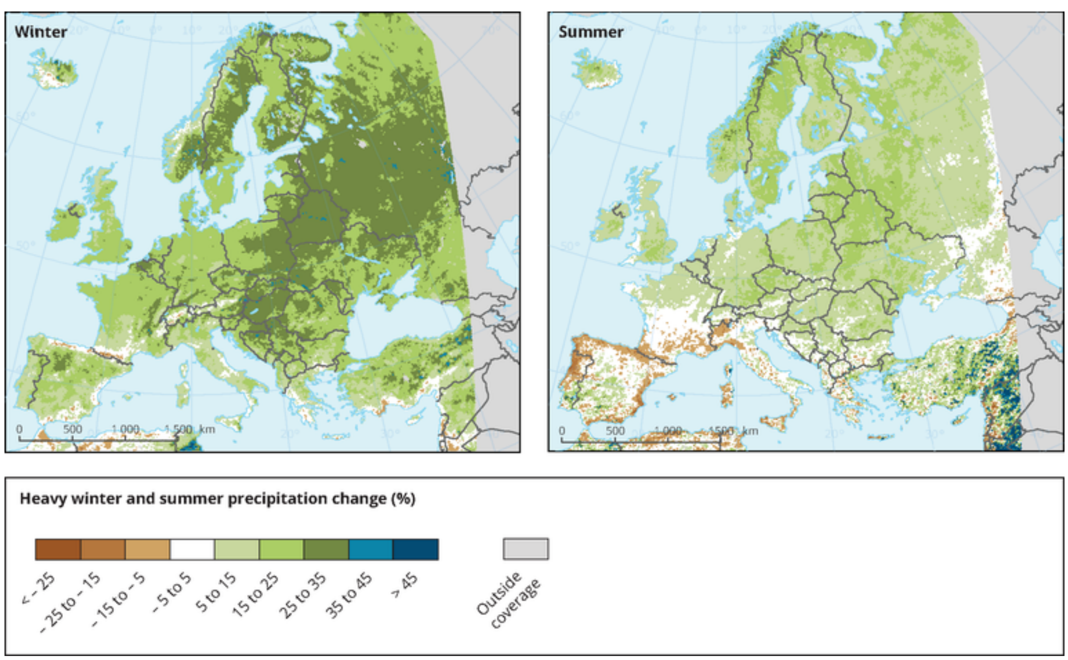


Figure 14: Change in precipitation for the RCP 8.5 scenario (European Environment Agency, 2014)

The use of mean temperatures to assess climate variability may not be adequate for assessing extreme weather events, which have increased in frequency and intensity globally. These events, such as droughts and extreme precipitation, are particularly challenging to manage as their high peaks can result in flooding. It has been observed that the frequency of extreme events is projected to exceed the 25% threshold. (Orlowsky & Seneviratne, 2012)

2.5 Current policies

2.5.1 National policies and laws

Water management is a critical aspect of Dutch society, and stakeholders and authorities at national, regional, and local levels recognize its importance. To this end, they have joined forces to establish a binding administrative agreement known as the 'Bestuursakkoord water', which outlines four central principles: flood protection, fresh water supply, reduction of regional water nuisance, and chemical and ecological water quality (Rijksoverheid, 2011). An integrated approach involving commercial parties, authorities, and residents is adopted to ensure that these goals are achieved. This approach enhances the acceptance and quality of water management projects. In particular, surface water projects involve a collaboration between the provincial authorities, waterboard, municipality, and drinking water companies, and all interventions are transparently discussed to guarantee future-proof solutions. (Rijksoverheid, 2011)

Another national water management approach in the Netherlands is the Deltaprogramm, which was launched in 2015. It provides a government-funded long-term water vision and prioritizes storage and drinking water supply for regional fresh surface water (Rijkswaterstaat, 2021). The program employs stress tests to identify weaknesses in the Dutch water systems and adopts an integral approach to resolve these issues. The most recent version of the program emphasizes the measurability of interventions (Delta commission, 2015). To ensure the quality and integration of different parts of the water system, an independent Delta commission serves as the link between the waterboards, government, and provincial authorities. Technical guidelines for water management are provided by each waterboard or Rijkswaterstaat. (Delta commission, 2023)

2.5.2 Waterboard policies

Waterboards are governmental bodies responsible for water management in the Netherlands. They are bound to guidelines and agreements with local and regional authorities. The Waterboard 'Vallei en Veluwe' is responsible for water management of sections of the water system in the provinces of Utrecht, Gelderland, and Overijssel (Unie van Waterschappen, 2022). In accordance with a written agreement with the provinces, the water system needs to be operational during a precipitation event that occurs once in every 100 years (T=100). Moreover, the water stored in non-paved areas is expected to be 60mm per m^2. Any construction plans that increase the pavement area must include provisions for water storage to maintain this requirement. With the potential effects of climate change, this norm may be revised from 60mm to 90mm. (waterschap vallei& veluwe, 2013)

Methodology

The present study is structured around a methodology consisting of four distinctive segments. Firstly, section 3.1 expounds upon the process of model updating. The initial model used for analysis underwent frequent modifications to enhance its comprehensiveness. Each individual step involved in updating the model is delineated to provide a comprehensive understanding of the underlying processes. Subsections include section 3.1.3 and 3.1.4, which elaborate on the calibration and sensitivity, respectively. Secondly, the validation procedure is outlined in section 3.2, as it holds substantial significance in evaluating the reliability of the model and the resultant outcomes. Subsequently, the identification of system problems is presented in section 3.3, with the objective of pinpointing factors that contribute to the underutilization or unavailability of storage. In this section, hypothesis are proposed. Finally, potential interventions are proposed in section 3.4 to address the aforementioned storage issues. Each of these segments plays a crucial role in drawing conclusions pertaining to the research objective. The initial step offers insights into the necessary improvements required for accurate assessment of small urban systems. The validation step illuminates the reliability of the obtained results. The third step uncovers existing problems within the current system, while the final step proposes potential solutions to rectify these issues.

3.1 Model Update

The assessment model utilized in this study is derived from the waterboard and constitutes an integral component of the national administrative water agreement (NBW) (STOWA, 2012). Originally developed in 2012, the model incorporates an extensive range of data, which has been subsequently updated and expanded upon as necessary. Specifically, the assessment model focuses on simulating the ”South” district of the waterboard. For computational efficiency, the model is divided at its connection to the main water body, referred to as ”Het Vallei kanaal.” To enhance accuracy, the model has undergone modifications and updates, primarily targeting the upstream segment of the system with regard to the automatic weirs. The downstream system, which lies beyond the scope of this study, primarily serves as a water discharge buffer.

Within the SOBEK model framework, two distinct modules, namely the Rainfall-Runoff (RR) module and the Rural 1D Flow Module, are sequentially employed. The interaction between these models can be performed either subsequently or interactively at each calculation step. In order to optimize computational efficiency, the latter approach has been chosen. The process of updating the models is described for both modules, involving the utilization of diverse data sources. Section 3.1.1 elaborates on the update of the Rainfall-Runoff module and 3.1.2 on the update of the flow module. The data is collected based on Databases from the national government, waterboard, municipality, and SOBEK. They are integrated to construct a comprehensive and realistic model. The validity of the model was confirmed through on-site visits, and its accuracy was further assessed through validation against measured data.

3.1.1 Rainfall-Runoff Model

The Rainfall-Runoff model serves the purpose of determining the correlation between rainfall and runoff. Operating as a black box model, it computes the final values of the system. The core model, as expounded upon in Section 2.2.1, was employed to describe the water system. Pertaining specifically to the stormwater model, the pertinent input parameters for the rainfall-runoff module are presented in Table 1. The model requires different types of data input. The meteorological data needs to be implemented in the model using HIS files, this data is the same for all cases. The Paved and unpaved land-use are implemented as nodes. The nodes are displayed in Figure 49 in the Appendix. The red nodes indicate the paved nodes, each paved nodes needs as input: area, sewage pump capacity, overflow location and sewer type. The overflow location is the location of its connection to the surface water system. If the sewage is mixed, the nodes are connected to both the Waste Water Treatment Plant (WWTP indicated with the brown hour glass) and the surface water system. If the system is separated, then the nodes are only connected to the surface water system. The data is collected from the municipality and open sources maps. The green nodes are unpaved nodes, they require as input data for each node: Area, soil type, Groundwater computation area, Land-use division, drainage values, and infiltration capacity. The data is collected from open sources information and elaborates upon the data of the old assessment model.

Data location	**Type**	**Unit**
Meteorological data	Precipitation	mm/h
	Evaporation	mm/d
Paved land-use	Area	ha
	Sewage pump capacity	mm/h
	Overflow location(x,y,z)	(m,m,m)
	Sewer type	mixed, separated
Unpaved land-use	Area	ha
	Soil type	-
	Groundwater area	ha
	Land-use division	ha
	Fast drainage resistance	days
	Slow drainage resistance	days
	Drainage depth	m
	Infiltration	m

Table 1: Relevant input data Rainfall-Runoff module

3.1.1.1 Paved

Paved land use refers to impervious surfaces that discharge into either the sewage system or the surface water system. A series of steps were undertaken to determine the runoff from paved nodes into the surface water system. Firstly, a map indicating impervious areas was obtained. Secondly, the location and areas behind the overflow points were identified. These steps were then combined to define the area behind each overflow. Lastly, the sewer pump capacity was estimated for mixed sewer systems.

The map depicting impervious areas comprises a combination of layers, as illustrated in the diagram shown in Figure 15. Initially, the Dutch General Address Registration (BAG) (Municipality of Ede, 2015) was integrated with the General Topographical Registration (BGT) (Gemeente Ede, 2023). This integration produced a map representing publicly known impervious areas, excluding impervious areas on private property such as gardens. To incorporate this additional information, an infrared aerial view from 2016 was utilized (Kadaster, 2016). The infrared aerial view distinguishes between paved and unpaved areas based on infrared radiation. Different surfaces exhibit varying heat profiles, resulting in the reflection and radiation of different wavelengths. Consequently, a map encompassing all impervious areas was obtained.

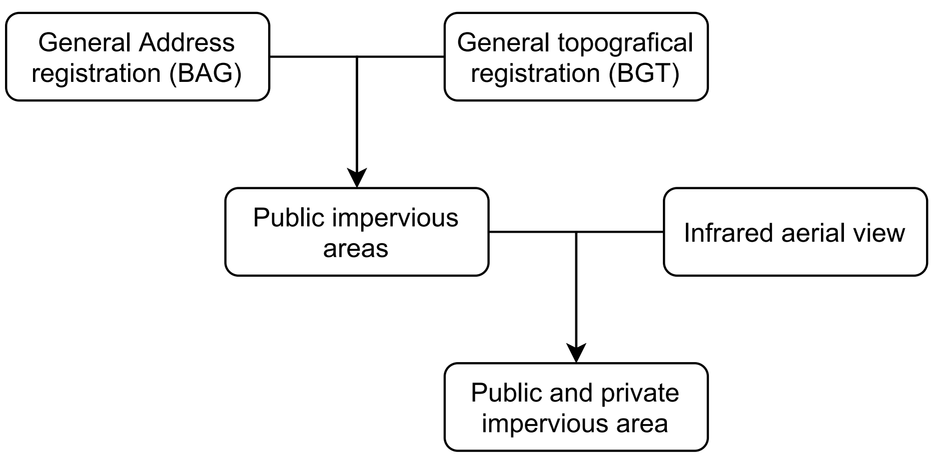


Figure 15: Layer combination for impervious areas

An AutoCAD model, provided by the municipality, was utilized to identify the areas that discharge into the sewer system and determine the corresponding sewage system types. The Rietkampen and Veldhuizen areas primarily consist of mixed sewage systems, while the Frankeneng area features a separated sewer system.

In the Veldhuizen area, which has a mixed sewage system, the overflow locations were obtained from an interactive database provided by the municipality, known as H2gO (Municipality of Ede, 2023). This database contains information on overflow locations, dimensions, and historical data. Each overflow has an upstream catchment area for which it operates. To delineate these catchment areas, the overflow locations from H2gO were merged with the publicly available current elevation database (AHN Nederland, 2023). The elevation map outlines the boundaries of the catchments upstream of each overflow location. The overflow locations, along with their corresponding catchment areas, are illustrated in Figure 16. Situated on a moraine with an elevation difference of 5 meters (AHN Nederland, 2023), Veldhuizen allows for gravity flow towards the wastewater treatment plant (WWTP). The sewage system's capacity is designed to accommodate an upstream area with a precipitation intensity of 0.7 mm/h, which is utilized for all overflows. Since the model focuses on heavy precipitation events, the uncertainty surrounding the pump capacity has a relatively smaller impact on the overflow volume into the surface water system. For precipitation events with a 10-year return period, there is 35.7 mm of rainfall in one hour. Out of this total, 35 mm overflows with a sewer capacity of 0.7 mm/h. In the case of doubling the sewer capacity to 1.4 mm/h, 34.3 mm overflows into the surface water, resulting in a mere 2% decrease (RIONED, 2019).

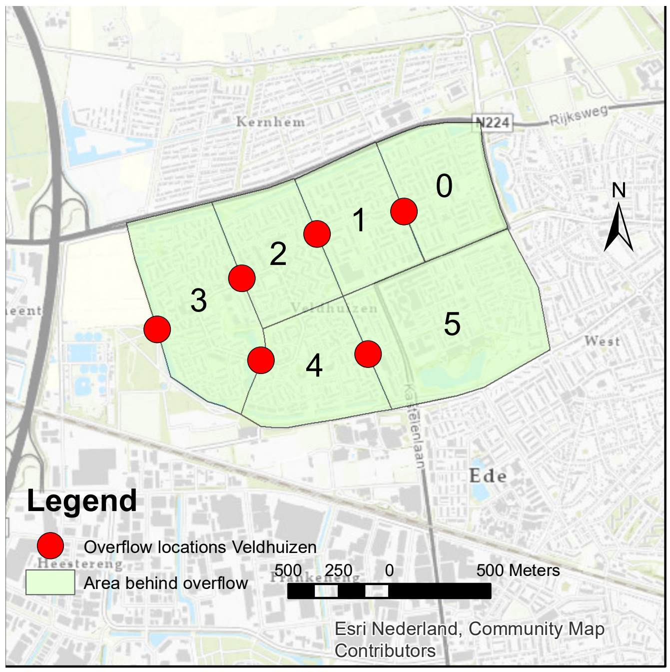


Figure 16: Overflow locations and area behind overflows Veldhuizen

In the Frankeneng area, a separated sewage system is implemented. Wastewater is directly pumped to the wastewater treatment plant and does not have any connection to the surface water system. Conversely, rainwater is collected through a stormwater sewage system and diffusely discharged into the surface water. To simulate the diffuse distribution of water from paved areas into the surface water system, a catchment area is defined for each surface water body. A total of 12 areas represent the influx of rainwater from paved areas into the surface water system. These areas can be observed in Figure 17. Since the separated sewage system does not involve any pumping in the model, pumping capacity is not taken into consideration.

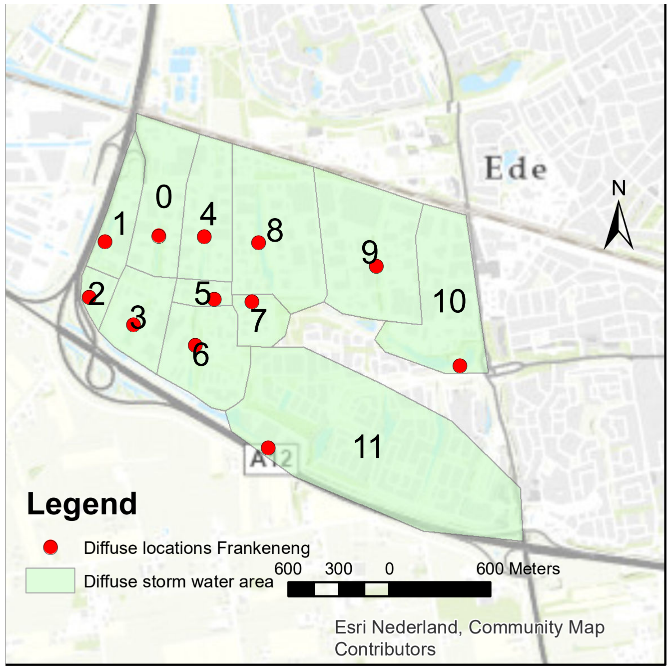


Figure 17: Area distribution and connection to water system of Frankeneng

In the case of Rietkampen, a mixed sewage system is implemented, and sewage water is pumped to the wastewater treatment plant (WWTP) using sewage pumps. The sewage pump has a capacity of 697 m3/h. In the event of overflow, the sewage is discharged into a pond, which subsequently overflows into the surface water system. The entire paved area of Rietkampen is encompassed within the area upstream of the overflow and pump (Municipality of Ede, 2023).

3.1.1.2 Unpaved Area

The unpaved area consists of all permeable land use, which includes all areas not classified as paved. Within the SOBEK model, 16 different land-use types are distinguished for unpaved areas. These land uses were categorized within the old assessment model, which forms the basis of the utilized model. The unpaved areas are obtained using LGN4, a nationally utilized land-use grid developed by Wageningen University and Research (WUR, 2000). The land uses from the land-use grid are converted into the 16 available land-use types in SOBEK and are diffusely integrated into the catchments. This integration process is based on the 2012 assessment. However, it is important to acknowledge the limited reliability of the 2012 data, considering that land use in the Netherlands undergoes continuous changes. Furthermore, the 2012 assessment was conducted on larger catchments, which did not allow for detailed validation of land uses. In order to achieve precise area measurements, it is necessary to account for the paved area by deducting it from the overall catchments and subsequently performing reverse aggregation. The reverse aggregation process involves expressing the division of land use as percentages and then allocating the new total area based on these percentages. By employing the reverse aggregation technique, the distribution of land use is maintained consistently, whereby all land-use categories are adjusted by a factor determined for each catchment. The modifications resulting from this approach are presented in Table 2.

	Old area	**New area**	**Change**
Veldhuizen	197,2 ha	81,7 ha	-58%
Frankeneng/ Rietkampen	290,7 ha	20,2 ha	-93%

Table 2: Unpaved area changes

To determine the soil types, an open-source soil map called 'Bodemkaart' was utilized. These soil types were already incorporated into the old model and are not prone to change (PDOK, 2017). The groundwater system extends beyond the unpaved area alone, as groundwater processes also occur beneath the paved areas. Therefore, the total area considered in the groundwater model encompasses both paved and unpaved nodes. The total area was divided among the different unpaved area nodes based on the area of each node. This approach ensured that the entire groundwater system is taken into account in the model.

The infiltration capacity is consistently set to 100 mm/h throughout the model. While this value may overestimate the actual infiltration capacity in certain nodes, reducing it may lead to the exclusion of the groundwater system from the model. Neglecting the groundwater system during calibration would focus solely on surface water runoff. However, when validation is performed using different precipitation events, the results can differ significantly. During prolonged droughts, the infiltration capacity decreases, but the occurrence of heavy precipitation events following a dry period is highly improbable (Stowa, 2019).

The drainage resistance is a crucial factor in groundwater modeling. The Ernst Drainage method is employed to simulate soil drainage in the model. The underlying equations of this method are included in the SOBEK model and are beyond the scope of this research (Ernst, 1978). The Ernst method utilizes two layers: a surface layer with fast drainage and a subsurface layer with slower drainage, as depicted in Figure 18. The depth at which the transition from fast to slow drainage occurs, known as the drainage depth, was obtained from the Alterra map collection produced by Wageningen Environmental Research (Alterra, 2015). During the calibration process, the drainage resistance values and drainage depth were adjusted to achieve a good fit with the measured data. The calibration input for the drainage resistance values can be observed in Table 3. Four values were calibrated for both fast and slow drainage resistances, with the calibration range based on the original model's calibration conducted by the consultancy firm HKV. As for the drainage depth, two values were considered in the calibration process. The first value of 1.20 meters for Ede was derived from a map that combines rules of thumb with area information, the second value of 0.8 meters for Ede is obtained from a nationwide agricultural assessment (Alterra, 2015). All possible combinations of these values (32 cases) are run in the model under identical conditions to determine the best fit.

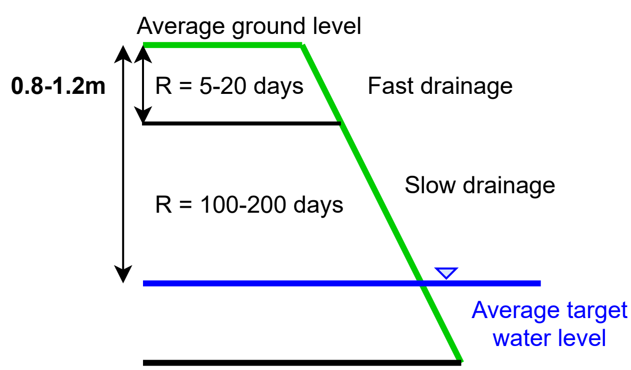


Figure 18: Schematic Ernst drainage resistance model (Ernst, 1978)

Fast drainage resistance [days]	Slow drainage resistance [days]	Drainage depth [meters]
5	100	80
10	125	1,2
15	150	
20	200	

Table 3: Parameter values for drainage resistance calibration

3.1.1.3 Meteorological data

Precipitation and Evaporation

Precipitation serves as the primary input to the water system. However, given that the model is based on the 2012 assessment, the precipitation data necessitates correction and elaboration to ensure its accuracy and relevance. The Royal Dutch Meteorological Institute (KNMI) plays a crucial role in monitoring precipitation patterns through a combination of automatic and voluntary-based measurements, thereby providing detailed insights into precipitation patterns at the neighborhood level (KNMI, 2023b). To enhance the accuracy of the measurements, KNMI employs radar data to correct the precipitation data, enabling precise measurements at a neighborhood level. This corrected data is integrated into the data information system of the waterboard, known as FEWS. Subsequently, the data was exported and adjusted to align with the requirements of the SOBEK model, with a temporal resolution of 5 minutes (KNMI, 2023a).

