Prinseneiland, Amsterdam: a water balance.

Master of Science Thesis

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Abstract

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A research on urban water balance was conducted for Prinseneiland, Amsterdam, for the period 04.08.2015 – 03.08.2017. The most important fluxes to determine were the recharge to the groundwater system and the amount of groundwater that is drained by the sewerage system. This was done by integrating three modelling techniques: statistical assessment of the data, groundwater modelling (through MicroFEM) and hydraulic modelling (through InfoWorks). It was estimated that 70% of the rainfall on the island will turn into runoff that will reach the sewerage system. A recharge rate of 11 m^3/d was estimated for the phreatic layer. Groundwater infiltration rate into the sewerage system was approximated to be between 9-10 m^3/d, however this might be an overestimation due to measuring errors. This value could be calibrated in the hydraulic model, however not in the stationary groundwater model due to a conceptualization error. A combined sewer overflow of approximatively 648 m^3/year was estimated to be discharged from the island. All the parameters of the urban water balance, except for sewerage exfiltration and transpiration from trees were estimated. In conclusion, this methodology is useful in obtaining an overall and robust overview on urban water balances and the interconnections between sub-systems. However, a series of iterative steps between the models should be undertaken to minimise the errors that propagated through the study and to complete the modelling process.

Acknowledgements

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Bianca M. Mogos

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List of abbreviations

CSO – Combined **S**ewer **O**verflows

DWF – **D**ry **W**eather **F**low

KNMI – **K**oninklijk **N**ederlands **M**eteorologisch **I**nstituut

MDW – **M**easured **D**rinking **W**ater Flow

NAP - Normaal **A**msterdams **P**eil

RMSE – **R**oot **M**ean **S**quare **E**rror

RSS – **R**esidual **S**um of **S**quares

RTC - Real **T**ime **C**ontrol

TSA – **T**ime **S**eries **A**nalysis

WWF – **W**et **W**eather **F**low

CHAPTER 1

Introduction

Population is constantly rising, urban areas are expanding, and climatic changes are leaving a mark on the hydrologic system. Therefore, there is need for an integrated and more sustainable water management to create a safe environment. An urban water balance is an important tool that provides comprehensive information on the water system of a city. It offers valuable information on the status of the current water management practices and can help in making predictions for future scenarios.

1.1. Problem description

An urban water balance consists of two interrelated sub-balances, one for each sub-system: the sewerage system balance and the groundwater balance. The inputs are the rainfall and the drinking water. The outputs are the pumped waste water, overflow, runoff, evaporation, and infiltration. Groundwater infiltration and exfiltration into/from the sewerage system are either input or output, depending on which sub-system is being investigated.

Waternet's objective is to determine groundwater recharge, the groundwater's interactions with the sewerage system and the amount of overflow. The amount of recharge through pavement is the main interest, as it can be further used in groundwater models for the entire Amsterdam. Having an estimate of the fluxes between groundwater and sewerage system is an important for the waste water sector, as infiltration into the network might increase the operational costs of waste water treatment. It is also important for the groundwater management sector, as infiltration in the sewerage system could mean lower groundwater levels. Overflows are important, as they have environmental impact on the receiving water bodies, which is proportional to the amount and composition of overflow water (de Moel, et al., 2006, p.25).

All the above-mentioned fluxes are important for Waternet, as they influence the groundwater levels, which are of high importance in Amsterdam. Groundwater levels can cause several inconveniences *to society* if too high or too low. Low groundwater levels, for extended periods of time, cause the degradation of wooden foundation piles, which will cause further damage to the structure of old buildings. High groundwater levels might mean flooding and mould in basements.

However, these fluxes are hard to determine and to get an overview on. Some of them cannot be measured, only determined through model estimates. Therefore, from *the scientific perspective*, the aim of this research is to try an explore various possibilities of modelling an urban water balance to obtain a better view on how the systems are interconnected and obtain better estimates for the most important parameters.

1.2. Research aim and objectives

Therefore, the **main objective** of this study is to find out *How much precipitation recharges the groundwater system and how much groundwater is drained by the sewer system?*

To answer the main question, a series of sub-questions will be addressed. Overall, they will give a complete overview of the urban water balance.

i. How does the measured rainfall data on the island compare to the data obtained through the radar and from the KNMI (Koninklijk Nederlands Meteorologisch Instituut) database?
ii. What is the amount of direct surface runoff to the surface water and the sewer system?
iii. What is the amount of groundwater that flows to the surface water and sewer system and can the hydraulic model be calibrated with the data?
iv. What is the amount of overflow from the sewerage system?
v. What is the amount of rainfall that infiltrates through the paved areas?
vi. What is the amount of evaporation from trees?

Question number vi) could not be answered because the piezometers to measure the drawdown from trees were installed too late in the study.

1.3. Thesis layout

The thesis is organised in 8 chapters, as it follows:

Chapter 1 presents the scope of this research and the main research question to be answered.

Chapter 2 provides an overview on the theory of water balances and presents past research and their findings.

Chapter 3 describes the study area and its characteristics. It offers background information on how the study area was formed, its geology, hydrogeology, information on its water supply and sewerage system.

Chapter 4 lists the available data sources and inputs for the research, including rainfall, evaporation, groundwater levels and flow measurements in the water chain of the study area.

Chapter 5 describes the methodology used in this research project. It is structured in 3 subchapters., Each one of them includes description of the modelling technique, sensitivity analysis or calibration methodologies.

Chapter 6 presents the results of the models described in Chapter 5, by following the same structure. The chapter concludes with a section on the overall water balance.

Chapter 7 describes the limitations that this research has.

Chapter 8 lists the conclusions of the study, and discusses future improvements and recommendations.

CHAPTER 2

Theoretical framework

2.1. The urban water balance

An urban water balance, as defined by Grimmond et al., (1986, p.1397) is *"a box with unit surface from horizontal area that extends from roof level to a depth in the ground below which no net exchange of water occurs over the period of interest"*. It can be expressed through the following equations:

- Surface water balance:

$$P = Q_r + E + In \qquad (2.1)$$

- Groundwater balance:

$$\pm\Delta S_g = In + Q_{se} - Q_{Si} - Q_c - Q_l \qquad (2.2)$$

- Sewerage system balance:

$$Q_p = Q_{Dw} + Q_r + Q_{si} - Q_{se} - Q_o \qquad (2.2)$$

where P - precipitation, In - groundwater infiltration, Q_r - net runoff, E - actual evapotranspiration, ΔS_g - net change in storage, Q_{se} - sewage exfiltration, Q_{si} – groundwater drainage into the sewage, Q_c – flow to the canal, Q_l – groundwater leakage to the underlying layers, Q_p – amount of waste water pumped, Q_{DW} – amount of drinking water and Q_o – amount of overflow. The soil water balance is not taken into consideration in this study. All these fluxes are represented in Figure 2.1.

Precipitation is the driving factor of the hydrological system. It is redirected via multiple pathways. It will be lost through evapotranspiration, interception, infiltration, direct drainage to the surface water and part of it will turn into runoff.

Evapotranspiration is the most difficult to estimate loss due to the lack of measuring devices and difficulties in correlating urban micro-climate patterns with a rural one (Mitchell et al., 2001). In the temperate zone it is expected that evapotranspiration will decrease due to extensive impervious areas (Grimmond et al., 1986). In term of initial losses on impervious and semi-pervious areas, the values range from 0.1 – 2.5 mm/event, 2.5 mm/event in extreme situations (Koot, 1977, p. 40), with the most common values being between 1 – 1.5 mm (Guo, 2006, Falk and Niemczynowicz, 1979, Smart and Herbertson, 1992).

