Efficiently designing sustainable urban drainage systems

Exploring trade-offs between 1D and 1D2D urban drainage models in sustainable flood-resilient design: A heuristic approach combining both models

Author: Stijn Overbeek

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By

Stijn Overbeek

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Student number:	5866707
Thesis committee:	
J.A. van der Werf (Job)	Delft University of Technology
T.A. Bogaard (Thom)	Delft University of Technology
P.J.C. Smit (Patrick)	Witteveen + Bos

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Preface

This thesis was written as the final project for obtaining my master's degree in Environmental Engineering at Delft University of Technology. During my bachelor's studies in Water Management, I gained substantial experience in urban climate adaptation. For my final thesis, I aspired to combine this topic with the modelling skills that I acquired during my master's programme. This culminated in a study on the use of various urban drainage models for designing sustainable urban drainage systems to prevent flooding—a subject I have enjoyed exploring in great depths.

The project was carried out as an internship at Witteveen+Bos. I would like to thank my colleagues for their inspiration and feedback, which helped me to elevate this study to a higher level. Special thanks go to Patrick Smit, who acted as my supervisor and taught me a great deal about the use of urban drainage models. Job van der Werf, the chair of my supervisory committee, inspired and challenged me during our weekly meetings to get the most out of this project. My supervisory committee was completed by Thom Bogaard, who offered a fresh perspective on my work and provided valuable input for reflection.

I am very pleased with the results I was able to obtain during this project and proudly present this thesis on the trade-offs between 1D and 1D2D urban drainage models in sustainable flood-resilient design.

Stijn Overbeek, 25 July 2025

Abstract

Climate change increases the likelihood of extreme rainfall events, while ongoing urbanization leads to greater surface imperviousness. Together, these trends result in more frequent and severe urban flooding. Sustainable Urban Drainage Systems (SUDS) are implemented to enhance the resilience of urban drainage infrastructure and mitigate urban flooding. Among the available modelling approaches, coupling a one-dimensional (1D) sewer system model with a two-dimensional (2D) surface model (1D2D) is considered the most accurate method to assess urban flooding. However, the practical application of 1D2D models in the design of SUDS is limited by their high computational requirements. In comparison, a 1D urban drainage model demands significantly less computational power, allowing for many more simulation iterations to be completed within the same timeframe.

This study investigates the trade-offs between 1D and 1D2D models in the design of SUDS for flood prevention. It proposes a heuristic approach that integrates both a 1D and a 1D2D model (method 1). This approach aims to leverage the speed of the 1D model and the accuracy of the 1D2D model to optimize SUDS design. The methodology was applied to an urban drainage model of Bloemendaal, the Netherlands. To evaluate the efficiency of the proposed method, its results were compared to those obtained using a second approach that relies solely on the 1D2D model (method 2).

Method 1 was more effective than method 2 in reducing the number of flooded buildings. Specifically, method 1 achieved the greatest reduction in areas affected by higher flood levels (>0.30 m), while method 2 was more effective at decreasing the area exposed to lower flood levels (>0.10 m).

Method 1 may assist decision makers in selecting and implementing SUDS more effectively for flood prevention, ultimately leading to more resilient urban drainage systems. Future research could expand method 1 to incorporate additional benefits of SUDS, enabling a multi-objective design approach.

List of Acronyms

- AMPE Absolute Mean peak flow Error
- BS Bioswale
- DEM Digital Elevation Model
- DTM Digital Terrain Model
- GR Green Roof
- LIDS Low Impact Development systems
- MPE Mean Peak flow Error
- NSE Nash-Sutcliffe efficiency
- PP Permeable Pavement
- RB Rain Barrel
- SUDS Sustainable Urban Drainage systems
- SWMM Storm Water Management Model
- UDS Urban Drainage system

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1. Introduction

Over the past centuries, urbanization has steadily increased, resulting in a 150% increase in the surface area covered by urban development over the past 40 years (Tian et al., 2022). At the same time, global warming is expected to cause more extreme precipitation events in many regions worldwide (Martel et al., 2021). These trends lead to both a greater frequency and severity of urban flooding events (Mignot & Dewals, 2022). Urban infrastructure—such as transportation and electricity networks, as well as other critical facilities—is becoming increasingly vulnerable to urban flooding. Addressing this challenge requires more resilient urban drainage systems, which can be achieved through the implementation of Sustainable Urban Drainage Systems (SUDS) (Dharmarathne et al., 2024).

In recent years, the focus in urban drainage systems has shifted from relying solely on conventional infrastructure to combining it with Sustainable Urban Drainage Systems (SUDS). In addition to enhancing flood resilience, SUDS offer further benefits such as pollutant load reduction, increased biodiversity, and heat mitigation—all of which address issues arising from climate change and urbanization (Montoya-Coronado, et al., 2024).

Urban drainage models are frequently used to quantify the effects of SUDS on urban drainage systems and to support the design of these systems. Designing SUDS that effectively prevent flooding requires a methodology in which the impacts of multiple options are quantified, allowing for the selection of the most effective measures to be implemented. One-dimensional (1D) urban drainage models coupled with two-dimensional (2D) surface flow models are considered the most suitable for simulating flooding, as they provide accurate simulations and detailed information that can be used to assess flood hazards (Cea & Costabile, Flood Risk in Urban Areas: Modelling, Management and Adaptation to Climate Change. A Review, 2022). However, these 1D2D models demand substantial computational resources, resulting in long model run times and making iterative SUDS design impractical. In contrast, 1D urban drainage models require significantly less computational capacity, enabling far more simulations to be performed within the same timeframe compared to 1D2D models.

Recognizing that both 1D and 1D2D models have specific advantages and disadvantages, this thesis investigates the trade-offs between these model types in the design of SUDS for reducing flooding in urban drainage systems. A methodology is presented that combines both a 1D model and a 1D2D model in a heuristic approach to designing SUDS for the village of Bloemendaal in the Netherlands. This methodology is designed to leverage the strengths of each model: the 1D model is employed for the iterative design process and to assess the efficiency of individual SUDS measures, while the 1D2D model is utilized to verify the hydrodynamic interactions between SUDS and overland flow. The performance of the resulting design is then compared to a design created using an alternative heuristic methodology based solely on the 1D2D model.

1.1. Theoretical background

The process of designing SUDS using urban drainage systems has been thoroughly studied in the past. First I will give a description of the different processes that are simulated in an urban drainage model, were I will give much attention on the different models that can be used to calculate flood propagation. Next I will give an overview of the processes regarding SUDS and their effects on flooding. Lastly, I will describe the types of models and methodologies that have been used for SUDS designing.

1.1.1. Urban drainage models

Urban drainage models simulate the drainage systems of urban catchments and are used for a variety of applications, including flood risk assessment, emergency management, urban renewal, and construction (Zeng et al., 2025). This section provides an overview of the processes represented in urban drainage models and discusses their application in both 1D and 1D2D urban drainage models.

Urban drainage models generally simulate four distinct processes: runoff generation, sewer pipe flow, channel flow, and overland flow. These processes are typically represented by separate models, which are integrated to form a comprehensive urban drainage model.

Runoff generation is typically simulated using hydrological models, in which hydrological processes are conceptualized through various parameters. Surface characteristics incorporated into these models include surface roughness, slope, infiltration capacity, and depression storage. The generated runoff is routed into the sewer system via manholes, which are represented as nodes in the urban drainage model. Sewer pipe flow is simulated using a one-dimensional (1D) numerical model along the direction of flow. Pipes are modeled as conduits that connect the nodes, and flow through these conduits is calculated using the 1D Saint-Venant equations.

Overland flow is typically simulated using a two-dimensional (2D) finite element numerical model, which solves the 2D shallow water equations. The surface is defined by points, known as nodes, which are connected to each other through elements. For each node, elevation, roughness, and infiltration capacity are specified based on a digital terrain model (DTM), land use, and soil data (Lanzafame et al., 2024). Water is transferred from the conduits to the surface via the 1D-model nodes when water levels exceed the maximum water depth at a node. Due to the complex geography of urban catchments, a small mesh element size is required to achieve sufficient model accuracy. This results in high computational demands for running the model (Hu et al., 2018).

1D urban drainage models consist of only the hydrological model and the 1D pipe flow model. When water levels exceed the maximum allowable depth, excess water is temporarily stored in a storage reservoir until the pipes are able to convey the water again (Murla & Willems, 2015).

A 1D2D model consists of a 1D model coupled with a 2D overland flow model. This coupling can be performed either dynamically or non-dynamically. In a non-dynamically coupled 1D2D model, the 1D model is only used to generate input data for the 2D model, without interaction in the opposite direction; as a result, both models can be solved independently. In a dynamically coupled model, both the 1D and 2D components interact with each other in real time, and the governing equations for both components are solved simultaneously. Dynamically coupled models are often more accurate than non-dynamical models (Bulti & Abebe, 2020).

1D2D models can be further categorized based on how runoff is generated. Some 1D2D models use a hydrological model to generate runoff at the nodes, while others simulate the rainfall-runoff process directly on the mesh. The latter approach requires even greater computational capacity but can improve model accuracy, especially during high-intensity precipitation events (Smit, 2021).

A 1D2D model can provide detailed information about flood depths and locations. This level of detail is important, as water managers often tolerate surface water up to a few centimeters deep; such low levels typically do not damage buildings or block infrastructure (Rioned, 2019), Additionally, the water depths calculated by a 1D2D model can be used to estimate flood damage by applying flood damage curves (Sahol, et al., 2014). In comparison, a 1D model can only identify the specific manholes where water levels exceed the maximum possible level, and gives an indication of the flood level at that manhole (Ferrans et al., 2022). A 1D model thus offers a rapid indication of potential flooding locations and extents.

An alternative method for flood modelling involves simulating surface streets as 1D conduits. However, the applicability of this approach is limited, as the surface of urban catchments is often far more complex than a simple 1D channel (Kourtis et al., 2017).

1.1.2. Sustainable urban drainage systems (SUDS)

Sustainable Urban Drainage Systems (SUDS)—also known as Low Impact Development (LID), Water Sensitive Urban Design (WSUD), Best Management Practices (BMPs), or Green Infrastructure (GI)—are a subset of urban drainage systems designed to restore the natural hydrological functions of urban catchments by increasing surface roughness, permeability, and storage capacity. SUDS reduce runoff and peak flows, and they enhance water quality (Fletcher, et al., 2014). Additional benefits include improved biodiversity, recreation opportunities, increased infiltration, heat reduction, and enhanced evaporation (Krivtsov et al., 2022; Chan et al., 2019; Rathnayake et al., 2017). By providing these services, SUDS increase the resilience of cities and reduce their vulnerability to climate impacts (Sulis et al., 2024). Common types of SUDS include green roofs, rain barrels, bioswales, and permeable pavement.

This study focuses on the development of SUDS for flood reduction. SUDS can reduce flooding in several ways, which can be categorized into three main functions: runoff reduction, runoff harvesting, and flood attenuation:

- Runoff reduction involves capturing precipitation at the location where it falls. This is achieved by increasing surface roughness, enhancing permeability, or removing flow paths, thereby minimizing the volume of runoff generated.
- Runoff harvesting refers to collecting and containing runoff by diverting it to locations where it can be stored, such as rain barrels or storage ponds.
- Flood attenuation entails capturing stormwater that has overflowed from the sewer system.

The first function is referred to as a hydrological mechanism, while the second and third functions are considered hydrodynamic mechanisms, as they relate to overland flow. Different types of SUDS provide one or a combination of these functions (Table 1). These distinctions have implications for the accuracy of SUDS simulation in urban drainage models. Hydrological mechanisms can be simulated within the hydrological component present in both 1D and 1D2D models. In contrast, hydrodynamic mechanisms require a 1D2D model for accurate simulation (Sandoval et al., 2023).

While a large number of different SUDS exists, The SUDS that have been selected for this study are based on different hydrological and hydrodynamical processes and are located in different locations in the urban drainage system. The processes, locations and functions of SUDS are important because they require different capabilities of urban drainage models when their effectivity is modelled and they can be combined with each other in urban drainage systems. The functions performed by the SUDS that were selected for this study are in Table 1.

Table 1 Functions performed by different SUDS types

	Hydrological	Hydrodynamical	
Type	Runoff reduction	Runoff harvesting	Flood attenuation
Green Roof	X		
Permeable pavement	X	X	X
Rain barrel		X	
Bioswale	X	X	X

1.1.3. Modelling SUDS using urban drainage models

Urban drainage models are often used to simulate the effects of SUDS on flooding within urban drainage systems. They can assess the effectiveness of both existing and proposed SUDS measures in reducing flooding, as well as help determine the optimal locations and dimensions for implementing SUDS.

Previous studies have assessed the effectiveness of SUDS in reducing flood risks. Huang et al. (2019) found that the relative reduction in stormwater runoff is greatest for rainfall events with a 15-year return period, and that more extreme events (with longer return periods) result in only a marginal increase in flood volume reduction. Similarly, Fiori & Volpi (2020) observed that infiltrating SUDS are more effective at reducing floods during precipitation events with shorter return periods.

When urban drainage models are used to simulate the interaction between SUDS and the drainage system, it is important that all flood-reducing functions of SUDS are accurately represented. Many studies, however, do not incorporate the hydrodynamic functions of SUDS, as interactions with overland flow are often not integrated into the model. For example, the commonly used LID control module of the Storm Water Management Model (SWMM) is not capable of simulating the hydrodynamic effects of SUDS in this way (Sandoval et al., 2023).

Neglecting the hydrodynamic functions of SUDS means that their effectiveness at reducing floods is not accurately estimated and may even be underestimated. This calls into question the validity of conclusions suggesting that SUDS are only effective for storm events with short return periods. In contrast, Haghighatafshar et al. (2017) used a 1D2D model to simulate the effects of SUDS and found that they were effective at reducing flooding during an event involving 100 mm of rainfall within 220 minutes. If 1D models were able to properly conceptualize the interactions between overland flow and SUDS, they could be more effectively used to simulate the impact of SUDS on flood hazards.

Previous studies often simulate the hydrological functions of SUDS using a 1D urban drainage model, employing downstream peak runoff reduction as a proxy for flood risk reduction (Fiori & Volpi, 2020; D'Ambrosio et al., 2022) or assessing effectiveness based on the reduction in the number of flooded nodes (Li et al., 2019). However, the information provided by these metrics is incomplete. A reduction in peak runoff does not necessarily correspond to less water on the streets—it may simply indicate less water within the drainage system. Similarly, the number of flooded nodes offers no insight into actual flood depths on the street, which is important since minor flooding, such as a few centimeters of water, is often not problematic.

1.1.4. Designing SUDS on the catchment scale

The process of designing SUDS at the catchment scale differs from merely studying their effectiveness, as it involves selecting the optimal locations and types of SUDS to achieve the most effective, and often the most cost-efficient, design (Wu et al., 2024). Common design strategies include scenario comparison (D'Ambrosio et al., 2022) and multi-objective optimization (Li et al., 2019; Karami et al., 2022). However, design scenarios are often not detailed, and the sheer number of possible scenarios makes it difficult to determine the best option. Studies using 1D models can assess a larger number of scenarios, while those applying 1D2D models are limited to a few due to computational constraints. Although optimization techniques can theoretically determine the optimal design, they further increase the computational burden, especially with already demanding 1D2D models. This is evidenced by the fact that most optimization studies utilize 1D urban drainage models (Ferrans & Temprano, 2022).

Ferrans & Reyes-Silva et al. (2022) presented a methodology that combines optimization with a 1D2D model, but their study was limited to a small area (22 ha) and used a grid element size of 10 m^2. For more detailed results, grid sizes should be an order of magnitude smaller (Yalcin, 2020). Even then, their optimization simulation required nine hours to complete, making optimization with smaller mesh sizes impractical. As a result, both optimization and multiple scenario analyses currently face significant limitations. Therefore, there is a need for alternative methods to effectively design SUDS at the catchment scale.

Martí (2022) proposed that when reaching an optimal solution is unfeasible due to excessive computation times, a heuristic approach is often a better alternative. Heuristic methodologies are characterized by more manageable computational demands and can yield solutions that closely approximate the true optimum. This study aims to evaluate the potential of using a heuristic methodology that leverages the rapid computational capabilities of 1D models alongside the accurate flood simulation and detailed output provided by 1D2D models.

Scenario studies are also generally unable to assess the effectiveness of individual SUDS measures, or can only do so at a highly aggregated level, where the catchment is divided into large subareas, each implemented with SUDS independently (D'Ambrosio & Balbo et al., 2022). A methodology that can evaluate the effectiveness of individual SUDS would enable users to identify a preferred order of implementation (Creaco et al., 2025).

A second important factor in SUDS design is the impact of the hyetograph shape on the outcome. Pritsis et al. (2024), found that using only a single hyetograph can result in designs with low robustness, which may lead to flooding during events with different rainfall patterns. Incorporating multiple rainfall events in the design process is only feasible when the computational demands per event are low, meaning this approach is currently only practical with 1D models.

1.2. Research questions

For this study, the main research question is: *What are the trade-offs between 1D and 1D2D models for designing SUDS for flood reduction?*

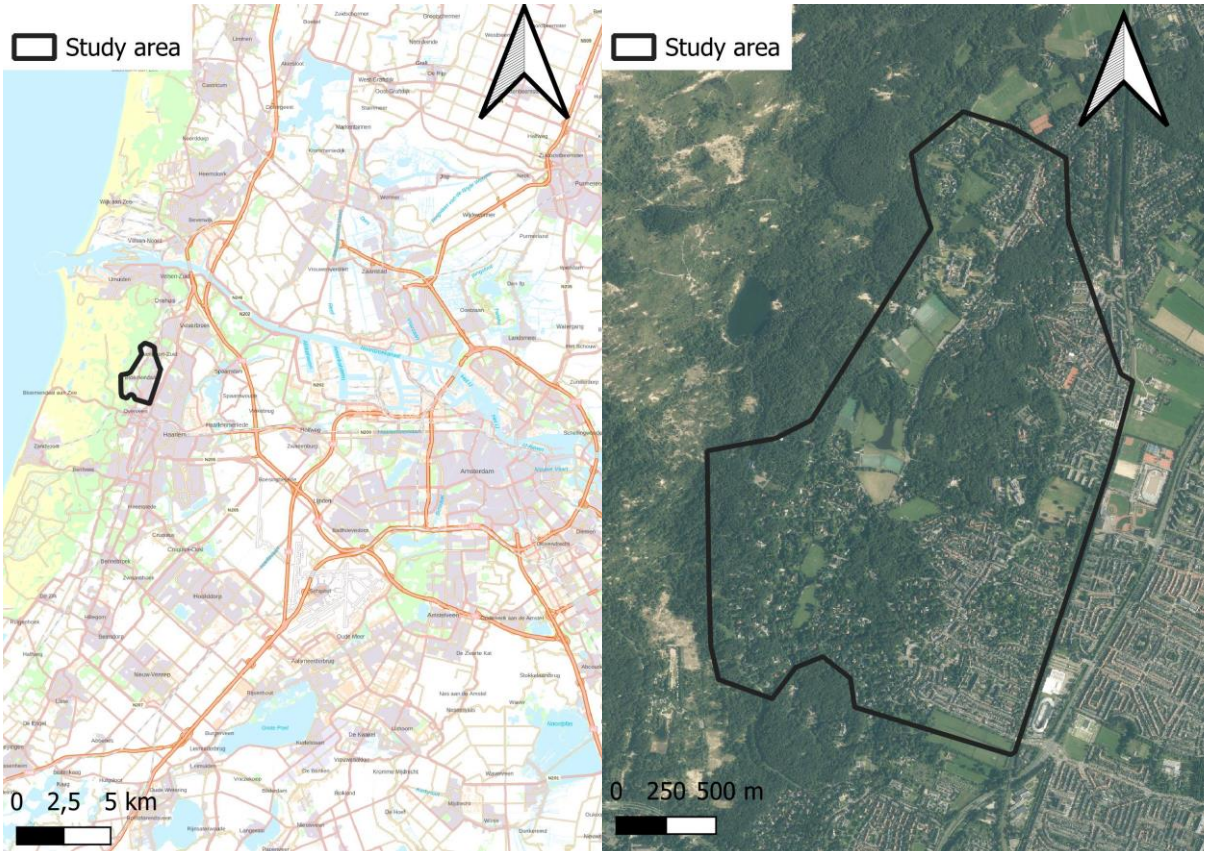
This question will be answered with the following sub-questions:

1. How can a heuristic methodology be used to design SUDS preventing flooding in 1D and 1D2D models?
2. How can a 1D model be used to quantify the effects of SUDS on flooding?
3. How can a 1D2D model be used to quantify the effects of SUDS on flooding?
4. How effective are SUDS designed with a heuristic method combining 1D and 1D2D models at reducing flooding compared to SUDS designed with a heuristic method using only a 1D2D model?

2. Study site: Bloemendaal

The research questions will be answered by applying the methodology described in Chapter 3 on the case study of Bloemendaal. Bloemendaal is a village in the municipality of Bloemendaal located in the Province of North-Holland in The Netherlands. It is located west of Haarlem and east of the North Sea beaches and Dunes (Figure 1a). The location has been chosen because of a diverse geology and a densely build urban center.

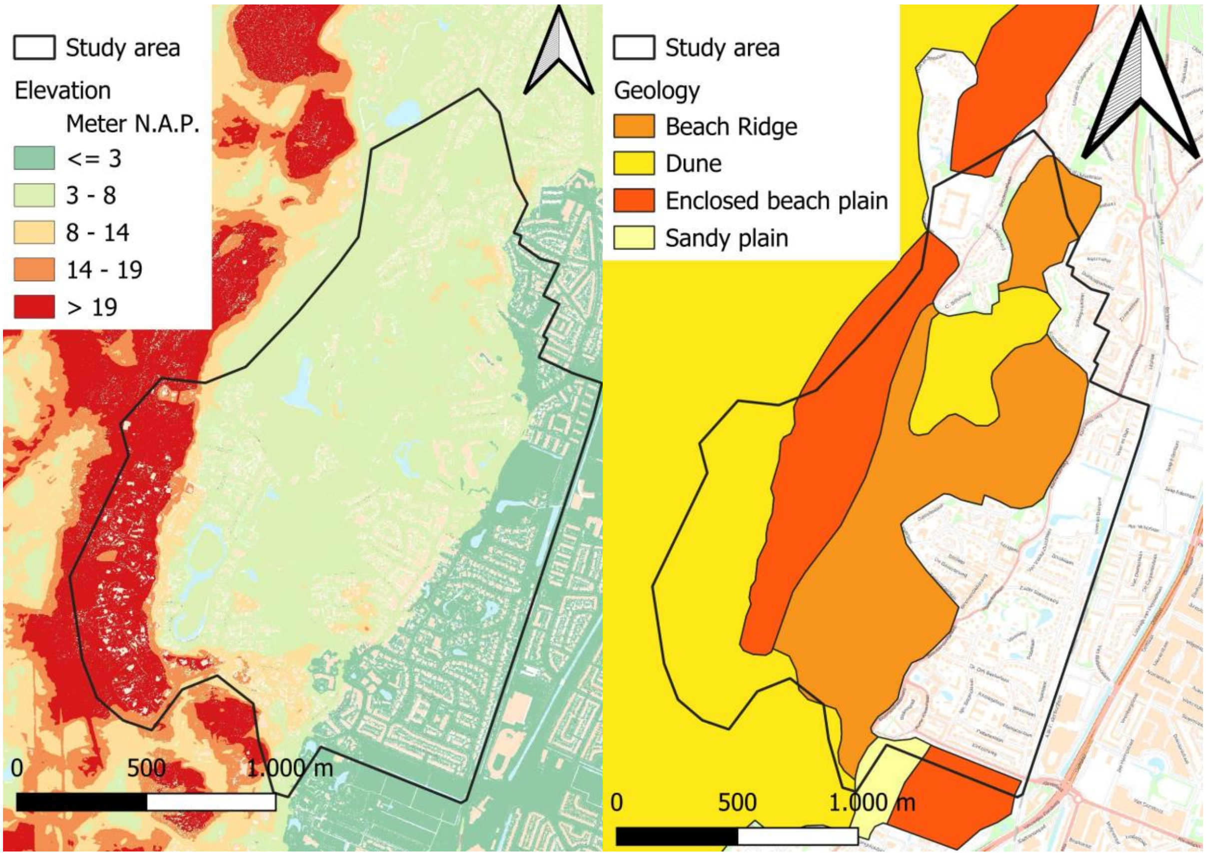
Figure 1: A. locator map of Bloemendaal, B. Areal image of Bloemendaal



The study area has a temperate oceanic climate (Cfb) and experiences a yearly average temperature of 10 C and 846 mm of rainfall (KNMI, 2025). Climate change is projected to increase the average temperature and increase the probability of extreme precipitation events in the summer. Winters will experience more precipitation on average while summers will see more periods of droughts (KNMI, 2023).