For calibration and validation purposes, precipitation events were chosen from January 2018 until April 2023, as discharge data from the automatic weir is available during this time interval. The focus of the project is the performance during heavy precipitation events. The events were selected based on both cumulative and peak discharge criteria, with a minimum of 150 days between events to ensure separate events and account for the duration of groundwater computations.

Evaporation is another influential natural factor that affects the discharges and storage within the model. Evaporation data is obtained from various measurement stations operated by the KNMI. Similar to precipitation data, the evaporation data is integrated into the FEWS system and undergoes radar correction for different neighborhoods. Daily evaporation values exhibit seasonal variations, with maximum rates occurring during summer (approximately 4 mm per day) and minimum rates during winter (approaching 0 mm). The evaporation data is available at 24-hour intervals (KNMI, 2023b, 2023a).

Terrain
Terrain data is included in specific nodes. The paved and unpaved nodes provide information on land use and average surface levels. Cross-sections depict the levels and dimensions of the surface water system. Elevation data is included in the sub-catchments for each neighborhood.

3.1.2 1D Flow Model

The Rural 1D Flow module within SOBEK is utilized to simulate and calculate one-dimensional flow in wastewater and stormwater systems. This module, known as SOBEK-rural 1DFLOW, is a sophisticated tool that performs well for area drainage, sewer overflow frequencies, and storage assessments (Appsolutelydigital, 2013; Deltares, 2013; Betrie et al., 2011). The decision to use the 1D version of the module is primarily driven by the need to reduce computation time, as the flows in the urban water system mainly occur within channels and culverts, while overland flow is not considered within the scope of the model.

The flow module relies on various input parameters, as outlined in Table 4. Weirs and culverts are among the most significant and abundant structures within the urban water system. The structure data was updated accordingly to ensure an accurate representation. Additionally, the pond profiles were updated to capture the flow characteristics within the system. All parts of the flow model were updated by utilizing the register data from the waterboard (waterschap vallei en veluwe, 2023).

Data location	Type	Unit
Cross-section	Profile (x,z)	(m,m)
	Level	m +NAP
	Friction (Chezy)	$m^1/2s^-1$
Culvert	Length	m
	Upstream height (z)	m +NAP
	Downstream height (z)	m +NAP
	Cross-sectional dimensions	øm
	Location (x,y)	(m,m)
Weir	Crest level (z)	m +NAP
	Crest width	m +NAP
	Location (x,y)	m +NAP

Table 4: Relevant input SOBEK Urban 1D flow module

3.1.2.1 Structure data

The structure data used in the model was updated based on the "Legger water system" register maintained by the waterboard (waterschap vallei en veluwe, 2023). This register contains essential information such as geographical data, dimensions, roughness coefficients, and accompanying pictures of the structures. It is crucial to critically evaluate the accuracy and reliability of the register data, and the site pictures are utilized to ensure the correct interpretation of the information. It should be noted that the register data is subject to incremental updates, as the information available in 2012 may differ from the current data due to changes in the water system over time and advancements in measurement techniques.

Culverts are among the prominent structures in the urban water system, serving as conduits to connect surface water bodies and facilitate integration within the interconnected water system. In the study area, all culverts are constructed using concrete materials, and their roughness coefficient is estimated at 75 ks based on Stickler's friction formula (Fu, 2012). The dimensions of the culverts have been updated to accurately reflect their specifications. Table 5 presents the most relevant parameters for the culverts. It is important to note that the location parameter in the model represents a single point in the middle of the culverts, as the SOBEK model incorporates additional resistance at specific points. For longer culverts, the structure node is replaced with a profile to ensure a more precise simulation of storage within the model.

Parameter	**unit**	**Measurement**
Inlet height	m +NAP	GPS z-value
Outlet height	m +NAP	GPS z-value
Length	m	GPS difference (x,y)-value
Diameter	m	GPS difference z-value

Table 5: Culvert important parameters

Weirs play a pivotal role in the control of water levels and regulation of flows within the urban water system. In urban settings, rigid rectangular weirs are commonly employed, serving to store water in urban ponds and facilitate overflow during and after heavy precipitation events. The key parameters for rectangular weirs, including crest width, crest height, and location, have been updated in the model to accurately represent the characteristics of the weirs.

Both catchments in the study area are bounded by an automatic weir. These weirs feature a dynamic crest height that can be adjusted to regulate water levels and discharge. However, automatic weirs are relatively costly and not found abundantly within the urban system. The weirs are controlled using a Proportional-Integral-Derivative (PID) controller, as described in the literature review and Figure 8. The TMX remote control system is utilized to acquire information about automatic weirs. The remote system operates based on a target water level, which may vary seasonally or in response to extreme weather conditions. Additionally, a maximum discharge limit can be implemented to mitigate downstream impacts. The on-site management team in Barneveld oversees the input conditions for controlling the weirs (waterschap vallei& veluwe, 2013).

To accurately simulate the behavior of the automatic weirs in the SOBEK model, the data from the remote system is converted into PID input parameters. The PID controller requires information such as the initial crest level, target crest level, crest width, and the minimum and maximum crest levels. Moreover, integral parameters are calibrated to achieve a good fit with the measured data and minimize oscillations.

The automatic weir that bounds the 'Frankeneng' and 'Rietkampen' systems can be observed in Figure 19. It has a crest width of 2 meters, a target crest level of 6.65 meters, a minimum level of 6.0 meters +NAP, and a maximum level of 7.0 meters +NAP. The proportional gain is set to 1, the integral gain is 0.005, and the differential gain is 1. The relatively low value of the integral gain helps reduce oscillations that may occur in certain scenarios.

(a) Weir inlet

(b) Weir top view

Figure 19: Frankeneng automatic weir pictures from site visit

The automatic weir governing the Veldhuizen system is presented in Figure 20. It possesses a crest width of 1.8 meters, a target crest level of 8.20 meters, a minimum level of 8.06 meters +NAP, and a maximum level of 8.66 meters +NAP. The proportional gain is established at 1, the integral gain is set to 0.01, and the differential gain is assigned a value of 0. In this particular case, the weir does not exhibit vulnerability to oscillations, hence the integral gain is adjusted to a higher value to facilitate a swifter response to variations in water levels.

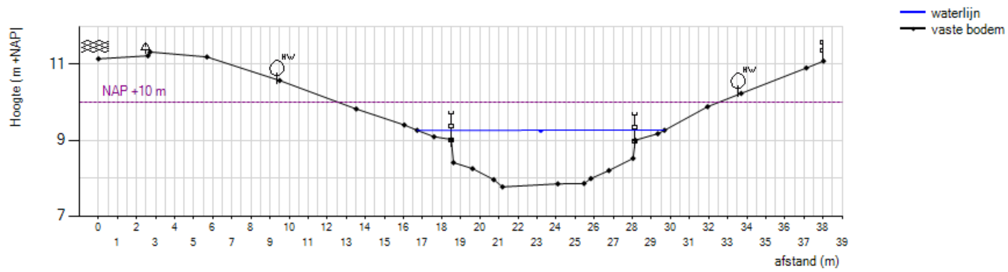
(a) Picture from legger (waterschap vallei en veluwe, 2023)

(b) Picture from site visit

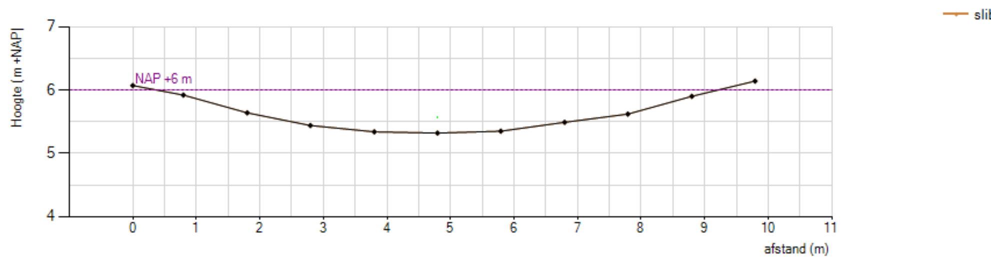
Figure 20: Veldhuizen automatic weir pictures

3.1.2.2 Cross section updates

In order to enhance the reliability of the model output and achieve a more accurate simulation of the storage capacity within the water system, additional field measurements were conducted in the years 2014 and 2015 for the examined catchments. These measurements aimed to capture the characteristics of the channel beds. The bed frictions of the profiles are typically assumed to have a Chézy coefficient of 30 (Megawaty, Susanto, Suryadi, & Ngudiantoro, 2012). A GPS gauging device was utilized during the field measurements to obtain precise x, y, and z coordinates for each measurement point. By combining these coordinates, cross-sectional profiles of the water system bed at specific locations were established, as depicted in Figure 21. Each cross-section represents the profile of the water system bed at a distinct point along the watercourse.



(a) New cross-section profiles (2014)



(b) Old cross-section profiles (2008)

Figure 21: Old and new profiles of cross-sections (waterschap vallei en veluwe, 2023)

The newly acquired cross-sections offer more detailed and comprehensive profiles compared to the previous ones, which only depicted the channel bed without including the sides. To create a comprehensive representation of the integrated water bed, the newly obtained cross-sections were interpolated. However, it was not possible to apply interpolation in the cross-sections that serve as culverts since they do not function as open channels. In order to account for the reduced storage capacity in these areas, the connection nodes between the cross-sections of culverts and open water were designated as 'no interpolation'. This approach ensures that the model accurately represents the flow and storage characteristics within the water system, considering the specific configuration of culverts and their impact on the overall system dynamics.

ArcGIS Pro was utilized as the software tool for extracting cross-sectional profiles from the register data. The cross-section database was imported into ArcGIS, and the catchments of interest were isolated and converted into shapefiles. Microsoft Excel and Access tools were employed to convert the data into DEF (Digital Elevation Format), DAT (Data Table), and BNA (Boundary File) files, which served as input for the SOBEK model, incorporating the new cross-sections. The DEF file described the cross-sections using GPS data points, the DAT file specified the assignment of profiles to each node along with bed and maximum water level information, and the BNA file positioned each cross-section with its corresponding x and y coordinates. The DEF and DAT files were added to the background files of SOBEK, while the BNA file was imported into the SOBEK interface. To validate the new cross-sections, the 'side view' tool was employed, which provided visual representations of the bed and maximum water levels along the length of the water system. It is noteworthy that the 'cross-section tool' was not compatible with the new version of SOBEK, necessitating the use of SOBEK 21204, which was subsequently converted back to SOBEK 216 for further analysis.

3.1.3 Calibration

The calibration process involves the utilization of automatic weirs, which were previously described in section 2.2.3, as the boundaries of each catchment. These weirs are remotely controlled and rely on water level measurements obtained from upstream and downstream water level sensors. Since the crest level of the weirs is controlled and known, and the width of the weirs is rigid and predetermined, the discharge over the weirs can be calculated using Formula 1, under the assumption that the weirs are not drowned.

The control of the weirs is facilitated through the remote control program TMX. This program performs real-time calculations of the discharges over the weirs and stores the resulting data in a continuous discharge database. The discharge data calculated by the program is then incorporated into the water information system of the waterboard, known as FEWS, which serves as a platform for analyzing and extracting the data series obtained from the TMX discharge database.

The calibration process involves comparing multiple data series using a MATLAB script provided by MathWork. This script enables the evaluation of different calibration scenarios and assesses their fitness. Two indicators, namely the Nash–Sutcliffe model Efficiency coefficient (NSE) and the Root Mean Square Error (RMSE), are employed to gain insights into the performance of each calibration scenario (Motovilov, Gottschalk, Engeland, & Rodhe, 1999).

The NSE is a metric specifically designed for assessing the predictive capability of hydrological models. It is calculated using Equation 7, where Q_o^t represents the observed discharge at time t, Q_m^t represents the modeled discharge at time t, and $\bar{Q_m}$ denotes the mean of the observed discharge (Jain & Sudheer, 2008; Zeybek, n.d.).

$$NSE = 1 - \frac{\sum_{t=1}^{T}(Q_o^t - Q_m^t)^2}{\sum_{t=1}^{T}(Q_o^t - \bar{Q_m})^2} \quad (Jain \ \&Sudheer, 2008) \tag{7}$$

The NSE formula calculates the squared differences between observed and modeled discharges and normalizes them using the mean of the observed discharge. This normalization allows for a distinction between dynamic and non-dynamic systems. The resulting NSE value ranges between 1 (indicating a perfect fit) and $-\infty$. A value of 0 suggests that the mean value of the observed discharge would describe the system better than the model. However, interpreting NSE scores can be challenging, so a general statistical error test is applied to validate the NSE values (Zeybek, n.d.; Jain & Sudheer, 2008). In addition to the NSE, the RMSE is used as a complementary indicator to assess the correlation of the data series. Although not specifically designed for hydrological models, the RMSE provides insights into the differences between observed and modeled discharges. It is calculated using Equation 8, where n represents the number of data points, Q_o^t is the observed discharge at time t, and Q_m^t is the modeled discharge at time t (Motovilov et al., 1999).

$$RMSE = \sqrt{\frac{\sum_{i=1}^{n}(Q_o^t - Q_m^t)^2}{n}} \quad (Motovilov \ et al., 1999) \tag{8}$$

The RMSE value is obtained by taking the square root of the sum of squared differences divided by the number of data points. It serves as a validation tool for the NSE scores and helps assess the correlation of the data series. The values which are calibrated are displayed in Table 3. The discharge and cumulative discharge were calibrated to ensure a proper water balance and corresponding ground and surface water systems.

3.1.4 Sensitivity of parameters

The model incorporates a comprehensive set of parameters sourced from literature, authoritative references, and calibration. Uncertainties are introduced into the model to account for variations in data quality and reliability. To assess the influence of these uncertainties on the model output, a sensitivity analysis is conducted. During this analysis, parameters are systematically adjusted by a specified percentage, considering that different parameters exhibit varying levels of estimated uncertainty. The uncertainties are classified based on the type of data, which includes calculated, exact, estimated, calibrated, single measurement, and measured data series. The following classification and associated uncertainties are considered:

1. Calculated data: This category encompasses spatial information obtained from validated governmental references. The maximum uncertainty attributed to calculated data is estimated to be 10%, accounting for potential variations in land use.

2. Exact data: This data type represents parameters with very low uncertainty or non-numerical values. No changes are made to these parameters during the sensitivity analysis, as doing so would inaccurately represent the system.

3. Estimated data: Estimated data is derived from literature or expert knowledge within the waterboard, often involving rules of thumb or simplifications, which introduce larger uncertainties. An uncertainty of 40% is assigned to the estimated data.

4. Calibrated data: Calibrated data is obtained through the calibration process, which involves fitting the model to a specific rainfall event. Overfitting or underfitting of the data can result in larger uncertainties. A 40% uncertainty is considered for calibrated data.

5. Single measurement data: This data type originates from field measurements using precise gauging devices. However, uncertainties can arise due to human errors during the measurement process. A 10% uncertainty is assigned to single measurement data.

6. Measured data series: Measured data series are obtained from gauged and validated sources, with radar correction applied to ensure reliable values.

Each parameter is classified according to the aforementioned categories, and positive and negative uncertainties are assigned accordingly. The sensitivity analysis is then performed, taking into account the different classifications and associated uncertainty values assigned to the parameters. For more detailed information on the parameter classification and uncertainties, refer to Table 6.

Model location	**Type**	**Data type**	**Uncertainty**
Meteorological data	Precipitation	Measured data serie	-
	Evaporation	Measured data serie	-
Paved land-use	Area	Calculated	+/-10%
	Sewage pump capacity	Estimated	+/-40%
	Overflow location	Exact	-
	Sewer type	Exact	-
Unpaved land-use	Area	Calculated	+/-10%
	Soil type	Exact	-
	Ground water area	Calculated	+/-10%
	Land-use division	Exact	-
	Fast drainage resistance	Calibrated	+/-40%
	Slow drainage resistance	Calibrated	+/-40%
	Drainage depth	Calibrated	+/-40%
	Infiltration	Estimated	-40%
Cross-section	Profile (x,z)	Exact	-
	Level	Single measurement	+/-10%
	Friction (Chezy)	Estimated	+/-40%
Culvert	Length	Single measurement	+-10%
	Upstream height (z)	Single measurement	+/-10%
	Downstream height (z)	Single measurement	+/-10%
	Cross-sectional dimensions	Single measurement	+/-10%
	Location (x,y)	Exact	-
Weir	Crest level (z)	Single measurement	+/-10%
	Crest width	Single measurement	+/-10%
	Location (x,y)	Exact	-

Table 6: Classification and uncertainty per relevant parameter

3.2 Validation

Validation is a crucial step in ensuring the reliability and accuracy of the model. The process consists of multiple stages, each addressing different aspects of the model's performance. The initial step was verification, which involves a qualitative assessment to examine the content and structure of the model. This assessment compares the model against databases and existing knowledge to identify any discrepancies or oversimplifications. Subsequently, the model undergoes validation using a precipitation event that differs from the one used for calibration. This approach ensures that the calibration process is based on the underlying hydrological models rather than solely relying on parameter fitting. By subjecting the model to a validation event, its robustness is tested by pushing it beyond the calibrated boundaries, thereby increasing its reliability.

The verification step can be further subdivided into two stages. First, the simulated model is evaluated to determine its correspondence with the real-life system. This evaluation assesses how well the model captures the characteristics and behavior of the actual system. The input data for the model already incorporates a diverse range of verified inputs, including GeoWeb data provided by the waterboard and additional databases from the municipality, which form a reliable background model. However, the dynamic nature of the water system presents challenges in maintaining up-to-date data. To validate the background data, pictures stored in GeoWeb are utilized, and a site visit conducted in collaboration with supervisor Dimitri van Dam provides insights into the remaining uncertainties in the simulated system. The site visit involves a comprehensive assessment of the overall system and specific bottlenecks. Subsequently, side views and flow directions are verified to lay bare structural problems within the model.

Once the model has undergone verification and calibration, it proceeds to the validation stage to assess its robustness. Calibration aims to fit the data for a specific event, but the complexity of the model may result in overfitting to that particular event rather than accurately calibrating the underlying hydrological parameters. Therefore, validation is performed using a different precipitation event to evaluate the effectiveness of the hydrological processes captured by the calibration. If the model outcomes align with the measured outcomes during validation, it indicates that the hydrological model has been appropriately incorporated. Conversely, if the modeled outcomes deviate from the measured outcomes, it suggests inadequacies in representing the hydrological model, with parameters fitted solely for the specific event. Such a scenario undermines the robustness and reliability of the model when simulating other precipitation events or system changes. The fitness of the model is evaluated using both the Nash-Sutcliffe Efficiency coefficient (NSE) and visual inspection of the analysis to ensure accuracy and reliability (Zeybek, n.d.; Motovilov et al., 1999).

3.3 Identifying system problem

To assess the under-utilization of the storage capacity in the system, a comprehensive system analysis is conducted. This analysis aims to identify the root causes of the problem and explore potential sub-problems contributing to the under-utilization. By examining the entire system, both at a macro and micro level, a holistic understanding of the issue can be obtained. The first step in the analysis is to identify and allocate the problem areas within the system. This involves pinpointing the specific components or sections that are not performing according to the design norms. This allocation process provides valuable insights into both the causes of underutilization and potential starting points for addressing the issue. Furthermore, the actual utilization of the storage capacity is assessed. While the storage may be theoretically available, it is important to evaluate whether it is being utilized in practice. Furthermore, the analysis investigates the discharge and emptying of the system. If the system is constantly operating at full capacity, the effective storage capacity is significantly reduced. Therefore, it is crucial to examine the factors influencing the discharge and emptying of the system. To identify the potential origins of the under-utilization problem, hydrological experts from the waterboard, an area manager, and a hydrological expert from the municipality are consulted. Their collective expertise and knowledge provide valuable insights into the system dynamics and potential factors contributing to the problem.

3.3.1 Problem allocation

Determining the key bottlenecks within the system is crucial for understanding the underlying causes of the capacity problem. In this study, the calibrated and validated SOBEK model is utilized to gain insights into the system dynamics. A relevant indicator used for evaluating the bottlenecks is the discharge per upstream connected area, also known as the relative discharge. This indicator is incorporated in the current waterboard policy, which sets an indicative norm of 3 l/s/ha for urban areas based on the 24-hour average of a T=100 event (waterschap vallei& veluwe, 2013).