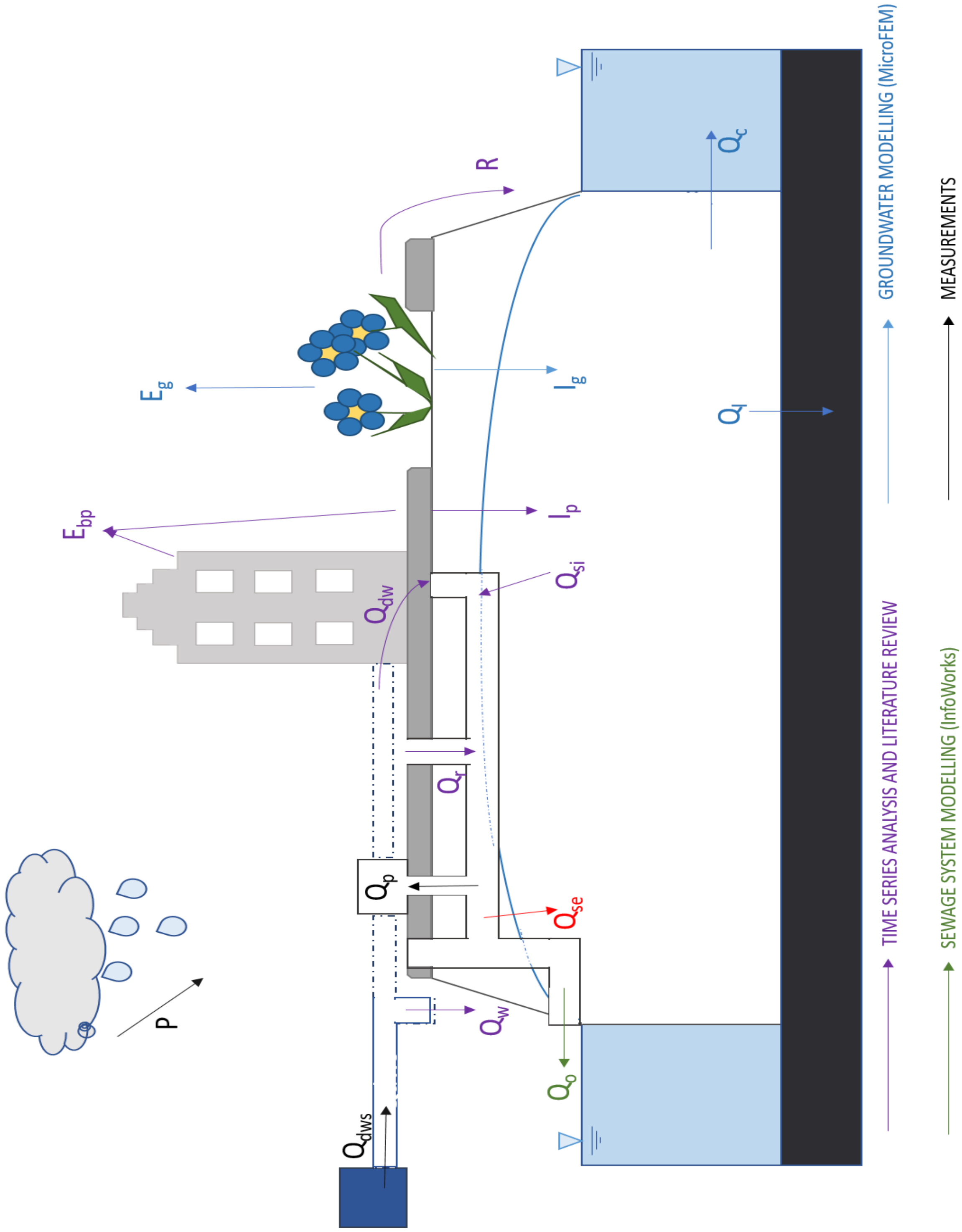


Figure 2.1 A schematisation of the urban water cycle at the study area.

Runoff is generated by impervious or semi-pervious surfaces, such as roofs, pavement and parking areas. The amount of runoff generated depends on the characteristics of the surfaces, the proportion of a certain surface type to the total contributing area to the sewerage system, slope, perviousness, amount of precipitation and initial weather conditions.

Losses through infiltration act as recharge for the groundwater storage (Mackay, Last, 2010). This happens through pervious areas. Infiltration occurs also through semi-pervious areas, such as through the connections of the pavement. The lost amount highly depends on the state of the pavement (an old pavement will allow less infiltration than a newer one, due to clogging of the inter-connections) (Smart, Herbertson, 1992, p.121). Therefore, the lost amount can only be estimated.

Recharge of the groundwater system happens through infiltration from the pervious and semi-pervious surfaces but also from sewage and drinking water supply systems which can provide a significant amount of recharge through leakages (Lerner, 2002). In Amsterdam, the amount of losses through the drinking water system are insignificant (internal communication), therefore they will not be taken into consideration in this study.

2.2. Previous research done

One study on Prinseneiland's water balance was done previously by Rutten (2013), under Deltares. The research's aim was to observe the processes in the urban water cycle and their interconnections, with a special interest in the possible effects of droughts (as an effect of climate change) on the elements of the urban water cycle. The research was based on a lumped model which used daily averages as input. The island was considered a homogeneous area to avoid long calculation times. Groundwater levels were simulated with two scenarios: completely paved area or unpaved. The model was validated on groundwater levels and sewer discharges. The research concluded that the effects on the groundwater levels will be proportional to the severity of the drought, due to less infiltration and recharge to the groundwater system (Rutten, 2013).

In literature there are few studies that focus on urban water balances: Grimmond and Oke, 1986, for Vancouver, Canada, and its suburban area, Hogland and Niemczynowicz, 1979, 1980 for Lund, Sweden and Van de Ven, 1985, Uunk and Van de Ven, 1984, Oldenkamp, 1988 for Lelystad, The Netherlands. Also, in 1990, Van de Ven makes a comparative study for water balances of Lund and Lelystad. What all these studies have in common is the estimation of evapotranspiration, which is usually the most inaccurate term of a water balance (Van de Ven, 1990).

Grimmond and Oke, 1986, developed a model for urban water balances which can be applied for different time ranges, and was later applied for Vancouver. The highlight of this model is that the evapotranspiration sub-model can be used with easily retrievable data.

CHAPTER 3

The study area

3.1. Background information

The area of study is Prinseneiland. It is a small island (~3 ha), located in the northwest of Amsterdam's city centre (Figure 3.1). It is part of the Westelijke Eilanden (Western Islands).

It was created at the beginning of the 17th century. In the construction process, material from the surrounding canals and peat from the IJ were used. Initially, warehouses that stored wood and tar were built on the island, and on its verges shipping wharves were constructed (Rutten, 2013, p.8). Nowadays, it is a residential area, with approximatively 694 inhabitants (§ 3.3.1) and a couple of small businesses.

It was chosen as research area due to its well-defined boundaries, the simplicity of the sewage and drinking water systems and due to the measurements availability. A schematisation of the island with all the measuring points can be seen in Annex A.

The surrounding canals form a well-defined physical boundary, but also give a constant head boundary condition for the groundwater model, which is the canal level. The sewer system and drinking water systems are simple, containing one pump installation with the correspondent measuring equipment, one overflow structure, and two flow meters for the drinking water supply. Therefore, having all the flows measured, Prinseneiland became the ideal area for Waternet's research on the urban water balance.

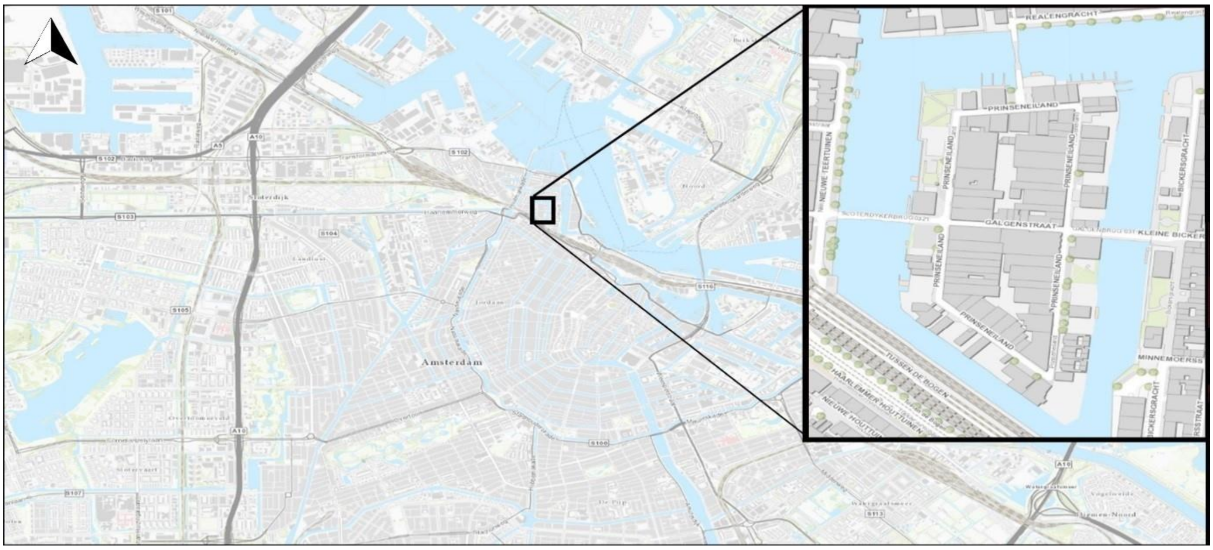


Figure 3.1 Location of Prinseneiland (Source: Waternet).

3.2. Geology and geohydrology

3.2.1. Geology

The island is situated on the western side of a Middle Pleistocene glacial basin. The basin was formed during the second to last glacial period (Saalian) (Schokker et al., 2015, p.2-4). The basin was later filled with deposits, resulted from alternating sea levels (Rutten, 2013, p.15). Therefore, the subsoil is an alternation of deposits of various origins (glacial, fluvial, marine, aeolian and organogenic) (Schokker et al., 2015, p.2). An extensive description on the geology can be found in Annex B.