The study area has a diverse geology and relief (Figure 2) In the West of the study area lies the North sea dune system. This consists of sandy marine sediments. The terrain is hilly and the elevation of this area reaches up to 50 meters above sea level. Next to the dune system lies an enclosed beach plain with an elevation of 5 meters above sea level. Further east lie more hilly Beach ridges were the elevation is between 5 and 10 meters above sea level. In the east the elevation drops to around 0 meters above sea level, there layers of peat soil can be found between the sand layers (TNO, 2025).

Figure 2 A. Elevation map of the study area, B. Geological map of the study area



Development in the study area is mainly concentrated in the East, with development generally becoming more sparser further to the West. Development mainly consists of a urban center with buildings and medium sized roads and streets. Undeveloped area's in the west consist mainly of forest areas.

2.1. Water system

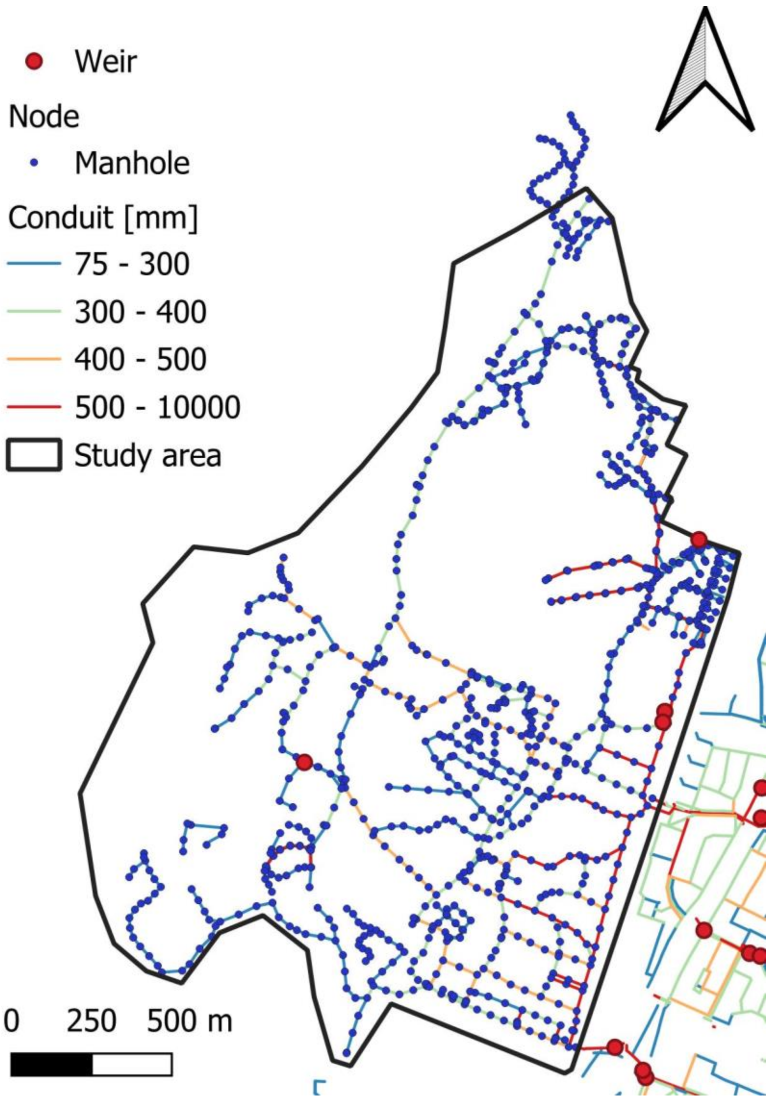
The dune area has a naturally fluctuating groundwater system, the phreatic groundwater level is located a few meters under the surface. The groundwater level fluctuates over a meter during the year. The groundwater system further east is regulated and more shallow (Wareco Ingenieurs , 2021).

In the East of the study area, the waterboard maintains a summer and winter water level of -0.61 and -0.64 meter relative to sea level (Hoogheemraadschap Rijnland, 2025). These canals are directly connected to the main water system of the water board. In the west water levels are dynamic and connected to east with a series of canals and weirs.

In total, the study area consists of 35 km of sewer pipes and 986 manholes. 29 km of the sewer network is combined sewer, 3 km is storm sewer and 2 km is infiltrating sewer. The sewer system has two connections towards the system of Haarlem. Water drains towards the system of Haarlem under gravity.

The sewer system has combined sewer overflows (CSO) that connects to the open water system in case of high discharge. The system is also connected to a storage settling tank with a capacity of 1,0 mm, the storage settling tank has a CSO as well. The storage settling tank is designed to be emptied in 48 hours.

Figure 3 Overview of the drainage system of Bloemendaal

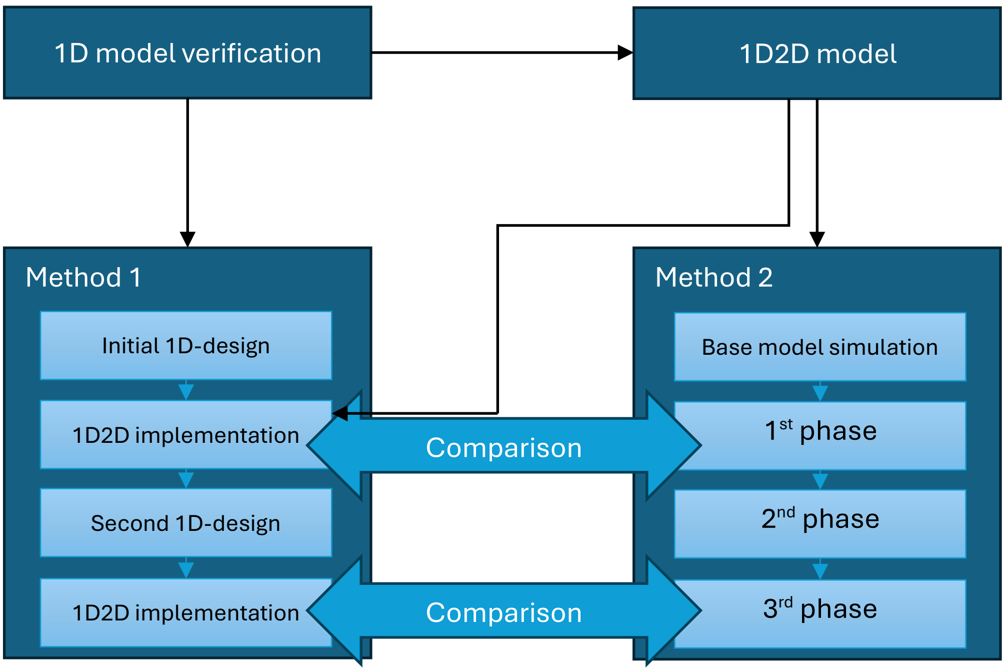


3. Methodology

The methodology involves developing two SUDS designs, they are called method 1 and method 2. Figure 4 provides a schematic overview of the process. Before starting the design steps, the 1D model was verified using measured data, the verification method is explained in Section 3.2. The 1D2D model was then validated as described in Section 3.2.1. The 1D and 1D2D models were used in the SUDS design processes.

Both SUDS designs were developed under the same constraints and conditions, which are outlined in Section 3.3. The first design was created using method 1, which combines the usage of the 1D model with the usage of the 1D2D model. The heuristic approach used in this method is explained in Section 3.4. The second design was developed using method 2, which relies solely on the 1D2D model and is described in Section 3.5. After the design phase, the results of both designs—simulated using the 1D2D model—were compared to determine which method is more efficient. The performance metrics used for this comparison are presented in Section 3.6.

Figure 4 Methodology schematic overview



3.1. Model description

During this study, two types of urban drainage models have been used, a 1D model and a 1D2D model. In this section, the characteristics of both models will be described, starting with the 1D model, next the 2D model is described before finally the process of coupling the 1D model to the 2D model is described.

The 1D urban drainage model was simulated using Storm Water Management Model (SWMM) software. SWMM uses a 1D numerical model to simulate conduit flow. Runoff routing was modeled using a hydrological approach, in which subcatchments drain towards manholes represented as nodes within the model. The connected surface areas were categorized into four types: flat roofs, sloped roofs, open paved areas, and closed paved areas (Table 2). Unpaved surfaces were assumed not to drain towards the urban drainage system.

Table 2 List of different types of area's

Type	Area (ha)	Depression storage (mm)
Closed paved	15,2	0,5
Open paved	13,6	0,5
Rooftop flat	5,7	2
Rooftop sloped	18,0	0

Evaporation was assumed to be negligible during short peak rainfall events. The dry weather flow was estimated at 120 liters per person per day, based on a population of 6,900 inhabitants (Mogos et al., 2023). This dry weather inflow was assumed to be evenly distributed across all nodes with dry weather connections. Flooding occurs when the water level in a node rose above ground level. In such cases, floodwater was stored in a 0D ponding area, representing the total street surface area connected to the respective node.

Simulating urban flooding requires a 1D2D urban drainage model. To create such model, the 1D model described in the previous section was coupled to a 2D surface model. Iber software has been used to simulate the 2D surface model. Iber uses the 2D Saint-Venant equations in a numerical finite volume model were the surface is represented as a 2D mesh. The surface mesh was generated in Iber with a mesh element size of 1.5 m^2, which is sufficient to create reliable results (Yalcin, 2020). Surface elevation was implemented using a digital terrain model from AHN 5 (Appendix I). Infiltration was modelled using a linear infiltration model, applying a K-value of 0.1 m/day for peat (Wong, 2009) and 1 m/day for sand (Ku, 2013), The soil map used for surface classification is provided in Appendix J. Unpaved surfaces were identified using a land use map from the Basisregistratie Grootschalige Topografie (Appendix K).

The 1D model from SWMM was coupled with the 2D surface model in Iber, using the IberSWMM plugin developed in 2024 by Sañudo et al. (2025). This plugin dynamically coupled the 1D sewer model from SWMM with the 2D surface flow model from Iber. Water was transferred from the 1D model to the 2D model over the manholes, represented in SWMM by the nodes.

3.2. Verification of the urban drainage model

The urban drainage model, originally developed and verified by Mogos (2022) using the urban drainage modelling software InfoWorks, was reverified using the Storm Water Management Model (SWMM). The verification was necessary to ensure a realistic scenario for designing SUDS measures against flooding. The verification was conducted using seven precipitation events, with precipitation data collected at one location using a rain gauge and water level measurements recorded at seven locations (Figure 5). The verification results were compared with those obtained by Mogos (2022). The connections of the 1D model to sewer systems outside of the study area were simulated using a time series of measurements at the outflow points.

Seven events were selected based on the highest precipitation intensity. Four of these events were used for trial-and-error configuration of the model parameters, while the remaining three events were used to validate the results (Table 3). Full hyetographs of the precipitation events are provided in Appendix A.

Figure 5 Measurement locations

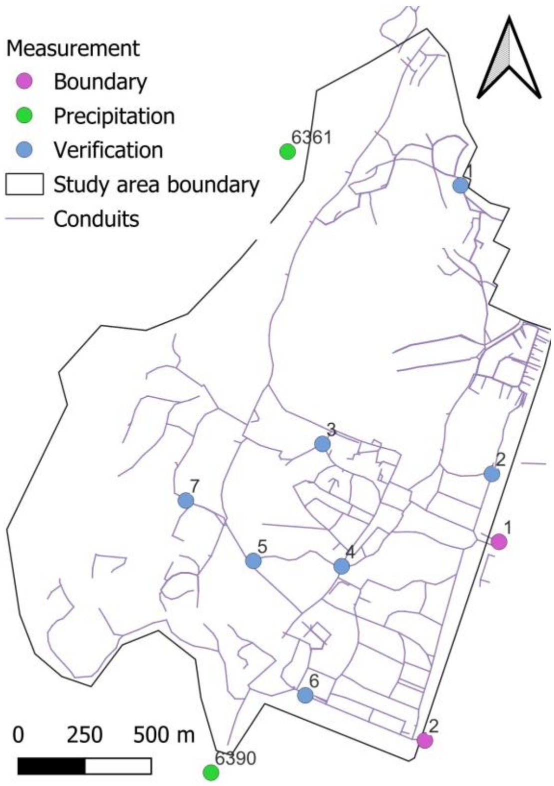


Table 3 Events used for verification

#	start time	End time	Highest intensity [mm/h]	Total precipitation [mm]	Trial-error/Validation
1	2019-06-10 21:00:00	03:00:00	23.00	24.0	Trial-error
2	2021-06-27 18:00:00	00:00:00	21.40	23.0	Validation
3	2019-06-05 21:00:00	03:00:00	19.40	31.2	Trial-error
4	2019-10-01 14:00:00	20:00:00	18.20	34.0	Validation
5	2019-06-15 03:00:00	09:00:00	17.80	37.0	Trial-error
6	2020-08-16 17:00:00	23:00:00	14.20	16.0	Validation
7	2021-10-21 00:00:00	06:00:00	13.80	41.0	Trial-error

To assess the performance of the model, the Nash-Sutcliffe Efficiency (NSE) (Nash & Sutcliffe, 1970) (Equation 1) was calculated. An average NSE of less than 0.65 was considered unsatisfactory; a value between 0.65 and 0.80 was considered acceptable; between 0.80 and 0.90, good; and between 0.90 and 1.00, very good (Ritter, 2013).

Equation 1 Nash-Sutcliffe efficiency

$$NSE = 1 - \left(\frac{\sum_{i=1}^{n} \left(H_i^{obs} - H_i^{sim}\right)^2}{\sum_{i=1}^{n} \left(H_i^{obs} - \overline{H^{obs}}\right)^2} \right)$$

- H_i^{obs} : Observed water levels [m]
- H_i^{sim} : Simulated water levels [m]

Since peak flow is especially important for flood modelling, the mean peak error (MPE) (Equation 2) and the absolute mean peak error (AMPE) (Equation 3) of the modelled peak flows were calculated relative to the measured flow. The verification aimed to minimize both performance metrics, with ±0.1 meters for the MPE and +0.2 meters for the AMPE considered acceptable.

Equation 2 Mean peak flow error

$$MPE = \frac{\sum_0^n \sum_0^i max\left(H_{n,i}^{sim}\right) - max\left(H_{n,i}^{obs}\right)}{N \times I}$$

- $H_{n,i}^{obs}$: observed water levels [m]
- $H_{n,i}^{sim}$: simulated water levels [m]
- N : number of verification locations [-]
- I : number of verification events [-]

Equation 3 Absolute mean peak flow error

$$AMPE = \frac{\sum_0^n \left| \frac{\sum_0^i \max\left(H_{n,i}^{sim}\right) - \max\left(H_{n,i}^{obs}\right)}{I} \right|}{N}$$

- $H_{n,i}^{obs}$: observed water levels [m]
- $H_{n,i}^{sim}$: simulated water levels [m]
- N : number of verification locations [-]
- I : number of verification events [-]

3.2.1. 1D2D model validation

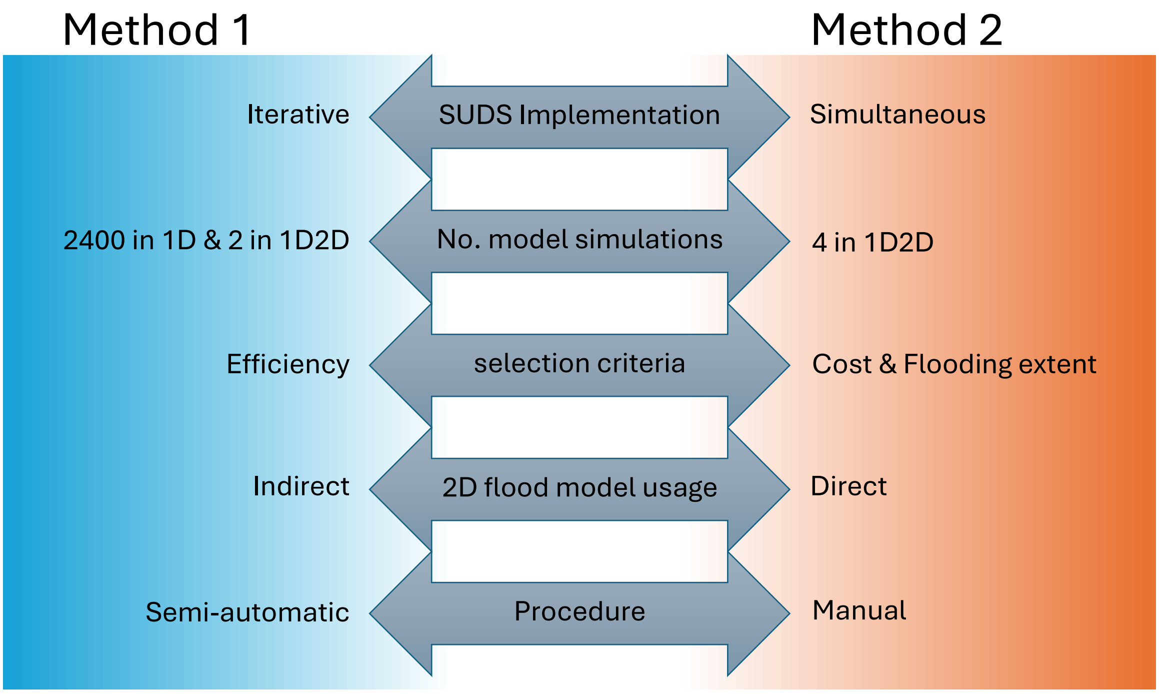
To validate the 1D2D model, the simulation results of a 50 mm, homogenous one-hour rainfall event were compared to the results of the 1D2D InfoWorks model (Mogos et al., 2023). InfoWorks was used for comparison because it is a more established software package compared to the newly developed IberSWMM coupling plugin. For both model simulations, the number of flooded buildings was calculated. A building was considered flooded when the water level against the wall exceeded 10 cm, which was assumed to represent the average height of a door sill. Flooded buildings were included in the validation process because they are a important aspect of the design process in this methodology.

3.3. Constraints and characteristics of SUDS design methods

To study the trade-offs between using 1D and 1D2D models for designing SUDS against urban flooding, two methodologies for designing SUDS have been used: one using both a 1D and a 1D2D model (Method 1), and one using only the 1D2D model (Method 2). This section outlines the shared characteristics and constraints under which both designs were developed, in order to ensure a fair comparison. Section 3.4 and Section 3.5 describe the procedures of both methods in detail, while Section 3.6 explains how the results of the two approaches were compared.

Both designs were developed to harness the potential of the respective models used. Figure 6 illustrates the differences between the two models. Method 1 exploits the capability to iteratively assess individual SUDS, and uses a greater number of simulations, whereas Method 2 implements all SUDS simultaneously. In its iterative process, Method 1 selected SUDS based on efficiency, while Method 2 considered both cost and the locations with the greatest extent of flooding. Moreover, Method 2 directly incorporated 2D flooding results into the design process, whereas Method 1 included these results only indirectly. Finally, Method 1 was carried out semi-automatically using a heuristic algorithm, whereas Method 2 was performed manually.

Figure 6 The differences between both methods

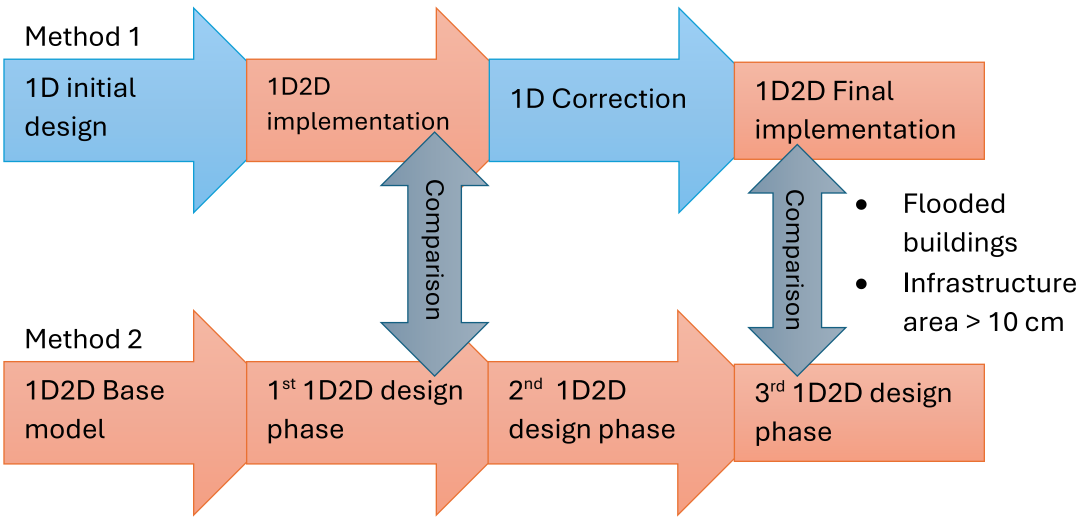


The designs were developed based on a homogeneous 50mm design event. According to the policy of the municipality of Bloemendaal, surface flooding of less than 10 cm is considered acceptable, and buildings must not flood during a 50 mm rainfall event (Haren, 2021), A building is considered flooded when more than 10 cm of water accumulates against its walls. A 50 mm, one-hour rain event statistically occurs once every 50 years in the Netherlands, but in the most extreme climate scenario, it could become as frequent as once every 25 years by 2100 (Nicolai, 2024).

Both designs were developed within the same development time, design value, and SUDS design constraints. The comparison is performed on both the initial and final designs (Figure 7).

While multiple types of heuristic methods exist, this study used a constructive method, in which the final design is developed step by step (Martí & Reinelt, 2022).

Figure 7 method 1 and method 2 design scheme's



The SUDS available for both designs included rain barrels, permeable pavements, green roofs, and bioswales. Their potential for implementation was defined beforehand (Table 4), A geographical overview of the SUDS potential is provided in Appendix L.

Table 4 Potential SUDS to be implemented in the study area

SUDS	Price /m^3	Area [ha]	Count
Rain Barrel	€ 220	0.85	36
Permeable pavement	€ 575	15	681
Green Roof	€ 3300	0.11	6
Bioswale	€ 202	4.5	30

The selection of the potential SUDS area in Table 4 was based on the following conditions:

- Green roofs and rain barrels were considered eligible only on public buildings, which includes schools, government buildings, sports facilities, and train stations.
- Rain barrel volumes were calculated to be able to capture the complete volume captured by the roof area.
- Green roofs were considered only for flat roofs; roof slopes were derived using a digital elevation model.
- Bioswales were considered only on public, unpaved land. Unpaved areas were identified using the land use map in Appendix K. Private land was defined by the presence of barriers such as fences, gates, or hedges.
- Surfaces with slopes greater than 3% were excluded from bioswale suitability due to reduced effectiveness at higher gradients (Seyedashraf et al., 2021).
- Rooftops were considered suitable for connection to bioswales if located within 25 meters of the bioswale.

- Permeable pavement was considered only for road surfaces, identified using the *Basisregistratie Grootschalige Topografie*.
- Only road surfaces with slopes below 3% were considered suitable for permeable pavement, as effectiveness decreases on steeper inclines (Seyedashraf et al., 2021).
- Both bioswales and permeable pavements included an underdrain with sufficient capacity to empty the structure within 48 hours.

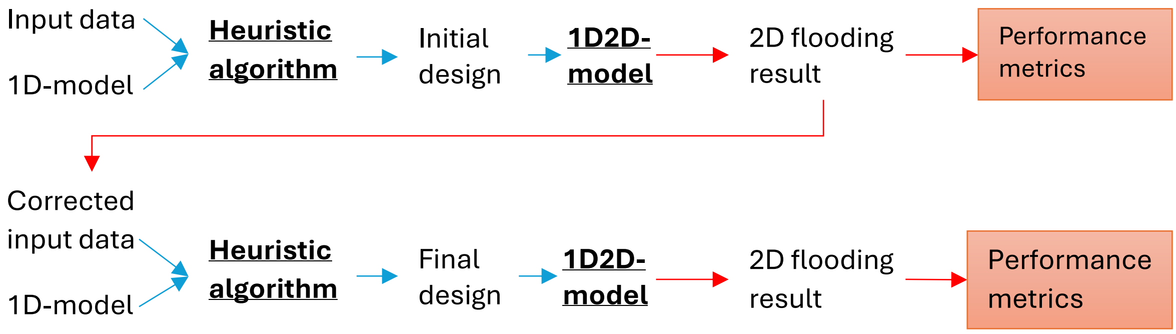
Both designs were constrained by a maximum design value of €2 million. A limited budget of €2 million was chosen to evaluate the design methodologies' ability to prioritize the most efficient SUDS. The SUDS have an average cost of €492 per cubic meter of storage, which implies that the designs should be able to provide at least 8 mm of storage capacity. The cost per cubic meter of storage for each SUDS type is provided in Table 4, and the detailed cost calculations are included in Appendix B.