The sub-catchments and ponds within the system are equipped with weirs that regulate the flow. The relative discharge at each of these weirs was evaluated to identify areas of concern. To determine the upstream area associated with each weir, a Matlab script is employed. The areas of both paved and unpaved nodes are assigned to the respective weirs. For simulating a T=100 event, a design precipitation event based on research of Stowa, HKV and KNMI (Stowa, 2019) was incorporated into the precipitation series. The simulation period extends beyond the duration of the event to ensure proper filling of the surface water system and generation of realistic groundwater values. The 24-hour average discharge was calculated for each weir, and the relative discharge was computed by dividing the average discharge by the upstream paved and unpaved area.

A map which can be seen in the results section 4.2.3 was produced using ArcMap 10.8.1 by ESRI for visual analysis of the system bottlenecks. The location data of the weirs was exported from the SOBEK model and converted to the appropriate GEO-reference format. The relative discharge and absolute discharge of each weir were included in the analysis. Different symbologies were utilized to represent the data visually. The absolute discharge is incorporated as the size element, while the relative discharge was displayed using a color ramp. Furthermore, the labels on the map provide information on the relative discharge. This map serves as a valuable tool for identifying neighborhood-specific issues and assessing the impact of the bottlenecks within the system.

3.3.2 Current storage utilization

The water system is a combination of ponds, culverts, and weirs. The weirs are the most important factor in regulating the water level. The storage can be available theoretically while the weirs only keep the water to a specific height. To assess this, side views and free board means are used. The freeboard mean is the difference between the water level and the ground level, this is displayed in Figure 22. This level indicates how much higher the water can get before flooding occurs. Samples are taken from the water system, higher values indicate more possibilities for storage. Additionally, the trends in the free board values indicate which factors are influential for water height and storage. A constant general level would indicate that the ponds are fully regulated by the weirs.

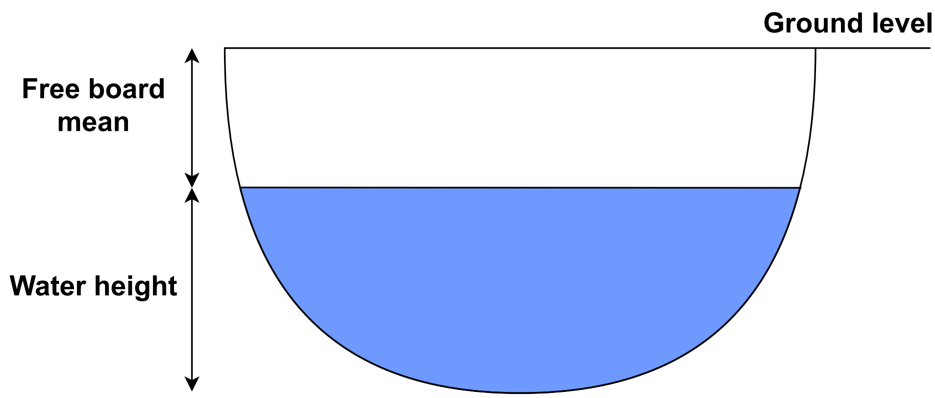


Figure 22: Free board mean definition

3.3.3 Emptying of the water system

Another potential cause of storage underutilization is the discharge of stored water. If the actual storage capacity is consistently full, it may result in the inability to retain heavy precipitation events, leading to continuous discharge. To assess this aspect, the water levels are compared to the bed levels within the system. Well-functioning storage systems should exhibit a peak in water height during and after precipitation events. Subsequently, the water height should gradually decrease and return to the pre-event level before the occurrence of the next event. Additionally, the water levels should normalize within 24-48 hours to maintain groundwater levels. The ponds within the system also serve the purpose of lowering the water level to ensure the dry condition of basements. Currently, the ponds are over-dimensioned due to the implementation of additional pipe drainage at a later stage. Consequently, the water height in the ponds can be higher than the current levels. The municipality has established a 24-hour norm, and a hydrologist from the waterboard has conducted research on the relationship between surface and groundwater levels, concluding that emptying the system within 48 hours does not significantly impact the groundwater system. The assessment of water heights in ponds for both the Frankeneng and Veldhuizen systems is performed to identify potential issues within the system.

To validate the results, measured data from the municipality's available monitoring points, which record surface water levels, is utilized. These measurements provide a valuable tool for validating the model and verifying the hypothesis that the water system is not adequately emptied.

3.4 Modeling Interventions

To address the issue of storage underutilization, various potential interventions are identified, modeled, and evaluated. These interventions are based on expert consultations with professionals from the waterboard and municipality who possess hydrological expertise or practical knowledge of the specific urban areas. This knowledge is combined with the conclusions drawn from the site visit and the iterative modeling process. Throughout these activities, it becomes evident that the dimensions and shapes of weirs play a critical role in the storage capacity upstream of the weirs. Moreover, more radical interventions, such as constructing additional ponds, are not feasible due to constraints in urban planning, budget limitations, and conflicts with the Dutch ambition to build 900,000 houses by 2030 (Ministry of the Interior and Kingdom Relations, 2022). By ruling out alternative solutions, modifying the weirs emerges as the most viable and cost-effective measure. Most weirs are situated within manholes, which presents an additional challenge. If new weirs are to be installed, the culvert system must be replaced, which entails similar construction issues as those encountered in the construction of additional ponds. Therefore, the remaining option is to modify the height and shape of the weirs, given that the width is constrained by the dimensions of the manhole. Figure 5 illustrates a concrete manhole structure with a rectangular weir, where the yellow section represents the fixed concrete structure and the blue section represents the weir.

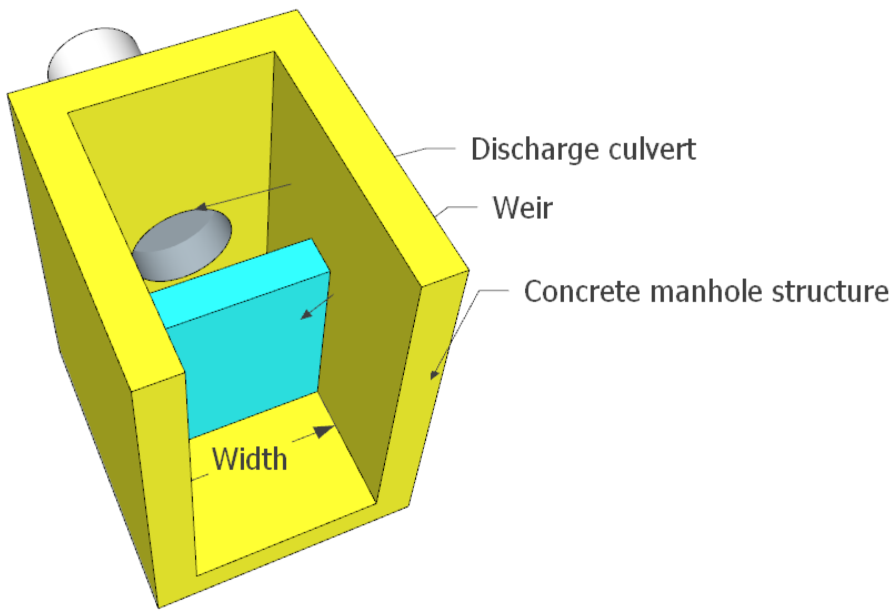


Figure 23: Rectangular weir in concrete manhole

3.4.1 V-notch weirs

V-notch weirs are selected due to their ability to drain higher discharges for higher water heights. During peak events, the discharge should be greater than during non-peak events. If the discharge is insufficient, the v-notch may overflow, leading to suboptimal utilization of the storage capacity. It is important to consider the width limitation of the concrete structure. Figure 24 illustrates how the proposed V-notch weirs are integrated into the system.

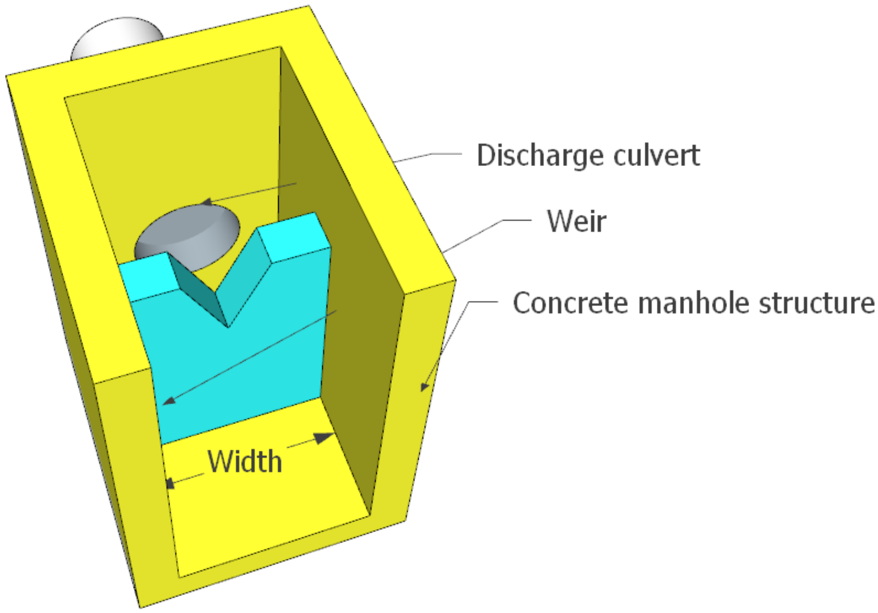
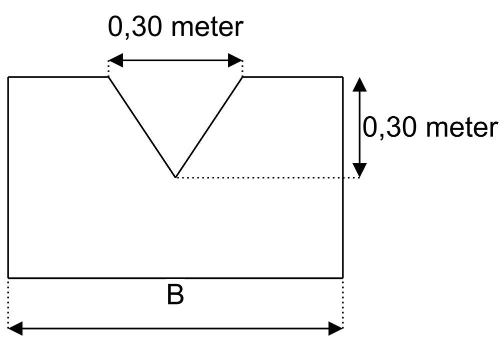
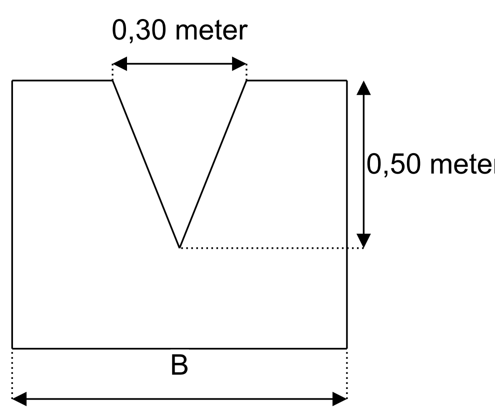


Figure 24: V-notch weir in concrete manhole

The width of the V-notch top is standardized at 0.30 meters to ensure compatibility with all weirs. Various heights are modeled to evaluate their effects on discharge rates and upstream water levels. Specifically, heights of 0.30 meters and 0.50 meters are compared to the reference rectangular weir. Figure 25 depicts the designs of the V-notch weirs.



(a) V-notch weir with a 0,3 meters height



(b) V-notch weir with a 0,5 meters height

Figure 25: V-notch weir interventions

Results

This section presents the results obtained through the steps outlined in the methodology. The results are accompanied by detailed descriptions and explanations to enhance comprehension. The quality and interpretation of these results will be discussed in the subsequent section. First, section 4.1 expounds on the results of the model update. For each update the change in data input and the change in results are displayed. Subsection 4.1.3.2 gives an overview of the model performance of each update step. Section 4.1.4 displays the outcomes of the sensitivity analysis, these results can be used to assess the model reliability. Subsequently, the validation results are displayed in section 4.1.5, these results are used to assess the robustness of the updated model. Section 4.2 displays the results of the hypothesis regarding the research problem. The results section is concluded with the results from the proposed interventions, which can be found in section 4.3. Following these steps displays all results of the research as described in the methodology in section 3.

4.1 Model Update

4.1.1 Meteorological Data

The meteorological data was updated using the integrated KNMI databases in the FEWS. It should be noted that the radar-corrected database was initiated in 2019, and a new measuring program has been implemented since then. However, the integration of these two databases is still in progress. Figures 26 and 27 illustrate the calibration and validation events, showcasing the complete event timeline of the Rainfall-Runoff (RR) and Flow modules. The plot provides a visualization of cumulative precipitation, indicating the volume of water entering the RR and Flow modules. It is worth mentioning that the Rainfall-Runoff module has a longer duration compared to the Flow module. This extended duration allows for the filling of the water system and the generation of realistic groundwater levels. The flow module for the calibration event started on 08-12-2022, while the validation event started on 20-02-2020.

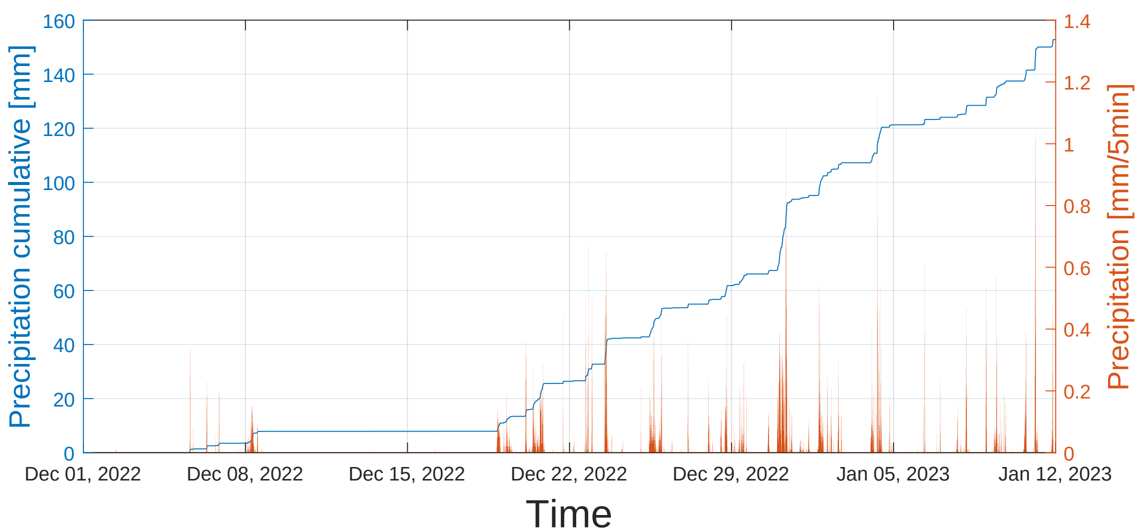


Figure 26: Calibration event

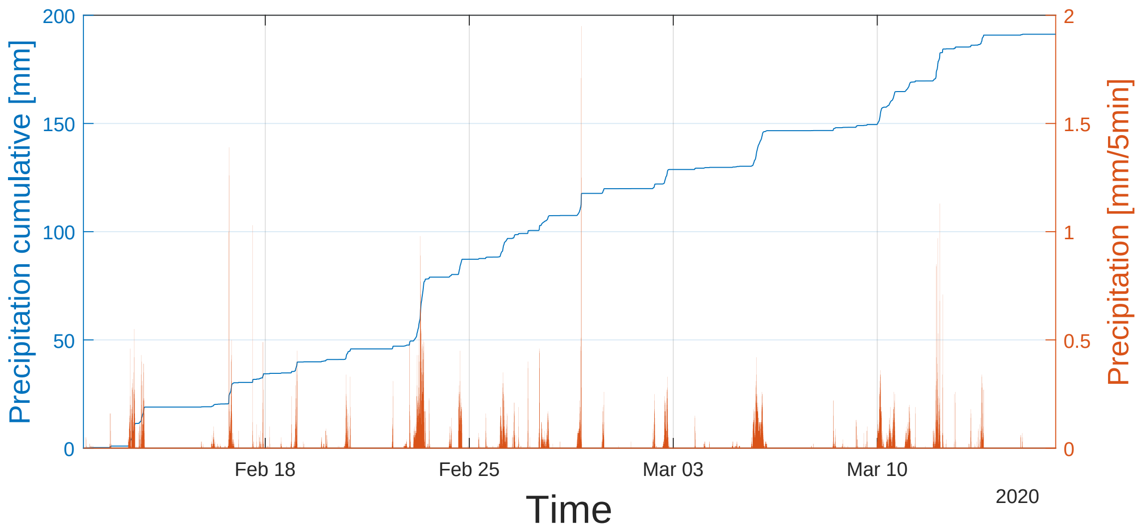
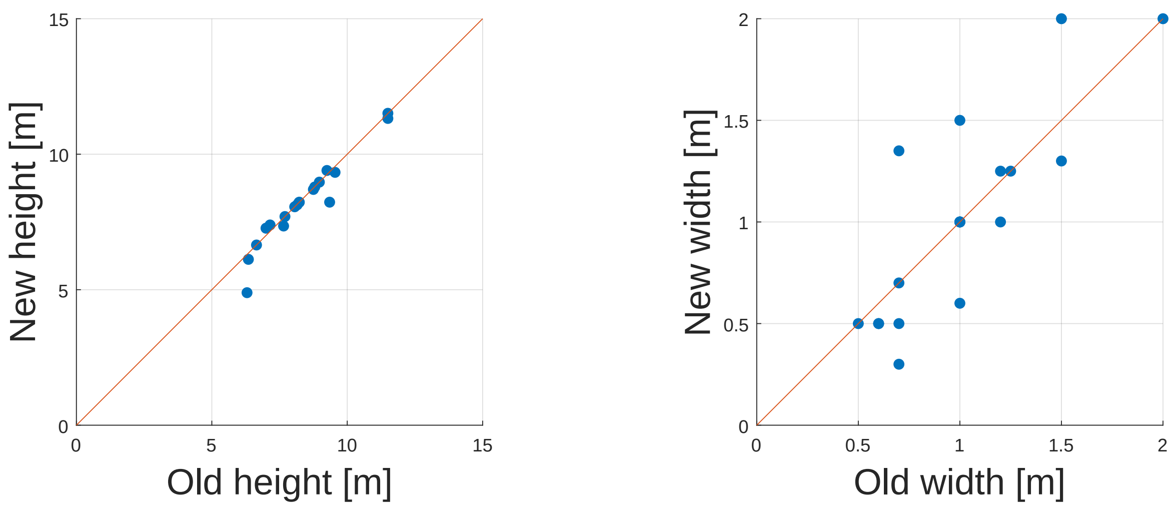


Figure 27: Validation cum

4.1.2 1D flow model

4.1.2.1 Structure data

The structure results entail two parts, the changes in input and the changes in results. The section aims to identify influential factors that improve the model outcome significantly. The model is updated according to the newly available data and verified with the site visit and site pictures. In Figure 28 the weir crest height and crest width can be seen. The crest height has small fluctuations between the old and the new model. The crest width fluctuates more dramatically, this can be caused by a higher accuracy of measurements or a different definition for crest width. The new model contains both lower and higher values, no systematical change can be seen.