The covering layer of Prinseneiland has the highest importance for this study. It consists of a mixture of peat, clay, construction material, wood residuals and sand which were used to lift the surface levels (Rutten, 2013, p.15). The classification made by TNO (2013) indicates that a layer of marine deposits from Middle Holocene should be present near the surface. However, the drilling descriptions (Annex B) indicate that that layer does not exist on the island and the depth of the fill material extends all the way to the bottom of the phreatic layer (approximatively NAP -3 m). Table 3.1 shows a detailed overview of the underlying layers of the island and their characteristics.

3.2.2. Geohydrological classification

In this study, the base of the hydrogeological system is taken as the Second Eem Clay (approximatively 15 m thick). Atop of it lies, from bottom up, the second aquifer, part of the Kreftenheye Formation. In between the second and first aquifer lies the "tussen laag", or the "in between layer", 2 m of clays of the Boxtel Formation. This confining layer is not expected to have much resistance. The clay and peat layer between the phreatic zone and first aquifer have a much higher resistance.

The phreatic layer is at a depth of approximatively NAP -3 m. Due to the heterogeneity and the relatively high clay content of the top layer, the hydraulic conductivity of this layer is expected to be very low. The urban setting of the island might have an impact on the groundwater flow in the phreatic layer: depending on how deep they are, the house basements might create resistance for the groundwater flow. Moreover, the bedding for a sewerage system consists of sand/gravel, which have a higher hydraulic conductivity.

Groundwater flow

Locally, in the first aquifer, at depths of NAP -10 m, the groundwater flows from the IJ towards the polder areas situated in the S, SW of the city, on an NW – SE direction. The isohypse pattern is shown in Figure 3.2.

Recharge of the phreatic aquifer happens due to excess rainfall. Infiltration occurs through the pervious and semi-pervious surfaces, which have a total surface of 14365 m^2 (MicroFEM). The water will drain towards the canals, which have a fixed water level of -0.4 m (Figure 3.3).

Table 3.1 Classification of the hydrogeological layers and their characteristics and representation in MicroFEM.

Model layer	Geological unit	Dominant lithology	Depositional environment	Age	Hydrogeological characterisation	Thickness [m]	Approximate upper and lower level [NAP m]
1	Anthropogenic layer	sand, very fine to very coarse; clay; humus; construction material and domestic waste	Anthropogenic	Late Holocene	Phreatic aquifer	variable	Top
	Formation of Naaldwijk	Medium coarse sand	Marine	Middle to Late Holocene			-3 m
2	Formation of Naaldwijk	Clay and very fine sands	Marine	Middle to Late Holocene	Aquitard	9,75	-3 m
	Formation of Nieuwkoop - Hollandveen	Peat, sometimes clayey	Organogenic	Middle to Late Holocene			
	Formatie of Naaldwijk	Clays	Marine	Middle Holocene			
	Formation of Nieuwkoop - Basis Veen	Peat	Organogenic	Early to Middle Holocene			-12.75 m
	Boxtel Formation	Sand, medium fine	Aeolian	Late Pleistocene	1st Aquifer	4	-12.75 m -16.75 m
3	Boxtel Formation	Clay, sandy clay, loam, peat	Aeolian, fluvial, lacustral and organogenic	Late Pleistocene	Aquitard	2,25	-16.75 m -20 m
	Kreftenheye Formation	Sand, medium fine to very coarse	Fluvial (Rhine Deposits)	Late Pleistocene	2nd Aquifer	8,25	-20 m -28.25 m
Base	Eem Formation The 2nd clay layer and the sandy layer	Sand, very fine to medium coarse; loam; clay, sometimes sandy, humic, peat	Marine	Late Pleistocene		16.75 m	-28.25 m -45 m
Base	Drente Formation	Sand, very fine to very coarse, sometimes clayey; clay, sometimes sandy or varved	Glacial (meltwater deposits)	Middle Pleistocene		~9 m	-45 m
	Drente Formation Gleten Member	Loam, clayey to gravelly; sand; very fine to very coarse; gravel; boulders	Glacial (till deposits)	Middle Pleistocene			~54 m

** Classification made as per Schokker et al., 2015, p.3

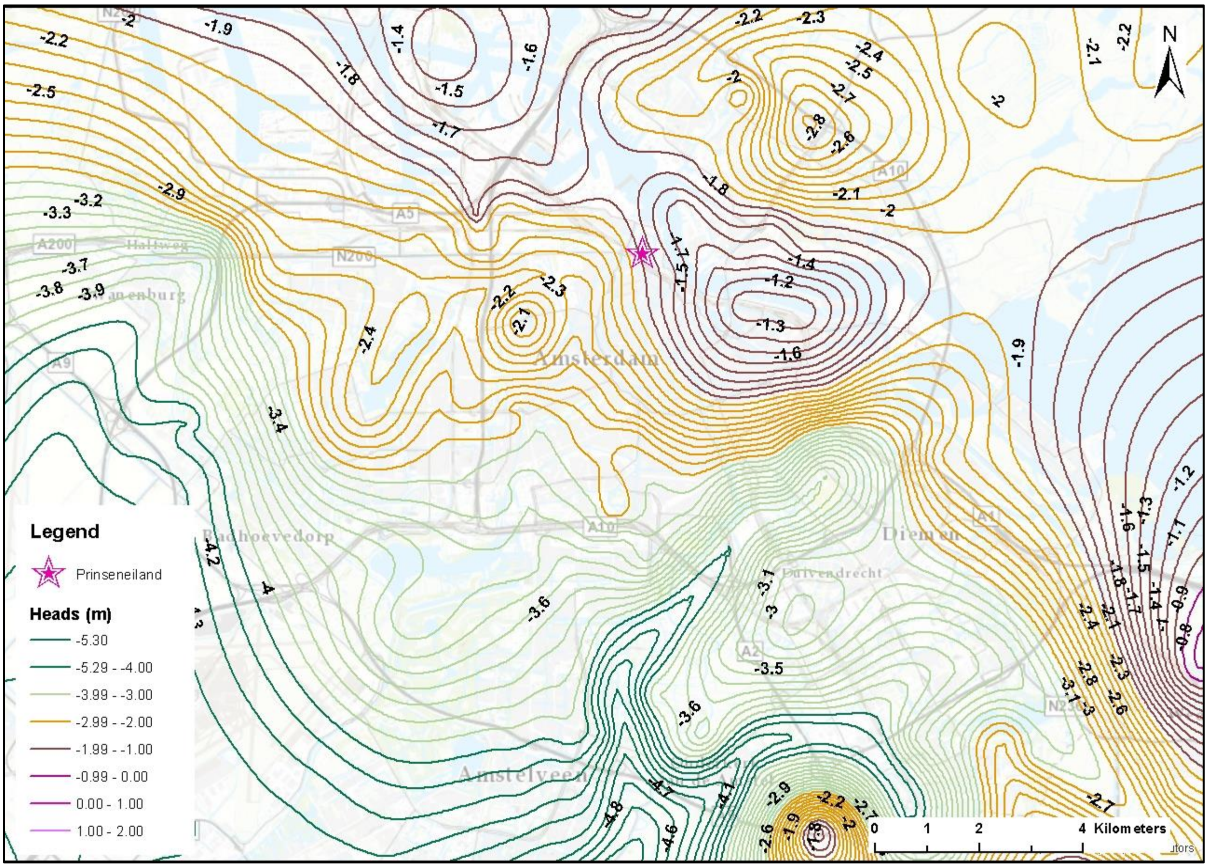


Figure 3.2 Isohypse map of the Amsterdam area, first aquifer. The star represents Prinseneiland. The map was generated in ArcMap by using data points with the average for groundwater levels over 2 years. The piezometers used in making the map and the data can be seen in Annex C.

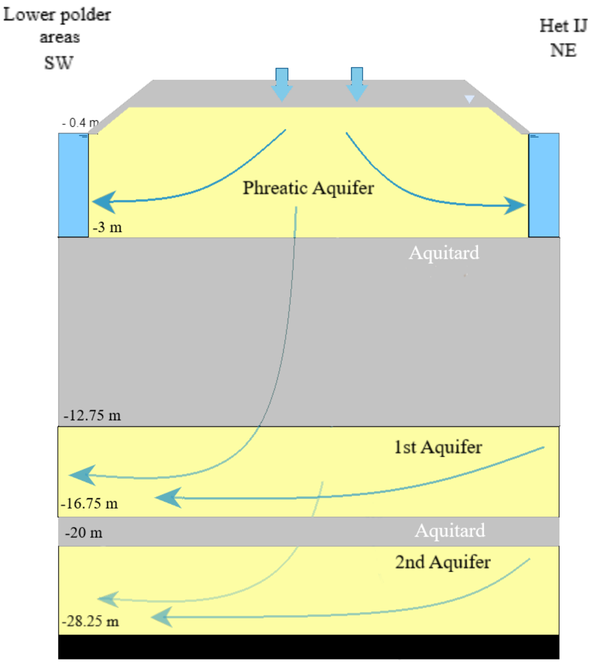


Figure 3.3 Groundwater flow at Prinseneiland.