Both design methods were developed within a predetermined maximum time limit of 21 hours, which is equivalent to four runs of the 1D2D model. The initial designs were completed within a predetermined maximum of 11 hours.

3.4. Method 1: using both 1D and 1D2D

Method 1 combined the use of the 1D model and the 1D2D model and consisted of two parts. Both parts began with the 1D model and concluded by implementing the results from the 1D model into the 1D2D model. These parts can be seen as the initial design and the final design phases. This method was developed to leverage the strengths of both model types. A schematic overview of method 1 is provided in Figure 8, which contains the initial design phase on top and the final design phase near the bottom.

Figure 8 Method 1: using both 1D and 1D2D models



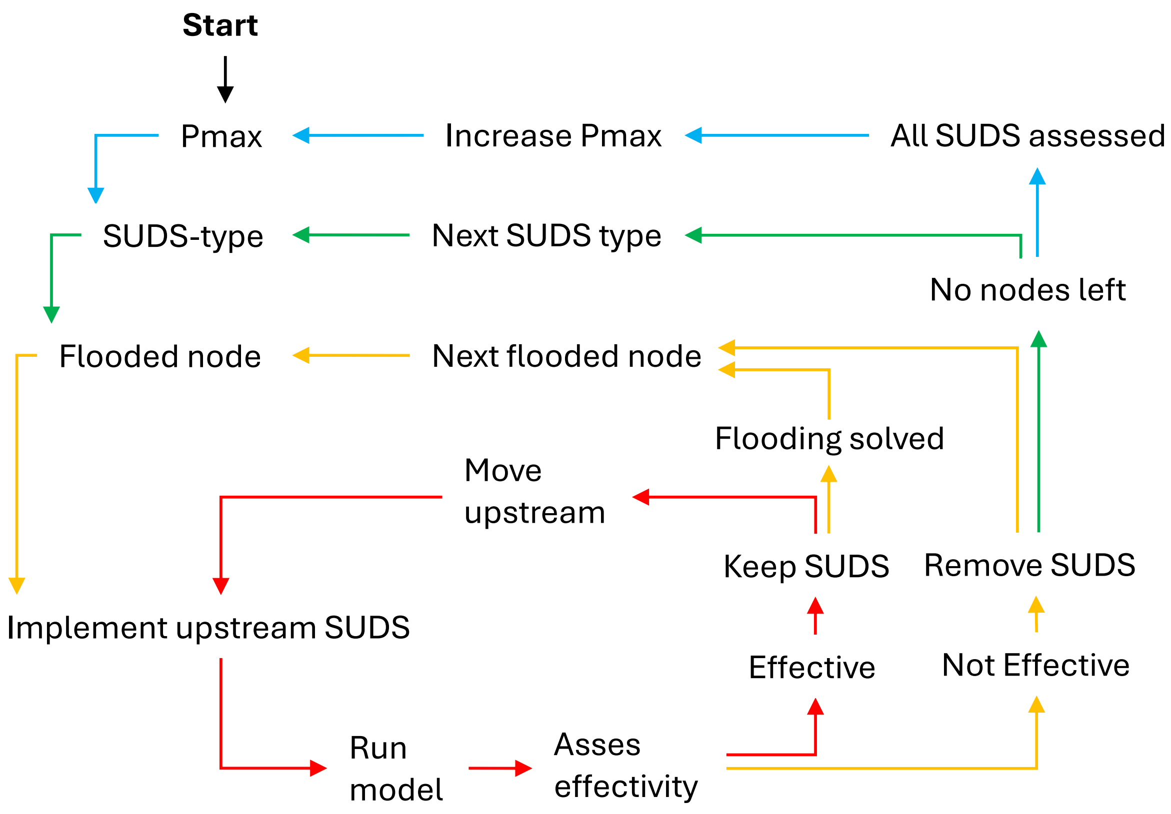
3.4.1. Initial SUDS design

In the initial design phase, SUDS were iteratively selected, implemented, and evaluated using a heuristic algorithm based on their flood reduction effectiveness in the 1D model and their asset value. The algorithm was designed to take advantage of the 1D model's ability to perform a large number of iterations within a short time frame.

A schematic overview of the heuristic algorithm is presented in Figure 9.Each loop will be described in more detail in this section. The algorithm has no predetermined endpoint; it terminates either when the maximum allowable SUDS design value is exceeded or when the algorithm runs longer than five hours. Exceeding the five-hour limit would leave insufficient time to execute the 1D2D implementation of the initial design. The algorithm consists of four nested loops, each represented by a different color:

1. Defining the maximum SUDS value per unit of flood reduction (P_{max}) (blue).
2. Selecting SUDS types to be implemented (green).
3. Selecting the flooded node near which SUDS should be implemented (yellow).
4. Implementing and assessing the SUDS (red).

Figure 9 Heuristic algorithm of method 1



Each loop will be described in more detail in this section:

1. The first and outermost loop was used to define P_{max}, which represents the maximum SUDS value in euros per cubic meter of flood reduction. This value was employed in the inner loops to assess the effectiveness of the SUDS. The initial P_{max} was set low to prioritize the selection of the most efficient SUDS. In each iteration, P_{max} was increased to allow the implementation of less effective SUDS if the most efficient ones were insufficient to reach the maximum design value.

2. The second loop was used to select the SUDS type to be implemented and assessed in the model. The implementation order of SUDS types is presented in Table 5. This order was primarily determined to prioritize upstream SUDS over those located at flooded nodes, since SUDS positioned at flooded nodes infiltrate water from the combined sewer system, potentially causing pollution. A secondary factor in defining the implementation order was the SUDS asset price per cubic meter of storage, to ensure that the most cost-effective SUDS were implemented first. The next SUDS type was selected only after the current type had been assessed for every flooded node in the system.

Table 5 SUDS implementation and assessment order

	Type	position relative to flooded node	Main Flood reduction principle
1	Bioswale	Upstream	Runoff capturing
2	Rain barrel	Upstream	Runoff capturing
3	Green roof	Upstream	Runoff reduction
4	Permeable pavement	Upstream	Runoff capturing
5	Permeable pavement	Flooded node	Flood attenuation

3. The third loop was used to select the flooded node upstream from or at which SUDS would be implemented. The selection order of flooded nodes was based on their position within the urban drainage system, starting with the most upstream node and proceeding downstream. The algorithm began upstream because this approach has been shown to be the most effective for designing SUDS (Haghighatafshar, et al., 2017). The next flooded node was selected when the required flood level reduction was achieved, all effective SUDS had been implemented, or a SUDS was assessed as ineffective.

4. The fourth and innermost loop was used to implement the SUDS, run the model, and assess the effectiveness of the SUDS. A SUDS was considered effective when its asset value (C) in euros divided by the total weighted flood volume reduction in cubic meters was lower than the maximum allowable value per volume reduction (P_{max}) (Equation 4).

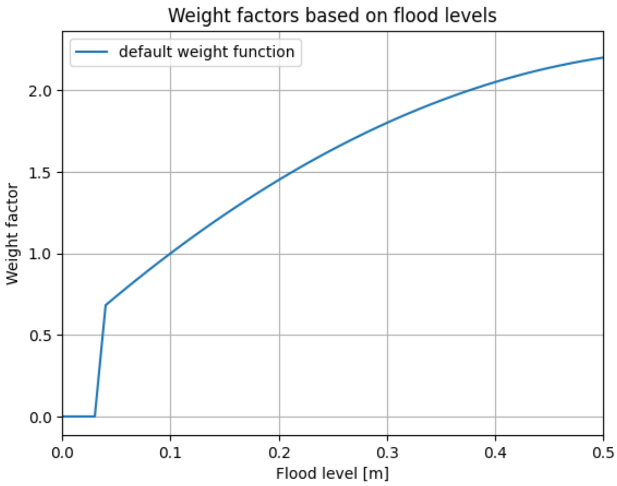
Equation 4 Assessment formula

$$\frac{C}{\sum_{n=0}^{N}\sum_{h_{i,n}}^{h_{i-1,n}} A_n \Delta h\, w(h+c_n)} < P_{max}$$

- C : SUDS asset value [€]
- N : number of nodes [-]
- $h_{i,n}$: water level at node n after implementation of SUDS [m]
- $h_{i-1,n}$: water level at node n before implementation of SUDS [m]
- A_n : ponding area of node n [m^2]
- Δh : water level step size [0.01 m]
- w : weight factor of water level h [-]
- c_n : correction level of node n [m]
- P_{max} : maximum SUDS asset value per unit volume of flood reduction [€/m^3]

The weight factor (w) was defined as a function of flood level (Figure 10). This function assigns a higher weight to higher water levels. This approach was adopted because the performance metrics only consider flood levels above 10 cm, making the influence of lower water levels negligible.

Figure 10: Weight factors based on flood levels.



When a SUDS was found to be effective, it was retained in the model, and the heuristic algorithm proceeded to implement and assess the next potential SUDS further upstream on the drainage branch where the SUDS was located. The algorithm searched up to 300 meters upstream from the flooded node for potential SUDS of the selected type. The upstream distance was determined based on the cumulative length of the conduits.

Rain barrels and green roofs were implemented in the 1D model using the LID Controls module of SWMM. Permeable pavement was represented as a storage node corresponding to the storage media volume, with linear infiltration applied to the node to simulate infiltration into the natural soil and the capacity of the underdrain. The interaction between the surface and the permeable pavement was conceptualized using weirs connecting the permeable pavement to the storage nodes. Bioswales were implemented through triangular conduits, with dimensions calculated to represent both surface storage and storage media volume. A linear infiltration model was used to simulate the natural infiltration capacity and the underdrain.

The initial design was implemented once in the 1D2D model using the IberSWMM plugin. SUDS were implemented according to the methodology described in section 3.5.1. Because the 1D2D model has higher accuracy in calculating flood volumes compared to the 1D model , the 1D2D flood volume was regarded as the baseline against which the 1D flood volumes were compared to assess the accuracy of the design.

3.4.2. Final SUDS design

The results of the 1D2D implementation were used to correct the input data of the heuristic algorithm. The strengths of 1D2D models lie in their ability to simulate accurate flood depths, flow between different area's and the hydrodynamic interactions between SUDS and overland flow. The adjustment procedure of method 1 was designed to leverage these strengths. After implementing the adjustments, the algorithm was rerun to generate a new design, which was also implemented in the 1D2D model. The adjustment of the input data consisted of four parts:

1. Permeable pavement infiltration assessment.
2. Assessment of decoupled surfaces to permeable pavement.
3. Node flood weight adjustment.
4. Bioswale storage assessment.

Each part of the adjustment will be explained in this section.

1. The first adjustment involved the assessment of permeable pavement infiltration. In the 1D model, infiltration through permeable pavement was represented using zero-dimensional nodes, implying that water spreads evenly across the permeable pavement surface. In reality, and as captured in the 1D2D model, geographical barriers cause water to spread unevenly, resulting in lower infiltration rates in certain areas. To address this, infiltration performance of the permeable pavement was assessed by dividing the simulated infiltration from the 1D2D model by the potential infiltration capacity, which included the permeable pavement storage media volume, natural soil infiltration capacity, and drain capacity. If the relative infiltration performance was lower than 0.7, the corresponding permeable pavement area was reduced accordingly.

2. The second adjustment involved assessing the decoupling of surfaces draining to permeable pavement. Paved areas draining towards nodes where permeable pavement had been implemented were decoupled from the sewer system and assumed to drain directly to the permeable pavement. To simulate this, the 1D2D model generated runoff directly on the 2D mesh at the locations of permeable pavement implementation, bypassing the hydrological model. The effectiveness of this decoupling was then assessed through visual inspection to identify geographical barriers that prevented runoff from the decoupled areas from reaching the permeable pavement surface. Subcatchments where decoupling caused flooding were excluded from the set of potentially decoupled subcatchments.

3. The third adjustment is node flood weight adjustment. In 1D, flooding does not flow laterally between nodes, while In the 1D2D model, floodwater flow is governed by ground elevation derived from the digital terrain model, causing water to move from areas of higher elevation to lower elevation. This can result in situations where nodes with higher water depths receive flow from nodes with lower water depths. This behavior was not captured in the initial 1D model design, where lateral flow between nodes is not simulated and which does not prefer reducing low water depths. To address this, the weighting factors of nodes were adjusted in the final design. Nodes with lower water depths that contribute flow to nodes with higher water depths had their weights increased to match those of the receiving nodes. This adjustment was implemented by modifying the coefficient c_n in the assessment formula (Equation 4) to reflect the difference in water depths between connected nodes.

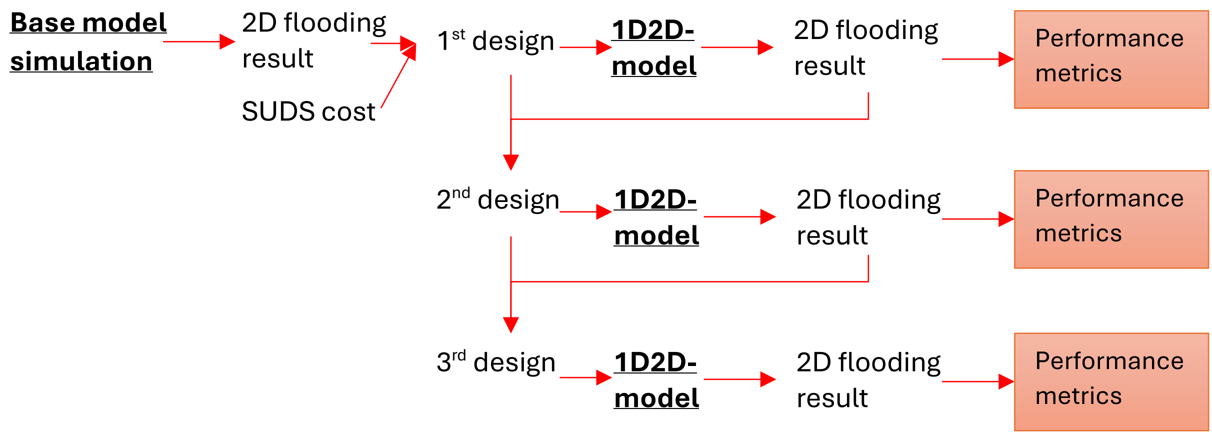
4. The fourth adjustment is the bioswale adjustment. The topographic generalization that occurs when the digital terrain model (DTM) is translated into a 2D surface mesh causes some bioswales to have lower storage capacity in the 1D2D model compared to the 1D model. To address this, hydrographs were used to determine the overflow volume of the bioswales. Bioswales for which the total final 1D2D storage was less than 80% of the water volume draining towards them were adjusted by reducing the total contributing area. The reduction was determined by the total overflow volume identified from the hydrographs.

The heuristic algorithm described in Section 3.4.1, with the implemented corrections in the input data, was re-run to create a final design, which was subsequently implemented in the 1D2D model.

3.5. Method 2: Three-stage 1D2D model design

Method 2 consisted of a simulation of the base model in 1D2D, followed by three design phases. SUDS were selected and implemented based on the locations where flooding occurred in the 2D flooding results and the SUDS cost (Table 4) . After each design simulation, corrections were applied using a method adapted from the correction procedure described in Section 3.4.2. Figure 11 provides a schematic overview of the design process of method 2.

Figure 11 Method 2: using only the 1D2D model



3.5.1. The SUDS design process of method 2

In the initial design, SUDS were implemented starting with the least expensive types and progressing to more costly options once the potential of the less expensive SUDS was exhausted. Implementation was prioritized at locations with the largest flooded areas exceeding 10 cm depth and at sites with the highest number of flooded buildings. The available budget was allocated across the study area proportionally to the ratio of flood volumes.

Infiltration from the bioswales and permeable pavement systems was modelled trough defining an initial infiltration representing the storage media, the hourly natural infiltration capacity of the underlying natural soil and the hourly drain capacity of a drainage pipe with the capacity to drain the SUDS in 48 hours. The Bioswale surface storage was implemented by adjusting the DTM to create a depression.

In general, runoff generation was modelled using the hydrological subcatchments in SWMM, However, in areas that are disconnected from the sewer system and drain toward the permeable pavement, the 2D mesh was used to generate runoff. This approach was adopted to simulate the infiltration of both runoff and flooding water. It also enabled the assessment of whether water flows toward the permeable pavement. Rooftops that drain toward the bioswales were connected to the surface model using outfalls that drain into the mesh in the IberSWMM plugin. Rain barrels and green roofs were modeled using the LIDS module of SWMM, with the parameterization of these Sustainable Urban Drainage Systems (SUDS) provided in Appendix M.

3.5.2. Correction of the design in method 2

Based on the results of the 1D2D design simulation, the heuristic method was adjusted to improve the design. The adjustment comprised of three parts:

1. Permeable pavement infiltration assessment.
2. Assessment decoupling to permeable pavement.
3. Bioswale storage assessment.

1. The first adjustment is the permeable pavement infiltration assessment. The infiltration of permeable pavement has been assessed in order to remove the areas were infiltration was too

low. This has been done by dividing simulated infiltration in 1D2D by the potential infiltration consisting of permeable pavement storage media capacity, the infiltration capacity of the natural soil and the drain capacity. In case of a relative performance lower than 0.7, the permeable pavement area has been reduced.

2. The second adjustment is the assessment of decoupled surfaces to permeable pavement. The paved areas draining towards nodes were permeable pavement has been implemented have been decoupled from the sewer system and have been assumed to drain towards the permeable pavement. The decoupling of permeable pavement was then assessed through visual inspection to identify geographical barriers that inhibit runoff from the decoupled areas from reaching the permeable pavement surface. Subcatchments where decoupling caused flooding were recoupled to the sewer system.

3. The third adjustment is the bioswale adjustment. The topographic generalization that occurs when the Digital Terrain Model (DTM) is translated into a 2D surface mesh results in some bioswales having lower storage capacity in the 1D2D model than initially calculated. To address this, the bioswales were adjusted using hydrographs to determine the overflow volume. Bioswales where the total final storage was lower than 80% of the water volume draining toward them have been modified by reducing the total contributing area to the respective bioswale in accordance with the total overflow indicated by the hydrograph.

The value reduction of the design caused by the adjustments was reinvested by continuing with the design process of section 3.5.1.

3.6. Comparison of methods 1 and 2

To address the fifth research question, which seeks to determine the superior method, the designs created using Method 1 and Method 2 have been compared. This comparison utilized the results from the 1D2D model, which was employed in both methods, facilitating a valid comparison. Two performance metrics were used to evaluate both designs:

1. The total area of infrastructure with more than 10 cm of flooding.
2. The number of flooded buildings.

These performance metrics were based on the policy of the municipality of Bloemendaal (Haren, 2021). The locations of infrastructure and buildings were defined using a land-use map from the *Basisregistratie Grootschalige Topografie*. A building was considered to be at risk of flooding when the water level against its walls exceeded 10 cm, which was assumed to be the average height of a door sill. The method that performed best according to these metrics was concluded to be the most effective.

The comparison of Methods 1 and 2 occurred twice: once after both methods had produced an initial design and once after the final designs of both methods had been completed. Both design processes had used an equal amount of implementation and simulation time at the comparison moments. Performing two comparisons allows for the distinction between the initial design phase and the later design phase, enabling a conclusion about which approach is more effective at each stage of the design process.

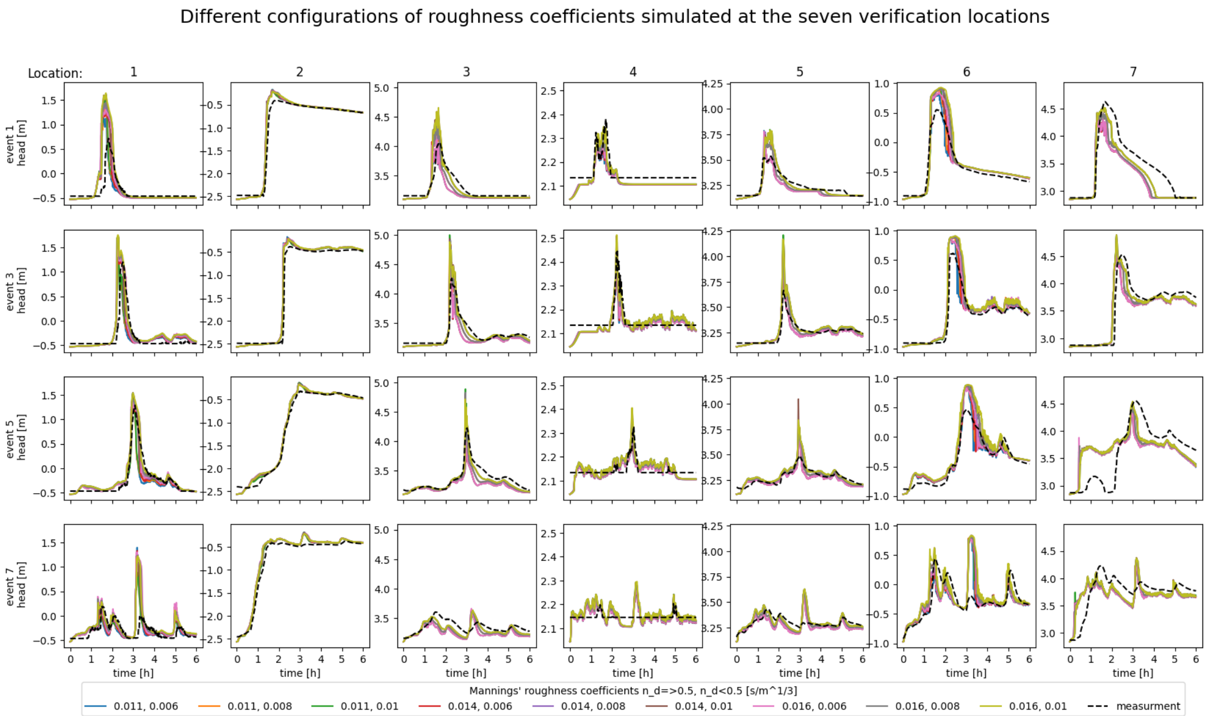
4. Results

This chapter shows the results and the interpretation of the results that led to the answering of the research questions. Before the design phase, the models were verified and validated. The verification of the 1D model is presented in Section 4.1. The verified 1D model was then coupled with a 2D flooding model to create a 1D2D model, which is validated in Section 4.2 These models were subsequently used in the design processes of Methods 1 and 2, the results of which are presented in Sections 4.3 and 4.4 , respectively. Finally, the results of these methods are compared in Section 4.5 to determine their effectiveness in reducing flooding.

4.1. Verification of the 1D urban drainage model

Verification was performed by applying different roughness coefficients to the conduits, using separate coefficients for conduits with a diameter less than 0.5 meters and for those with a diameter of 0.5 meters or more, in order to account for differences in conduit material. This resulted in nine different roughness coefficient configurations. Figure 12 displays the hydrographs showing both the simulated and observed water heads during four of the verification events that have been used for trial-and-error parameter selection. The Manning's roughness coefficients used are indicated in the legend.

Figure 12 Different configurations of roughness coefficients simulated at the seven verification locations.



The simulations showed high sensitivity to different configurations of roughness coefficients at several nodes, caused by the relatively high flow velocities that occur during peak precipitation events. Sensitivity was lower at locations 2 and 7, where water levels are mainly influenced by the nearby weirs. The model simulations in the hydrographs generally followed the observed measurements, capturing the overall trends and dynamics of the events, although some deviations are present:

The first and most important deviation was the overestimation of the water head at location 6. A possible explanation for this discrepancy is that the model simulated surface ponding near this location during all events. In 1D models, surface ponding is represented by 0D ponding areas associated with nodes; however, the actual surface topography is much more complex. This complexity makes accurate model verification at locations with surface flooding more difficult. Simulating the verification event with a 1D2D model might improve model performance, but the IberSWMM plugin used in this study does not produce hydrographs that can be used for verification.

A second deviation was observed at location 4, where a significant difference in base flow occurred. This deviation is of lesser importance for this study, since base flow contributes little to the peak flow, which in turn leads to flooding.