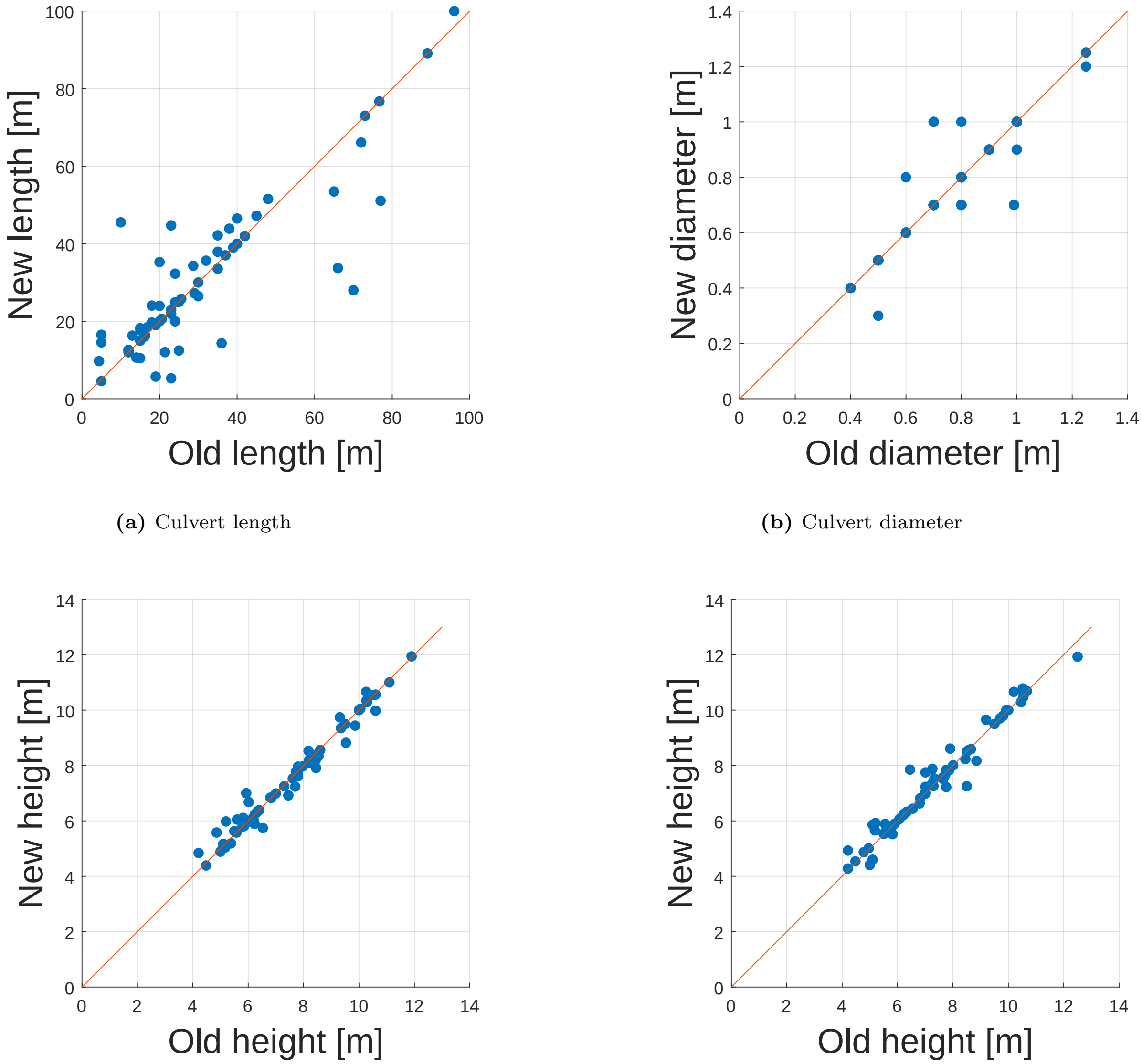
(a) Weir crest height

(b) Weir crest width

Figure 28: Changes in weir dimensions from old NBW to the updated model

The analysis of the structure update results consists of two components: changes in input and changes in results. This section aims to identify influential factors that significantly enhance the model's performance. The model is updated based on the newly available data and verified through the site visit and accompanying site pictures. Figure 28 illustrates the variations in weir crest height and crest width. The crest height shows minor fluctuations between the old and new model, suggesting a relatively consistent representation. On the other hand, the crest width exhibits more pronounced fluctuations, which could be attributed to improved measurement accuracy or a different definition of crest width. The new model encompasses both lower and higher values, indicating no systematic alteration in crest width.

Changes in the input data for culverts include modifications in length, upstream height, downstream height, and diameter. The corresponding results are depicted in Figure 29. Among these parameters, culvert length exhibits the most significant change. This can be attributed to several factors, including the difficulty in measuring culverts located below the water level and the potential evolution of measurement definitions over time. It is worth noting that some culverts in the system feature curved sections, which are enclosed by manholes. Different definitions may exist regarding culvert length, with some considering it as the distance between two manholes and others encompassing the entire length of the culvert from one open water body to another. Discrepancies in culverts arise based on their unique IDs, where multiple sections with different lengths may share the same ID.



(a) Culvert length

(b) Culvert diameter

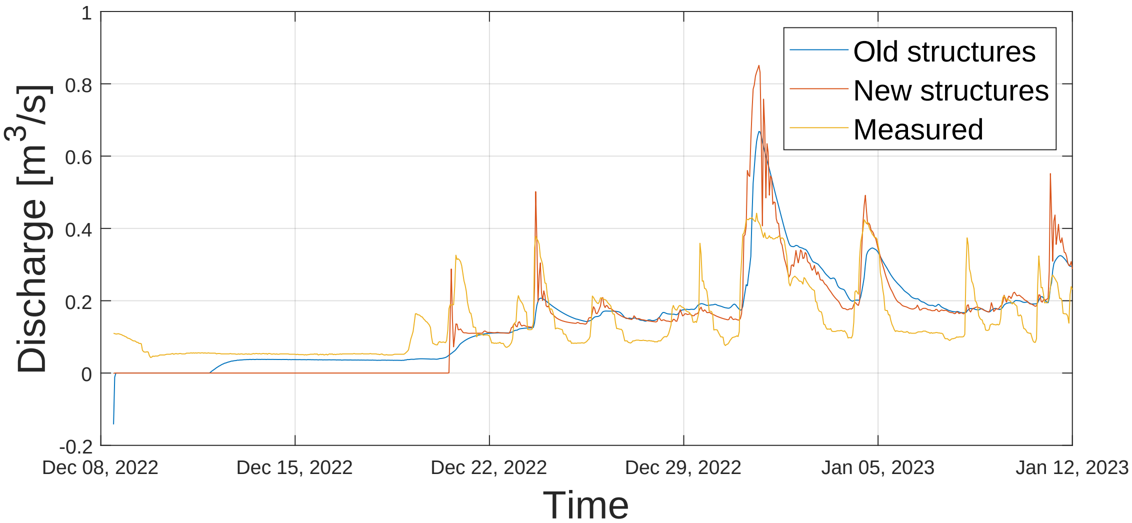
(c) Culvert upstream bottom level

(d) Culvert downstream bottom level

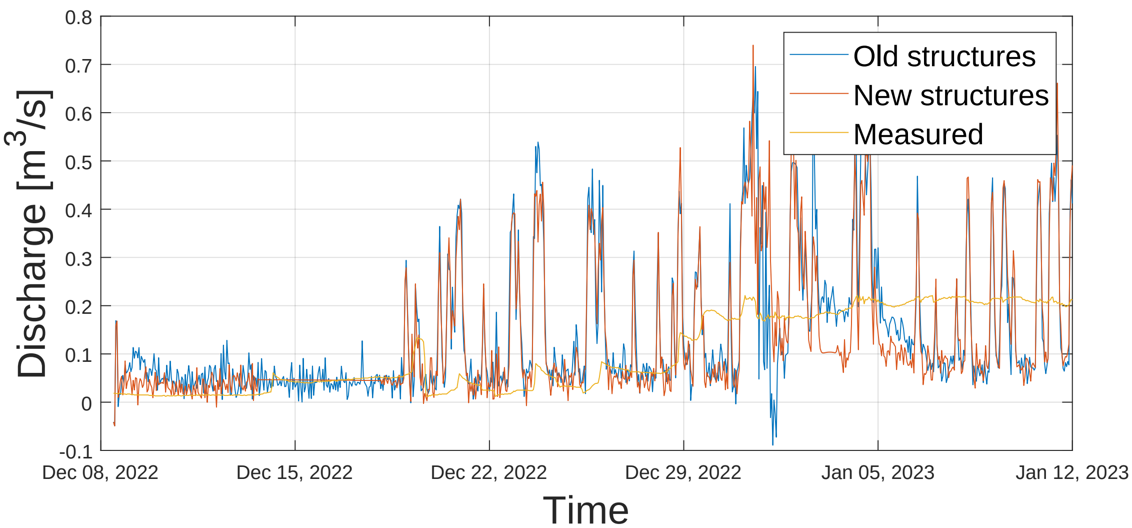
Figure 29: Changes in culvert dimensions from old NBW to the updated model

The analysis of inlet and outlet heights (i.e., upstream and downstream heights) reveals fewer alterations compared to other culvert parameters. The variations in height are relatively minor, as significant changes in geographical location may only result in slight adjustments in height. Height plays a crucial role in determining the storage capacity of different ponds within the system. On the other hand, the diameter exhibits more substantial fluctuations. This unexpected variability can be attributed to system changes over time. While most culverts adhere to standard sizes, it is possible that the system itself has undergone modifications. Changes in culvert diameter do not follow a clear trend; some culverts become smaller due to updates, as newer PVC alternatives can transport the same water volume with smaller dimensions owing to their reduced friction coefficient (Megawaty et al., 2012). Conversely, culverts may become larger when new neighborhoods with increased paved areas are connected, leading to higher peak flows, demanding necessary updates to accommodate the increased water volume.

The structure update of the model has resulted in changes in discharge patterns, as illustrated in Figure 30. For the Frankeneng system, the model demonstrates improved accuracy in capturing peak flows, although the highest peak is somewhat overpredicted due to the exclusion of the maximum discharge in the model. In the case of the Veldhuizen system, there is a slight improvement in the model's performance, but the peaks are still greatly over predicted. It is expected that the update incorporating paved and unpaved areas will reduce this overprediction, as existing literature emphasizes the substantial influence of these factors on model predictions (Pagotto, Legret, & Le Cloirec, 2000; de Carvalho, Iensen, & dos Santos, 2021).



(a) Frankeneng

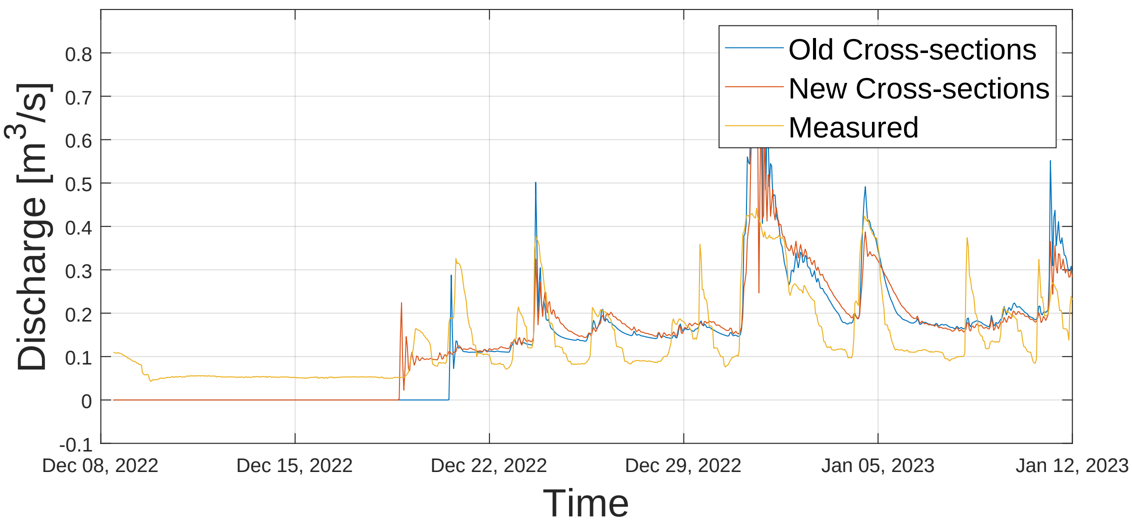


(b) Veldhuizen

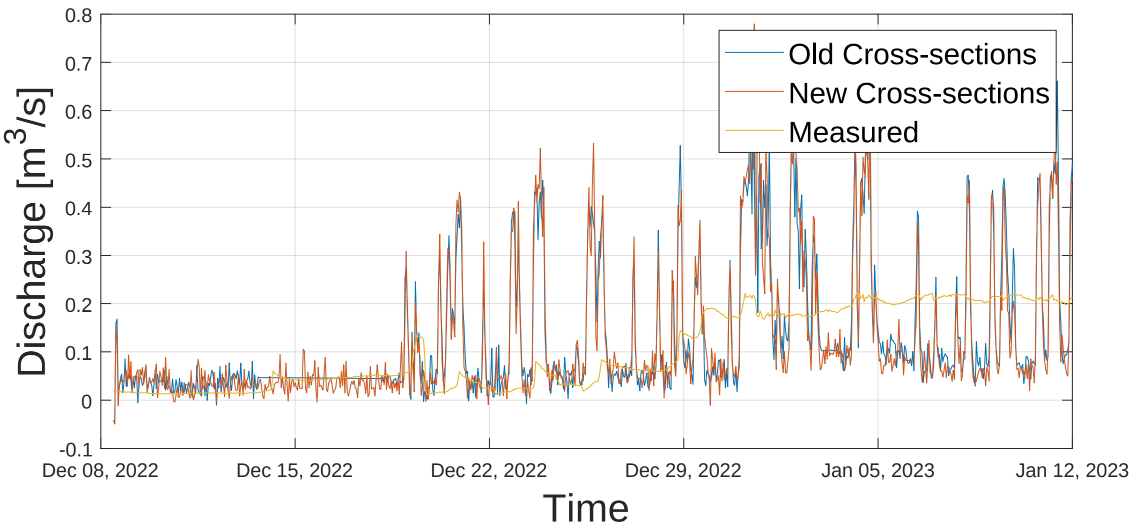
Figure 30: Discharge changes with structure update

4.1.2.2 Cross section

The cross-section profiles have been updated following the methodology outlined in Section 3.1.2.2. Figure 50 in the Appendix displays a map illustrating the changes in the number of cross-sections between the old and new models. The update of the cross-section profiles has led to alterations in discharge patterns, as depicted in Figure 31. The cross-sectional update displays limited change, the peaks are estimated slightly better. The cross-sectional update is mostly important for exploring the full storage capacity as displayed in Figure 21 in Section 3.1.2.2.



(a) Frankeneng



(b) Veldhuizen

Figure 31: Discharge changes with cross-section update

4.1.3 Rainfall-runoff

4.1.3.1 Paved& unpaved areas

The updated paved areas are derived from a combination of various data layers, including those from Figure 15, the overflow maps from Figure 17 and 16, and the sewer system. In the case of Veldhuizen, this process generates a paved map that encompasses the areas connected to the mixed sewage system behind each overflow, as depicted in Figure 32. Similarly, for Frankeneng, the updated paved map represents the areas connected to the separate stormwater system, as shown in Figure 33. These locations are diffusely integrated within the water system. In both maps, the purple sections represent the paved areas, while the green sections indicate the unpaved areas along with their respective catchment boundaries. The numbers on the labels indicates the specific catchment, corresponding with the Figures 16 and 17.

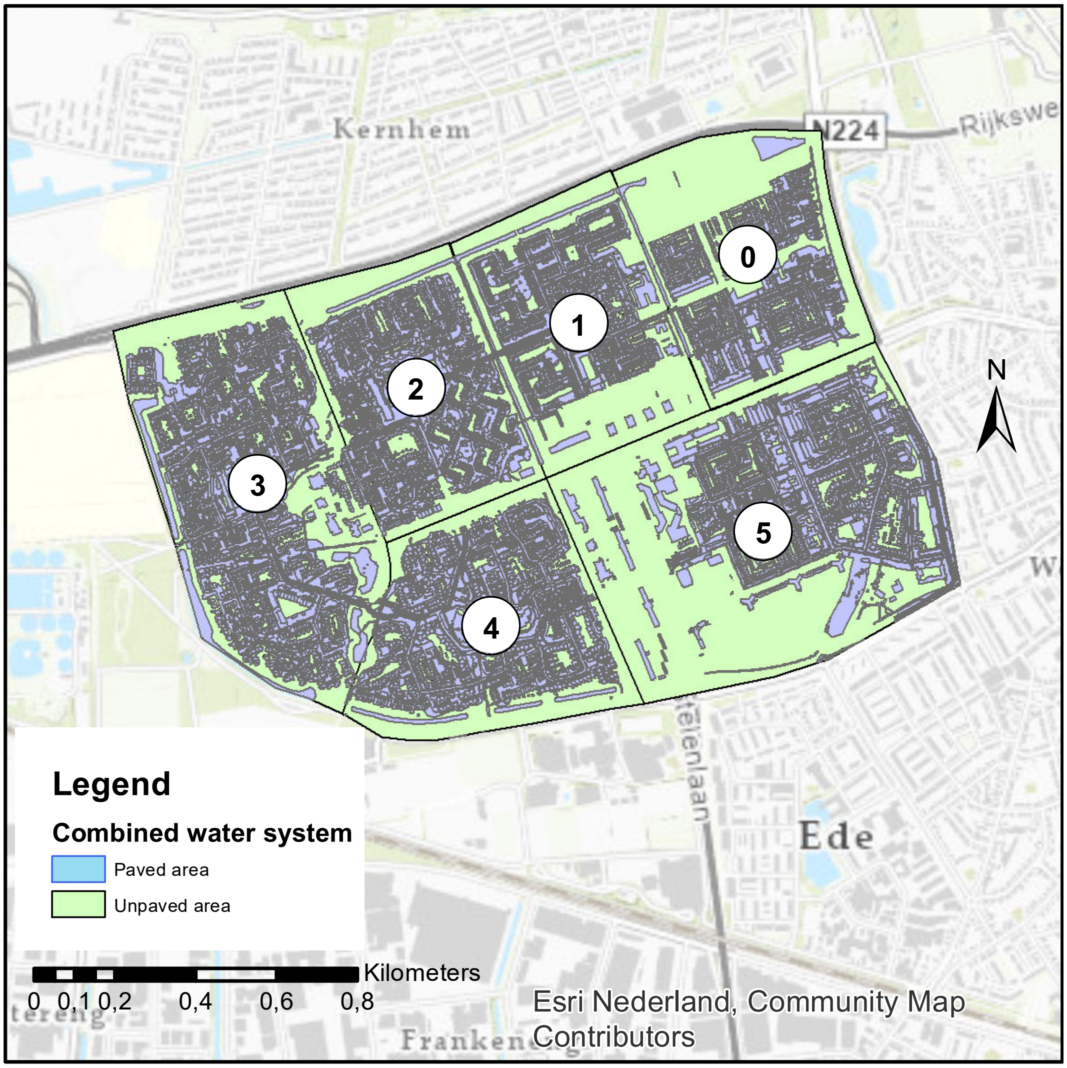


Figure 32: Paved area of the mixed sewage per overflow location in Veldhuizen

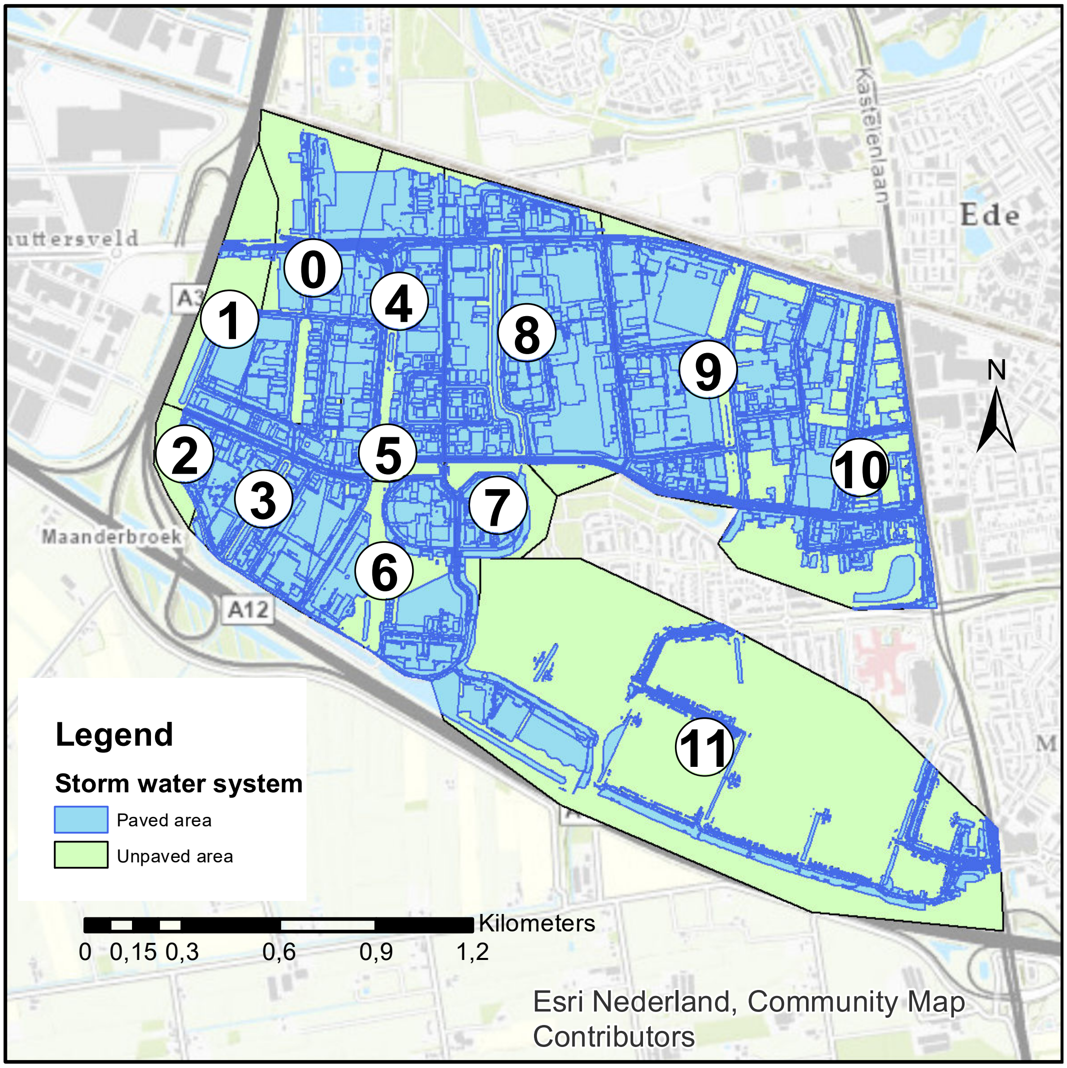
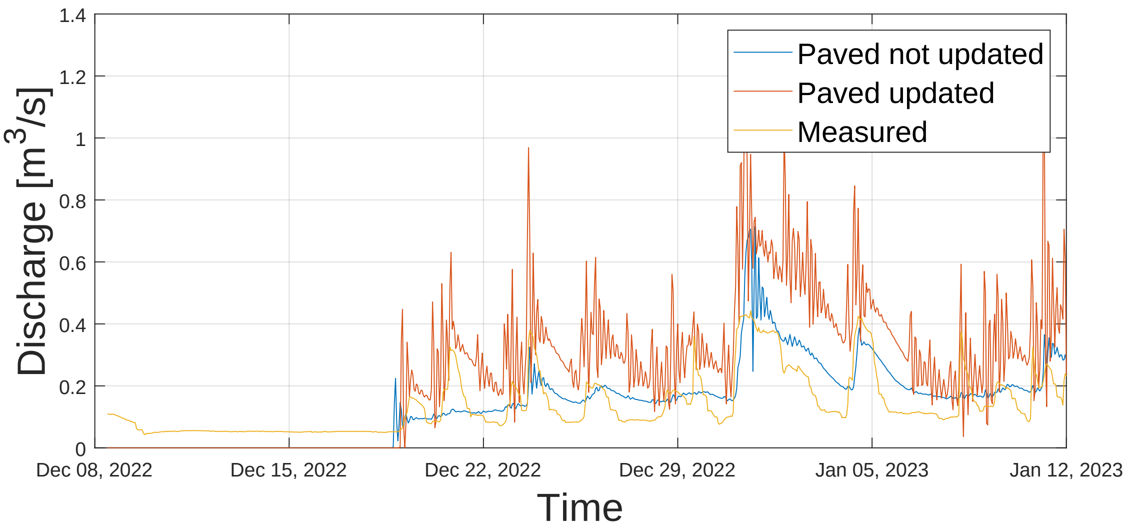
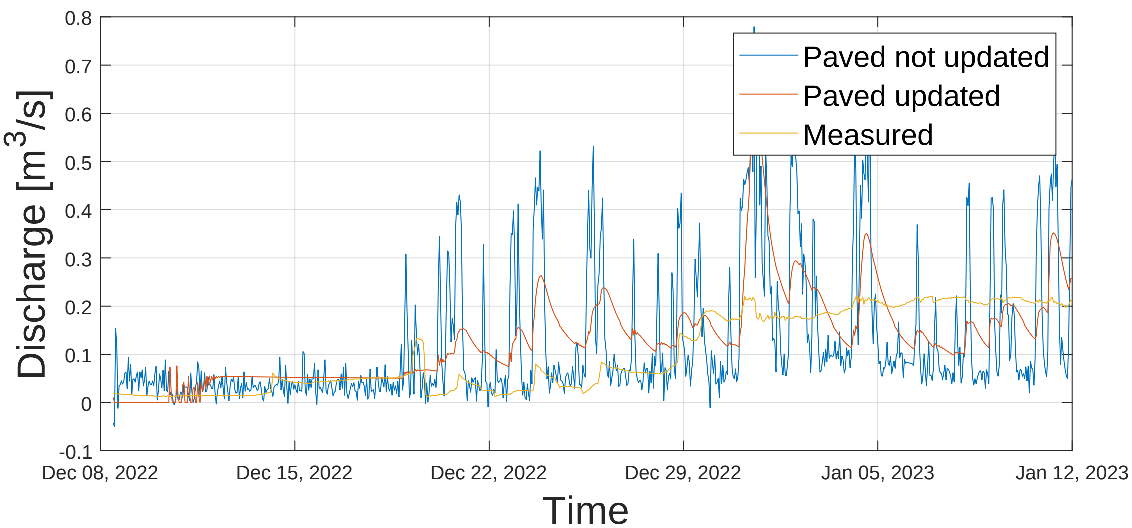