3.3. Urban water chain information

Bots (2008, p.281) defines the urban water chain as "*the infrastructure for the production, distribution, and consumption of drinking water, and the collection, treatment and disposal of waste water*".

3.3.1. Drinking water system

The drinking water supplied on the island is measured at two flow meters on Bickerseiland and reaches the customers through the piped system that was built in 1986.

Based on house connections, there were estimated 580 inhabitants (personal communication, Oscar Werner (Waternet)). However, the population derived from the measured drinking water consumption (MDW) of 2015, based on the average consumption of 140 l/p/d, is 694 persons. This number will be the one used further in the study. The difference comes from the fact that more people are using drinking water on the island (exitance of small business) than what is estimated through house connections.

The drinking water consumption follows a predictable daily pattern. The maximum consumption is in the morning (8-9 AM), followed by a second peak in the evening (7-8 PM). The lowest flows occur during the night. Weekend days follow a similar pattern; however, the morning peak is delayed by approximatively 2 hours (Figure 3.4).

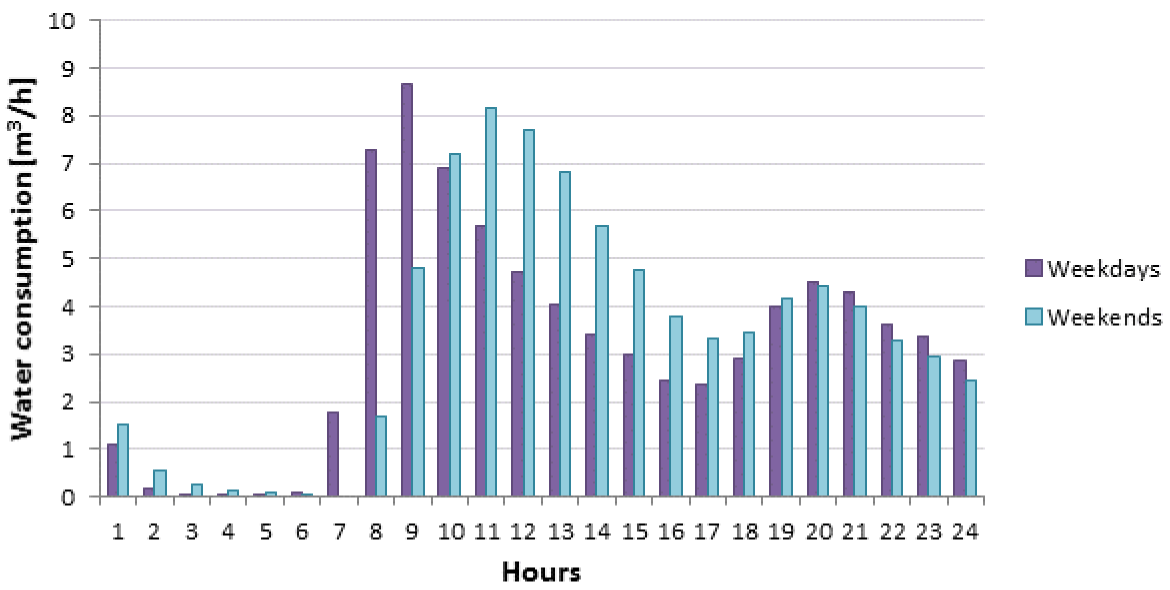


Figure 3.4 Patterns in drinking water consumption for Prinseneiland, Amsterdam, for the period 04.08.2015 – 03.08.2017.

When comparing the average hourly drinking water consumption with the dry weather flow (DWF), a delay in response of the latest can be observed (Figure 3.5). The time lag between the two is about an hour. The lag happens during the supply phase, between the flow meter and the customer's tap. The travel time between the tap and the collection system is only a couple of minutes (Koen Tromp, personal communication, March 2018).

During the night hours, an extra flux in the pump can be observed, of less than 1 m^3/h. This either suggests groundwater drainage by the sewerage system or simply inconsistencies in the low-flow measurements.

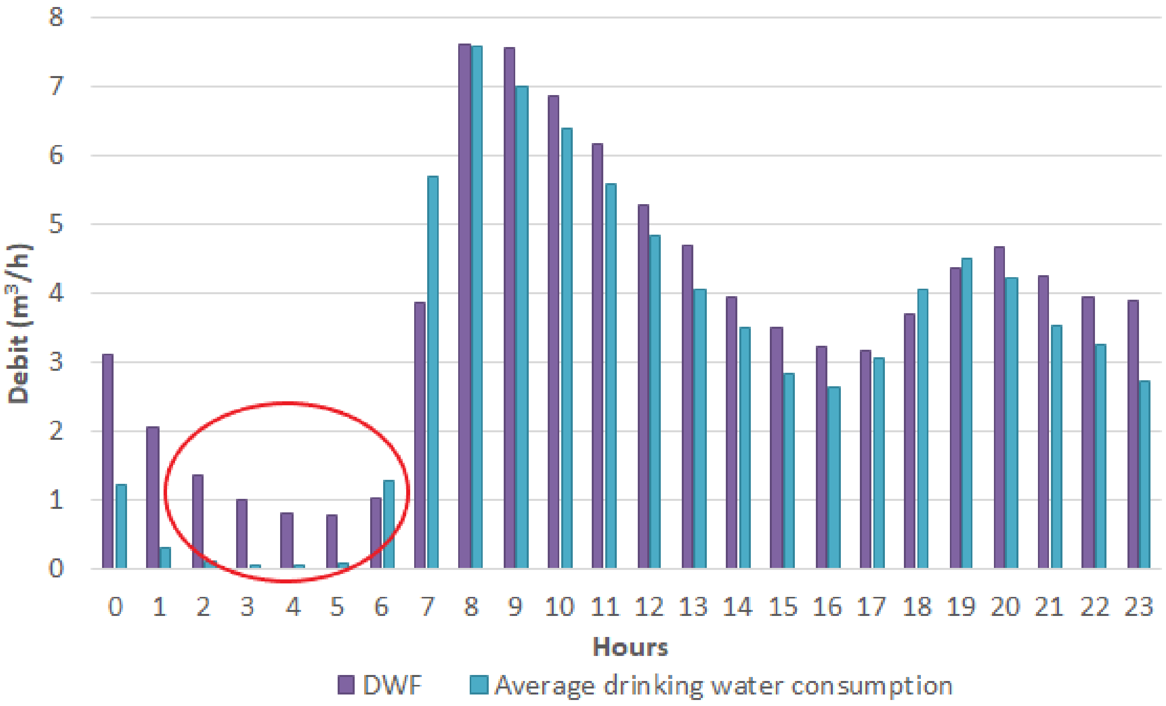


Figure 3.5 DWF compared to average drinking water consumption. The DWF was calculated by considering only the hours of no rain. If there was a shower registered at some point during the day, an extra dry day was taken after the end of the shower, as buffer until a period was considered dry.

3.3.2. Waste water system

Waste water system design

The waste water system consists of a combined sewerage system, a pump that transports the sewage water off the island and an overflow structure.

The system was built in 1988, with conduits with internal diameters between 400 mm and 600 mm. In total they measure 683 m in length (ArcMap measurement). It has a total storage capacity of 175.89 m^3, out of which 14.6 m^3 are in the manholes. The pump installation consists of two pumps that function alternatively and which have an individual maximum capacity of 12 l/s, or 43.2 m^3/h. The pump will "switch on" when the water level in the collection chamber rises to NAP -0.35 m and it will "switch off" when the water level drops to NAP -0.55 m. The overflow construction consists of a manhole which contains an asbestos-cement tube, with a diameter of 1100 mm. When the water level in the manhole exceeds the overflow's crest level (NAP 0.6 m), then a spill will happen into the inner tube, which is connected to the canal through an outfall.

Sewerage system faults

During inspection, it was observed that the sewer system has faults such as cracks and faulty connections, which allow interactions with the groundwater system (Dirksen, 2016).

Exfiltration, which leads to groundwater pollution, is thought to happen through displaced house connections to the main conduit, as seen in Figure 3.6 (personal communication, Jacqueline Flink, October 2017).