A third deviation was observed during event 7, where peak flow was significantly overestimated at locations 1, 3, 4, 5, 6, and 7. A possible explanation for this discrepancy can be found by comparing the rain gauge data with that from a second rain gauge located at approximately the same distance from the study area. The compared rain gauge data is in Appendix A, this comparison shows that the large spike in precipitation is not present in the second rain gauge. This indicates a significant uncertainty regarding the actual precipitation received by the catchment. This uncertainty has been addressed during the verification process by only focusing on systematic deviations that are present across multiple events.

The mean peak flow error (MPE), absolute mean peak flow error (AMPE), and Nash-Sutcliffe efficiency (NSE) were calculated for each combination of roughness coefficients to determine which configuration best fits the observations. The results of these calculations are shown in Table 6.

Table 6 mean peak flow errors, absolute mean peak flow errors and Nash-Sutcliffe efficiency for each of the parameter combinations

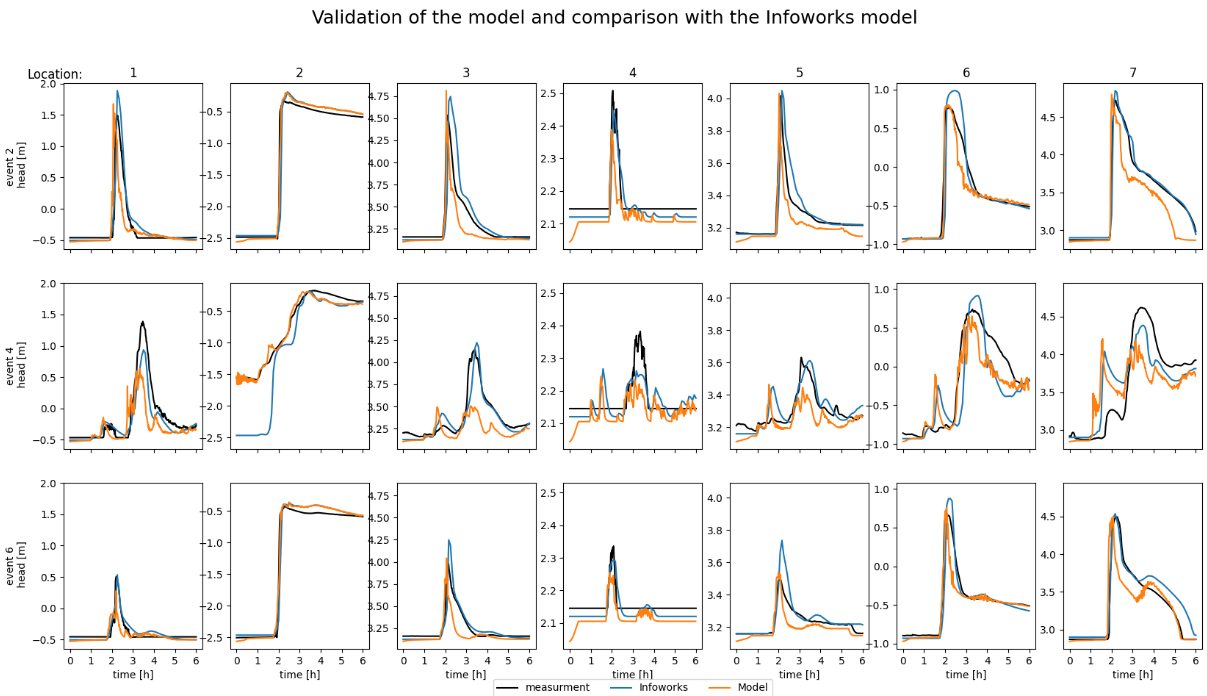
		$n_{d \geq 0.5}$ $\left[s/m^{1/3}\right]$		
$n_{d<0.5}$ $\left[s/m^{1/3}\right]$		0.011	0.014	0.016
0.006	MPE [m]	0.20	0.23	0.30
	AMPE [m]	0.23	0.23	0.30
	NSE [-]	<u>-0.03</u>	-0.05	-0.22
0.008	MPE [m]	<u>0.17</u>	0.26	0.31
	AMPE	<u>0.20</u>	0.26	0.31
	NSE [-]	-0.04	-0.09	-0.30
0.01	MPE [m]	0.20	0.25	0.31
	AMPE [m]	0.22	0.25	0.31
	NSE [-]	-0.09	-0.14	-0.36

The lowest absolute mean peak flow error (AMPE) and mean peak flow error (MPE) were obtained with $n_{d\geq0.5}$ = 0.011 and $n_{d<0.5}$= 0.008 The highest Nash-Sutcliffe efficiency (NSE) was achieved with $n_{d\geq0.5}$ = 0.011 and $n_{d<0.5}$= 0.006 (Table 6). Because peak flow is the most important characteristic when modelling floods, the configuration with $n_{d\geq0.5}$ = 0.011 and $n_{d<0.5}$ = 0.008 was selected as the best-performing configuration. This configuration will be used for validation with the validation events.

The AMPE and MPE performed much better than the NSE because the NSE is influenced by the timing of flood peaks and base flow. The timing of flood peaks was particularly influential in the drainage system of Bloemendaal, where the system response is fast and the difference between base flow and peak flow can be as little as 10 minutes. As a result, a small error in the timing of the flood peak has a large effect on the NSE. This can be illustrated by event 1, where at location 3 an acceptable MPE of 0.08 was simulated, yet the NSE only reached 0.29 because the event appeared slightly earlier in the simulation compared to the observations. The timing bias was not consistent across different events. The large number of events and verification locations benefited the MPE and AMPE compared to the NSE, since positive and negative errors can cancel each other out in the MPE and AMPE, but not in the NSE.

The trough trial-and-error selected parameters were validated using the validation events and compared with a previously validated model in InfoWorks by Mogos (2022) (Figure 13). This comparison was performed because the 1D2D InfoWorks model was used to validate the 1D2D IberSWMM model. Both 1D2D models were developed using the verified 1D models.

Figure 13 Validation of the model, comparison with the Infoworks model



The model simulations generally followed the observed measurements, capturing the overall trends and dynamics of the events. At location 6, the structural overestimation of water levels observed during verification was not present in the validation events. In contrast, this overestimation was still present in the InfoWorks model results. In events two and six, the simulated peak flows were close to the measured peaks, while in event four, peak flows were consistently underestimated compared to the observations.

Table 7 shows the performance of the verified SWMM model and the InfoWorks model on the performance parameters for both the trial-and-error events and the validation events. This comparison was carried out to interpret the differences between the 1D2D IberSWMM model and the 1D2D InfoWorks model, both of which were developed based on the verified 1D models.

Table 7: Performance parameters of the validated models, compared with the Infoworks model

	1D SWMM model			1D Infoworks (Mogos R. , 2022)		
	Trial-error events	Validation events	All events	Trial-error events	Validation events	All events
Absolute mean peak flow error [m]	0.2	0.09	0.15	0.28	0.09	0.20
mean peak flow error [m]	0.18	-0.08	0.07	0.28	0.06	0.19
Nash-Sutcliffe efficiency	-0.04	0.49	0.23	-1.06	0.24	-0.50

The SWMM model performed significantly better on all performance parameters compared to the InfoWorks model. The mean peak flow error was considerably higher in the InfoWorks model than in the SWMM model, indicating that the InfoWorks model would overestimate flood volumes more than the SWMM model. The InfoWorks verification was performed using only event 2 from the SWMM validation dataset and an additional, smaller precipitation event. Incorporating more events in the validation dataset improved the overall model performance.

Because the MPE and AMPE of the SWMM model were lower than the required thresholds of 0.1 and 0.2 meters, and the simulations generally followed the observations, the model was considered suitable for proceeding to the design phase, although some uncertainty should still be taken into account.

4.2. Validation of 1D SWMM coupled with 2D Iber

Both design methodologies require the use of a 1D2D model. This section presents the validation results of the 1D2D IberSWMM model. The verified 1D SWMM model was used to create a 1D2D model by dynamically coupling it to Iber. That model was used to simulate the design event of 50 mm in the 1D2D coupled IberSWMM model. validated was carried out using flood volume and the number of flooded buildings and comparing the IberSWMM model to the 1D2D InfoWorks model.

4.2.1. Flood volume

For verification, the maximum flood volume was compared with those of the 1D2D InfoWorks model (Mogos et al., 2023)(Table 8). The 1D2D InfoWorks model is suitable for verification, as the software is currently widely used for 1D2D modelling applications, whereas the 1D2D IberSWMM coupling is still a new development. It should be noted that the 1D InfoWorks model performed worse on the verification parameters than the 1D SWMM model. The 1D SWMM model flood volume is also included in Table 8 to allow a comparison of surface flooding between the 1D and 1D2D models.

Table 8 Water on surface in IberSWMM and Infoworks

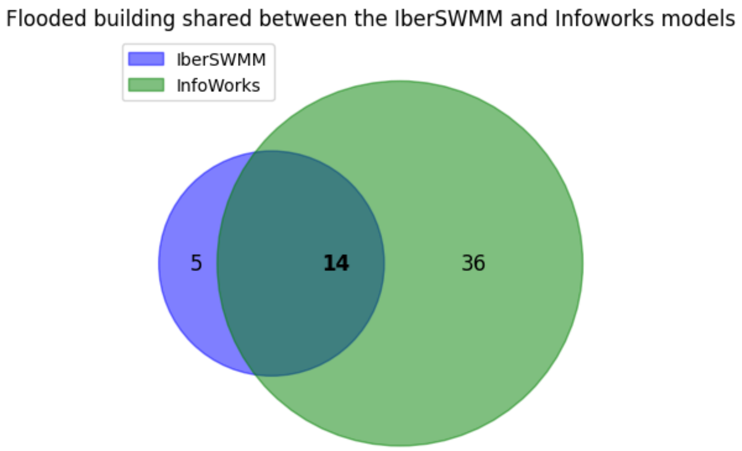
Model	Max volume water on surface [m^3]	Max volume water on surface [mm]
1D SWMM	7,981	15.1
1D2D IberSWMM	7,683	14.4
1D2D Infoworks (Mogos et al., 2023)	10,702	20.0

The IberSWMM model simulated significantly less surface ponding compared to the Infoworks model. This result was expected, considering that the Infoworks 1D model tended to overestimate peak flows significantly more during the verification events. Across all 2D cells containing water in either model, the 1D2D Infoworks model produced, on average, 2 cm deeper surface flooding than the 1D2D SWMM model. This difference can be used to gain insight in the sensitivity of 1D2D flood levels to the peak flows observed during 1D verification. During 1D model verification, the Infoworks simulation showed, on average, 12 cm higher water levels than the SWMM model, which ultimately resulted in an extra 2 cm of simulated flooding at the 2D surface. The error between the SWMM model and observed water levels was 7 cm, suggesting that a similar magnitude of error (around 2 cm) can be expected in averaged flood simulations by the IberSWMM model when compared with actual flooding observations.

4.2.2. Flooded buildings

Flooded buildings represent an important aspect of the design methodology in this study. The sensitivity of flooded buildings to the verification was studied by calculating the number of flooded buildings for both the IberSWMM and InfoWorks models (Figure 14). A building was considered flooded when water depths exceeded 10 cm against a section of its walls.

Figure 14 Flooded buildings shared between the IberSWMM and Infoworks models



The number of buildings classified as flooded varies significantly between the two models. The main discrepancies occur around the Kinheimweg (Figure 15 & Figure 16), At this location, the 1D Infoworks model simulated substantially higher water levels than the 1D SWMM model (Figure 13). This difference is problematic, as it could affect decision-making when upgrading drainage systems. While errors in the verification process did not result in major differences across the entire catchment, they did lead to significant discrepancies at one specific location—coincidentally, the area with the highest number of flooded buildings. Therefore, when models are verified for flood prevention purposes, greater attention should be paid to areas most susceptible to flooding.

Figure 15 Flooding and flooded buildings in Infoworks

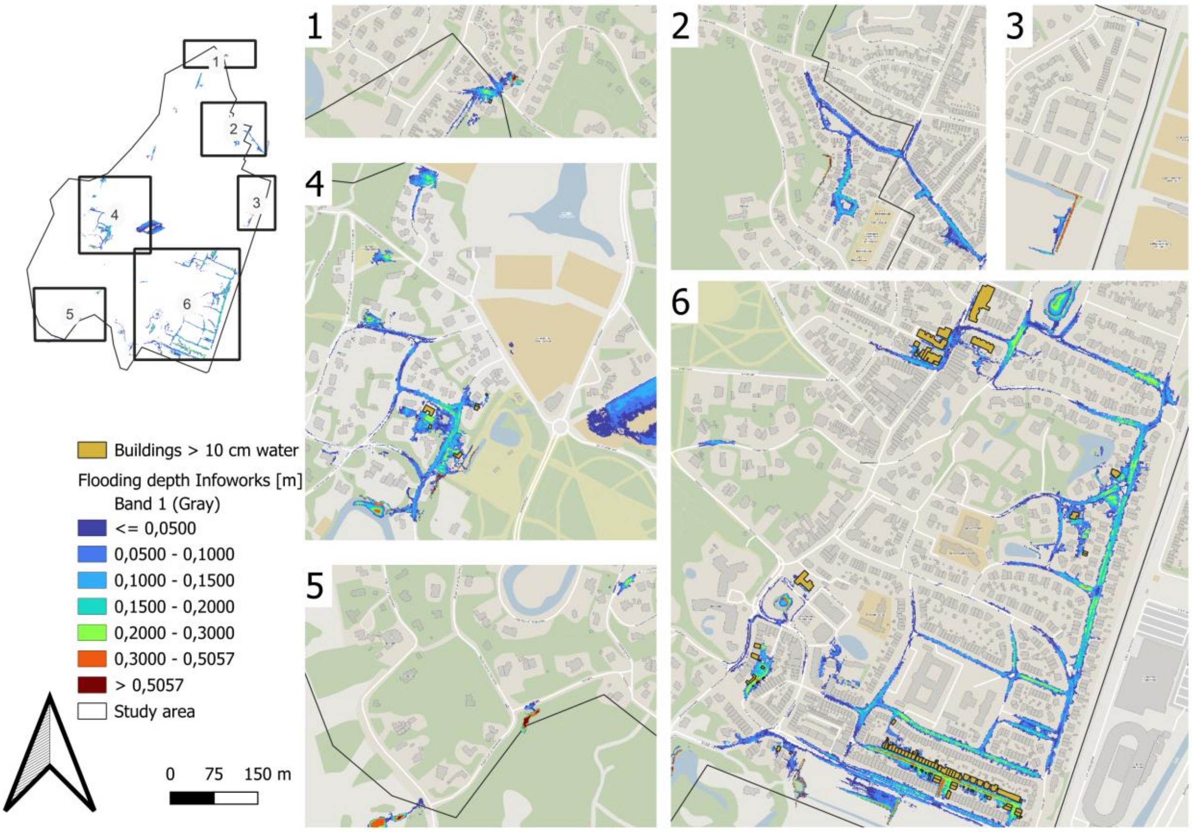
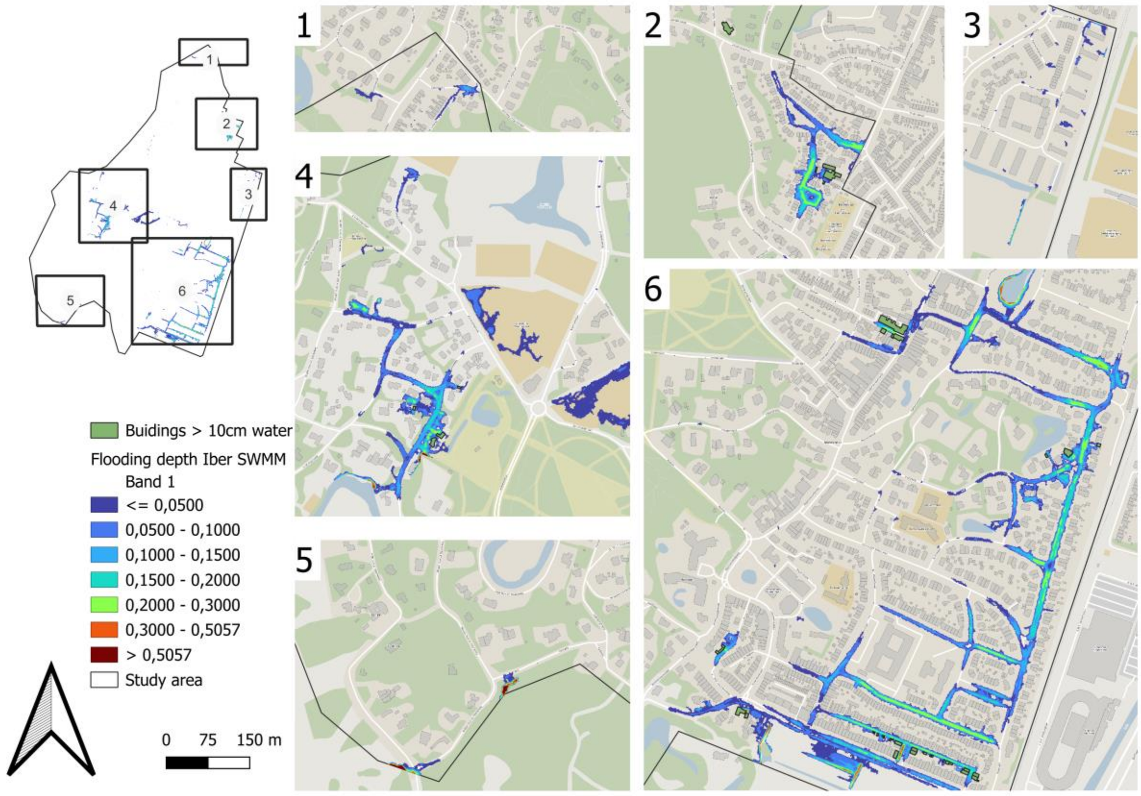


Figure 16 Flooding and flooded buildings in IberSWMM



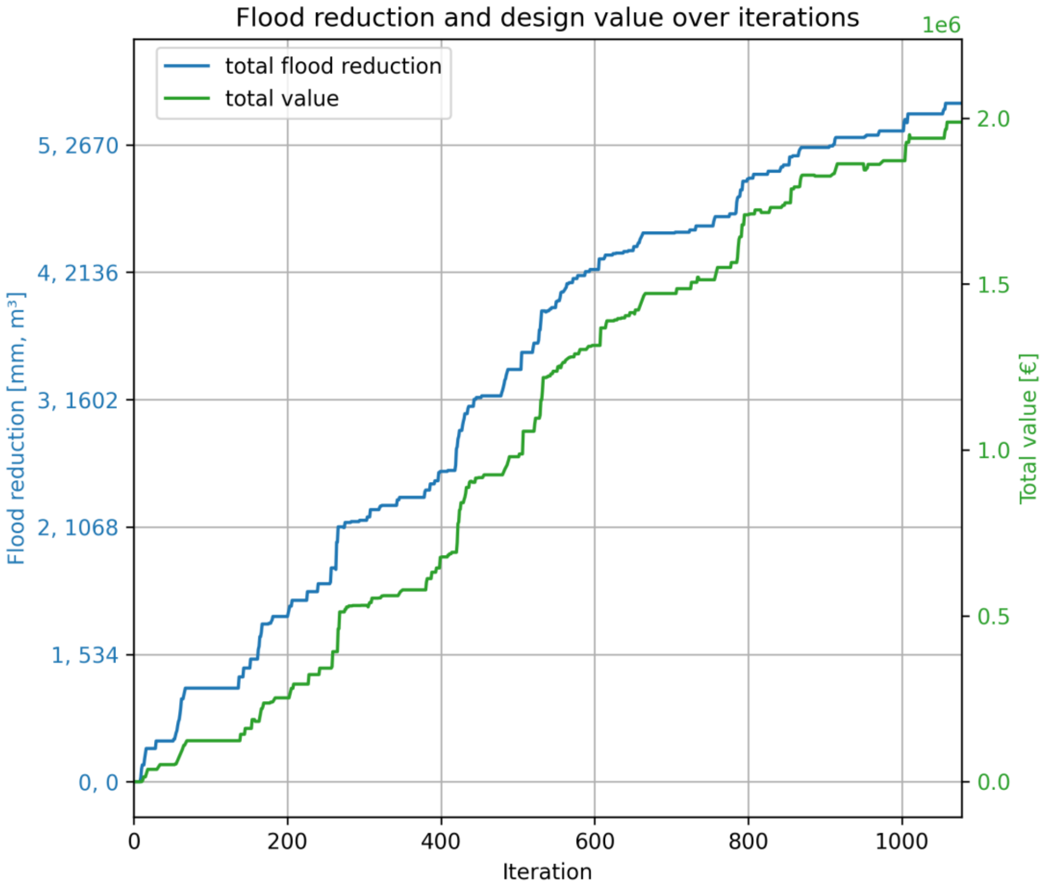
4.3. Design method 1: combining 1D and 1D2D

Design method 1 combines the use of both the 1D and 1D2D models. Method 1 begins with an initial design that is heuristically constructed in the 1D model and then implemented in the 1D2D model. The outcomes of these iterations, as well as the comparison to the 1D2D implementation, are discussed in section 4.3.1. In the second part, the heuristic input values are adjusted based on the results of the initial 1D2D implementation. These new heuristic input values are then used to generate a revised design, which is subsequently implemented in 1D2D. This is presented in section 4.3.2

4.3.1. Initial design:

Using the heuristic algorithm, an initial design was constructed. The total time required for selection, simulation, and assessment was 4 hours and 10 minutes. The evolution of this design is presented in Figure 17. This figure shows that, over the course of 1010 simulations, the total flood volume was reduced by 5.1 mm. It also demonstrates that the total design value increased to just under the maximum value of €2 million over the iterations. The heuristic algorithm was developed to select the most cost-effective SUDS. The figure has been used to analyze the relationship between flood reduction and design value.

Figure 17 1D total flood volume reduction and total design value over the iterations



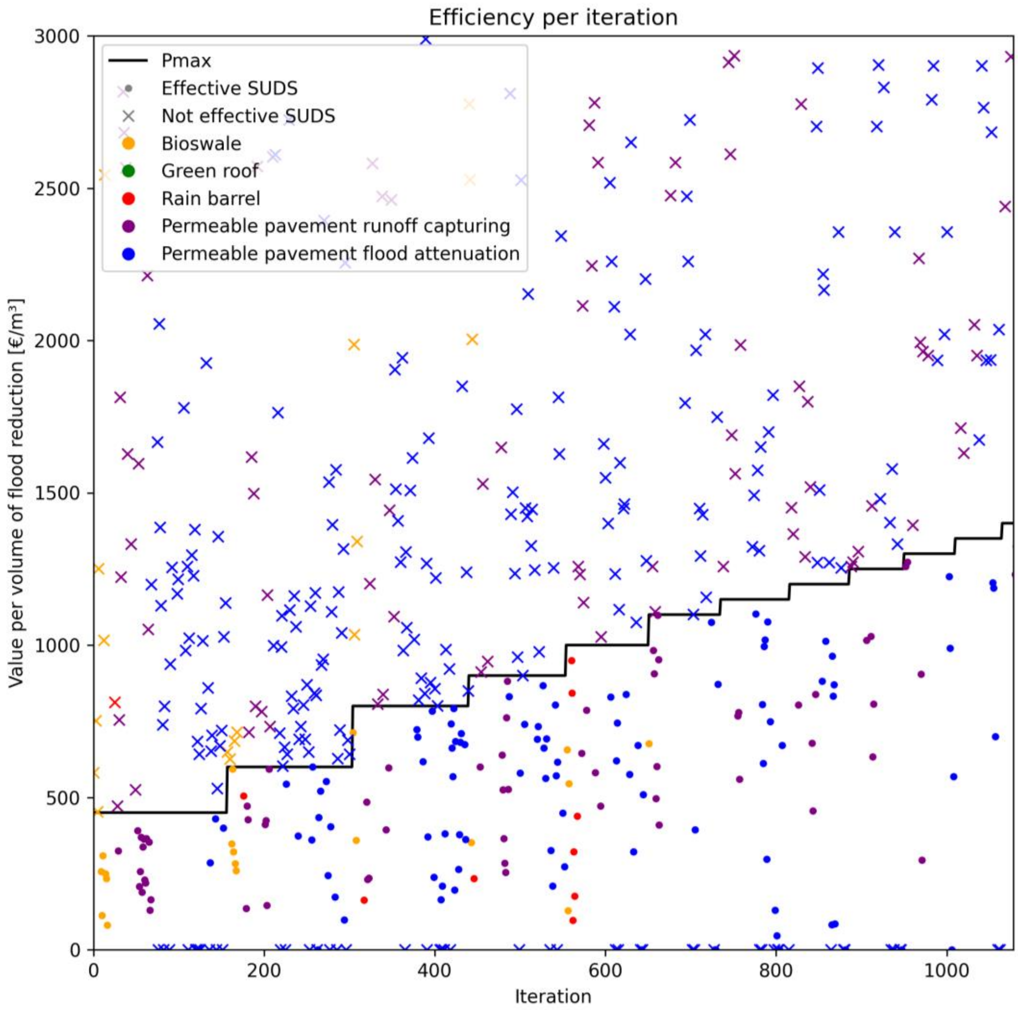
The total flood reduction increased more rapidly during the earlier iterations compared to the later ones. This is illustrated by the fact that a flood reduction of 2.3 mm was achieved in the first 400 iterations, whereas only 1.1 mm of reduction was attained in the same number of iterations between iterations 600 and 1000. This represents a decrease of 52%, while the total design value added over these periods decreased by only 16%, from €1.8 million to €1.5 million. This may indicate two possibilities: first, that the heuristic algorithm was able to select the more cost-effective SUDS initially, before moving on to less cost-efficient options; or second, that there is a diminishing return from certain SUDS as others are already implemented.