Figure 33: Paved area of the separate sewage per overflow diffuse point in Frankeneng

The impact of the updated paved areas on the model outcomes is depicted in Figure 34. It can be observed that the discharge tends to be overestimated due to the exclusion of updates in the unpaved area, resulting in the inclusion of certain areas twice in the model. The graphical representations substantiate the assertion that the updating of the paved area in isolation is not advisable, and instead, it should be updated together with the unpaved area. This error can arise from the practice of incrementally updating the model, as opposed to constructing the model anew.



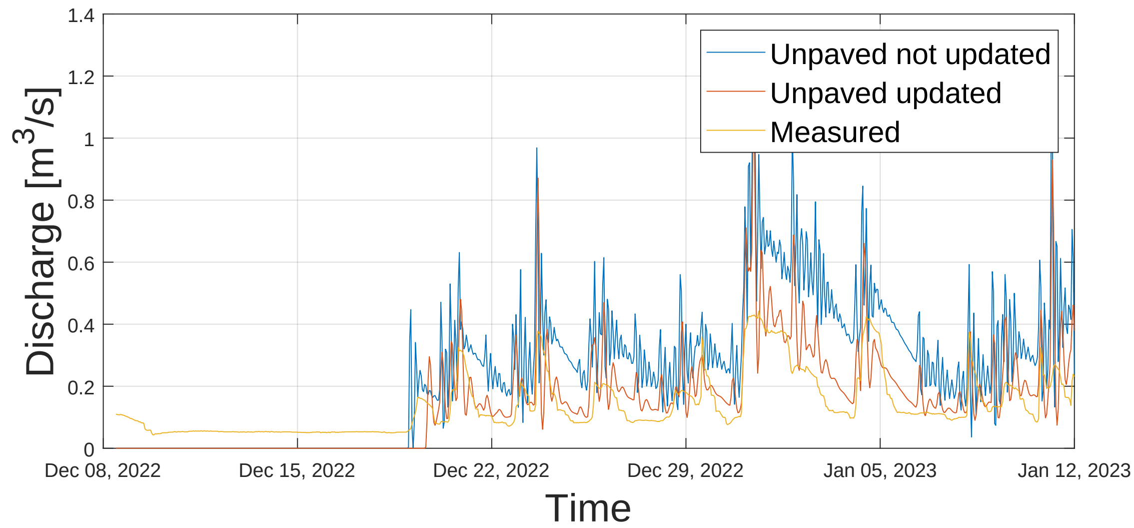
(a) Frankeneng



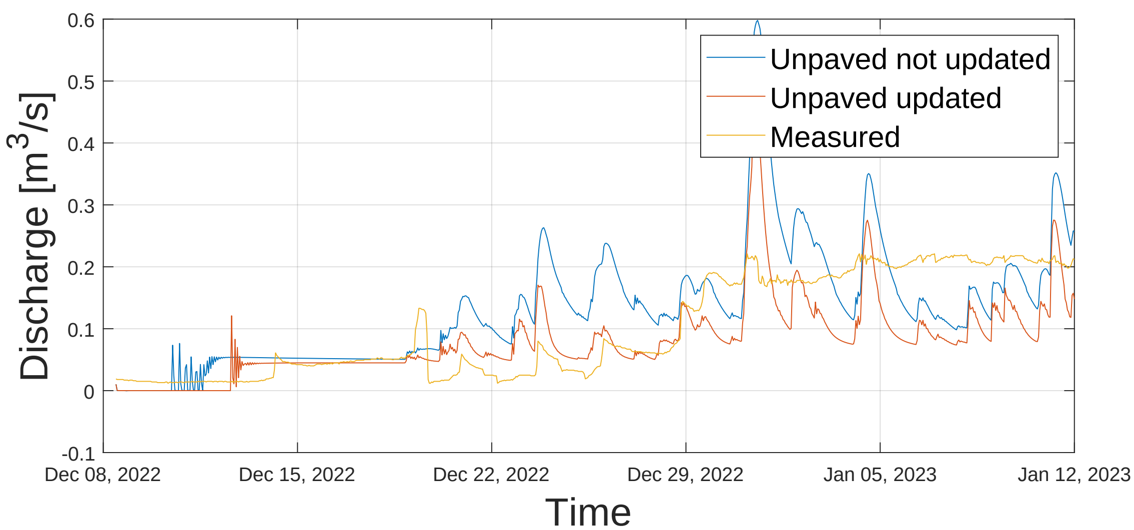
(b) Veldhuizen

Figure 34: Discharge changes with paved area update

The integration of unpaved areas contributed to a reduction in overestimated values, as evidenced by the improved fitness depicted in Figure 35. Furthermore, Figure 35a illustrates that the update resulted in a dampening of oscillations for the Frankeneng automatic weir.



(a) Frankeneng



(b) Veldhuizen

Figure 35: Discharge changes with unpaved area update

4.1.3.2 Model improvement overview

This section endeavors to provide a comprehensive overview of the incremental model update steps. In order to evaluate the model's improvement throughout the iteration process, two metrics, namely the Nash-Sutcliffe Efficiency (NSE) and Root Mean Square Error (RMSE), are utilized. The NSE score is a measure that ranges from $-\infty$ to 1, with a value of 1 signifying a perfect correspondence between the observed and modeled data, while lower values indicate poorer fitness. Conversely, the RMSE quantifies the disparity between the observed and modeled data, with smaller values denoting a lesser discrepancy between the two. Figure 36 provides a depiction of the NSE and RMSE values at each iteration step. Significantly, both metrics exhibit a trend towards enhanced accuracy in the models. It is crucial to note that the cross-section update yields a decrease in fitness. This can be attributed to the fact that the previous assessment model was calibrated based on the former cross-sections, whereas the new profiles encompass the entire cross-section from the bed level to the ground level, as illustrated in Figure 21. This expanded scope may facilitate increased groundwater drainage due to the heightened disparity between the ground level and water level. This assertion finds support in the Ernst calculation method, which is used in the model

(Ernst, 1978).

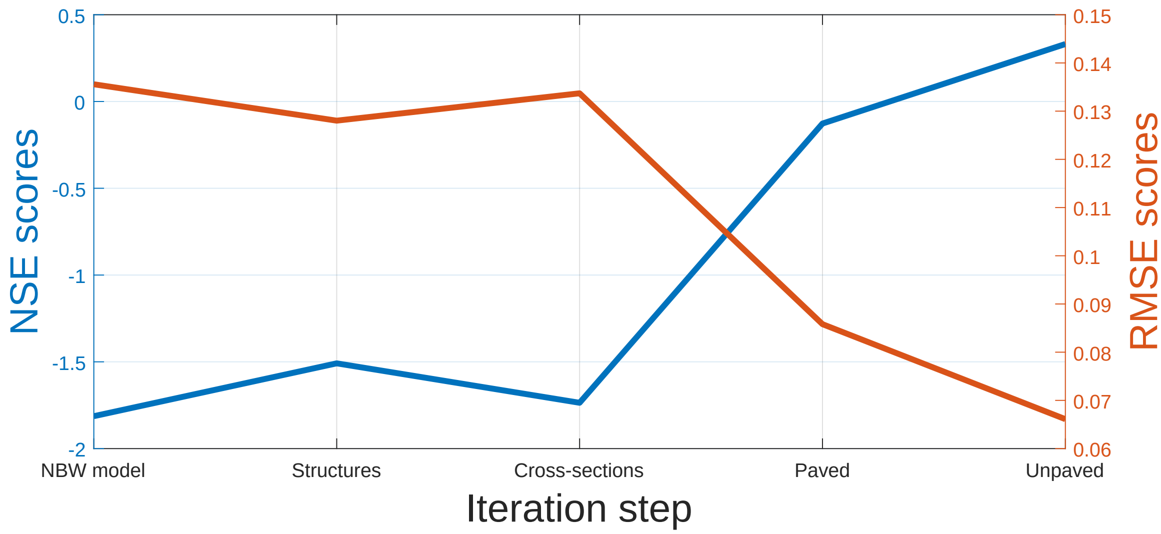


Figure 36: NSE and RMSE scores per model iteration

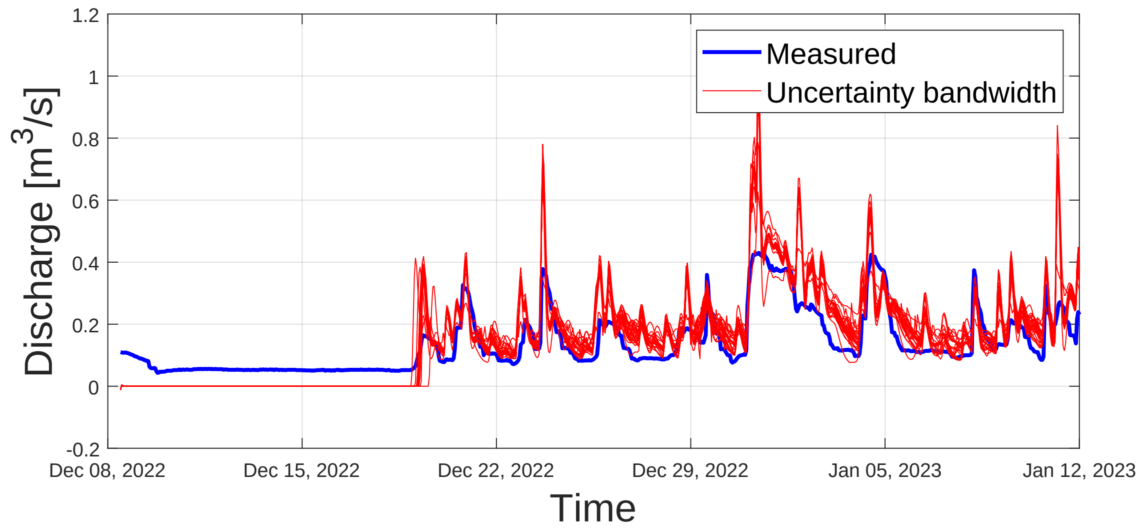
4.1.4 Sensitivity

The evaluation of sensitivity is conducted by considering the estimated uncertainty associated with input parameters or data. The results of the sensitivity analysis are presented in Table 7, which showcases the NSE fitness values for various parameters. The color scale employed in the table indicates the NSE scores relative to those obtained from the reference model used in the assessment. Specifically, a yellow color indicates a fitness level similar to that of the measured data, while shades leaning toward the green spectrum indicate a superior fitness compared to the reference model. Conversely, shades tending toward the red spectrum suggest a fitness level inferior to the reference model. It is worth noting that NSE scores should ideally be greater than zero, while the uncertainty associated with the model should have minimal impact on these scores (Knoben, Freer, & Woods, 2019). The Veldhuizen model exhibits greater robustness in terms of parameter variation, as all NSE scores exceed 0.2. Moreover, the extreme values fall within acceptable boundaries. In contrast, the Frankeneng model displays greater sensitivity to parameter changes, with the lowest NSE score falling below zero and one score approaching zero. This heightened sensitivity of the Frankeneng model could be attributed to its relatively higher proportion of paved area. Conversely, unpaved areas demonstrate greater resilience to changes due to their ability to retain and subsequently release water (de Carvalho et al., 2021).

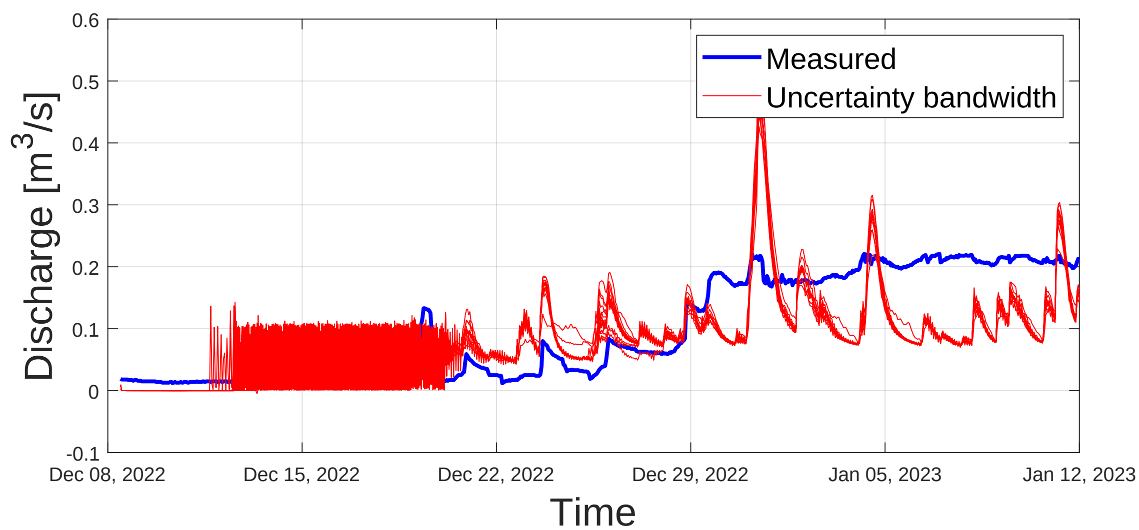
Parameter	Change [%]	Direction	NSE Frankeneng	NSE Veldhuizen
Slow drainage	40	+	0,214	0,311
		-	0,178	0,300
Fast drainage	40	+	0,195	0,263
		-	0,147	0,304
Drainage depth	40	+	0,164	0,255
		-	0,159	0,252
Paved	10	+	-0,076	0,265
		-	0,346	0,288
Sewage	40	+	0,208	0,287
		-	0,109	0,212
Unpaved	10	+	0,185	0,266
		-	0,206	0,259
Groundwater	10	+	0,165	0,266
		-	0,155	0,263
Infiltration	40	-	0,175	0,264
Cross-section	10	+	0,206	0,261
Culvert length	10	+	0,200	0,312
		-	0,171	0,305
Culvert height	10	+	0,427	0,387
Weir width	10	+	0,172	0,241
		-	0,194	0,323
Weir height	10	+	0,194	0,296
		-	0,134	0,211
Friction	10	+	0,007	0,263
		-	0,142	0,283

Table 7: NSE indicators for sensitivity analysis

The NSE provides an overall assessment of the model's fitness, taking into account the entire dataset. However, it is important to note that specific outliers in the data may have a limited impact on the NSE value but can significantly affect the maximum values predicted by the model. To capture the uncertainty in the model, all sensitivity cases are combined to create an uncertainty bandwidth, which is then compared to the measured data. Figure 37 illustrates that no implausible outliers are observed in the models. However, it is worth mentioning that the Veldhuizen model exhibits oscillations during the initial 10 days of the simulation. This behavior may be attributed to the filling time required for the deep ponds in the system.



(a) Frankeneng



(b) Veldhuizen

Figure 37: Uncertainty bandwidth compared to the measured data

The calibration process encompassed the assessment of model performance across the 32 cases specified in the methodology. The adequacy of fit was evaluated by considering the NSE and RMSE scores, which are presented in Table 11 in the Appendix. Among the various cases examined, the scenario featuring a fast drainage period of 20 days, a slow drainage rate of 150 days, and a drainage depth of 0.8 meters emerged as the most optimal fit for both catchments.

4.1.5 Validation

To validate the model, it was further tested using a different precipitation series. Figure 38 illustrates that the model adequately simulates the discharge peaks during the validation event. The NSE score obtained for the validation is 0.294, which is slightly lower than the score achieved during calibration. Nevertheless, this indicates that the surface and groundwater model remain robust even when pushed beyond their calibrated boundaries.

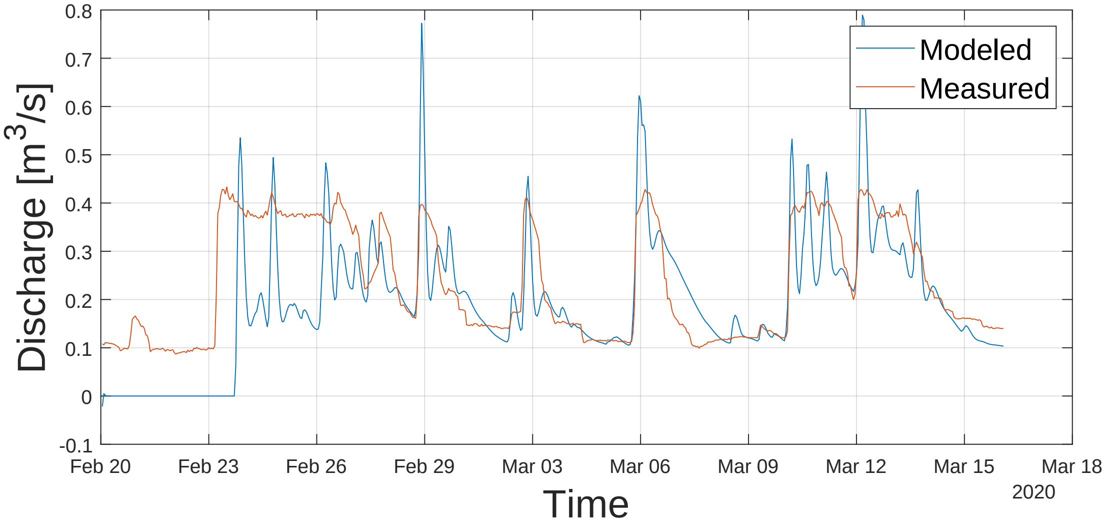


Figure 38: Validation event

In order to evaluate the comparative robustness of the new model in relation to the old model, a simulation for the validation event with the old model is performed. The results, depicted in Figure 39, indicate that the employment of the old model leads to fewer oscillations. Nevertheless, it should be noted that the model tends to underestimate peak values, as evidenced by the absence of two peaks in the simulated data compared to the measured data. This discrepancy poses a significant challenge for research pertaining to urban scenarios that place particular emphasis on accurately predicting peak discharge.