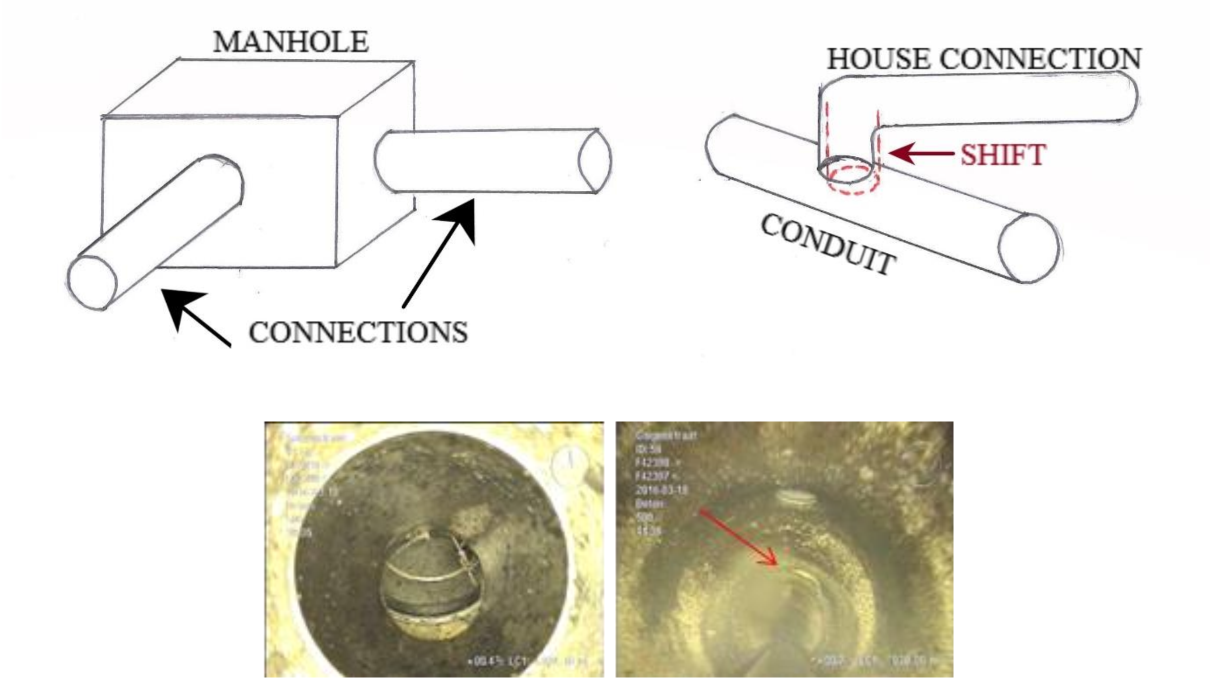


Figure 3.6 Top: Schematisation of the normal connection (upper left) compared to the connection on Prinseneiland (upper right), where the house connection happens directly at the pipe. Bottom: From left to right: displaced house connection and an obliquely drilled house connection pipe where there is water coming out (Dirksen, 2016).

Infiltration happens through cracks or displaced house connections, during high groundwater levels. It is facilitated by the higher hydraulic conductivity of the sandy material the conduits are buried in (Ellis, 2001, p.4). Infiltration in the sewerage system reduces the available storage in pipes, increases the operational costs of pumping and of waste water treatment, causes structural damage to the sewerage system and increases the chance of overflows.

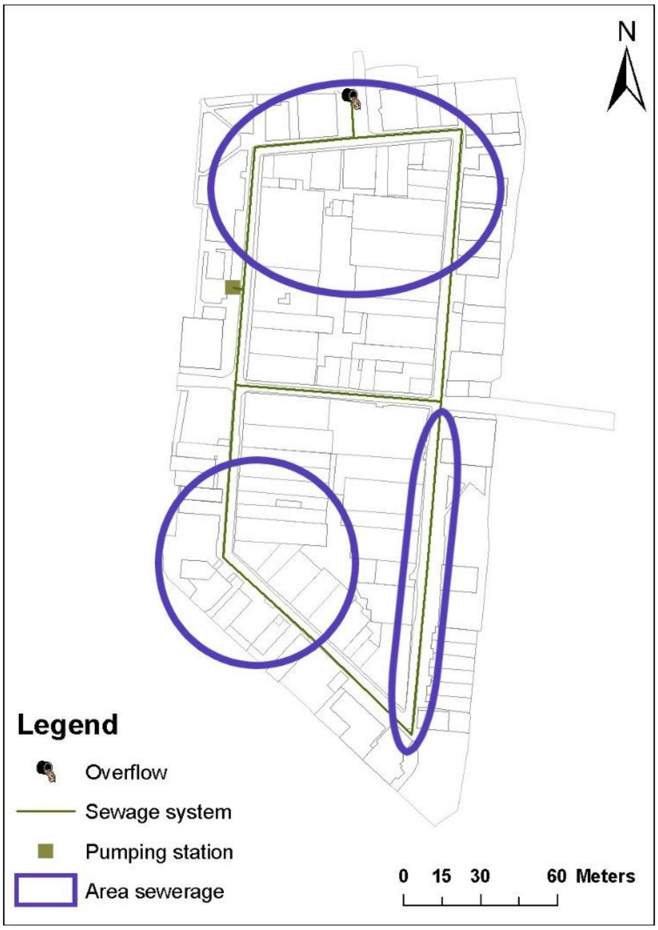


Figure 3.7 Approximate area of where the risk of infiltration is low or non-existent (Dirksen, 2016).

An assessment on the risk of infiltration into the sewerage system was done by Dirksen, 2016, through visual inspection of the sewerage system and through analysis of the groundwater level fluctuations. The document concluded that for the most part of the island the sewerage system is above groundwater levels (SE) or partly in groundwater (N, SW), therefore the risk of infiltration is small, as the groundwater is rarely completely submerging the pipes (Figure 3.7). However, in the centre of the island, the sewerage is more than a half below groundwater (groundwater at medium level), therefore there is a greater risk of infiltration through displaced house connections (Dirksen, 2016, p.4-6).

Contributing area

Since it is a combined system, it collects and transports both rainwater and waste water. While the amount of waste water depends on the consumption patterns and number of users, the amount of runoff depends on various factors: rainfall amount, the contributing area (or "active area"), type of surface, weather conditions.

Based on the type of cover and its permeability, there are, as seen in Figure 3.8:

- Unpaved areas: pervious areas (public or private gardens);

- Paved areas: partially pervious areas (streets, parking areas and sidewalks);
- Built areas: impervious areas.

Figure 3.8 Type of surfaces on Prinseneiland (areas in ha).

Based on the surface's contribution to the sewer with runoff, were defined, as seen in Figure 3.9:

- Active areas: the runoff generated by these areas will flow towards the sewerage system (streets or roofs which have the rain collectors connected to the sewerage system);
- Inactive areas: areas from which runoff does not flow towards the sewerage system (isolated paved areas, all the unpaved areas and the roofs or part of the roofs for which the rain collectors are redirected towards the canal).

Figure 3.9 Contribution to the sewerage system on Prinseneiland (areas in ha).

Figure 3.9 shows that most of the island's runoff is drained by the sewer system. Further, by determining the proportion of each type of surface on the active, it was concluded that 64.3% of the area is built, while the remaining 35.7% represents paved surfaces (Figure 3.10).

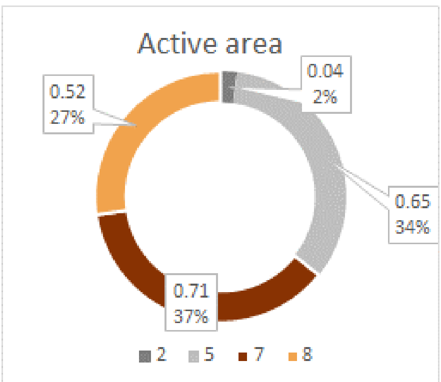


Figure 3.10 Active area and the surfaces types (areas in ha).

The active and inactive areas were determined based on Waternet internal documents, schematics of the house connections, aerial photography and field inspections. The areas were calculated by using InfoWorks and ArcGIS.

CHAPTER 4

Data

4.1. Data sources

Firstly, a literature review was done on the urban water balance. Secondly, data was collected for the chosen study period, 04.08.2015 – 03.08.2017.

The dynamic data was retrieved from Waternet's databases (FEWS, MIDAS and PIMS), for different time steps. The data was reviewed and sorted (for missing data, for outliers). The drinking water data needed correction for the winter flow (§ 4.5.).

Hydrogeological and geological information was retrieved from FEWS, GeoWEB (Waternet database) or generated from http://dinoloket.nl (TNO, 2013), by using the GeoTOP and REGIS II v2.2 models. REGIS II v2.2. provides ranges for hydrogeological parameters, while GeoTOP shows a detailed overview on geological units down to NAP -50 m (Schokker et al., 2015, p.2).

4.2. Rainfall

Rainfall measurements are available on the island from two rain gauges. However, due to their improper placement and frequent malfunctions, data is deemed not suitable for the study.

Data from Papaverweg rain gauge was used for the study (map with location in Annex A). It proved to be the most reliable and suitable rain gauge in the vicinity of the island. Data is available in various time steps (5 minutes, hourly, daily), there is missing data only for a short period (06.02-.2016 – 15.02.2016) and the correlation between the Papaverweg station and the other stations in the proximity of Prinseneiland (Schiphol and Zaandam) is very high for both hourly and daily steps (Annex D). However, very local showers might still not coincide.