The graph shows that, during certain sequences of iterations, flood reduction temporarily plateaued, particularly before iteration 400 and after iteration 800. Prior to iteration 400, these plateaus occurred due to the value differences between the SUDS: the more expensive SUDS types had only a small chance of being cost-effective. Potential improvement within the heuristic algorithm could be aimed at reducing the phases in which the flood reduction temporarily plateaued to reduce the simulation time.

The evolution of design value and flood reduction during the heuristic algorithm can be explained by the progression of P_{max}, which represents the maximum value per volume of flood reduction. This parameter was used to assess the efficiency of potential SUDS and was gradually increased from €450/m^3 to €1400/m^3.

Figure 18 shows how the P_{max} evolved over time in combination with the cost-efficiency of each assessed SUDS, based on the weighted flood reduction and the value of the SUDS. SUDS located below the Pmax line were implemented, while those above the line were excluded from the design.

Figure 18 Effectivity assessment for every iteration, with effectivity related to the shift in Pmax during the heuristic algorithm.

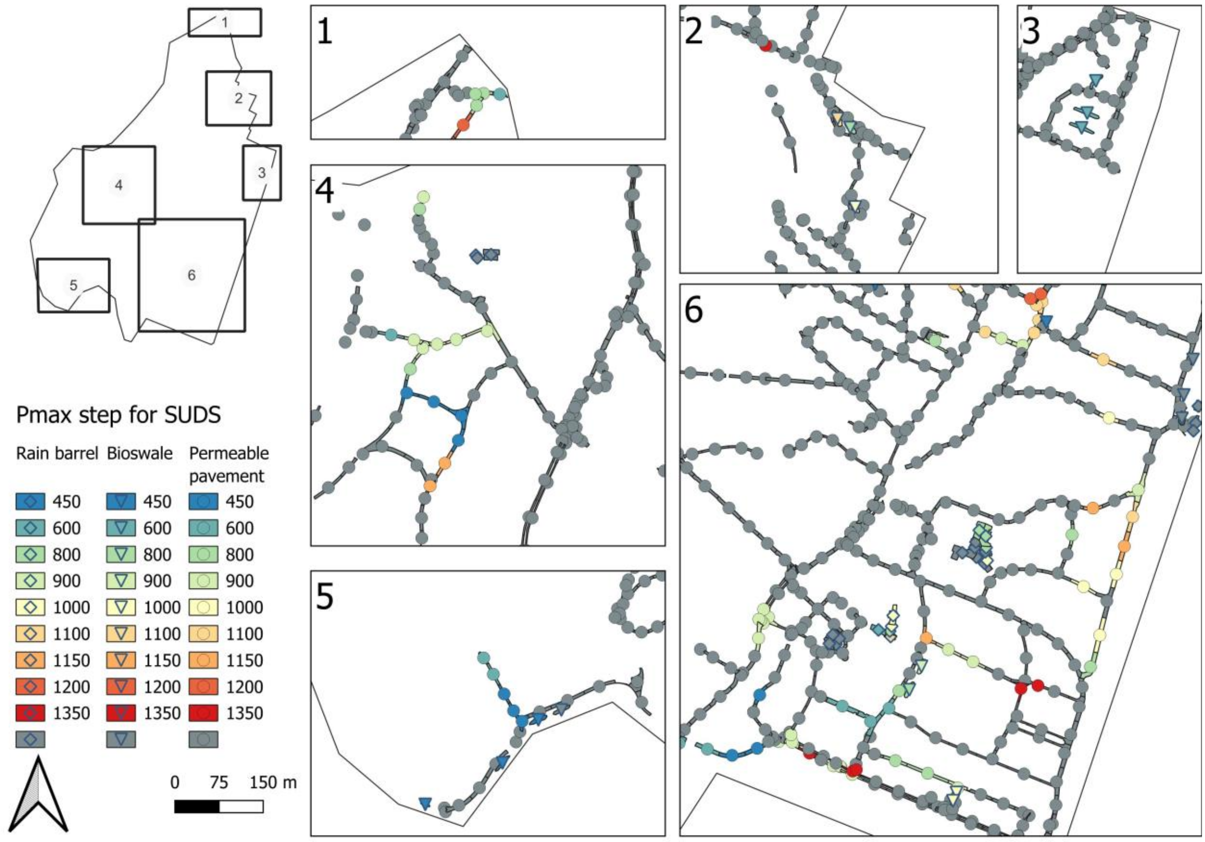


In the earlier iterations, the cheaper bioswales and rain barrels were implemented most frequently, while in the later stages, SUDS implementation was dominated by the more expensive permeable pavement. Green roofs were not implemented by the heuristic algorithm, as their costs were significantly higher compared to the other measures. In some locations, permeable pavement was also implemented in the earlier stages, when it could benefit from the large infiltration capacity of the underlying natural soil. The figure shows that, during the later stages of the heuristic algorithm, some SUDS were still implemented with a value per volume of flood reduction lower than the initial Pmax of €450. This is due to the interdependency between different SUDS; SUDS implemented in previous iterations made these subsequent SUDS much more efficient.

Further analysis of the interdependency between SUDS can be performed by examining the geographical locations of the SUDS in combination with the Pmax step at which each SUDS was implemented, as shown in

Figure 19 Figure 19. A higher Pmax—represented by a redder color—indicates that the SUDS was implemented during a later stage of the heuristic algorithm. The different SUDS-types are represented by different symbols.

Figure 19 Implemented SUDS and their heuristic step. Upstream SUDS are implemented earlier and downstream SUDS later in the process



During the initial steps of the Pmax evolution, SUDS were primarily implemented in the upstream parts of the catchment, which especially visible in areas 4 and 5. Later on, SUDS in downstream areas such as area 6 were also found to be effective. While Figure 18 demonstrated that there was interdependency between SUDS within the study area, Figure 19 reveals that this interdependency extends from upstream to downstream, since downstream SUDS were only effective after upstream SUDS were implemented. Implementing SUDS upstream before implementing them downstream proves to be an effective strategy.

The Pmax step at which each SUDS is implemented can provide information on both interdependency and effectiveness. This information can be used to formulate a preferred order of implementation when the full design cannot be realized all at once.

The heuristic algorithm resulted in an initial design based on the iteration in which the design value exceeded the maximum allowed value. Table 9 presents the design obtained from the heuristic algorithm, while a complete geographic overview of the design can be found in Appendix C.

Table 9 Initial design results

SUDS	Storage volume [m^3]	Percentage of potential storage [%]	Area [m^2]	Value [€]
Rain Barrel	97	23	97	€ 21,340
Bioswale	797	61	2,585	€ 159,947
Permeable Pavement	2,995	8.3	23,333	€ 1,722,125
Green Roof	0	0	0	€ 0
Total	3,889	-	25,642	€ 1,903,412

Table 9 shows that most of the storage was realized by permeable pavement, which is due to the greater potential for permeable pavement storage in the study area. Bioswales and rain barrels had the largest proportions of their potential areas implemented, as they were the most cost-effective measures. A computational factor influencing the outcome is that flood volumes were calculated by multiplying the water levels above the nodes by the associated ponding area. SWMM rounded the water levels to the nearest centimeter. This made it challenging to accurately calculate the effects of smaller measures, as the impact of a SUDS were distributed across multiple nodes and often result in vertical changes of only a few centimeters. This may have favored the larger bioswales over the much smaller rain barrels, which could help explain their difference in the final result.

Moving towards the next phase, the initial design developed in the 1D model was implemented in the 1D2D model. A water balance comparing the 1D and 1D2D models for this design is presented in Table 10. This comparison was conducted to assess the accuracy of the 1D simulation in terms of volumes, using the 1D2D model as a benchmark. The table includes total flooding and infiltration, as well as the performance of the SUDS implemented in 2D, which are the bioswale and permeable pavement. In the 1D2D model, the bioswale provides both surface storage and infiltration into the storage media and natural soil. In the 1D model, no distinction is made between these mechanisms; only the total value has been calculated.

Table 10 Water balance comparing the 1D and 1D2D simulation of the initial design

	Initial design 1D		Initial design 1D2D		Difference:	
	m^3	mm	m^3	mm	m^3	mm
Precipitation	26700	50.00	26700	50.00	0	0
Flood volume	4555	8.53	5397	10.11	-842	-1.58
Flood infiltration	0	0.00	493	0.92	-493	-0.92
PP Infiltration	3672	6.88	3432	6.43	240	0.45
Bioswale storage			299	0.56		
Bioswale Infiltration			420	0.79		
Total Bioswale	797	1.49	719	1.35	78	0.15
Total SUDS:	4469	8.37	4151	7.77	318	0.60

Bioswale storage and permeable pavement infiltration performed significantly worse in the 1D2D model compared to the 1D model, potentially resulting in increased surface flooding. The difference in permeable pavement infiltration between the 1D and 1D2D models was caused by geographical barriers that prevented surface water from reaching the permeable pavement. The difference in bioswale storage was due to lower-than-expected storage capacity in some bioswales, which resulted from the mesh element size of 1.5 m^2 being too coarse to accurately represent the bioswale design. A smaller mesh element size locally at the locations of bioswales could improve bioswale simulation.

An important difference between the permeable pavement and the bioswale is that for the bioswale, runoff is first diverted to the bioswale and leaks away while for permeable pavement, water never reaches the pavement. This means that permeable pavement primarily causes lower flood levels over a wide area while leakage from the bioswale causes higher concentrations and deeper flood levels.

The water balance reveals that the 1D2D model simulated 1.58 mm more flooding than the 1D model. This difference cannot be fully explained by the reduced performance of the SUDS, as that difference is much smaller. Instead, it is primarily due to the higher accuracy of the 1D2D model in simulating floods. The total flood reduction calculated in 1D was 5.1 mm (see Figure 17), resulting in a relative difference in flood volume reduction between 1D and 1D2D of 31%. This highlights one of the major drawbacks of using a 1D model: it leads to significant uncertainty in calculating flood volumes.

The results of the 1D2D model were used to evaluate the performance of the initial design using the performance metrics. The number of flooded buildings decreased from 21 to 14, and the area experiencing more than 10 cm of flooding was reduced from 2.18 ha to 1.37 ha. Two buildings that did not flood in the base model experienced flooding in the 1D initial design; these floodings were caused by decoupled areas that did not drain sufficiently toward the permeable pavement and by a bioswale that flooded. For the remaining twelve buildings, the maximum water level against the walls, averaged over the remaining buildings, decreased from 20 cm to 14 cm (Appendix G).

4.3.2. Correction of heuristic input values

In the second part of Method 1, the results of the previous 1D2D implementation were used to adjust the input values of the heuristic algorithm. The correction aimed to compensate for the shortcomings of 1D relative to 1D2D. These revised input values were then used to generate a final design, which was subsequently implemented in the 1D2D model. The correction of the input values consisted of four components:

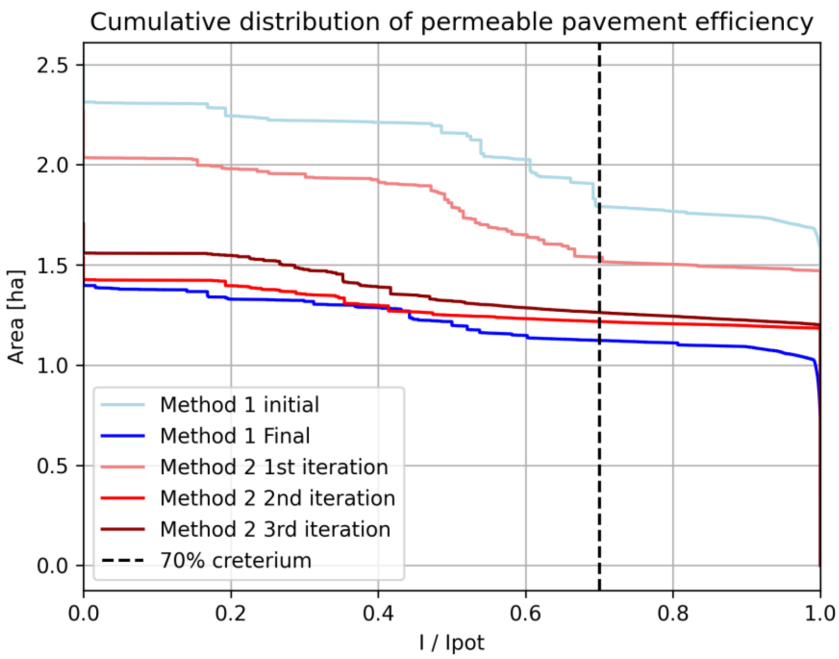
1. Permeable pavement infiltration assessment.
2. Assessment of decoupling to permeable pavement.
3. Node flood weight adjustment.
4. Bioswale storage assessment.

The results of each part of the correction will be presented in this section.

1. Permeable pavement infiltration was assessed to identify locations where runoff and flooding could not reach the permeable pavement and to improve its performance. The assessment was conducted by dividing the actual infiltration by the potential permeable pavement infiltration, which included the storage capacity of the permeable pavement's media, the infiltration

capacity of the natural soil, and the drain capacity. Surfaces with a relative performance lower than 0.7 were removed from the potential permeable pavement area. As a result, 1.8 ha was considered effective, while 0.7 ha was deemed ineffective. A cumulative distribution of the effectiveness assessment is shown in Figure 19; areas to the left of the striped line are effective, whereas areas to the right are ineffective. The figure also displays the results of the other designs, which will be discussed later.

Figure 20 Cumulative distribution of permeable pavement efficiency for the different designs



The figure shows that the sensitivity of the 0.7 threshold is relatively low, as the full capacity is utilized for most of the permeable pavement area. Locations where permeable pavement infiltrated less than 70% are shown in Appendix D. These locations are primarily where permeable pavement is used to capture upstream runoff, and to a much lesser extent, where it is used for flood attenuation. This is due to the higher water levels present at those flood attenuation locations.

2. The assessment of decoupled surfaces toward permeable pavement was conducted to reduce flooding caused by geographical features that prevent water from reaching the permeable pavement. This assessment was carried out through visual inspection. Subcatchments where decoupling led to flooding were set to not decouple in the final design. As a result, 580 m^2 (1.3%) of the total decoupled surface area was removed from the potential decoupled area (Appendix D). This indicates that, for the majority of the decoupled areas, decoupling surfaces toward permeable pavement did not cause flooding.

Generating runoff directly on the 2D mesh, instead of through the hydrological model, made it possible to carry out the first two corrections with only a limited impact on the simulation time: the model run time increased from 4 hours and 10 minutes to 5 hours.

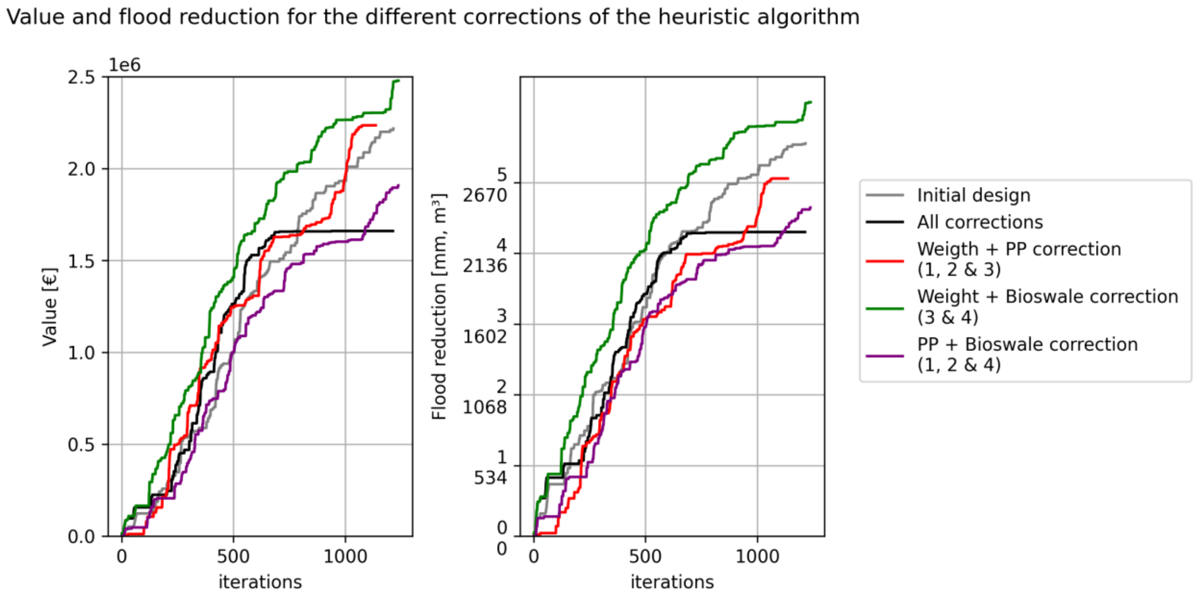
3. The node flooding weights were adjusted to compensate for the lack of surface flow between different areas in the 1D model. Hydrodynamical connections were analyzed and incorporated by modifying the flood node weights. Street cross-section hydrographs were used to determine

the main flow direction and the water level raster. The weights of nodes with lower water depths—but with a principal flow direction toward areas with higher water depths, due to elevation differences—were adjusted to match the weight of the higher water levels. As a result, the weights of 31 nodes were adjusted, with an average increase of 7.5 centimeters (Appendix D).

4. The bioswales were adjusted using hydrographs to determine the overflow volumes. Bioswales for which the total final 1D2D storage in the water balance was less than 80% of the water volume draining toward them were modified by reducing the total contributing area to the bioswale by the amount of overflow indicated by the hydrograph. As a result, 1,490 m^2 (0.6%) of the potential decoupled area was removed from the heuristic algorithm, affecting 4 out of the 19 bioswales.

These four corrections were implemented in the input values, and the heuristic algorithm was rerun, resulting in a final design. The development of this design, with all corrections applied, is presented in Figure 21. The figure shows how the design value increased over the iterations (left) and how the total flood reduction improved (right). The line representing the design with all corrections implemented is accompanied by the initial design for comparison, as well as three additional algorithm runs, each applying different combinations of corrections to study the effects of each individual adjustment.

Figure 21 1D total flood volume reduction and total design value over the iterations



With the corrected input values, the heuristic algorithm was unable to reach the maximum design value of €2 million. After 700 iterations, almost no additional flood reduction was achieved, which is indicated by the line stagnating. The implemented corrections created a soft lock in the process, where no effective SUDS could be found. Implementing different combinations of corrections revealed the sensitivity of the algorithm to the various aspects of the adjustments. It was found that the correction for permeable pavement primarily led to lower flood reduction, while a combination of weight and bioswale corrections resulted in more SUDS being implemented.

The final design resulted from the rerun of the heuristic algorithm, an overview of the design is presented in Table 11, a map of the final design is in Appendix E.

Table 11 Overview of the final design produced by method 1

SUDS	Storage volume [m^3]	Percentage of potential storage [%]	Area [m^2]	Investment [€]
Rain Barrel	50.2	11	50.2	€ 11,044
Bioswale	847	61	2,200	€ 159,947
Permeable Pavement	2364	8.3	14,239	€ 1,359,300
Green Roof	0	0	0	€ 0
Total	3261	-	25642	€ 1,530,291

Compared to the initial design, the areas allocated to permeable pavement and rain barrels changed, while there was no difference in the implementation of bioswales. Besides the lower design value, the final design retained the same characteristics as the initial design, with permeable pavement continuing to dominate in terms of value and storage volume. However, what did change was the specific locations where permeable pavement was implemented. By correcting the permeable pavement areas that were previously implemented, while leaving the other potential permeable pavement areas unchanged, the former were not selected in favor of the latter.

The final design was implemented in the 1D2D model and evaluated using the performance parameters. Despite a decrease in design value of €376,000 compared to the initial design, the area of infrastructure with more than 10 cm of water increased only slightly, from 1.37 ha to 1.39 ha, while the number of flooded buildings decreased further from 14 to 10. For the 10 buildings that still experienced flooding, the average water level decreased from 14.7 cm to 13.8 cm between the initial and final design, indicating a reduction in flood damage for the remaining affected buildings.

Once again, the 1D and 1D2D models were compared using a water balance to assess the accuracy of the 1D model, with the 1D2D model serving as the benchmark. The water balance is presented in Table 12. This comparison can be used to evaluate the effectiveness of the corrections implemented for the permeable pavement and bioswales by comparing the differences in this water balance to those in the initial design's water balance. This is shown in the last column of the table, where the change in water balance difference between the initial and final designs is displayed.

Table 12 Water balance comparing the 1D and 1D2D simulation of the final design

	Method 1 final 1D		Method 1 final 1D2D		Difference		Change
	m^3	mm	m^3	mm	m^3	mm	mm
Precipitation	26700	50.00	26700	50.00	0	0	0
Flood volume	5402	10.12	5218	9.77	184	0.34	1.92
Flood infiltration	0	0.00	537	1.01	-537	-1.01	0.09
PP Infiltration	2682	5.02	2345	4.39	337	0.63	0.18
Bioswale storage			285	0.53			
Bioswale Infiltration			414	0.78			
Total Bioswale	725	1.36	699	1.31	26	0.05	-0.10
Total SUDS:	3407	6.38	3044	5.70	363	0.68	0.08

The water balance shows that the total flood volume in the 1D2D model was 0.34 mm lower than in the 1D model. This indicates that the implemented SUDS were more effective than the 1D model had estimated. This result is a significant difference compared to the initial design, where the 1D model overestimated the effect of the SUDS, as the flood volume in the 1D2D model was 1.58 mm higher than in the 1D model. Therefore, it can be concluded that the difference in flood volume between the 1D and 1D2D models is independent of previous designs constructed within the same models.

The difference in bioswale storage between the 1D and 1D2D models decreased from 0.15 mm in the initial design to 0.05 mm in the final design. This indicates that the bioswale correction was successful in reducing the discrepancy between the two models, although it still resulted in one additional flooded building compared to the base model (Appendix G). However, the difference in permeable pavement infiltration increased by 0.18 mm, from 0.45 to 0.63 mm. This suggests that the correction applied to the potential permeable pavement area was unsuccessful. The reason for this was that permeable pavement was implemented in different locations compared to the initial design, and the infiltration capacities of these newly selected areas had not yet been assessed. Given that the correction for permeable pavement reduced the ability of the heuristic algorithm to select effective SUDS and did not improve the water balance, removing this aspect of the correction may enhance the overall effectiveness of the methodology.

With the construction of the final design, method 1 has been completed. Although this method successfully reduced both the flooded area and the number of flooded buildings in the initial and final designs, its actual efficiency can only be determined by comparing these designs to alternative designs produced using method 2.

4.4. Design method 2: only the 1D2D model

The heuristic design method 2 consisted of four steps: first, a simulation of the base 1D2D model was conducted to identify flooding locations, followed by three design phases in which SUDS were implemented and simulated simultaneously. Each phase produced a design that was based on the results of the previous step and involved adaptations to the earlier design. A map showing these designs can be found in Appendix F.

SUDS were implemented starting with the least expensive options and progressing to more expensive measures once the potential of the less costly SUDS near flooded infrastructure and buildings had been exhausted. As a result, rain barrels and bioswales were implemented first, followed by permeable pavement. Initially, permeable pavement was installed upstream of flooded locations to capture runoff; later, it was applied directly at the flooded sites to store floodwater. The implementation of permeable pavement was prioritized at locations with the largest areas experiencing more than 10 cm of flooding and with the greatest number of flooded buildings. Table 13 provides an overview of how the design evolved over the various phases.