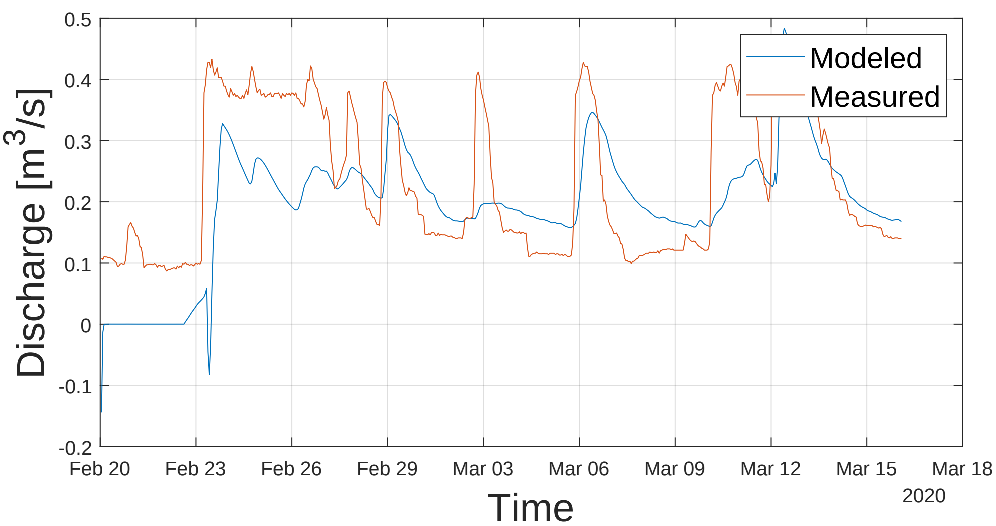


Figure 39: Validation event original model

4.2 Problem origin identification

4.2.1 Storage utilization

The comparison between the actual storage and the theoretical storage reveals a significant disparity. The design of the weirs, which serve to regulate water levels, is a contributing factor to this underutilization of storage capacity. Specifically, the weir height is considerably lower than the maximum water level, leading to the observed discrepancy.

To gain further insight into the actual storage, the freeboard height and side views are examined. The freeboard height is the difference between the water level and the ground level, as displayed in Figure 22.

Figure 40 illustrates the free board heights observed at various sampling locations within the water system. Each line on the graph represents a different pond, displaying the fluctuations in free board height (measured in meters) over the calibration period. The ponds are strategically positioned along the water system, ranging from far upstream to near the automatic weir. The variations in free board heights at different locations indicate distinct storage capacities for each pond. The underlying assumption is that a temporary increase in water height leads to a rise in storage utilization (Mitchell, McMahon, & Mein, 2003). The graph reveals that the potential increase in water height varies across the locations, ranging from 0.2 to 2 meters. Furthermore, the peaks in the graph exhibit both limited magnitude and duration, suggesting that the ponds store only a limited amount of water before it overflows over the downstream weir. This observation is further supported by examining side views of the system, presented in Figure 51 in the appendix. These side views specifically depict the Veldhuizen, Rietkampen, and Frankeneng ponds, providing visual evidence that the ponds are entirely regulated by the weirs, thereby aligning with the initial hypothesis.

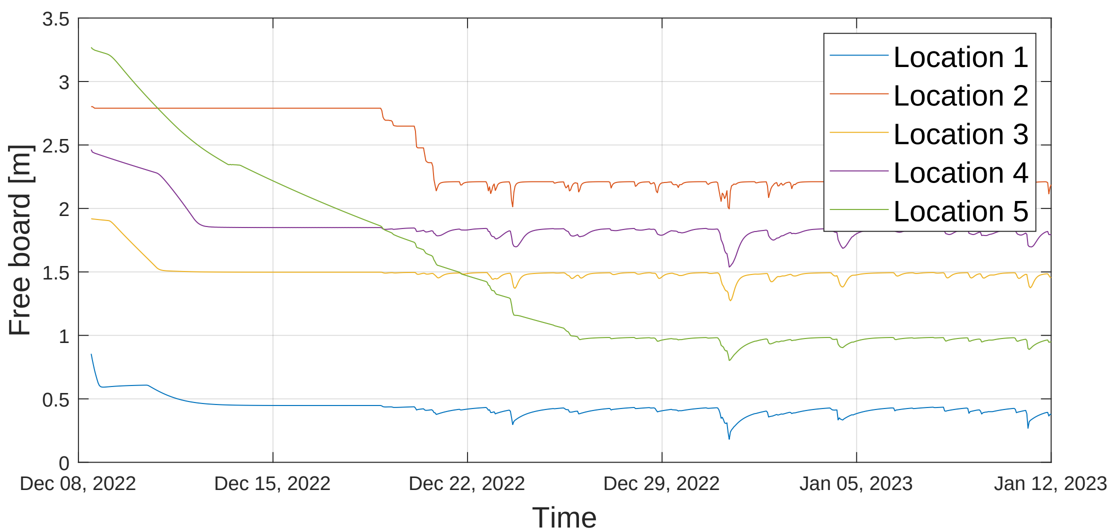


Figure 40: Free board mean at sample locations

4.2.2 Drainage of system

To illustrate the potential for improvement, Figure 41 depicts the change in water height of 0.30 meters, which is solely caused by the overflow at the weirs. A more desirable scenario would involve a system with a lower water level, indicating a greater storage capacity that can be utilized during precipitation events. The Figure demonstrates that the water height can increase more when the system is emptier, this increase in water height during peak events increases the storage utilization (Mitchell et al., 2003)

The capacity of the ponds to store water is reliant on the presence of storage capacity within the water system, as explained in the preceding section. Once the storage capacity threshold is reached, any surplus water must be discharged downstream, evaporated, or infiltrated. The consistent water levels observed in Figure 40 suggest that the ponds operate consistently at near-full capacity. During precipitation events, minor fluctuations in water height are observed, with a maximum magnitude of 0.30 meters. This finding confirms the hypothesis that the ponds are consistently filled to their maximum capacity.

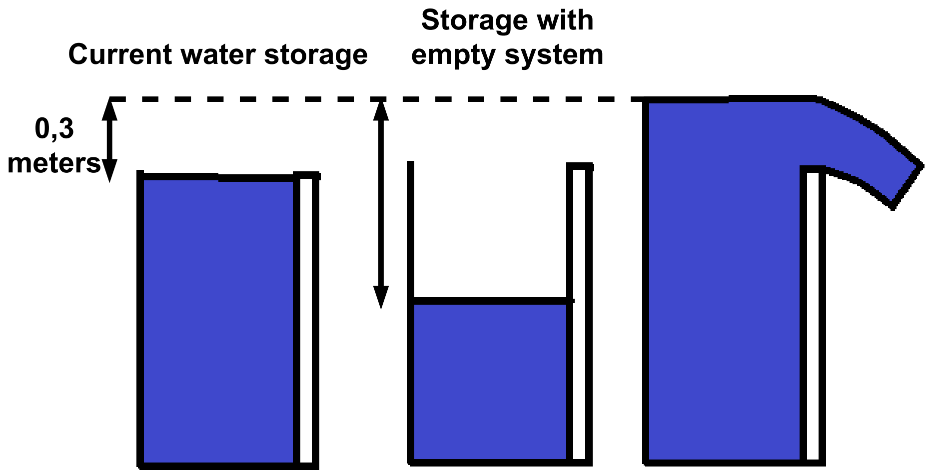
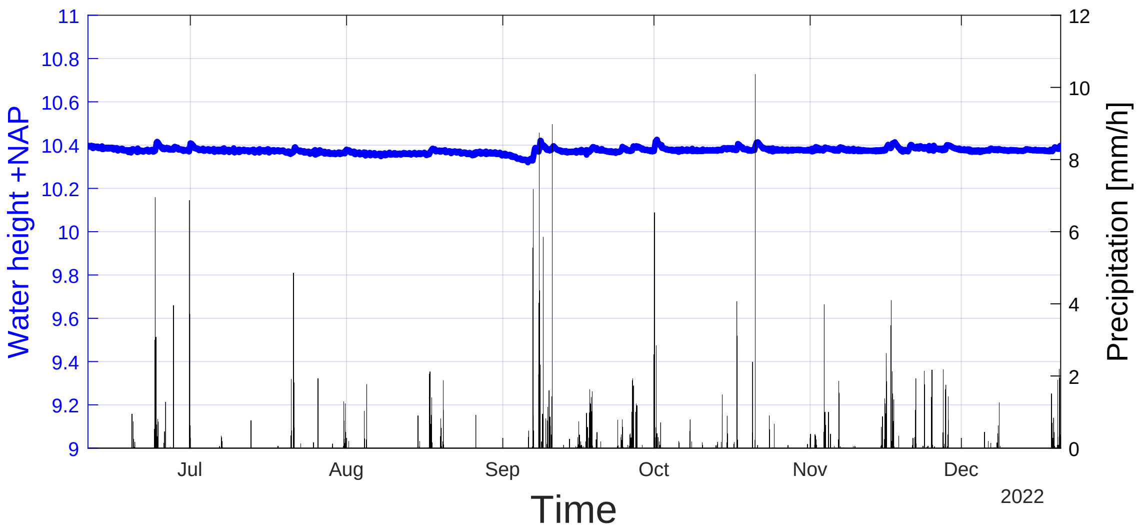
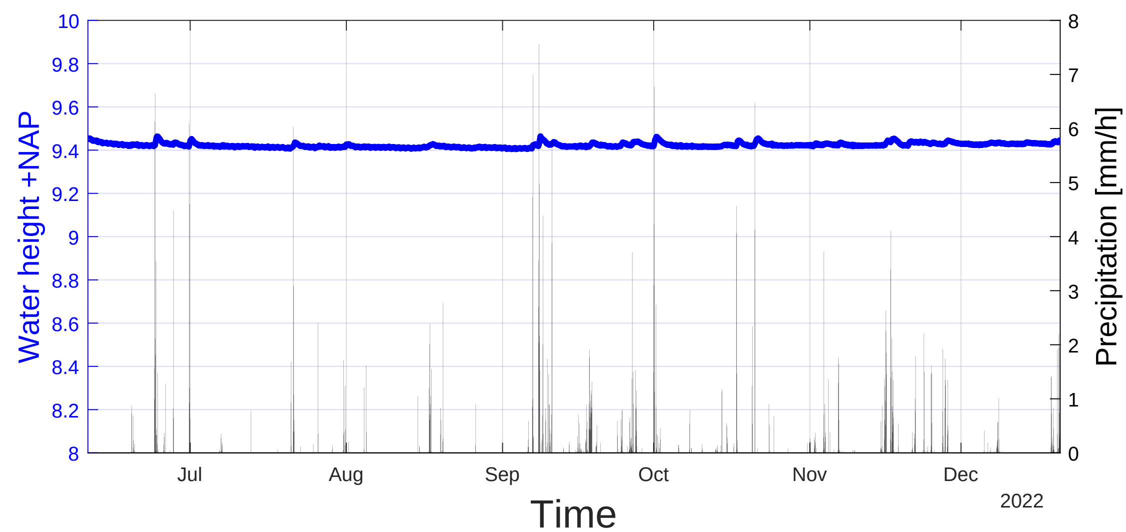


Figure 41: Current and potential storage capacity

To validate the aforementioned hypothesis, measured data obtained from water height gauging devices installed downstream of the overflow locations by the municipality are employed. These devices accurately capture the water height of the surface water. In order to enhance the reliability of the findings derived from the SOBEK model, which is calibrated and validated downstream at the automatic weir, the measured water heights within the system are incorporated. It should be noted that most municipal overflow locations solely measure the sewage side of the water system. However, the two selected overflow locations provide measurements of the surface water height and offer consistent data series. Figures 42a and 42b illustrate the measured data, which aligns with the hypothesis and reveals minimal fluctuations in water height. These findings provide further support for the assertion that the ponds consistently operate at or near their maximum capacity. Moreover, they underscore the imperative for enhancing the utilization of the storage system.



(a) Overflow Arendshorst



(b) Overflow Munnikenhof

Figure 42: Water height measurements at overflow locations

4.2.3 Problem allocation

The relative discharges per weir were computed, leading to the generation of the map presented in Figure 43. The symbol size in the map corresponds to the absolute discharge measured in cubic meters per hour (m^3/h), while the color ramp represents the relative discharge expressed in liters per second per hectare (l/s/ha). The labels on the map indicate the specific values of the relative discharge. This map serves as a valuable tool for identifying potential bottlenecks within the system. The weirs highlighted in red on the diagram indicate instances with the most problematic relative discharge, these values deviate significantly from the waterboard norms (waterschap vallei& veluwe, 2013). The size of each symbol corresponds to the magnitude of the underperformance associated with the respective weir. By prioritizing intervention at the larger red symbols, it is possible to enhance the overall capacity of the system more efficiently.

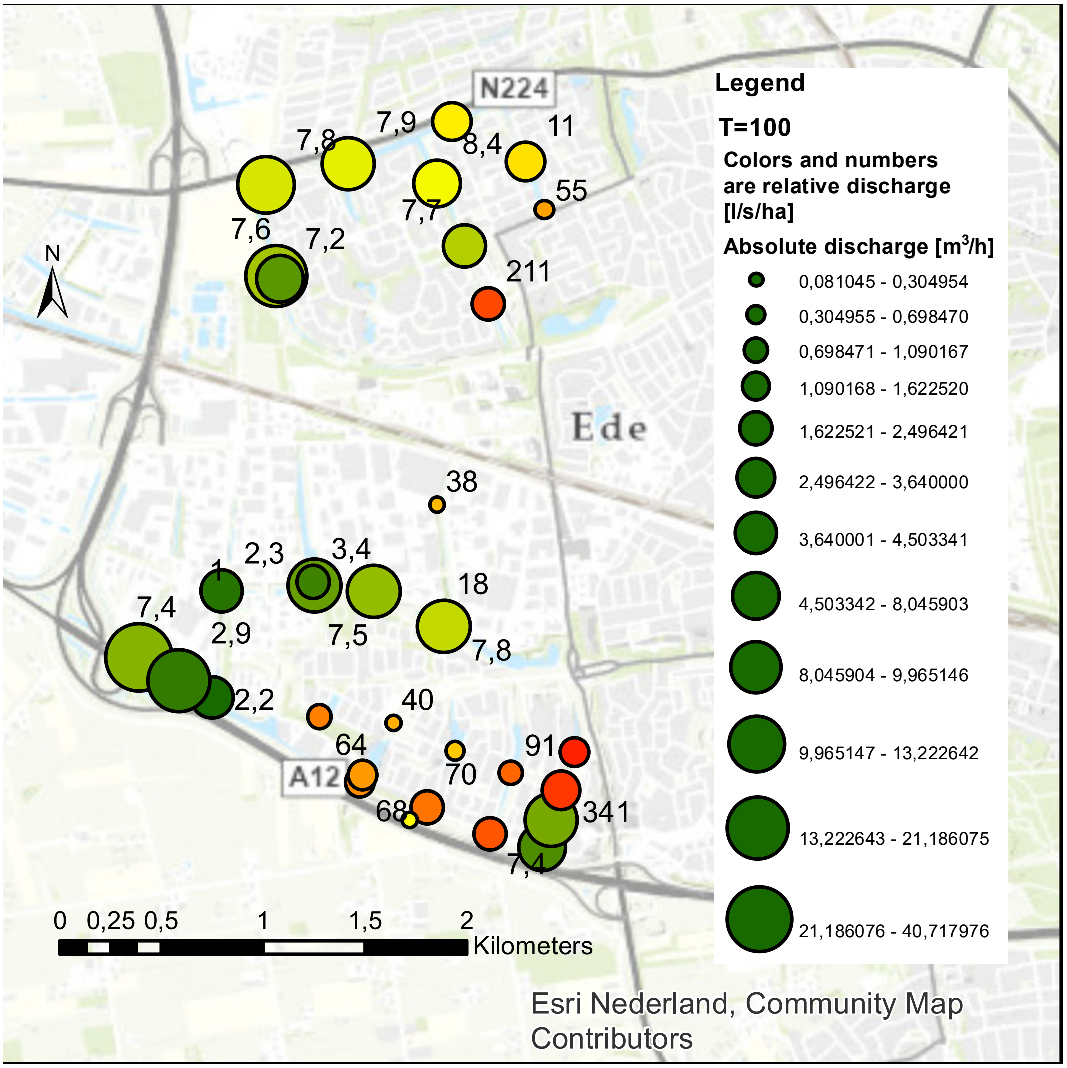


Figure 43: Relative discharge per weir

4.3 Interventions

Based on the findings from the preceding section, potential interventions aimed at increasing storage capacity through temporary elevation of water levels in ponds are being considered.

4.3.1 Influence on discharge

Utilizing the methodology outlined in Section 3.4, several criteria have been established for selecting interventions. Among these criteria, the following are deemed particularly significant: maintaining the integrity of the concrete structure surrounding the manhole weirs, facilitating storage capabilities across various discharge events, and adhering to budget constraints. Based on these criteria, the intervention chosen is the implementation of V-notch weirs. This selection is justified by the ease of construction, the ability to accommodate different discharges at varying water heights, and the cost-effectiveness of the construction process. To assess the impact of implementing V-notch weirs on discharge, Figure 44 presents the results. The peak discharges were found to decrease by 20% and 40% for the 0.3-meter and 0.5-meter weirs, respectively. However, it should be noted that the discharges following the peak levels remain relatively high. This observation implies that the model is functioning correctly, enabling effective water exit from the system, while reducing the peak discharges.

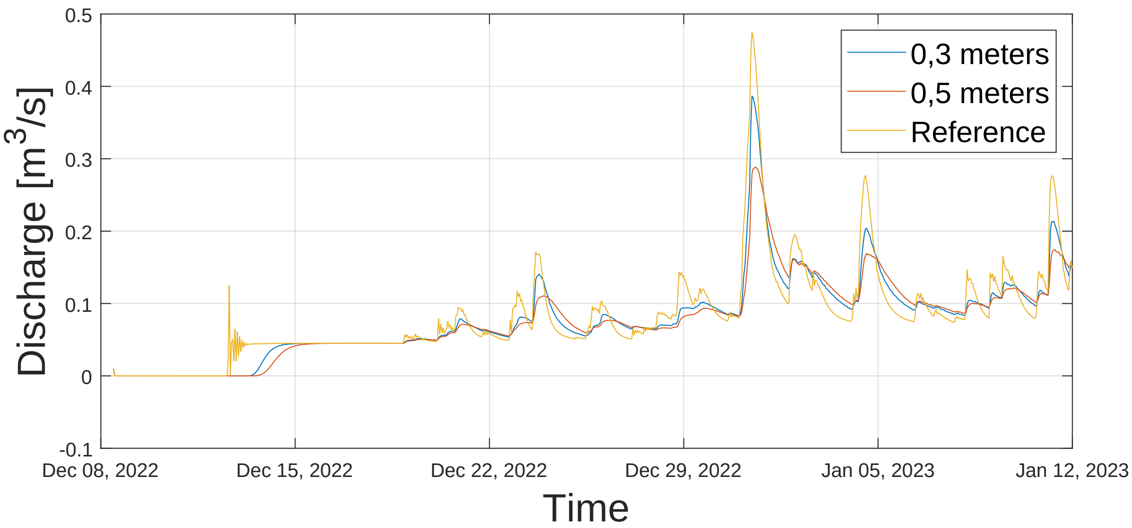


Figure 44: Discharge of potential V-notch weir interventions

4.3.2 Influence on water height

The purpose of the basins is not primarily to store water, but rather to lower the groundwater levels in the surrounding neighborhoods. The presence of additional pipe drainage ensures that the groundwater level is effectively reduced. Therefore, increasing the water levels in these ponds is unlikely to cause significant issues for the groundwater level. However, it is important to note that the water levels should be lowered within 24-48 hours to mitigate any potential effects on the groundwater level. Figure 45 illustrates the water heights in sample ponds located upstream of the modified weirs. The response of the weirs to the changes varies depending on the specific location. In the first two graphs, the normal water level remains relatively stable while the peaks become higher. In the remaining graphs, a more systematic change can be observed. However, it is worth noting that the change in water height is much smaller compared to the change in weir height. Additionally, the available freeboard storage capacity, as depicted in Figure 40, is not exceeded.