4.3. Evaporation

The daily evaporation data for the models was obtained from KNMI for the station of Schiphol. They measure the Makkink reference evaporation. Makkink's formula requires only the incoming shortwave radiation and daily mean temperature (Hiemstra, Sluiter, 2011, p.7) and it is expressed as:

$$ET_{ref} = C * \frac{s}{s+\gamma} * \frac{S^{\downarrow}_{day}}{\lambda * \rho} \tag{4.1}$$

where ET_{ref} is the reference evaporation (m/d), C is a constant equal to 0.65, s is the slope of the saturation water pressure curve (kPa/°C), γ is the psychometric constant (kPa/°C), $S_{day}^{\downarrow}$ is the incoming shortwave radiation (J/m^2/d) and ρ is the bulk density of water (Hiemstra, Sluiter, 2011, p.7-8).

Figure 4.1 shows the precipitation deficit for years 2015 - 2017. The precipitation deficit is a measure of drought and is calculated by KNMI as the difference between potential evaporation and precipitation, from 1st April until 30th September. An ascending line means an increase in deficit, as evaporation exceeds precipitation, while a descending line indicates that the amount of precipitation is exceeding evaporation (KNMI, n.d.). It can be observed that 2015 was the driest year of the three, while 2016 was wetter compared to the other two.

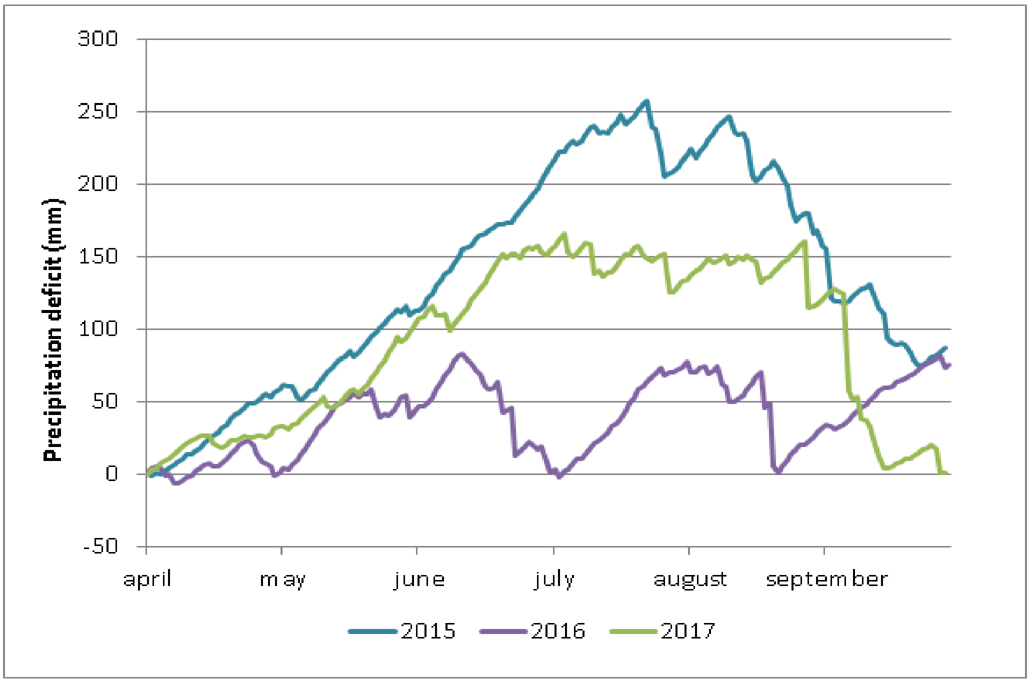


Figure 4.1 Precipitation deficit, calculated with data from Schiphol.

4.4. Groundwater levels

Groundwater levels on the island are measured at 9 piezometers (Annex A). Four additional piezometers were installed in July 2017 to observe possible drawdown near trees. However, due to the late installation of the data loggers (mid-August), results will not be discussed in this study.

Figure 4.2 shows the phreatic groundwater levels. It can be observed that the groundwater levels are responsive to rainfall and evaporation due to the shallow depth of the aquifer.

The surface cover is an influential factor on the groundwater levels. The piezometers placed in unpaved areas (C05271, C05268, C05270) have much higher groundwater levels and display more and larger fluctuations compared to the ones placed in paved areas, where there is minimum infiltration. Piezometer

C06322 (situated in the south-eastern part of the island) displays the lowest and least responsive groundwater levels.

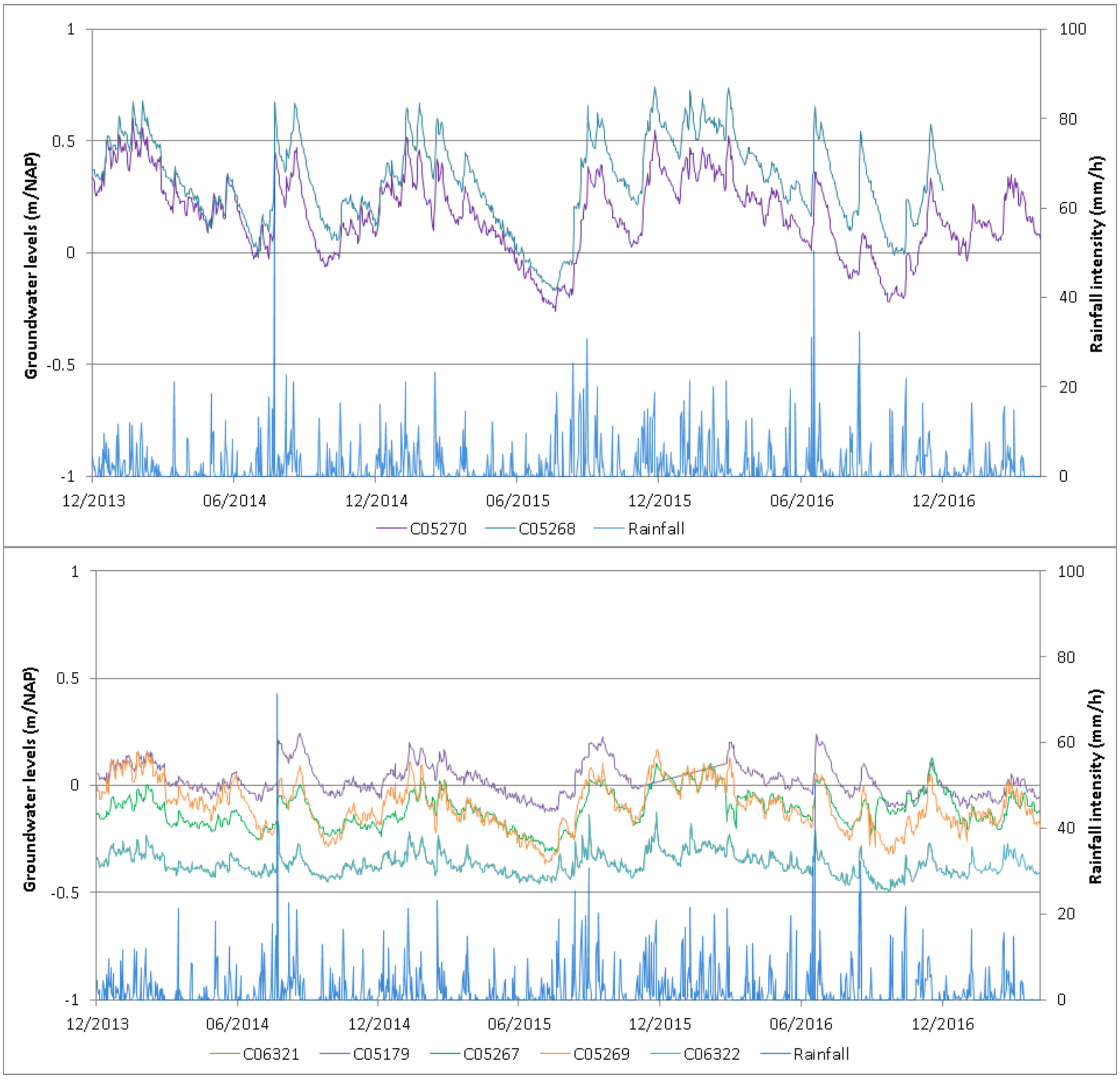


Figure 4.2 Groundwater levels on the island. The top graph shows groundwater levels from piezometers placed in gardens. The bottom one the piezometers placed in paved areas. The end date of the study is 03.08.2017. However, at the time of the study, measurements were available only until 11.04.2017.

Regardless of placement, all piezometers display the same seasonal pattern, with the highest levels during the wet winter months. The levels decrease during spring, and reach a minimum during summer, due to high temperatures and evapotranspiration. The effects of the dry summer of 2015 can be clearly noticed in Figure 4.1.

Surface water levels were retrieved from the SGravenhekje measuring point, and they have an average value of NAP -0.4 m (Annex A).

4.5. Flow measurements

4.5.1. Drinking water measurements

The drinking water flow measurements were corrected for the “winter flow”. Drinking water is supplied to the island via pipes that are attached to the bridge, which makes them susceptible to freezing. To prevent that, some of the drinking water is discharged into the canal, thus creating a continuous flow of water. Therefore, this water will never appear in the pump measurements. The total drinking water amount measured at the flow meters, before correction, is 66728.04 m^3. After correction, the amount of supplied drinking water is estimated to be 57333.38 m^3. The difference is 9394.66 m^3, 14% of the total supply.