Table 13 Overview of the designs made in method 2

SUDS		First design	Second design	Third design
Rain barrel	Storage volume [m^3]	109	109	109
	Percentage of potential storage [%]	32	32	32
	Area [m^2]	109	109	109
	Value [€]	23,320	23,320	23,320
Bioswale	Storage volume [m^3]	847	789	789
	Percentage of potential storage [%]	60	56	56
	Area [m^2]	2,200	2,200	2,200
	Value [€]	148,350	148,350	148,350
Permeable pavement	Storage volume [m^3]	3159	3328	3388
	Percentage of potential storage [%]	8.8	9.0	9.0
	Area [m^2]	23,333	17,920	17,700
	Value [€]	1,822,631	1,805,350	1,822,750
Green Roof	Storage volume [m^3]	0	0	0
	Percentage of potential storage [%]	0	0	0
	Area [m^2]	0	0	0
	Value [€]	0	0	0
Total	Storage volume [m^3]	4115	4226	4286
	Area [m^2]	25,642	20,229	20,009
	Value [€]	1,994,301	1,977,020	1,994,420

Large portions of the potential rain barrels and bioswales were implemented since these SUDS are the least expensive options. Rain barrels and bioswales that were not implemented were either located downstream or too far upstream from flooded locations. Green roofs were not implemented due to their high cost compared to the other SUDS.

After the first design phase, the second and third designs were created after corrections were made to the previous design based on the results of the 1D2D model. The newly available design value caused by these corrections was then used to implement additional SUDS. The correction process consisted of three steps:

1. Permeable pavement infiltration assessment.
2. Assessment decoupling to permeable pavement.
3. Bioswale storage assessment.

1. Permeable pavement infiltration was assessed by dividing the actual infiltration by the potential infiltration, which included the storage capacity of the permeable pavement's media, the infiltration capacity of the natural soil, and the drain capacity. Permeable pavement areas with a relative performance below 0.7 were removed from the design. As a result, 1.5 ha was considered effective and 0.7 ha ineffective in the first design, while 1.2 ha was assessed as effective and 1 ha as ineffective in the second design. A cumulative distribution of the effectiveness assessment for all designs is shown in Figure 20.

2. The decoupling of permeable pavement was assessed through visual inspection to identify geographical barriers that prevented runoff from decoupled areas from reaching the permeable pavement surface. Subcatchments where decoupling caused flooding were reconnected to the 1D sewage system. As a result, 671 m^2 (1.3%) of the total decoupled surface area was reconnected to the sewage system in the first phase. In the second phase, there were no areas that needed to be recoupled.

3. The bioswales were adjusted using hydrographs to determine the overflow volumes. Bioswales for which the total final 1D2D storage was less than 80% of the water volume draining toward them were modified by reducing the total contributing area in line with the total overflow indicated by the hydrograph. As a result, 1,170 m^2 of the total implemented area draining toward bioswales was recoupled to the sewer system, affecting 3 of the 16 bioswales. After the second phase, no further corrections to the bioswales were necessary.

In terms of design composition, little changed after the second phase. This is due to the high degree of similarity between the first and second designs. Therefore, it may be more time-efficient to limit the process to two phases.

The designs resulting from each phase were assessed using the performance metrics, as presented in Table 14. These performance parameters will later be used to determine which of the two methods is most effective at reducing flooding. By examining how the parameters evolve over time, it becomes possible to identify which parts of the process are more effective than others.

Table 14 Performance of the designs made using method 2

	Base model	First phase	Second phase	Third phase
No of flooded buildings	21	18	16	16
Area > 10cm on infrastructure [ha]	2.18	1.36	1.18	1.42

The number of flooded buildings started at 21 in the base model and decreased to 18 and 16 after the first and second phases, respectively. After the second phase, the number of flooded buildings did not decrease further. The area of infrastructure with more than 10 cm of flooding decreased from 2.18 ha to 1.18 ha in the second phase, which proved to be the best performing design, as the area increased again in the third phase. Within the three iterations, the model simulations did not provide sufficient information to explain the differences between the designs. The fact that the third iteration performed worse than the second and first illustrates the lack of effective assessment mechanisms for the SUDS in the 1D2D heuristic method. To compare method 2 with method 1, the most effective design will be used, in this case that is the one resulting from the second phase.

4.5. Comparing method 1 and 2

The comparison between method 1 and method 2 takes place at two moments: after the simulation of the initial design in method 1 and after the simulation of the final design. This comparison is used to answer the fourth research question and to determine which design method is the most effective.

4.5.1. Initial design method 1 - first phase method 2

After the initial design for method 1 and the first phase for method 2, both methods have been made within approximately 10.5 hours of modeling and implementation time. This comparison is relevant because it allows for an analysis of how the designs evolve and which parts of the design methodologies are most effective. Moreover, at this stage, no information generated by the 1D2D model has been used to create the method 1 design, meaning that using only this stage of the design method can reduce the potential cost of the 1D2D modeling software, as well as the effort and information needed to construct the base 1D2D model.

For the comparison of the design methods, two performance metrics were proposed based on the policy of the municipality of Bloemendaal: the number of flooded buildings and the total area of infrastructure with more than 10 cm of surface flooding. The results for these performance metrics are presented in Table 15.

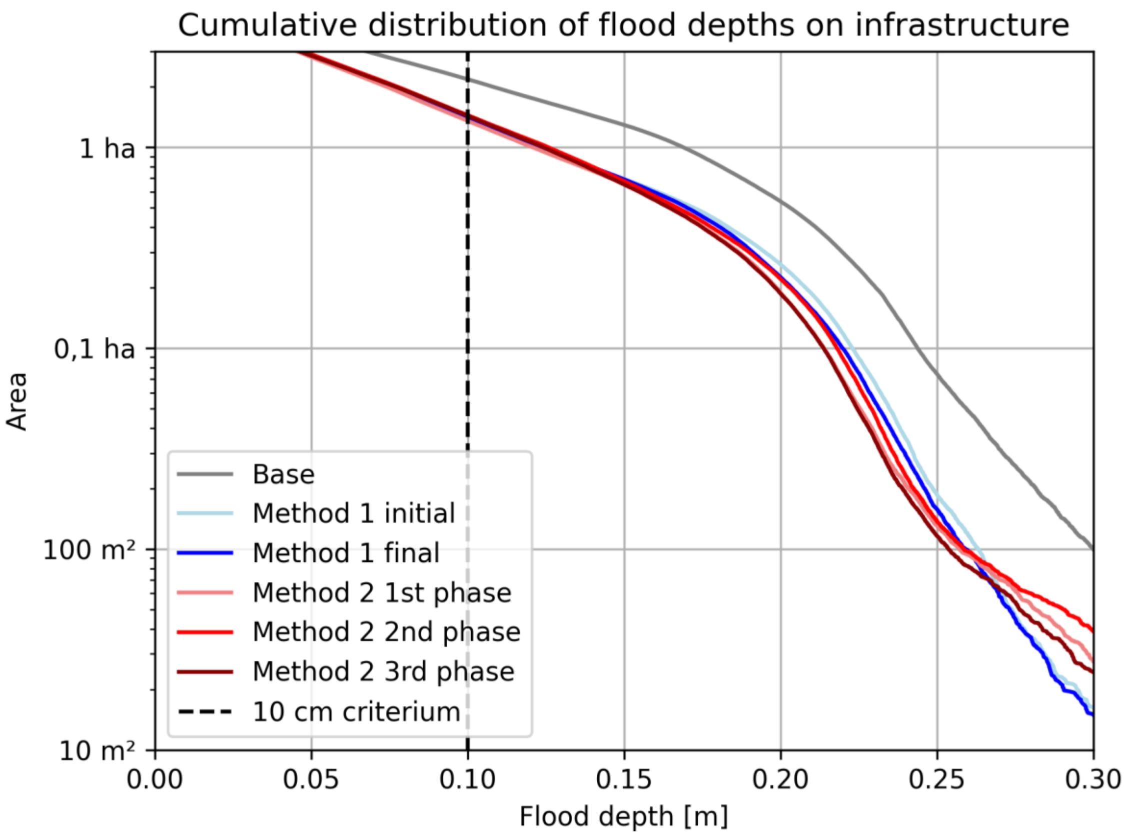
Table 15 Performance metrics for both initial designs

	Base	Method 1 initial design	Method 2 phase 1 design
No of flooded buildings	21	14	18
Area > 10cm on infrastructure [ha]	2,18	1,37	1,36
Design Value	€ 0	€ 1,994,301	€ 1,903,412

The total flooded surface area on infrastructure was nearly equal for both designs, but the total number of flooded buildings was higher in the method 2 design compared to the method 1 design. The average water level in the remaining flooded buildings for method 1 was 14.4 cm, whereas for method 2 it was 15.9 cm. This indicates that method 1 was more effective at reducing flooding than method 2 in the initial stages. Considering return on investment, method 1 prevented flooding at a cost of € 284,000 per building, while method 2 prevented flooding for a cost of € 634,000 per building.

To better understand the underlying patterns behind the performance metrics, it is useful to examine the distribution of flood depths on infrastructure across the different designs. This is presented in Figure 22, which shows a cumulative distribution of flood depths on infrastructure for each design as well as for the base model. The figure also includes a line indicating the 10 cm flooding threshold.

Figure 22 Cumulative distribution of flood depths between different designs



The figure shows that method 2 performed better at reducing flood depths up to approximately 26 cm, whereas method 1 was more effective at reducing the flood depths larger than 26 cm. This pattern can be explained by the shape of the weight factor function, which favored the reduction of higher flood depths. This also accounts for why method 1 was more effective at reducing the number of flooded buildings, as buildings are generally situated at higher elevations than the surrounding infrastructure. Adjusting the shape of the weight factor function could influence the distribution of flood depths.

4.5.2. Final SUDS designs

After the initial designs, the processes from both methods progressed into their respective final designs. After the final design phase, both methods required approximately 21 hours of modeling and implementation time. For each method, the final design selected was the one that performed best according to the performance metrics: for method 1, this was the design obtained after correction of the input values, while for method 2, it was the second iteration. A comparison of both designs based on the performance metrics is presented in Table 16.

Table 16 Performance metrics of the final designs

	Base	Method 1 final design	Method 2 final design
No of flooded buildings	21	10	16
Area > 10cm on infrastructure [ha]	2,18	1,38	1.18
Value	€ 0	€ 1,530,291	€ 1,977,020

Method 1 was more effective at reducing the number of flooded buildings, while method 2 was better at reducing the area of infrastructure with more than 10 cm of flooding. As explained in the previous section, this difference is due to method 1 assigning greater value to reductions in higher flood depths. Given the 23% lower design value, the final design from method 1 provides a better return on investment compared to method 2. Method 1 prevented flooded buildings at a rate of € 139,000 per building while method 2 had a rate of € 395,000 per building

One of the main differences between the two design methods is that method 1 can calculate the efficiency of each individual SUDS, whereas method 2 cannot. This allows the data obtained from method 1 to be applied to the SUDS implemented in method 2. Table 17 compares both designs based on the (weighted) flood reduction according to the heuristic algorithm. The SUDS implemented in method 2 are categorized according to how they were assessed by the heuristic algorithm in method 1. The values presented in the table are the maximum values obtained for each SUDS in the heuristic algorithm. The bottom rows show the SUDS that method 1 assessed as efficient but that were not implemented in method 2. Table 17 divided over every SUDS type is in Appendix H.

Table 17 Comparison of final designs based on heuristic algorithm results

Method 2 design:	Method 1 assessment:	Efficient	Not efficient	Not assessed
Implemented	Weighted reduction [m^3]	1266	-6	
	Reduction [m^3]	940	130	
	Value [€]	859,806	454,388	834,496
Not implemented	Weighted reduction [m^3]	1878		
	Reduction [m^3]	1208		
	Value [€]	850,047		

The table shows that, in the final design of method 2, €454,000 worth of SUDS have been implemented that would result in very little or even negative weighted flood reduction according to the heuristic algorithm. Additionally, €834,000 worth of SUDS in method 2's design were never assessed by the heuristic algorithm. This indicates that these unassessed SUDS are either located upstream of an inefficient SUDS or not situated within 300 meters upstream of a flooded node, making their efficiency unknown.

This table highlights the limitations of method 2. Implementing SUDS based solely on their proximity to areas with flooding did not necessarily result in effective solutions; in several cases, their impact was even negative.

5. Discussion

In this chapter, the results of this study will be discussed, the discussion will compare the results of method 1 to methodologies proposed by other studies, it will discuss the transferability of this methodology on other drainage systems and the performance metrics that were used in this study.

Of particular interest is the performance of the method used in this study compared to methods used in other studies. Especially studies were optimization has been performed, since that methodology aims to reach the most optimal design and a heuristic methodology aims to approach an optimal design (Martí & Reinelt, 2022). Comparing the result of this heuristic methodology to optimal designs could theoretically give an estimate of how well the methodology was able to reach an optimal design.

Optimization SUDS design for pollution control, flood reduction and cost was performed by Karamai et al. (2022). It used only a 1D model and calculated flood risk reduction based on the flood volume weighted by the probability of occurrence of the event using multiple events. The study decreased flood risk by 27%. This result is comparable to this study were a 26% reduction in flood volume was reached. The main difference is that Karamai et al. (2022) only used a 1D model and did not quantify the hydrodynamical effects of SUDS.

Optimization for maximum flood depths, life cycle cost and land occupation was performed by Wang et al. (2022). The urban drainage model used was a simplified 1D sewer model and a 1D model was used to represent the street surface. By incorporating green roofs, permeable pavements and bioretention sells, this study reached a reduction of 56% of the maximum flood depths. The main difference comparing this study to Wang et al. (2022) is that SUDS were only modelled in the hydrological model and the interactions with overland flow were not taken into account. Simulating the street surface using a 1D model also assumes that flooding is confined between the street boundaries, which was not the case in most parts of Bloemendaal in this study.

A study that was able to use 1D2D in optimization for flood reduction and design value was performed by Ferrans et al. (2022). Designing for a small catchment (22.9 ha), it was able to completely remove flooding at a cost of 1.5, 3 or 2 million euros for T= 50 years events lasting 30, 120 or 360 minutes respectively. The method used only land-use as a location constraint and was therefore able to implement SUDS on a much larger part of the catchment than in this study. The value of the design was comparable to the maximum value of the design in this study while the study area was more than twice as small.

The methodology that is proposed in this study is only used on a single drainage system. The specifics of this drainage system may have influenced the results in multiple ways which influences the transferability of the methodology on other catchments. Firstly, Bloemendaal does not contain a large share of public land, this reduces the potential of SUDS implementation compared to other catchments. Secondly, the evolution of the maximum value per cubic meter flood reduction (P_{max}) that is best suitable may depend on the study area, when the study area contains a large potential for very efficient SUDS, it may result in a more efficient design when the P_{max} is increased in smaller steps compared to larger steps when most potential SUDS in the study area are not verry efficient.

The performance metrics that are used in this study are based on the policy of the municipality of Bloemendaal (Haren, 2021). The policy states that flooding is acceptable on infrastructure when water levels are under 10 cm and are assumed not to flood if the water level against the walls is lower than 10 cm. This approach does not value reducing higher flood levels to flood levels just over 10 cm while that would in practice lead to significant reductions in the total damage. Implementing different performance metrics to determine the best design would lead to a better assessment .

During recent events, high ground water levels caused by high seasonal rainfall have led to the flooding of infrastructure and buildings in the study area (Witte & Van den Eertwegh, 2025). Increasing infiltration trough the SUDS proposed in this study might increase the flood risk caused by high ground water levels. This should be taken into account by incorporating sufficient drainage capacity when SUDS are implemented, avoiding area's with a risk of high ground water tables and incorporating the effect of SUDS on ground water tables in the assessment of the potential SUDS.

6. Conclusion

This thesis was conducted to study the usage of 1D and 1D2D urban drainage models in the design process of Sustainable Urban Drainage Systems (SUDS) to prevent flooding. While multiple methodologies exist for designing SUDS in urban drainage systems, this study focused on two heuristic methodologies: method 1, which uses both a 1D model and a 1D2D model, and method 2, which uses only a 1D2D model. To investigate the design process, a main research question was formulated:

“What are the trade-offs between 1D and 1D2D models for designing SUDS for flood reduction?” This was answered trough the following sub-questions:

1. How can a heuristic methodology be used to design SUDS preventing flooding using 1D and 1D2D models?

The first research question focused on how heuristic logic can be applied to design SUDS in urban drainage systems. This question was answered by analyzing the results of the different steps in the heuristic methodologies. Method 1 enabled the selection and implementation of the most efficient SUDS before implementing less efficient options. By implementing the design in 1D2D and using the results for further corrections, the number of flooded buildings was reduced. From this, it can be concluded that the heuristics were effective. However, there is still room for improvement because the infiltration performance of permeable pavement did not improve, and the corrections did not lead to a design that reached the maximum budget of €2 million. Method 2 did not provide enough information to objectively determine which SUDS should be implemented and which SUDS were effective.

2. How can a 1D model be used to quantify the effects of SUDS on flooding?

Answering the second research question consists of two parts: first, the ability of 1D models to simulate SUDS and their effects on flooding, using 1D2D as a benchmark; and second, the information the 1D model provides and its usefulness for the design process. This was examined through the application of method 1, where SUDS were implemented in 1D and then translated into 1D2D for comparison via a water balance. The results of the water balance comparison showed significant differences in SUDS storage and infiltration between the 1D and 1D2D models, with the 1D model overestimating total infiltration. In most cases, this only led to shallow flooding of less than 10 cm in the 1D2D model. More importantly, there was a significant difference in the simulated flood volume between the 1D and 1D2D models. For the second part of this research question, the 1D model was able to calculate the effects of individual SUDS on flood volumes, compensating for the lack of accurate flood depth and flooded building calculations. The model's scale made it hard to detect the effects of smaller-scale SUDS.

3. How can a 1D2D model be used to quantify the effects of SUDS on flooding?

Answering the third research question consists of two parts: first, the ability of the 1D2D IberSWMM plugin to simulate SUDS and their effects on flooding; and second, the usefulness of the information provided by the 1D2D model for the design process. For the first part, simulating SUDS in Iber had its limitations because the software does not provide a physical infiltration model that accurately simulates the storage media and natural infiltration of SUDS, simulating SUDS was only possible by using a conceptual model. Additionally, the mesh size of 1.5 m^2 caused some bioswales to have smaller storage than intended, leading to leakage. For the second part, the 1D2D model can calculate flood depths and identify which buildings are at risk of flooding. While this information allows for the identification of areas where flood reduction is needed, it did not provide insight into the most effective methods for achieving this. After implementation, the results yielded limited information on which SUDS were the most effective. Generating runoff directly on the 2D mesh at locations were permeable pavement was implemented enabled the assessment of hydrodynamic interactions of permeable pavement.

4. How effective are SUDS designed with a heuristic method combining 1D and 1D2D models at reducing flooding compared to SUDS designed with a heuristic method using only a 1D2D model?

The fourth research question was addressed by comparing the initial and final designs of both methodologies based on the results of the 1D2D model. The results indicated that method 1, which employs both a 1D and a 1D2D model, was more effective at reducing the number of flooded buildings. In contrast, method 2, which utilizes only the 1D2D model, was more effective at minimizing the total area of flooded infrastructure. The difference was caused by the larger weight method 1 puts on the highest flood depths that cause the flooded buildings. The flood depth distribution further endorsed the effectivity of method 1 at reducing larger flood depths. Notably, the difference in effectiveness between the two methods increased in the final design compared to the initial design. Considering the fact that larger flood depths cause the most damage, it can be concluded that method 1 is the most effective at reducing flooding.

Returning to the main research question, *'What are the trade-offs between 1D and 1D2D models for designing SUDS for flood reduction?'* 1D models are best-suited for assessing the effectiveness of individual SUDS in reducing flood volumes and for iteratively constructing a design. In contrast, 1D2D models provide more accurate calculations of total flood volume, flood depths, and identification of which buildings are at risk of flooding. This conclusion can help determine which model is most suitable at various stages of the design process: 1D can be used to construct a design, while 1D2D can be employed to assess whether and where flooding causes problems and to make adjustments to the design.

7. Recommendations

This study presents a novel methodology for designing SUDS in urban drainage systems. During the development and analysis of the Method 1 heuristics, several potential enhancements have been identified that could further improve performance:

1. *Incorporate multiple criteria in the heuristic methodology:* This study presents a methodology focused specifically on reducing flooding through SUDS. In practice, however, SUDS are often implemented to achieve multiple objectives, such as improving water quality, reducing drought risk, mitigating heat stress, and enhancing biodiversity. The heuristic methodology could be expanded to incorporate these additional criteria. For instance, integrating models for water quality or other relevant aspects alongside the urban drainage model would allow for a more comprehensive, multi-objective design approach.
2. *Adding more SUDS or different measures to the heuristic methodology:* Currently, the heuristic method incorporates four types of SUDS. With these four SUDS, the method is not able to achieve a significant further reduction in flood volume beyond what was accomplished by the initial €2 million design. To develop a methodology that can meet the municipality's defined flood reduction targets, additional measures could be included. These do not have to be limited to SUDS; the methodology could also be extended to incorporate grey infrastructure solutions, thereby broadening the range of possible interventions.
3. *Increase heuristic algorithm efficiency:* The efficiency of the heuristic algorithm can be improved, particularly by excluding the more expensive SUDS types during the initial reduction steps. Currently, the algorithm spends considerable time assessing permeable pavement, which seldom meets the efficiency threshold in the early stages of the process. By modifying the algorithm to focus only on less expensive SUDS during these steps, overall computation time and performance could be enhanced.
4. *Built the final method 1 design by amending to the initial design:* In Method 1, the heuristic input data is adjusted based on the 1D2D results, after which a completely new design is created and evaluated. This often results in a design that is very different from the initial plan. Effective SUDS identified in the initial design may be penalized during the correction process, while less effective SUDS may remain unaffected. Consequently, SUDS that were previously deemed less effective might appear more favorable despite their actual effectiveness still being uncertain, leading to a potentially less optimal final result. It may be more efficient and effective to develop the final design by amending the initial design—similarly to the approach used in Method 2. This adjustment could also accelerate the process toward the final result.
5. *Use multiple design events in the heuristic algorithm:* The heuristic algorithm used in this study currently considers only a single design event. Incorporating multiple design events with varying hyetograph shapes and return periods could enhance the robustness of the design, as their effects can be averaged (Pritsis, et al., 2024). Additionally, results for each event can be weighted by the probability of occurrence to estimate the reduction in flood risk (Karamai et al., 2022). Different flood weight functions can also be applied if regulators set specific objectives for different events. For example, the municipality of Bloemendaal requires the drainage system to fully handle a 35.7 mm rainfall event (Mogos et al., 2023). In such cases, a new weighting function could be formulated to place greater emphasis on lower flood levels, and the

combined effects of both the design event and the 35.7 mm event could be used to assess effectiveness.