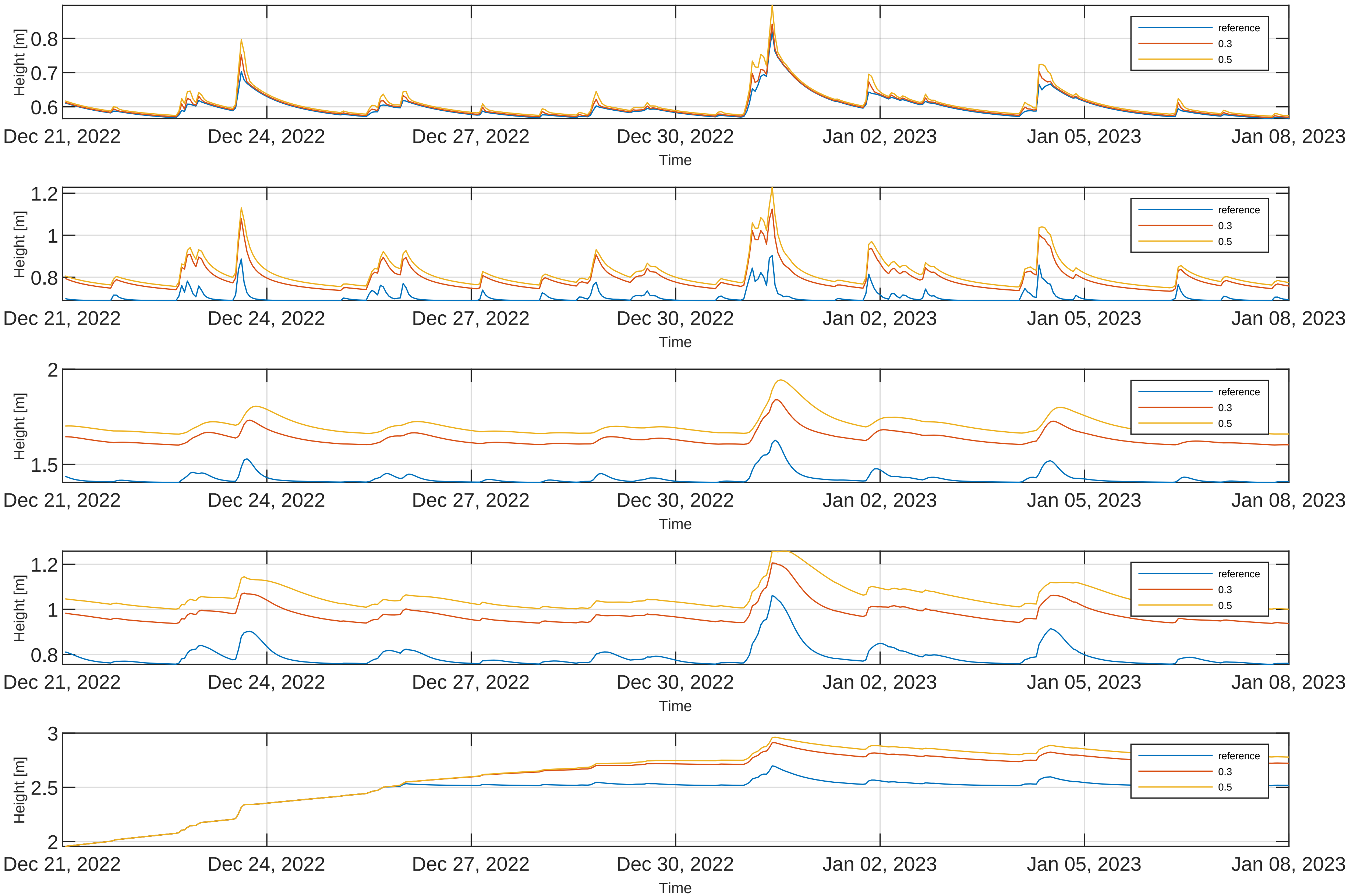


Figure 45: Sample water heights

Discussion

This discussion aims to put the result into context. The Discussion is divided into different segments. First, the data quality and acquisition are discussed. Subsequently, the model reliability followed by the validation and data integration.

5.1 Data quality

The quality of the data used in the model plays a crucial role in determining the accuracy of the model outcomes. Each type of data used in the model is evaluated in terms of its quality and reliability.

FEWS, which is a data collection program from the waterboard, provides a wide range of data accessible through a GIS interface. The reliability of the data sources integrated into FEWS is important, as any uncertainties or errors in these sources can impact the model outcomes. The weather data used in the model is obtained from the Royal Netherlands Meteorological Institute (KNMI) (KNMI, 2023b), which is considered a reliable source for weather data. The data is collected using both automatic and manual gauging devices, and the automatic devices, in combination with radar technology, provide neighborhood-level weather data. This data is continuously validated using manual voluntary-based gauging devices.

The water height and discharge data are integrated from TMX, the management software for automatic weirs. Water height gauging devices are located both upstream and downstream of the weirs, and the crest height is known. TMX stores real-time data for these three variables. However, there are some challenges with the discharge calculations. The discharge coefficient (C_d) used in the calculation is a general coefficient and may not accurately reflect the specific conditions of each case (Aydin, Altan-Sakarya, & Sisman, n.d.; Rehbock, 1929). Moreover, the measurement devices used for water height may degrade or get stuck, potentially affecting the accuracy of the discharge calculations. It is important to note that the primary purpose of automatic weirs is to regulate water heights and discharges, rather than gather precise discharge data.

The sewage system data is controlled by the municipality and stored in the H2gO interface, which is used for monitoring and maintaining the sewage system. The data from H2gO, including overflow locations, is up to date and validated by site managers from the municipality. However, there are limitations with the interface itself, as it does not allow for easy extraction of data in shapefile format. Additionally, the capacity of the sewage system is simplified to a norm of 0.7 mm/h based on the basic sewage plan (BRP) (Municipality of Ede, 2022). This simplification may not accurately represent the capacities of different sewage systems, as the sizes of the systems can vary significantly.

The surface water data is stored in the register data from the waterboard, accessible through the GeoWeb servers operating on ArcGIS online (waterschap vallei en veluwe, 2023). The register data provides information on the structural data, profile data, and site images of the water system. The accuracy of the data relies on the input provided by project planners, area managers, and measurement projects. However, human error, varying definitions, and difficulties in precisely locating measurements can introduce uncertainties into the data. The locations can be difficult to access due to their position in manholes or under the water level. Furthermore, the register data may not always be up to date due to the challenges of keeping track of the constantly changing water system.

The delineation of paved and unpaved areas is based on a series of map overlays conducted by the municipality. Data sources such as the 'Basic Registration of Addresses and Buildings' (Municipality of Ede, 2015), the Basic 'Registration of Large-Scale Topography' (Gemeente Ede, 2023), and infrared views are used for these overlays, and these sources are up to date and validated. However, a more accurate estimation of the water catchment area behind each overflow could be achieved by employing 2D modeling rather than relying on height maps. It is also worth mentioning that the changing nature of paved areas, particularly with the government's plan to build 900,000 houses by 2030, introduces challenges in accurately representing the evolving urban systems (Ministry of the Interior and Kingdom Relations, 2022).

The unpaved areas are derived from the old NBW model, and the calculations for these areas were limited by the available time and knowledge during the research. The data is based on the 'Landelijk Grondgebruik Nederland 4' (WUR, 2000) dataset, which uses data from the year 2000. Given that spatial data continuously changes, relying on information that is over two decades old significantly reduces its reliability. To estimate the current unpaved land use, the total area and paved area are used, with the distribution based on the land-use ratios from the old model. It is assumed that these ratios change less over time compared to the total unpaved area.

5.2 Model reliability

SOBEK 216 is a complex hydrological model developed by hydrological experts and validated by field experts. Over the years, numerous iterations have improved the model's performance and stability. However, due to simplifications and underlying assumptions, the model has inherent limitations in terms of its reliability. It is essential to note that while the model simulates the water system, it does not directly represent the actual water system. Conclusions drawn from the model should not be considered direct conclusions drawn from the real-world water system. The SOBEK model employs hydrological interactions based on continuous hydro-physical equations to simulate the water system (Deltares, 2013). In the context of larger hydrological problems requiring calibration, computation time emerges as a significant concern (Rouholahnejad et al., 2012) Consequently, it becomes imperative to assess and optimize each component of the model to reduce computation times while preserving the reliability and accuracy of the model (Salvadore, Bronders, & Batelaan, 2015). To reduce computation time and increase model stability, simplifications are employed. The RR (Rainfall-Runoff) and Flow models are used sequentially, excluding the interaction between water levels and groundwater modeling. Integrating these components into a single model would significantly increase computation time and introduce instability to the model.

In terms of culverts and cross-sectional data, different parties were responsible for their measurements, leading to instances where culverts were found below the bed level of the surface water system. This situation would close the system, so a solution was implemented in which the simulation module allows for the movement of culverts in height, up to a maximum of 1 meter. The assigned height is arbitrary and not directly related to the actual height, which results in inaccurate measurements.

The storage in the system is determined by the cross-sections, which presents two challenges. Firstly, capturing the complex nature of storage structures accurately in a 1D module is difficult. Figure 46 illustrates a non-linear reservoir, where the green lines represent available cross-sections and the blue line represents the 1D SOBEK model. Implementing these cross-sections in the model would not yield accurate storage calculations. To address this issue, the 2D module could be utilized, but this would significantly increase computation time and introduce instability to the model.

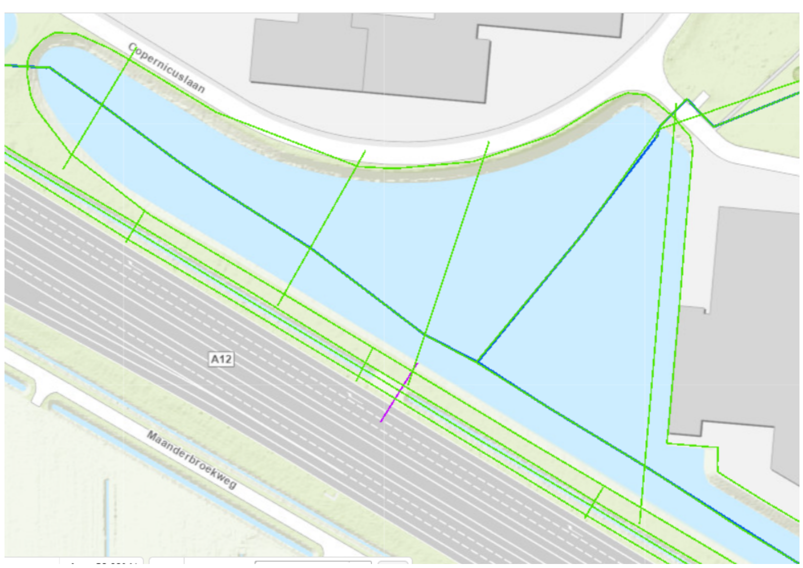


Figure 46: Complex nature of storage facilities

The subsequent issue relates to the inclusion of culverts in the model, which poses challenges in accurately representing the storage capacity. In the model, all components of the water system are assigned cross-sectional profiles, including the culverts. However, the model calculates the culverts based on an additional resistance at a single point, without considering the absence of water surrounding the culvert. This issue is illustrated in Figure 47, where the maximum storage capacity is depicted in orange. It is evident that the SOBEK model tends to overestimate the storage capacity due to this oversight. Additionally, Figure 29a demonstrates that the number and length of culverts can lead to significant deviations in the storage capacity. To address this problem, the culverts can be alternatively modeled as cross-sections, particularly for culverts longer than 50 meters. By employing cross-sections, the storage capacity can be accurately calculated, accounting for the lost potential storage capacity resulting from the inclusion of culverts.

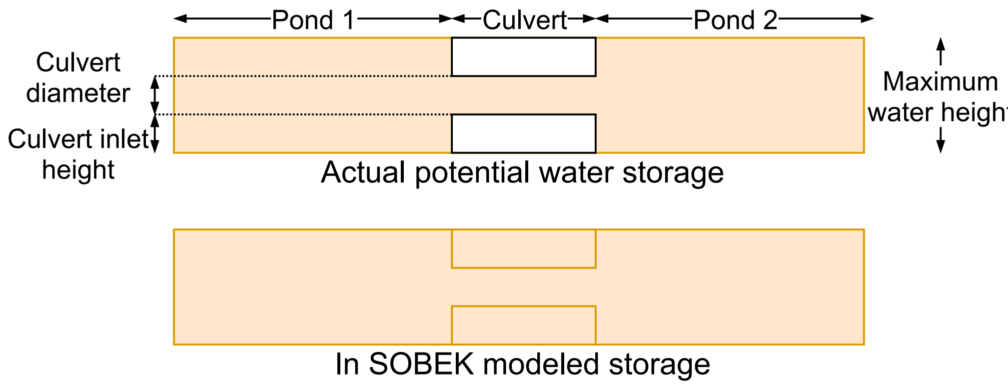


Figure 47: Water storage problem culvert

5.2.1 Groundwater

SOBEK, developed by Deltares, is primarily a surface water model that focuses on simulating the behavior of surface water systems. However, surface water interacts with other components of the water cycle, particularly the sewage and groundwater systems. The groundwater system operates on different hydrological timescales, spanning days or weeks rather than hours. Deeper groundwater and aquifer velocities are generally measured in meters per year, as opposed to the meters per minute scale of the surface water system (Mazor & Nativ, 1992). To account for this discrepancy and reduce computation times, assumptions are made regarding groundwater levels. The Ernst formula for drainage is employed as a simplification of the groundwater system. Although more sophisticated groundwater models such as AZURE and MODFLOW (Lourens, Bierkens, & Geer, 2015; Brunner, Simmons, Cook, & Therrien, 2010) exist for more accurate groundwater simulation, in SOBEK only other simplified groundwater models can be chosen.

The calibration of the model's drainage resistances for each layer presented some challenges. Three independent parameters were simultaneously calibrated, resulting in numerous combinations. To streamline the process, only four values were calibrated for drainage resistances and two values for drainage depth. A more comprehensive calibration with a wider range of values and additional data points in between would have been preferable. Employing a local optimum algorithm could have further refined the fit, but this was beyond the scope of the project. Such improvements would enhance the reliability of the groundwater model. The infiltration capacity in the model is set at 100 mm/h, which is an overestimation. This choice ensures that the groundwater system is effectively utilized in the model. Reducing the infiltration capacity could lead to relying solely on surface runoff toward the water system, thereby introducing deviations in the modeling of subsequent precipitation events. During heavy precipitation events, the surface becomes saturated rapidly. Maintaining a constant infiltration capacity of 100 mm/h throughout the entire event is unrealistic and can result in flawed model outcomes.

5.2.2 Results interpretation and validation

The model update described in Section 4.1 highlights the challenges associated with urban hydrological modeling. Existing research has extensively discussed the difficulties in data collection for larger urban catchments and the computational challenges encountered during model calibration (Salvadore et al., 2015; Ichiba et al., 2018). The studies emphasize that an integral and critical approach is crucial for reliable urban hydrological modeling.

The findings presented in Section 4.2 provide evidence to suggest that the current design of urban weirs is suboptimal for reducing peak discharges. This finding is consistent with previous research, which highlights the underestimation of storage requirements (McEnroe, 1992). Notably, this research emphasizes that the definition of peak storage should focus on the actual reduction in peak rather than solely considering the volume of storage.
The results obtained in Section 4.3 demonstrate the impact of replacing rectangular weirs with v-notch weirs on peak discharge. Previous studies on v-notch weirs have indicated that such interventions can effectively reduce peak discharges by enabling lower steady-state water levels (Martínez, Reca, Morillas, & López, 2005).

The model was calibrated for a specific event, while another event was used for validation. However, the period of measurements starting in 2018 encompassed a limited number of extreme precipitation events. Consequently, both the calibration (training) set and the validation (test) set were constrained. The model was validated against the existing water system, and its robustness was enhanced by considering the full infiltration capacity and groundwater system. Nevertheless, the model was pushed beyond its calibrated and validated boundaries. The outcomes of the proposed interventions should always be critically examined and not accepted as absolute truth. They provide insights into the system's processes and reactions to changes. However, implementing the proposed interventions does not guarantee the same system changes. To address this limitation, a potential solution is to plan incremental projects, implementing interventions in small sections of the catchment, measuring the actual changes, and feeding the measured data back into the model to enhance catchment-specific validation. This process requires careful planning and time.

Furthermore, certain verification steps performed in this research cannot be conducted at the scale of the waterboard's entire management area. Site visits and on-site inspections are time-consuming and not financially feasible for the entire management area. If the proposed methodology is applied, the data quality must be reliable and within the error boundaries specific to the project. To ensure reliable data new techniques can be used. Satellite data gets more diverse and more detailed, AI programs could be used to analyse the data to identify and verify water systems (Dawood, Elwakil, Novoa, & Delgado, 2020). Additionally, innovative field-measuring devices can be used to increase measurement reliability (Paul, Buytaert, & Sah, 2020)

5.3 Data integration

Expanding this research to other neighborhoods necessitates data integration. Currently, the integration of municipal data is carried out manually, which calls for more automated processes for data acquisition and conversion. Different authorities store data in various formats and use different modeling software, requiring interaction between the parties. Collaboration between the municipality and the waterboard can yield mutual benefits in terms of data and models. Figure 48 visually represents the potential data interaction among various entities involved in urban hydrological modeling. The diagram highlights that urban hydrological models can serve as a foundation for collaboration among these entities. The use of three distinct colors in the diagram indicates the respective authorities responsible for the data, products, and goals. Specifically, the waterboard assumes responsibility for constructing the model, while the municipality shares the responsibility for providing accurate data. Integrating data from the waterboard, municipality, and external sources enhances the reliability of the models, benefiting all involved parties. The lower section of the figure showcases potential benefits for each authority. By leveraging the collective expertise and resources of the waterboard, municipality, and external entities, the collaboration facilitated through the utilization of urban hydrological models can yield substantial advantages for each authority involved (Bach, Rauch, Mikkelsen, McCarthy, & Deletic, 2014).

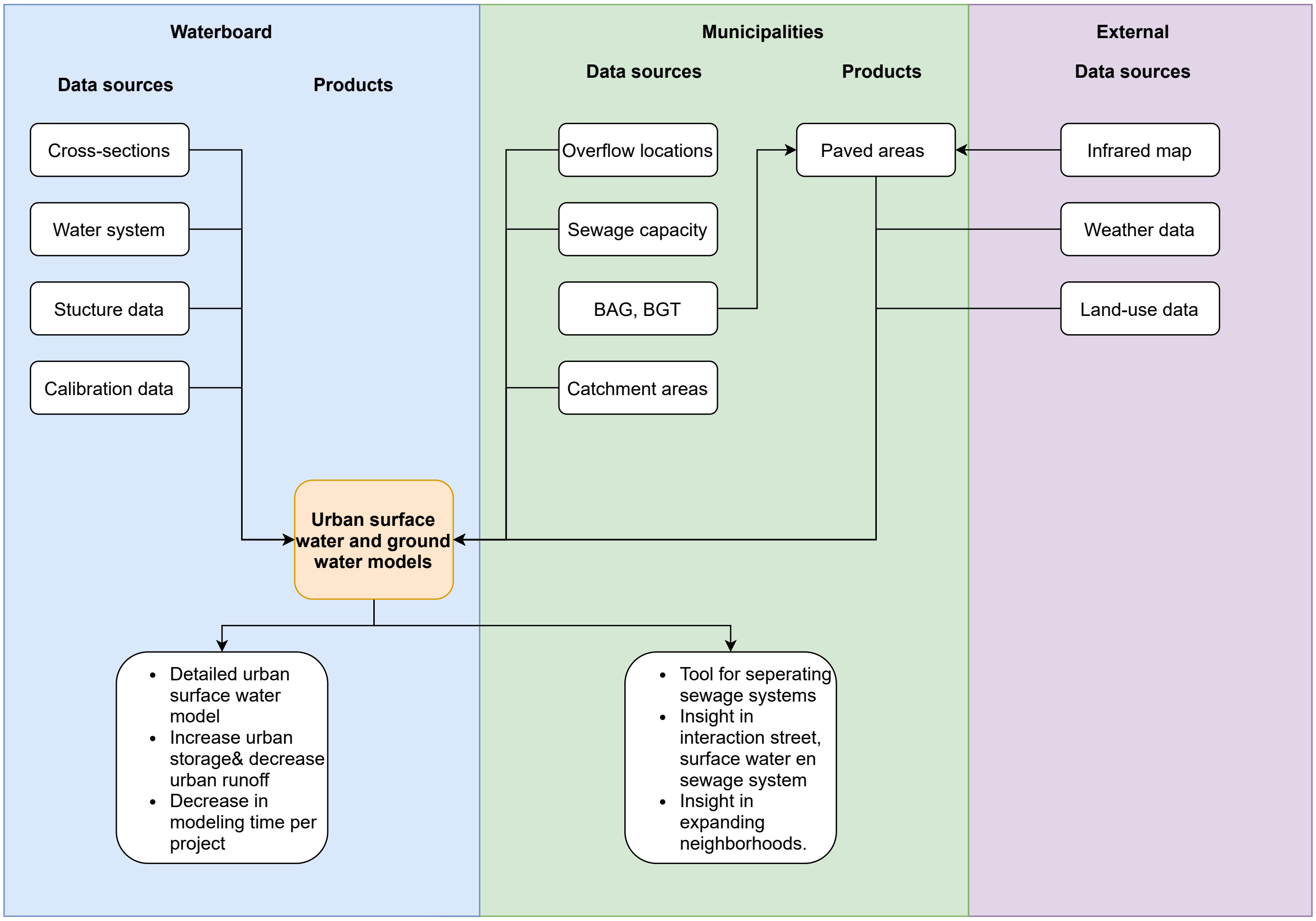


Figure 48: Data interaction waterboard and municipality

Collaboration is driven by the shared goal of improving the water system to make it more sustainable and resilient for the future. Nationwide agreements such as the "Administrative Agreement Water" (Rijksoverheid, 2011) and the "Delta Program" (Delta commission, 2023) have already been established to promote collaboration among water authorities. However, these agreements and programs do not specifically address the integration of data systems. Sharing data and models is a crucial step in informed decision-making, and a long-term program of data interchange would lead to higher-quality models.