The corrections were made for periods 16.01.2015 – 26.05.2016 and 25.10.2016 – 12.03.2017 by comparing the values during each winter period with the average values (hourly and daily) of the dry period before and correcting for the differences. A shift in data is created (Figure 4.3). A correction for the outliers in data was also made at the same time. These values did not appear in the sewerage measurements either. The correction method might affect further results in the study.

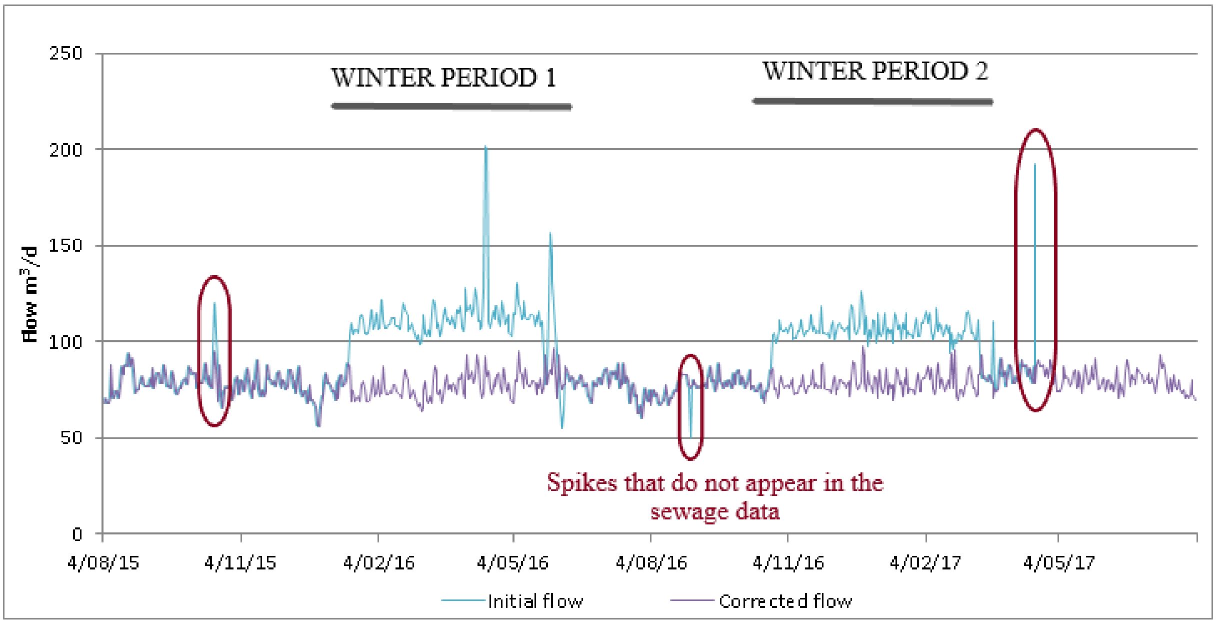


Figure 4.3 Drinking water flow before and after correction for the “winter flow”.

4.5.2. Waste water measurements

There is suspicion that the measurements registered at the sewerage pump are not completely correct due to the following reasons:

i. The level data for the switch “ON” and “OFF” states of the pump (§ 3.3.2.) differ for every measurement, which they should not, indicating that there are steps that are missing;
ii. The data is transmitted every 2.5 minutes which means there is data loss; the requested data under this time interval is obtained through interpolation between two points;
iii. Outside its best efficiency point, the measurements might not correspond to reality, especially at flows under 1 m^3/h;
iv. Due to the grease in the sewage water, the pump might register more volume being pumped;
v. During rainy periods the pump might pump air too, since this becomes trapped in the rainwater while swirling down the collection pipes, and can no longer escape.

All information was obtained through personal communication with Jacqueline Flink (Waternet) in January 2018 and Marcel van der Blom (Waternet) in March 2018.

Overflow measurements

Sensors were installed to measure the “On” and “Off” status of the overflow and the water levels in the structure. However, the measurements are not reliable, therefore they are not going to be used further in the research. A series of figures showing the faults in the measurements can be seen in Annex E.

CHAPTER 5

Research methodology

In the following chapter, the three methods used in this study will be presented (Figure 5.1). The double arrows between methods show the steps that were undertaken in this study. The dotted double-arrow shows an iterative process between methods that was not undertaken, but which is recommended to be done.

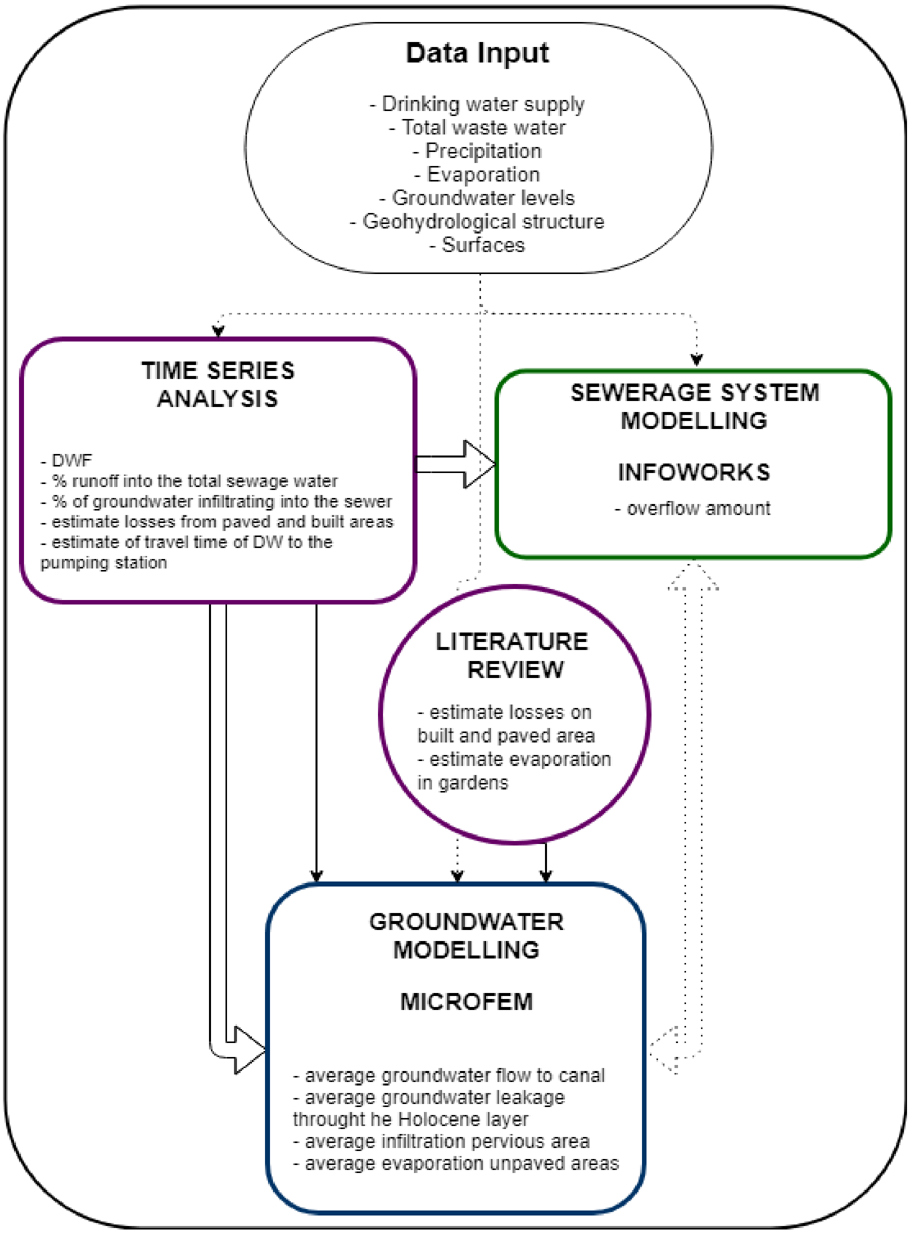


Figure 5.1 Theoretical framework used in this study.

5.1. Time series analysis

5.1.1. Aim of the time series analysis

The main purpose of time series analysis (TSA) is to get a quantitative approximation of the fluxes that contribute to the sewerage system and an approximate time for the fluxes to get to the pumping station. TSA is also used to determine to what extent groundwater level fluctuations can be explained by rainfall and evaporation. The results will serve as boundary conditions for the other models.

5.1.2. Theory and model building

The model is a linear regression model that tries to recreate the total sewage water by its constituent fluxes: rain, waste water and groundwater infiltration. The output is the modelled sewer flow. The general formula of the model is:

$$Flow = b * N_{s,ti} + DWF_{s,ti} + GW_{leak} \tag{5.1}$$

where Flow is the modelled flow at time i [m^3/h], N_{sti} is the rain [mm/h], DWF_{sti} is the dry weather flow [m^3/h], GW_{leak} is the interaction with the groundwater system [m^3/h] and b is a discharge coefficient [m^3/mm] which tells how much water is pumped in the station [m^3], as result of 1 mm of rain (Janse, 2012).