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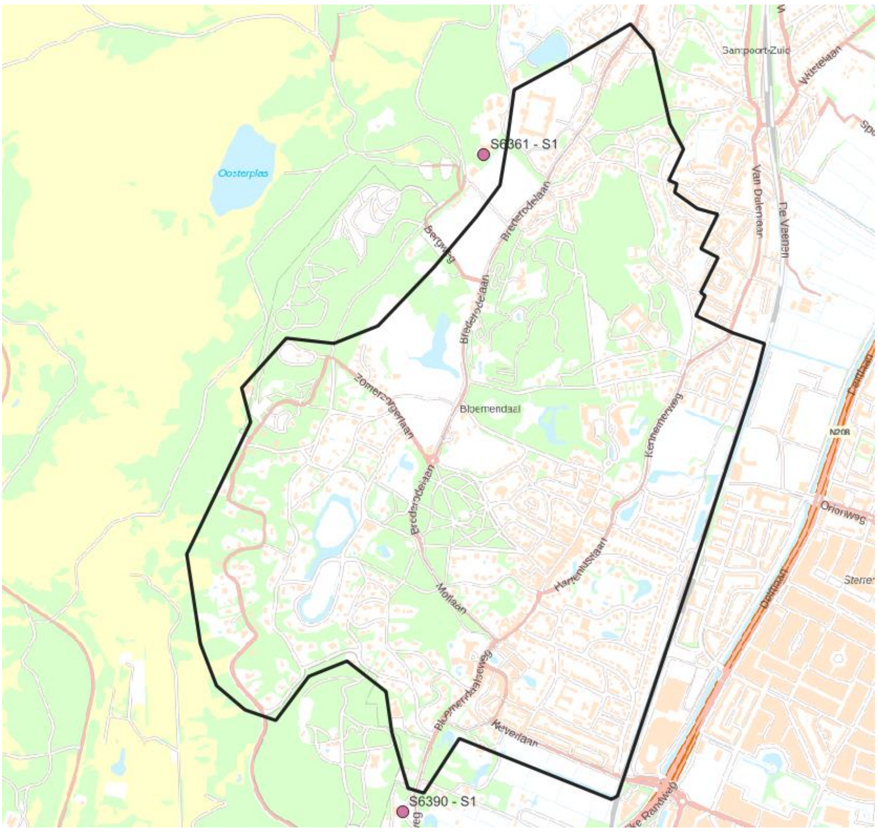
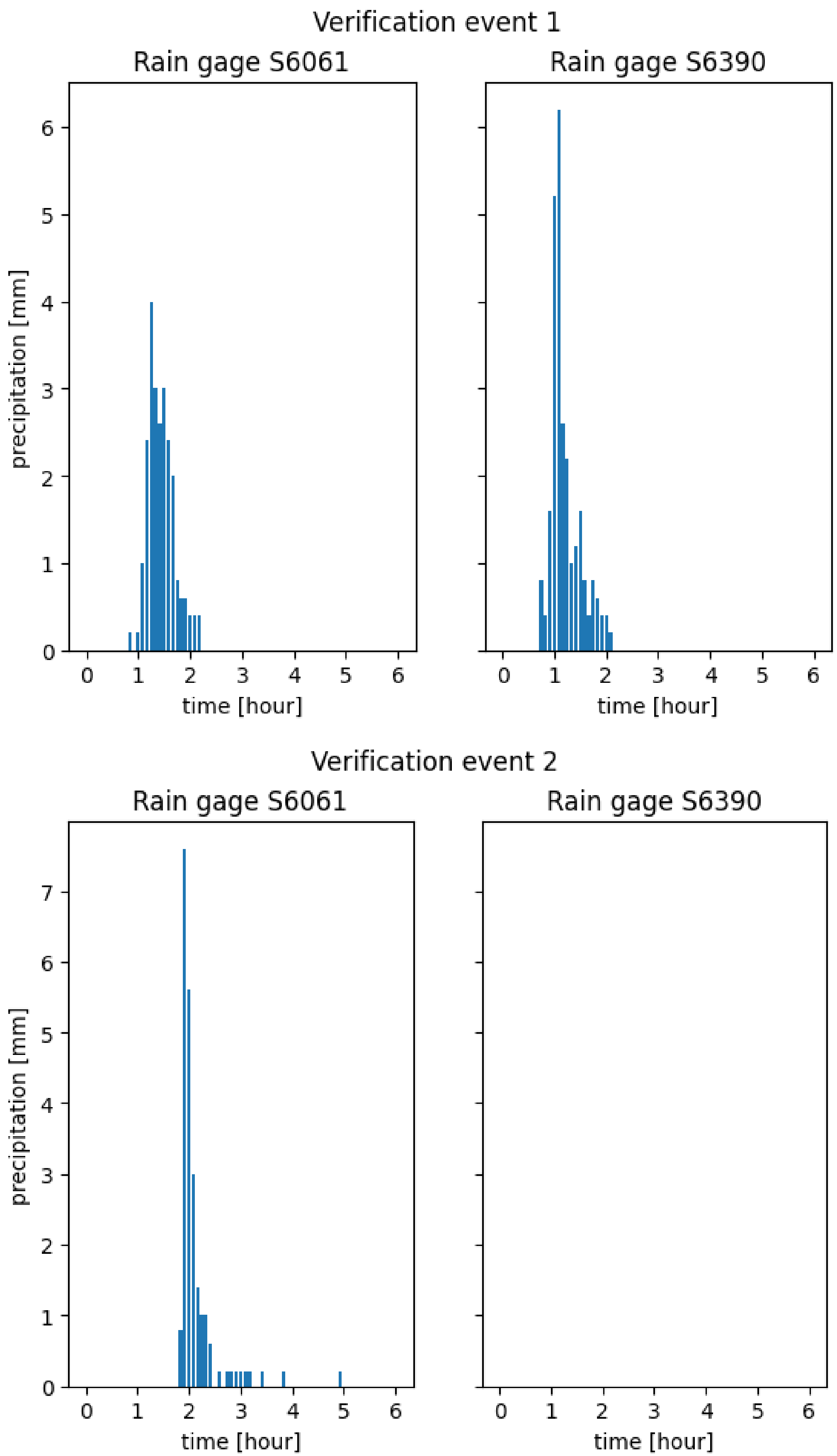
Appendix A Verification-validation events

Seven events have been used for trial and error followed by validation

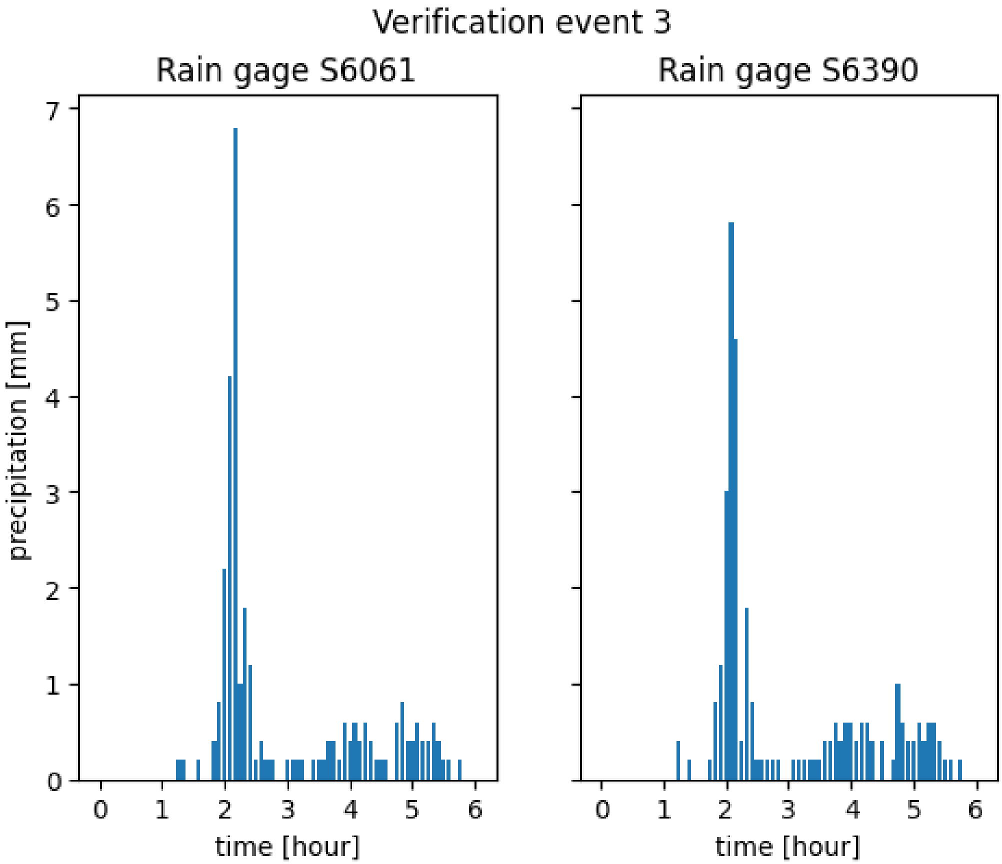
nr	start time	End time	Highest intensity	Total precipitation	Trial-error/Validation
1	2019-06-10 21:00:00	03:00:00	23.00	24.0	Trial-error
2	2021-06-27 18:00:00	00:00:00	21.40	23.0	Validation
3	2019-06-05 21:00:00	03:00:00	19.40	31.2	Trial-error
4	2019-10-01 14:00:00	20:00:00	18.20	34.0	Validation
5	2019-06-15 03:00:00	09:00:00	17.80	37.0	Trial-error
6	2020-08-16 17:00:00	23:00:00	14.20	16.0	Validation
7	2021-10-21 00:00:00	06:00:00	13.80	41.0	Trial-error

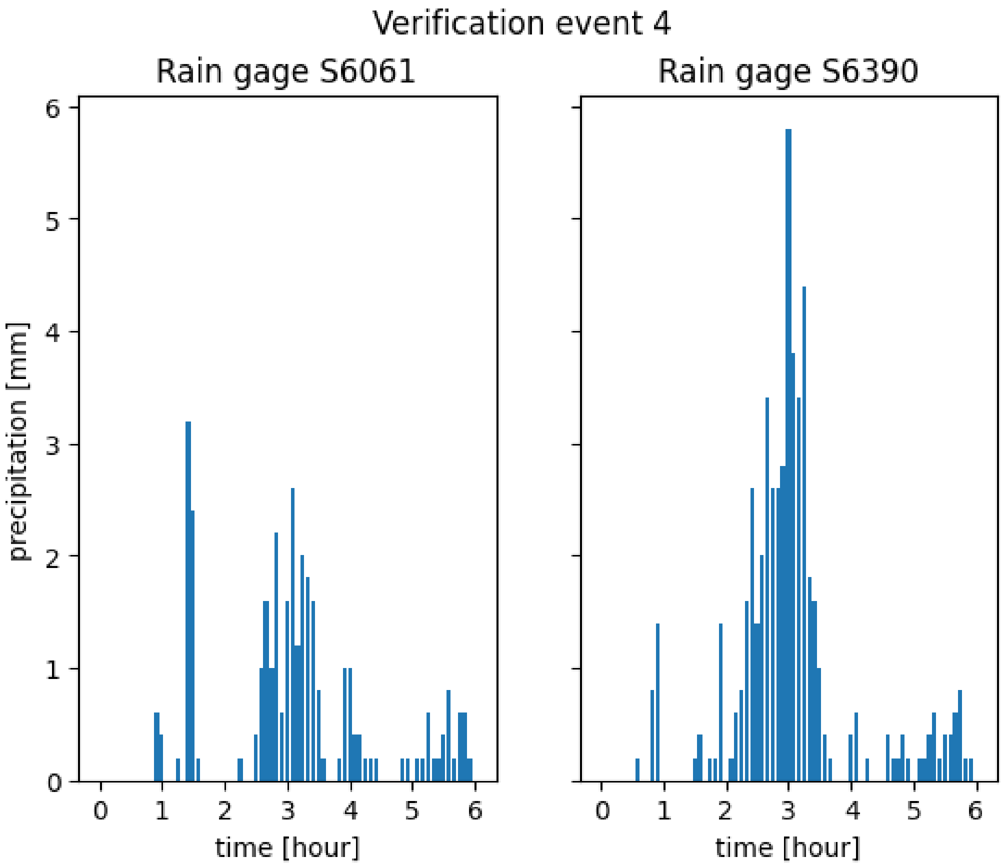
Rain gage S6361 has been used for validation. Data from the rain gage has been compared to rain gage S6390 just south of the study area, the data shows large differences between both rain gages.

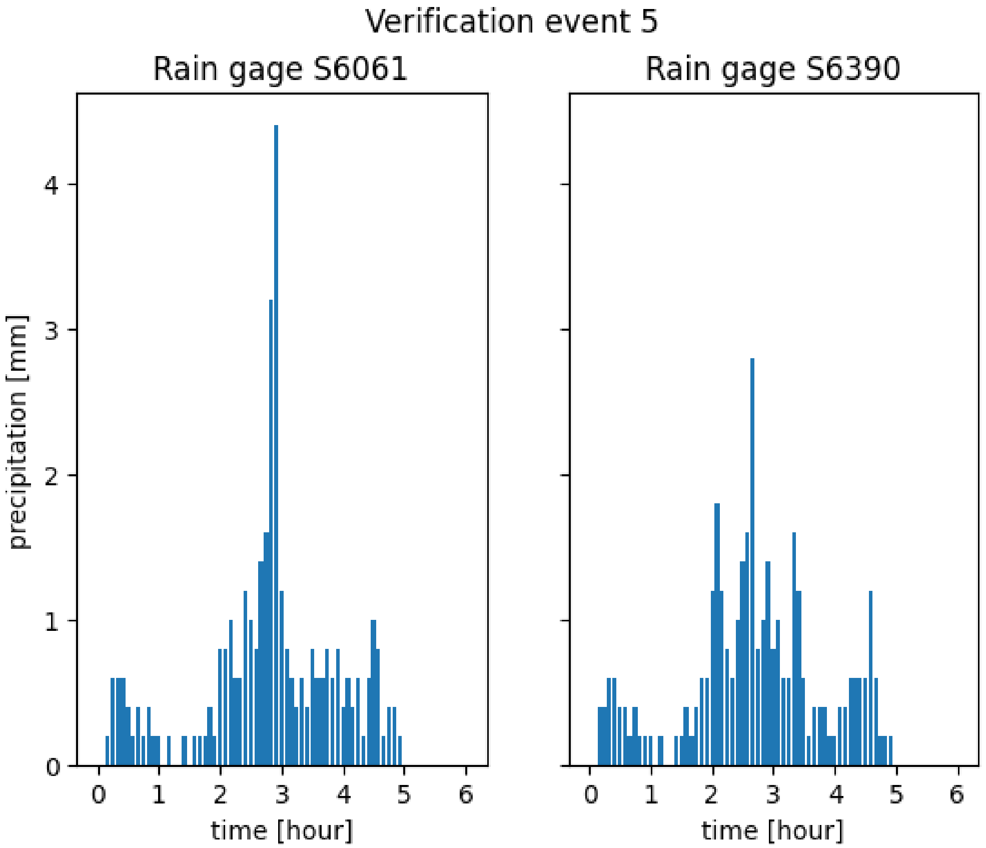


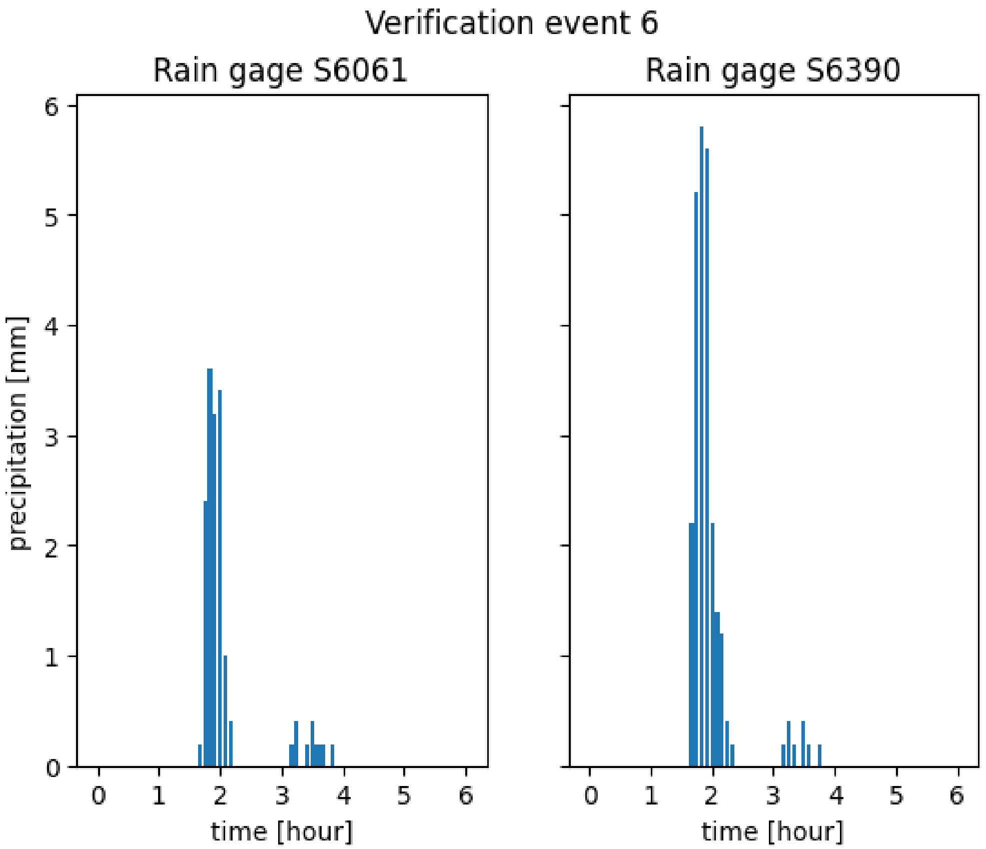


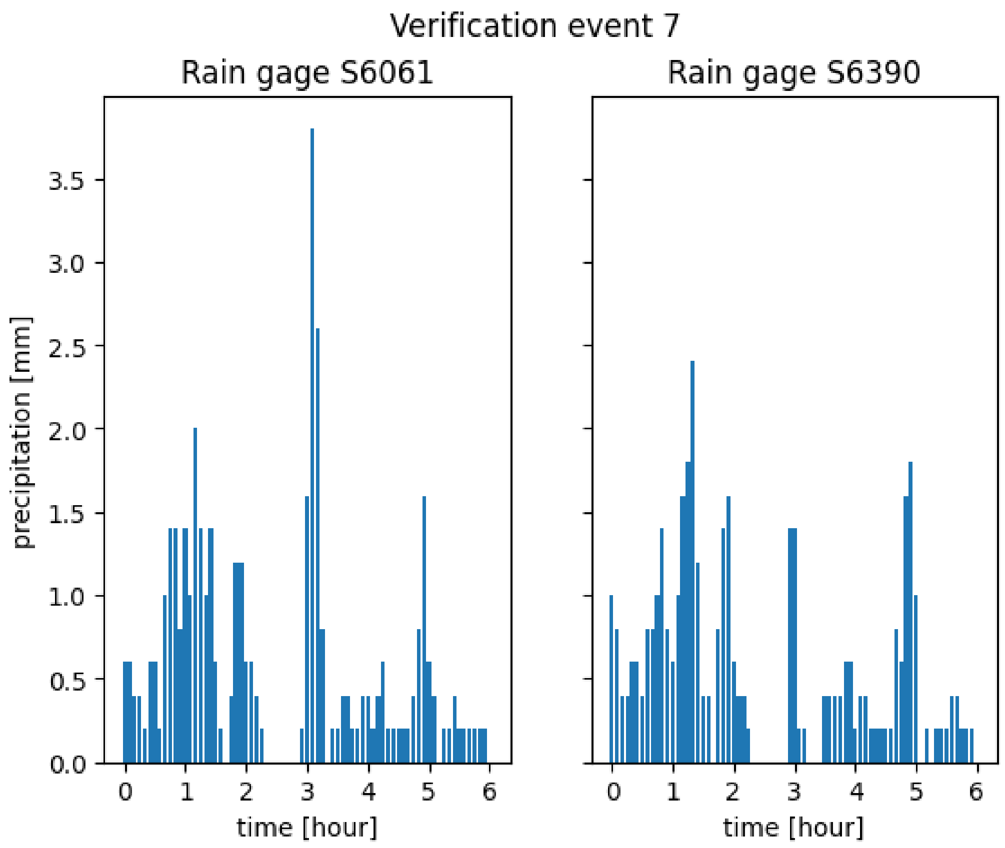
Verification event 1
Rain gage S6061
Rain gage S6390
precipitation [mm]
time [hour]
Verification event 2
Rain gage S6061
Rain gage S6390
precipitation [mm]
time [hour]


Verification event 3
Rain gage S6061
Rain gage S6390
precipitation [mm]
time [hour]
time [hour]
0
1
2
3
4
5
6
7


Verification event 4
Rain gage S6061
Rain gage S6390
precipitation [mm]
time [hour]
time [hour]
0
1
2
3
4
5
6


Verification event 5
Rain gage S6061
Rain gage S6390
precipitation [mm]
time [hour]
time [hour]
0
1
2
3
4
5
6


Verification event 6
Rain gage S6061
Rain gage S6390
precipitation [mm]
time [hour]
time [hour]
0
1
2
3
4
5
6


Verification event 7
Rain gage S6061
Rain gage S6390
precipitation [mm]
0.0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
0
1
2
3
4
5
6
time [hour]
time [hour]

Appendix B SUDS cost calculation

Rain barrel

Source: https://www.riool.net/kennisbank/financien-gemeentelijke-watertaken/kostenkengetallen/kostenkengetallen-per-rioleringsobject/regenton

Cost rain barrel	€ 40.00	/200L
Cost	€ 200.00	/ m^3
Inflation	10%	
Final cost	€ 220.00	/ m^3

Permeable pavement

Source: https://www.riool.net/kennisbank/financien-gemeentelijke-watertaken/kostenkengetallen/kostenkengetallen-per-rioleringsobject/infiltrerende-verharding

Tiles	€ 25.00	/m^2
Removing existing tiles	€ 2.30	/m^2
Construction	€ 18.70	/m^2
Storage foundation	€ 130.00	/m^3
Removing Existing foundation	€ 16.40	/m^3
Cost /m surface	€ 46.00	/m
Cost/m depth	€ 146.40	/m
Depth	0.4	m
Cost	€ 104.56	/m^2
Storage	0.2	m
Cost	€ 522.80	/m^3
Inflation	10%	
Cost	€ 575.08	/m^3

Bioswale

Source: https://www.riool.net/kennisbank/financien-gemeentelijke-watertaken/kostenkengetallen/kostenkengetallen-per-rioleringsobject/wadi

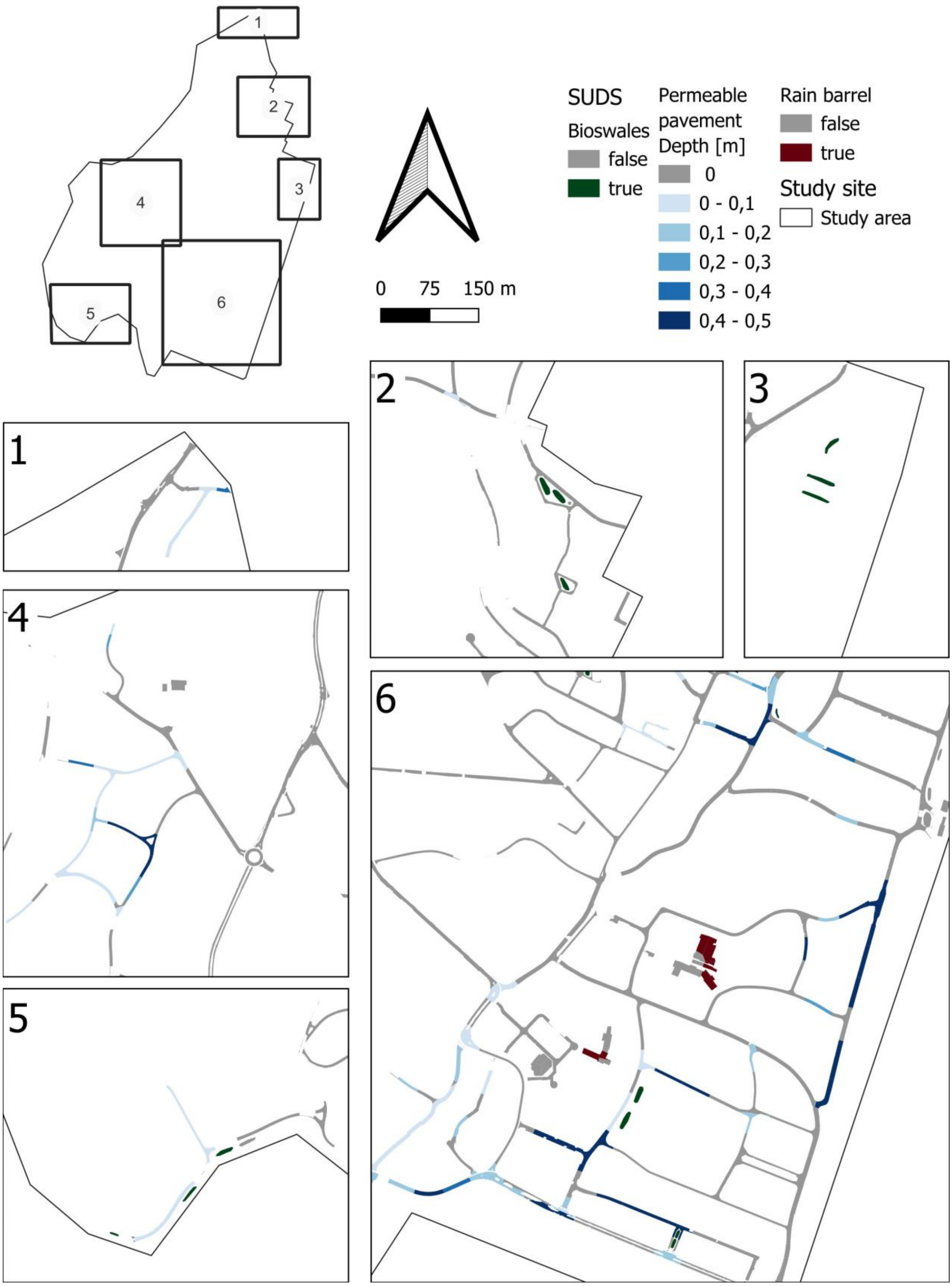
Top layer	€ 4.10	/m^2
Storage substrate	€ 130.00	/m^3
Construction	€ 15.00	/m^3
Digging	€ 15.00	/m^3
Drain	€ 6.00	/m
Width	5	m
Average depth	0.3	m
Substrate depth	0.3	m
Cost	€ 56.60	/m^2
Cost/m	€ 6.00	/m
Storage	0.45	m^3/m^2
Cost	€ 128.44	/m^3 storage
Inflation	10%	
Cost/m^3	€ 141.29	/m^3 storage