Conclusion

This section provides a summary of the findings and discussions presented in the previous sections. The first step involved updating the data and model of the urban hydrological system. This update incorporated data from multiple sources, including the waterboard, municipalities, and national research institutes. The model's components, such as cross-sections, structures, overflow locations, and paved and unpaved areas, were updated with measured data from these sources. The updated model demonstrated improved fitness, with the Nash-Sutcliffe Efficiency (NSE) score increasing from -1.4 to 0.4 (with 1 being a perfect score), indicating a better fit to the data. However, updating the cross-sections resulted in a decrease in fitness due to variations in groundwater modeling. While the updated data enhances the accuracy of the model, it serves as a foundation rather than a final product. Further calibration, verification, and validation processes are necessary to improve the model's reliability. Case-specific measurements can be used for calibration and validation purposes, but achieving verification on a larger scale requires increased automation, as extensive on-site visits are currently not feasible with the available resources. Exploring alternative methods, such as increased automation and data integration, can help address this challenge.

Once the validated and calibrated model was obtained, it could be utilized to investigate the underutilization of storage capacity. The analysis of water levels in various ponds revealed that they were primarily designed for groundwater drainage from urban areas rather than storage during peak events. Consequently, the available storage capacity remains consistently full, leading to direct overflow during precipitation events. Although natural processes such as evaporation and infiltration contribute to water depletion, the constant drainage of groundwater maintained the water level relative to the weir height. Automatic downstream weirs regulate local water levels and discharges but do not address the underlying issues, merely displacing the problems to other areas. Mapping discharge per hectare for each weir provided valuable insights into the actual utilization of storage capacity. The relative discharge showed that most sections of the urban system are not in accordance with the design requirements.

To effectively address heavy precipitation events, the implementation of v-notch weirs was proposed. By raising the water level, the peak storage can be increased, decreasing the discharge during peak events. The v-notch design allows the water level to return to a lower level after an event. The extent of peak discharge reduction depends on the height of the v-notch. A v-notch height of 0.3 meters achieved a reduction of 20%, while a v-notch height of 0.5 meters resulted in a reduction of 40%.

Increased utilization of storage capacity may give rise to upstream issues such as water accumulation on streets, impacts on groundwater levels, and system saturation during a series of events. However, the proposed interventions resulted in limited increases in water heights. Most parts of the urban system have water levels more than two meters below street level, and the proposed interventions raised the water level by a maximum of one meter during peaks. Additionally, the short duration of peak events allowed for minimal effects on groundwater levels. Further measurements of ground and surface water levels provided additional insights into their relationship. The interventions ensured that the system can be emptied, making the calculated capacity available during each precipitation event and reducing downstream issues.

Expanding the interventions to more urban catchments requires a data management plan. The lack of integration of data into general formats complicates the data acquisition process for larger catchments. Different authorities with diverse urban responsibilities often collaborate on a project basis, making it challenging to implement shared data and modeling programs. Obtaining precise data necessitates intensive collaboration between the municipality and the waterboard, driven by a shared pursuit of mutual benefits and model interchange.

The findings cannot be directly translated into general policy recommendations. It is concluded that the current methodology for assessing storage capacity is inaccurate, attributable to flawed storage definitions and unrealistic urban models in the existing policy. Before implementing a new policy, the effects of the proposed interventions need to be measured and documented. Subsequently, a new storage definition can be formulated, taking into account the actual storage and drainage capacities of the model, while theoretical storage can either be omitted or considered always full. The new storage definition should align with the environmental epoch and reflect the waterboard's stance within this epoch. Accurate models are crucial for implementing and assessing the new policy, and increased modeling of urban systems should precede the implementation.

Recommendations

The recommendations for the waterboard are derived from the results, discussion and conclusion. It seeks to convert the theory into action. The recommendations are a combination of the technical and organizational proposed changes.

- **Data integration**
  The foundation of every modeling project lies in effective data management. The waterboard possesses software and data managers who proficiently acquire, store, and organize data. The proposed improvement pertains to the data sourced from external parties. Given that the water system operates as an integrated framework with distinct authorities responsible for different aspects of the water cycle, it is crucial to recognize that the surface water model cannot function in isolation and necessitates the integration of sewage data. Future challenges demand a more comprehensive modeling approach. To address this, the waterboard can initiate a shared platform for municipalities, where data updates are automated or easily facilitated, even for smaller municipalities. This shared data would enhance the reliability of the surface water model, consequently improving the quality of advice provided to the waterboard. Moreover, acknowledging that enhanced modeling also benefits municipalities can increase internal motivation. By integrating data and sharing models, as depicted in Figure 48, it is possible to establish enduring collaborations, moving away from the prevailing project-based interactions. Key data elements crucial to successful implementation include paved land use, overflow locations, and sewage capacities.

- **Urban modelling**
  The Netherlands is undergoing increasing urbanization, leading to a higher demand for storage capacity within urban areas. Moreover, due to the interconnected nature of most urban water systems, it is not feasible to store water downstream of urbanized regions. The capacity of the water system can only be increased with more reliable models or more accurate data. This report proposes an integrated urban modeling project encompassing the entire management area's urban systems. The time required to implement and convert the data is generally independent of its volume. Modeling ten cities, for instance, incurs only a marginal increase in building time compared to modeling just one city. The automation advancements in the sector, coupled with the utilization of satellite data, can facilitate this process. While executing this comprehensive project in collaboration with a consultancy firm may incur costs, these expenses can be offset by the reduction in modeling time for urban area projects. Furthermore, the model's reliability improves, resulting in fewer flawed conclusions and projects. The currently employed models exhibit significant flaws and are occasionally treated as absolute truth rather than rough simulations. Therefore, particular attention must be given to specific factors in these urban models, including the incorporation of overflow data and the acquisition of precise and reliable information on paved areas.

- **Technical system changes**
  The urban system is not designed to store water during peak events, as the rectangular weirs retain the water within the system, and the ponds remain filled. However, it is possible to enhance the storage capacity while maintaining the existing water system. This can be achieved by redesigning the weirs into V-notch weirs, which effectively reduces the peak discharge. The implementation of this redesign can involve concrete add-ons or metal sheets. For this, it is important to consider the width of the manhole. Different v-notch shapes result in varying storage capacities. Engaging in discussions with area managers can provide further insight into the technical possibilities associated with these weirs.

- **Implementation strategy**
  In addition to technical limitations, other constraints may arise during the project. The project aims to develop a future-proof urban water design for all urban areas within the waterboard's management area. However, this endeavor is time-consuming and costly. In order to ensure the proper functioning and long-term viability of the water systems following the implementation of interventions, initial tests must be conducted. The relative discharge shows the most influential and critical weirs to start with. By measuring the changes in water heights within these systems, insights can be gained into the water systems' reactions. These data series can be utilized to validate the reduction in peak discharge. Furthermore, these tests enable the identification of potential implementation issues and irregularities before embarking on the full project. By assessing the

water heights upstream of the modified weirs, valuable insights can be obtained regarding the behavior of the water and groundwater systems. The successful demonstration of the project's concept can facilitate funding for both the urban model and the v-notch intervention projects. All steps can be executed in collaboration with the municipality, and the increased storage capacity can aid in the transition toward separated sewage systems.

- **Policy change**
  Current definitions of storage capacity are primarily based on the volume of water that can be retained within the system. However, when considering peak discharges, an alternative form of storage becomes necessary. In this case, the focus shifts from storing water within the system to reducing direct runoff. These two aspects are not directly interrelated. Many neighborhoods were constructed in the previous century, but the current era of water management calls for resilient and climate-adaptive systems. Consequently, a reevaluation of system requirements is imperative. Adopting an integral and multipurpose approach to storage in urban systems can effectively address challenges related to floods, heat stress, and droughts. The new definitions should align with the waterboard's responsibilities for developing a climate-resilient water system, as well as the waterboard's strategic objectives to achieve this.

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Appendix

8.1 Simulation models

Classification	Model name	Strengths	Weaknesses
Metric	Unit Hydrograph (UH)	Straightforward and objective	Not robust
	Data based mechanics (DBM)	Identifying structures without constraints by hypothesis	Does not include all complex hydrological processes
	Artificial Neural network (ANN)	Nerves as different aspects to check what the driving parameters are	Nodes are not directly related to the hydrological process
Conceptual	Stanford watershed model	Different sizes and parameters possible	Hard to use, sensitive to errors
	TOPMODEL (Topographic model)	Comprehensive picture of the water balance and storages	Many assumptions, performs less with different land-use
	Hydrologiska Byråns Vattenbalansavdelning (HBV)	Easy to calibrate and fits for different catchments	Does not include spatial variability
Physics-based	Soil and water assessment tool (SWAT)	Simulate impact of land-use change, comprehensive picture of water balance	Detailed input data, computationally heavy
	MIKE SHE (SHE)	Includes a wide range of hydrological processes	Detailed input data, computationally heavy
Hybrid	Storm water management model SWMM	User friendly, urban focussed, compatible with modern GIS software	Detailed input data, computationally heavy
	Deltares SOBEK	Wide range of hydrological processes, compatible with GIS, urban focussed	Detailed input data, less literature available

Table 8: Potential models per classifiction

8.2 MCDA

The criteria are included in a Multi-Criteria Decision Analysis (MCDA) to identify which model fits best. First, each criterion is weighted based on importance. subsequently, literature is used to asses scores for each model alternative. The models are evaluated against the criteria according to the assessing format in table 9. The orange indicated sections are knockout scores, if the models earn this score the model cannot be chosen. Not all criteria have the same importance. Of the five criteria, the input and suitability for urban catchments are the key factors. All criteria are important so the weights are five for the key criteria and four for the other criteria.

Criteria/ weights	**1**	**2**	**3**
Suitable for urban catchments	Model has no use for urban catchments	Model can be used for urban catchments but not as primary goal	Model is a good fit for urban catchment
Input data	Does not use the data	Uses precipitation and register data	
Drivers	Excludes two or more	Includes all except one	Includes(Storage capacity, ground water/ surface water, stopping water, other side of the system, vegetation)
Output	Does not use the data	Output data is discharge and waterheight	
Easy applicable for other catchments	Not applicable for other catchments	Applicable for related catchments	Applicable to all urban catchments

Table 9: Assessment framework

Table 10 displays the result of the MCDA. Two models indisputably fit best with the maximum score of one. Of these two the SOBEK model is chosen for its compatibility with the Dutch water system.

	Weights	Stanford	TOP	HBV	SWAT	SHE	SWMM	UH	DBM	ANN	SOBEK
Suitable for urban catchments	5	2	1	2	3	2	3	1	3	1	3
Input data	5	2	2	2	2	2	2	1	2	2	2
Drivers	4	2	2	2	3	3	3	1	2	1	3
Output	4	2	2	2	2	2	2	2	2	1	2
Reproducable	4	1	1	2	2	1	3	3	2	2	3
Normalized score		**0,70**	**0,61**	**0,77**	**0,93**	**0,77**	**1,00**	**0,59**	**0,86**	**0,54**	**1,00**

Table 10: MCDA model choice

8.3 Paved and unpaved nodes

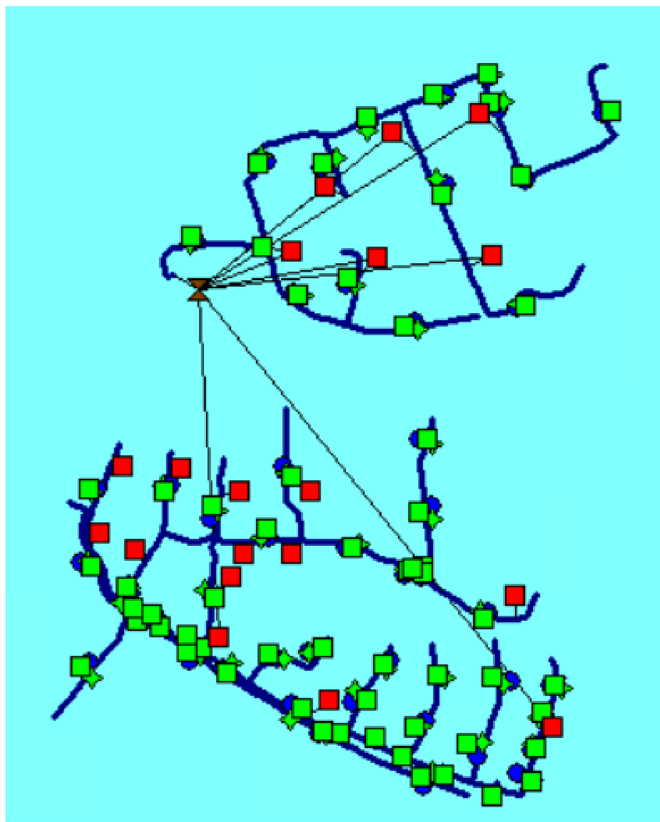
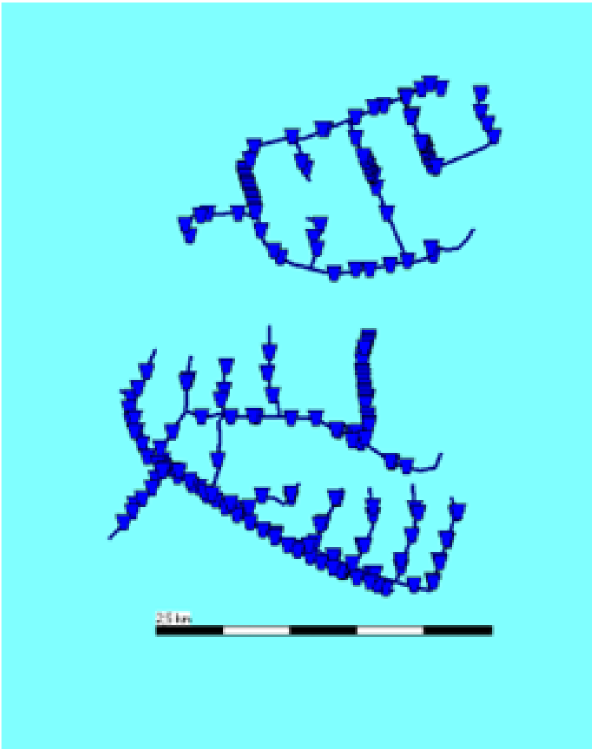
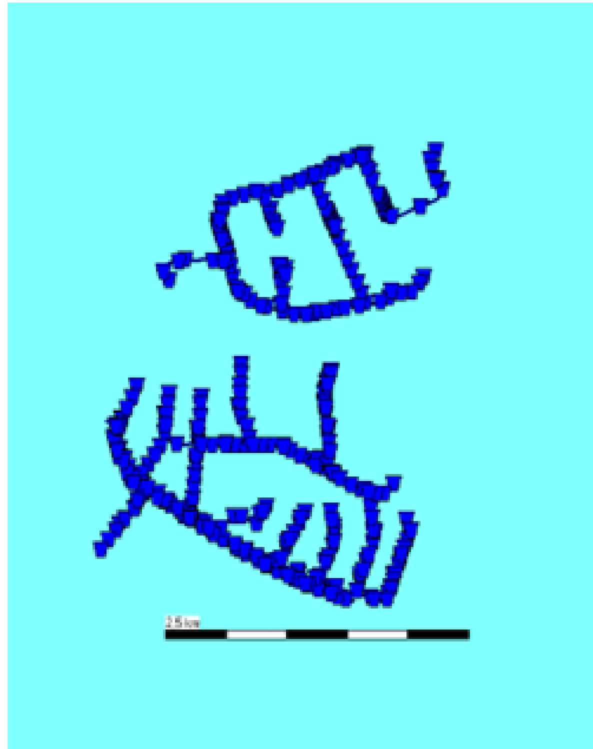


Figure 49: Rainfall-Runoff paved and unpaved nodes

8.4 Cross-sections



(a) Old cross-sections in catchments



(b) New cross-sections in catchments

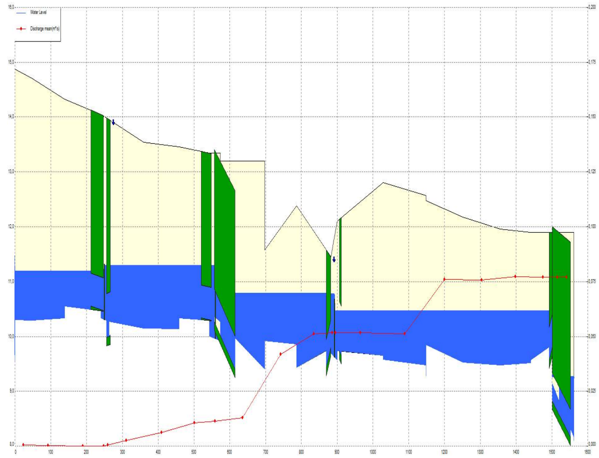
Figure 50: Old and new cross-sections of both catchments

8.5 Calibration results

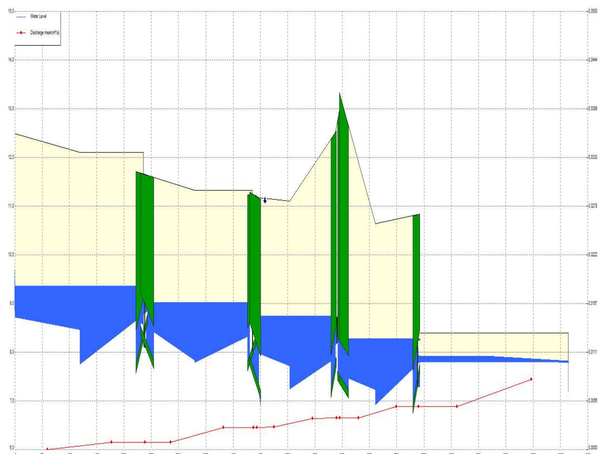
Scenario	Fast drainage [days]	Slow drainage [days]	Drainage depth [m]	NSE Frankeneng	NSE Veldhuizen
1	5	100	0,8	-3,00	-0,76
2	5	125	0,8	-2,95	-0,71
3	5	150	0,8	-2,92	-0,67
4	5	200	0,8	-2,97	-0,53
5	10	100	0,8	-2,95	-0,50
6	10	125	0,8	-2,93	-0,48
7	10	150	0,8	-2,90	-0,43
8	10	200	0,8	-2,94	-0,41
9	15	100	0,8	-2,92	-0,38
10	15	125	0,8	-2,89	-0,35
11	15	150	0,8	-2,86	-0,31
12	15	200	0,8	-2,87	-0,33
13	20	100	0,8	-2,85	-0,30
14	20	125	0,8	-2,83	-0,27
15	20	150	0,8	-2,79	-0,23
16	20	200	0,8	-2,94	-0,69
17	5	100	1,2	-2,93	-0,75
18	5	125	1,2	-2,92	-0,78
19	5	150	1,2	-2,92	-0,81
20	5	200	1,2	-2,93	-0,46
21	10	100	1,2	-2,92	-0,48
22	10	125	1,2	-2,91	-0,49
23	10	150	1,2	-2,89	-0,50
24	10	200	1,2	-2,91	-0,34
25	15	100	1,2	-2,89	-0,34
26	15	125	1,2	-2,88	-0,35
27	15	150	1,2	-2,86	-0,35
28	15	200	1,2	-2,83	-0,26
29	20	100	1,2	-2,81	-0,26
30	20	125	1,2	-2,81	-0,26
31	20	150	1,2	-2,79	-0,25
32	20	200	1,2	-3,14	-0,88

Table 11: Result calibration

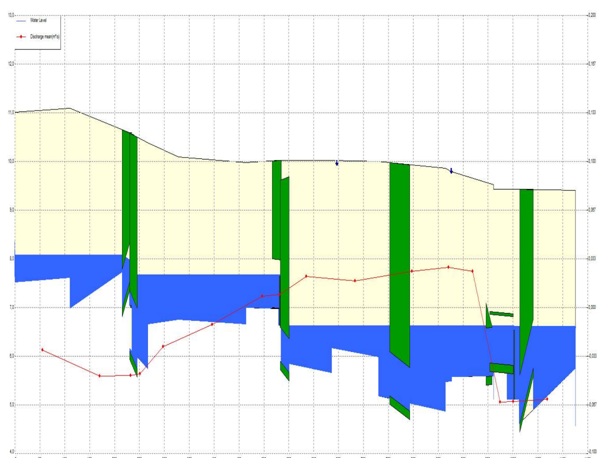
8.6 Side views



(a) Side view storage Veldhuizen



(b) Side view storage Rietkampen



(c) Side view storage Frankeneng

Figure 51: Side views of storage in ponds