Green roof

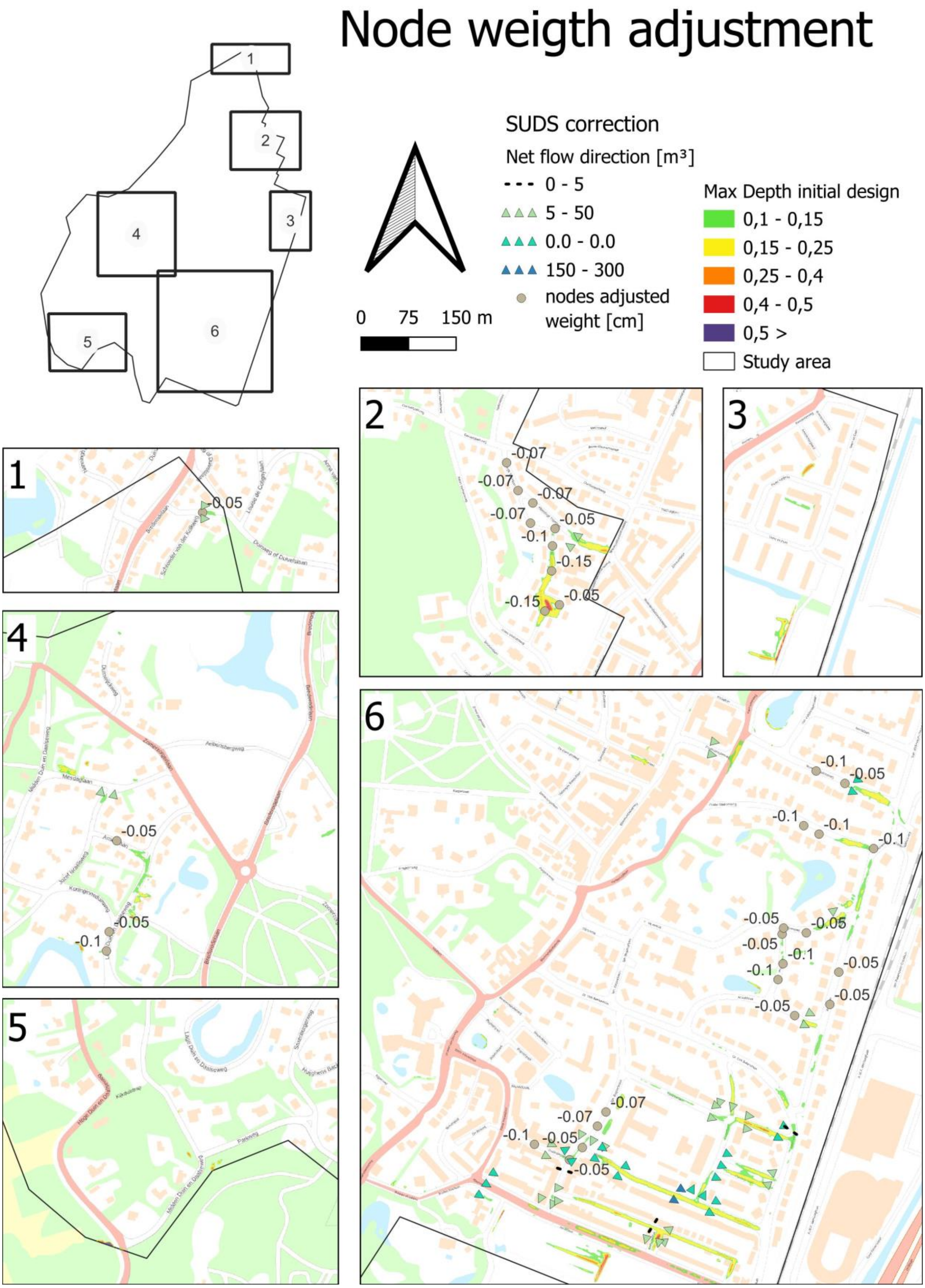
Source: https://www.homedeal.nl/dakbedekking/kosten-groendak/

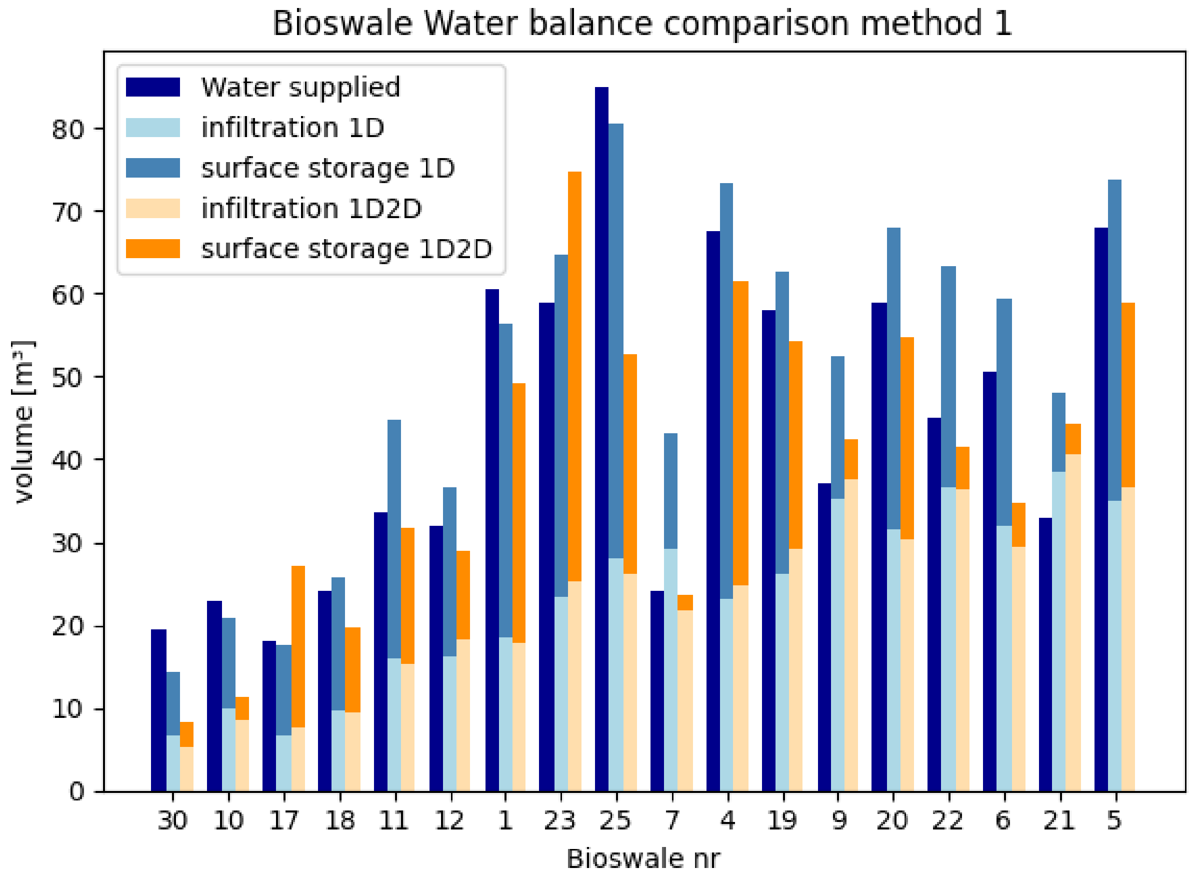
Intensive green roof cost	€ 150.00	/m^2
Storage	0.05	m^3/m^2
Cost per m^3 storage	€ 3,000.00	/m^3
inflation	10%	
final cost per m^3 storage	€ 3,300.00	/m^3

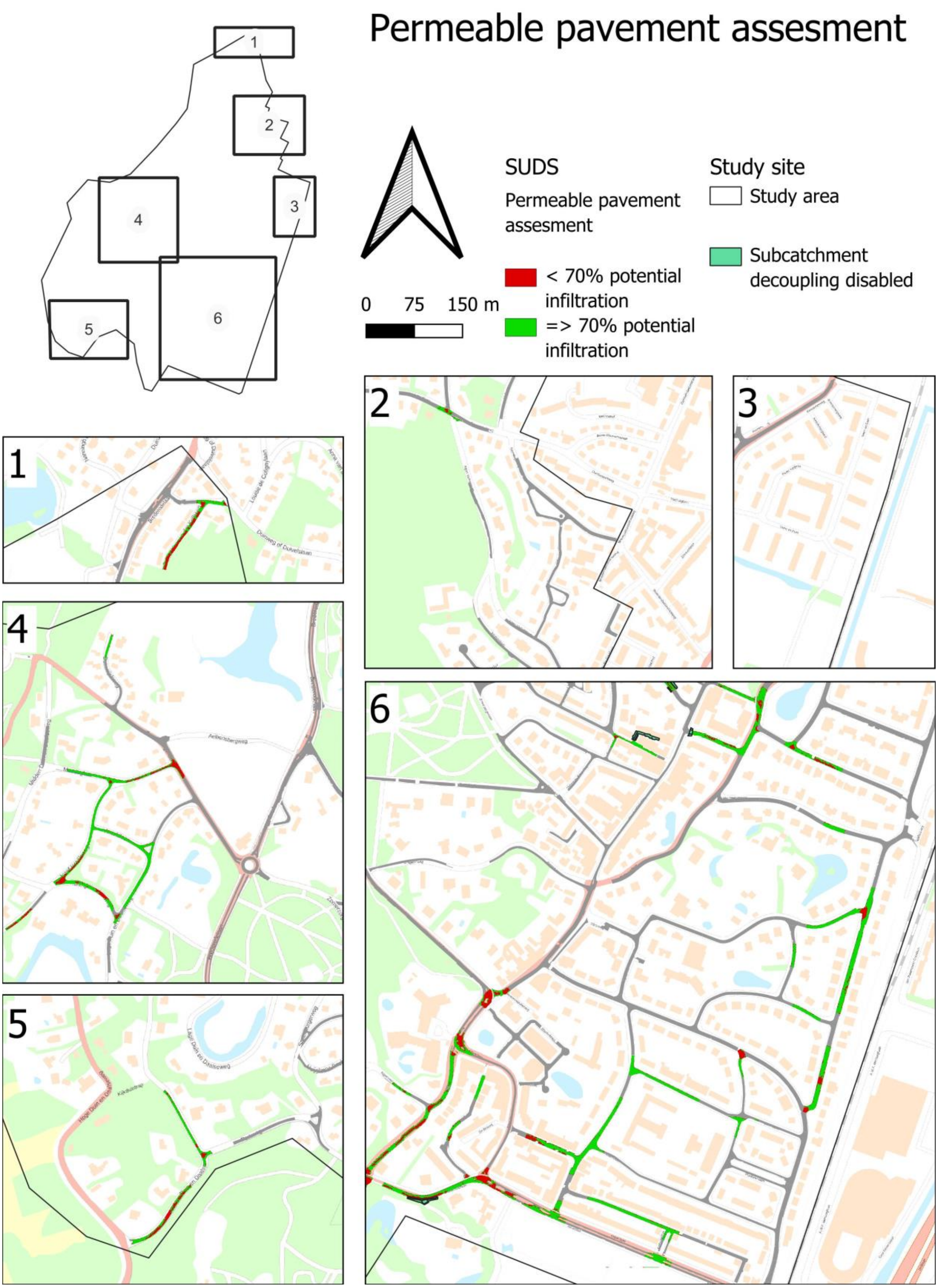
Appendix C Initial method 1 design



Appendix D Method 1 correction




Bioswale Water balance comparison method 1
Water supplied
infiltration 1D
surface storage 1D
infiltration 1D2D
surface storage 1D2D
volume [m³]
0
10
20
30
40
50
60
70
80
30
10
17
18
11
12
1
23
25
7
4
19
9
20
22
6
21
5
Bioswale nr

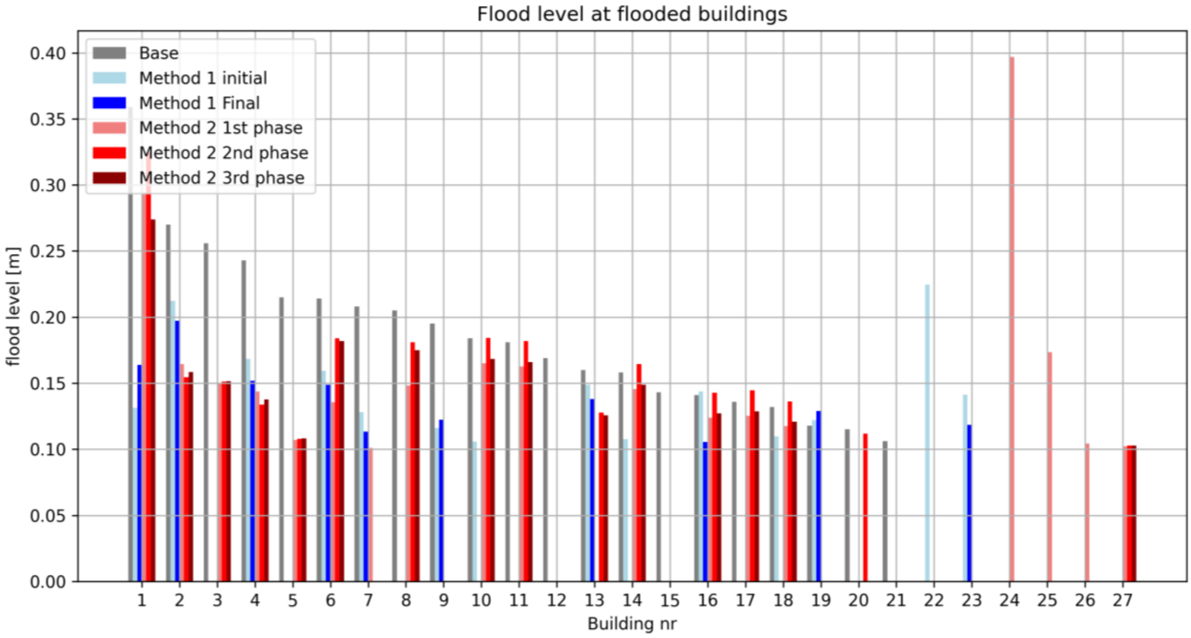


Permeable pavement assesment
SUDS
Permeable pavement assesment
< 70% potential infiltration
=> 70% potential infiltration
Study site
Study area
Subcatchment decoupling disabled
0
75
150 m
1
2
3
4
5
6

Appendix E Final method 1 design

Appendix F 1D2D design method

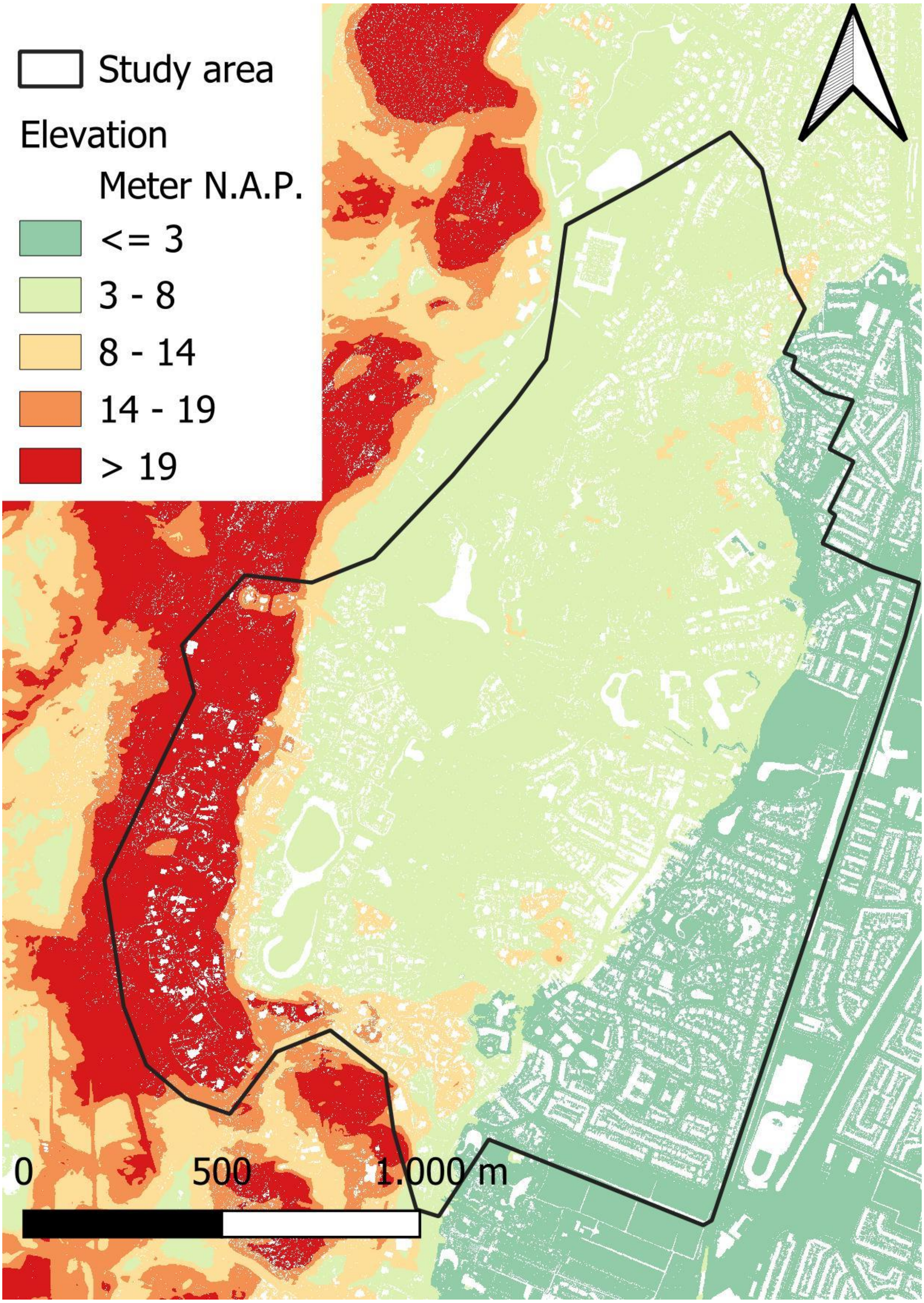
Appendix G Flooded buildings



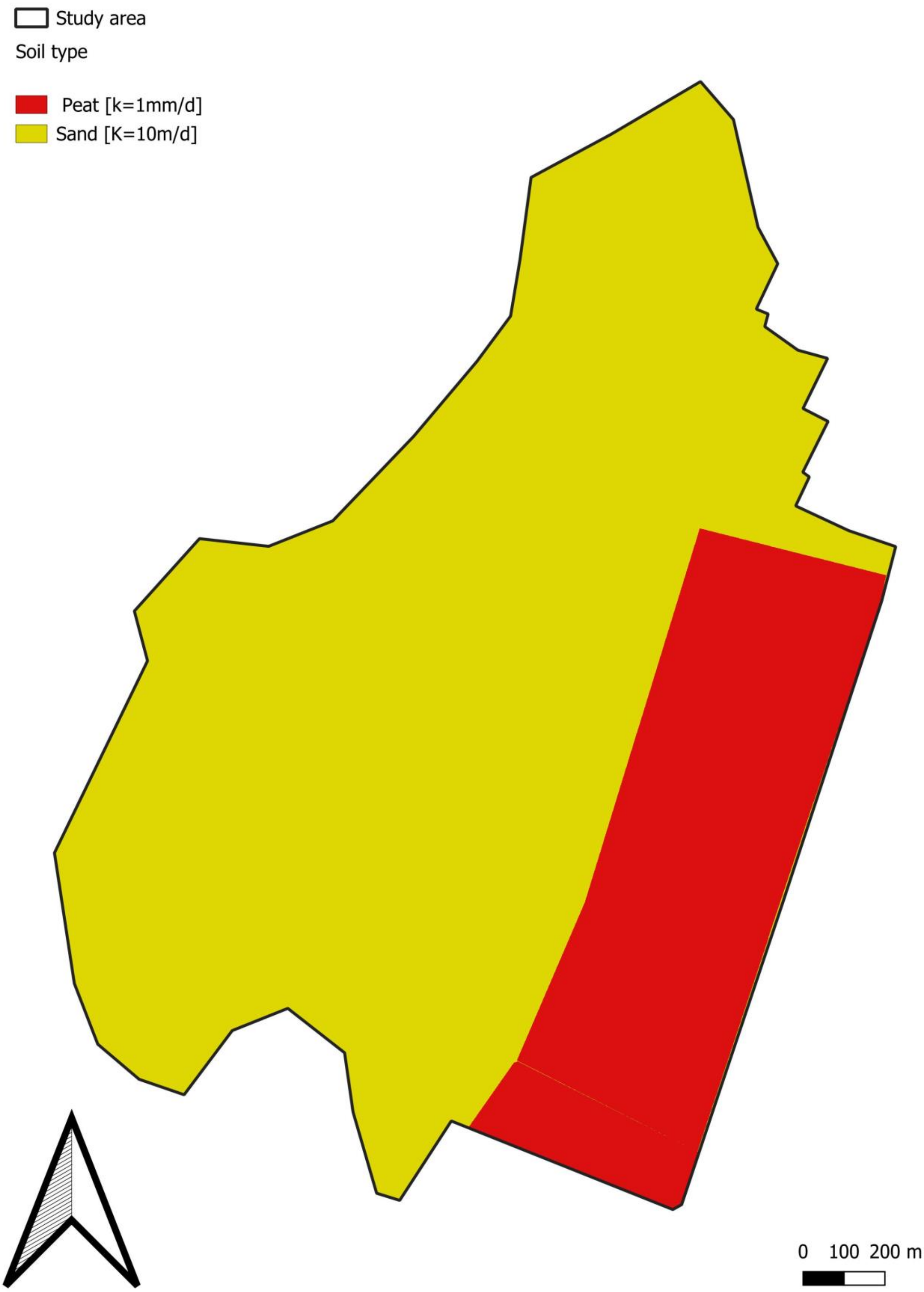
Appendix H Comparison final designs

		Implemented			Not implemented		
	Method 2:	Weighted reduction [m^3]	Reduction [m^3]	Value [€]	Weighted reduction [m^3]	Reduction [m^3]	Value [€]
	Method 1:						
Permeable pavement	Implemented	842	628	726645	1689	1097	796749
	Not implemented	-11	126	444708			
	Not assessed			805667			
Rain Barrel	Implemented	17	30	5830	10	8	4290
	Not implemented	5	4	9680			
	Not assessed			7810			
Bioswale	Implemented	407	282	127331	179	103	49008
	Not implemented	0	0	0			
	Not assessed			21019			
Total	Implemented	1266	940	859806	1878	1208	850047
	Not implemented	-6	130	454388			
	Not assessed			834496			

Appendix I DTM of the study site



Appendix J Soil map of the study site



Appendix K Land-use map of the study site

Appendix L SUDS potential

Appendix M LID control parameters

Rain barrel:

Barrel height:	1000	mm
Flow Coefficient:	0.5	

Green Roof:

Berm Height:	10	mm
Vegetated volume fraction:	0.3	
Surface Roughness	0.1	
Surface slope	1.0	
Soil Thickness	100	mm
Porosity	0.35	
Field Capacity	0.15	
Wilting point	0.1	
Conductivity	50	mm/hr.
Conductivity slope	10	
Suction Head	3.5	mm
Thickness	100	mm
Void Fraction	0.5	
Roughness	0.1	