5.1.2.1. Exponential smoothing

All the fluxes are smoothened fluxes. Exponential smoothing is a technique created to reduce the noise in the data, especially at hourly steps, without quantitatively affecting the fluxes. In the urban water chain there is usually a time lag between the input and the response at the pump. Exponential smoothing can be used to fit the time series and determine the time lag between two processes.

The general formula for exponential smoothing, as described by Janse (2001), is:

$$Y_{s,i} = (1 - \alpha) * Y_{s,i-1} + \alpha * Y_i \tag{5.2}$$

where $Y_{s,i}$ is the smoothed value at time i, Y_i is the original value and α is the exponential smoothing factor, which has the value of $e^{\frac{-1}{T}}$ where T is a time-constant. As an example, the formula used to smoothen the rainfall is:

$$N_{s,ti} = (1 - a) * N_{s,ti-1} + a * N_{reg,ti} \tag{5.3}$$

where $N_{s,ti}$ is the exponentially averaged precipitation at time ti [mm], $N_{reg,ti}$ is the original precipitation at time ti [mm] and a is the exponential smoothing factor. The same formula can be used for evapotranspiration, drinking water flow and groundwater levels.

5.1.2.2. Model building

The model is built in 6 stages, as suggested by Janse, (2012).

i. Recreate the sewage flow based on precipitation and a fixed DWF.

$$Q_{recon,ti} = DWF + b * N_{s,ti} \tag{5.4}$$

where $Q_{recon,ti}$ is the reconstructed sewage flow at time ti [m^3/h], DWF is the average hourly dry weather flow for the study period, determined with Excel's Solver [m^3/h], b is the sewage/drainage coefficient [m^3/mm] and $N_{s,ti}$ is the smoothened rain at time ti [mm/h].

ii. Determining the real DWF.

$$DWF = MDW - \%Loss * MDW \tag{5.5}$$

where DWF is the dry weather flow [m^3/h], MDW is the measured drinking water flow [m^3/h] and %Loss are the possible losses.

Even though the formula includes a loss for the drinking water system, based on discussions with Waternet experts, there are no known losses, therefore the value is considered 0, therefore MDW = DWF but with time lag.

iii. Reconstruction of groundwater levels based on smoothened rainfall and evaporation data, as shown by Janse and Pytrik (2015).

$$\mathrm{GWL}_{new} = \mathrm{c} + \mathrm{b}_{nslg} * N_{s,ti} + b_{evap} * E_{s,ti} + b_{surfw} * SW_{s,ti} \tag{5.6}$$

where GWL_{new} is the modelled groundwater level [NAP m], c is the intercept of the model, which represents the drainage level of the aquifer (groundwater level if there would be no influence from precipitation and evaporation) [NAP m] and b_{nslg}, b_{evap} and b_{surfq} are regression coefficients. They indicate the contribution of the measured precipitation, evaporation and surface water (Sw) to the groundwater level in percentages. For the surface water level, the deviations from the mean are taken. A "no-effect" on the base level (c) happens if the rainfall and/or evaporation are zero and if the surface water does not deviate from the mean. The regression and smoothening coefficients are calculated during calibration with Excel's Solver function.

Results of the TSA will be compared with Menyanthes results of linear computation for the groundwater levels (Von Asmuth et al., 2012).

iv. Transforming groundwater level into a quantity.

$$GWL_{leak} = c_{gw} * GWL_{new} \quad (5.7)$$

where GWLleak is the quantity of groundwater flow into the system [m^3/h] and c_{gw} is the groundwater coefficient which will the determined with the Solver.

v. Simulating an "overflow" on the recreated sewer flow, based on the maximum pump capacity of 12 l/s. This step was undertaken only for the hourly steps because a daily time step is too big compared to the time scale at which overflows happen (minute).
   - If $Flow_t + Overflow_{t-1} > Pump\ cap => Overflow_t = Flow_t - Pump\ cap + Overflow_{t-1}$;
   - Otherwise $Overflow_t = 0$; (5.8)
   - Construction of the final flow: $Final\ flow_t = Reconstructed\ flow_t - Overflow_t + Overflow_{t-1}$. (5.9)

5.1.3. Validation of the model

The reliability of the model is checked by using the following:

Correlation coefficient

This is a coefficient which describes the relationship between the calculated and observed values. The formula is:

$$r = \frac{\sum(x_i-\overline{x})(y_i-\overline{y})}{\sqrt{\sum(x_i-\overline{x})^2\sum(y_i-\overline{y})^2}} \quad (5.10)$$

where, $\overline{x}$, $\overline{y}$ are the averages of time series of N elements, x and y (Chatfield, 1978).

Root Mean Squared Error

The Root Mean Squared Error (RMSE) is used to calculate the accuracy of the values obtained from the model, compared to the measured ones. It is the simple standard deviation of the differences between the obtained values and the observed ones.

$$RMSE = \sqrt{\frac{\sum_{i=1}^{n}(\hat{x_i}-x_i)^2}{n}} \quad (5.11)$$

where $\hat{x_i}$ is the modelled value, x_i is the measured value and n is the sample size.

Method of least squares

To obtain the best fitted models with the originals, the calibration through Excel's Solver function of the smoothening and regression parameters is necessary. This is done by minimizing the RRS (residual sum of squares) of the model and measurements.

The RSS offers us an insight into the discrepancies between the measurements and the model. The larger the values, the higher the discrepancy.

$$RSS = \sum_{i=1}^{n}\left(y_i - f(x_i)\right)^2 \quad (5.12)$$

where y_i is the value to be predicted and $f(x_i)$ is the predicted value.

Confidence of the regression parameters

To obtain the 95% confidence levels for the regression parameters, standard statistics from the Regression Analyst Toolbox is used. However, the interpretation of the parameters must be made carefully as the observations are autocorrelated (not independent). To estimate the confidence intervals a correction should be made for the number of degrees of freedom: not the number of observations but the "effective number of observations" (personal communication Theo Janse, March 2018). He uses the following formulas:

$$N - effctive = N * (1 - a)(1 + a), \qquad \text{where } a = \exp(-\frac{1}{T_x}) \quad (5.13)$$

where N is the number of residuals and T_x is the time constant from the autocorrelation of residuals.

Reliability of smoothing parameters

It is complicated to compute intervals for the smoothing parameters or the time constants they are related to. To give an indication of the sensitivity of the model for these parameters a simple sensitivity analysis is performed.

5.2. Groundwater modelling

5.2.1. Aim of groundwater modelling

The aim of groundwater modelling is to recreate the average groundwater levels and observe the fluxes affecting the phreatic layer. Thus, by using this method recharge to the groundwater will be determined, as well as the flux from the island towards the canal and the leakage to the underlying layers.

The model was created in MicroFEM, a finite-element software for steady-state and transient groundwater flow modelling. The software calculates the heads by using a hybrid calculation method, between the finite element and finite difference methods. The finite element method is used to compute horizontal flow components, while the finite difference method is used to compute the vertical flow ones, allowing a high computational speed, while maintaining the flexibility of an irregular grid (Hemker, Nijsten, 1996).

5.2.2. Geohydrological schematisation

The model includes 3 aquifers, and a total of 5 layers. The first one is a phreatic aquifer with varying saturated thickness, while the other layers have fixed dimensions. The island is surrounded by the canal, which is assumed to be fully penetrating the unconfined aquifer, and it was modelled as part of it. The distinction between land and water is made by assigning large transmissivity values (1000 m^2/d) and a fixed head (NAP -0.4 m) to the nodes marked as "Canal". The unsaturated zone was not taken into consideration due to its small thickness.

The initial values for the layers of the model are presented in Table 5.1.

Table 5.1 Uncalibrated values for the groundwater model. Starting values suggested by Jos Beemster (Waternet) and Jacqueline Flink.

Layer no.	Geological unit	Hydrogeological characterisation	Thickness [m]	Upper and lower level [NAP m]	Hydraulic conductivity [m/d]	Resistance [d]
1	Anthropogenic layer Formation of Naaldwijk	Phreatic aquifer	variable	Top -3 m	Variable *	
2	Formation of Naaldwijk Formation of Nieuwkoop - Hollandveen Formatie of Naaldwijk Formation of Nieuwkoop - Basis Veen	Aquitard	9,75	-3 m -12.75 m		15000
	Boxtel Formation	1^{st} Aquifer	4	-12.75 m -16.75 m	3.75	
3	Boxtel Formation	Aquitard	2,25	-16.75 m -20 m		50
	Kreftenheye Formation	2^{nd} Aquifer	8,25	-20 m -28.25 m	17.5	

* $K_{building}$ - 0.05 m/d, $K_{pavement}$ – 0.1 m/d, K_{garden} – 0.1 m/d, T_{quay} - 0.45 m^2/d.

5.2.3. Model area

The model has a total area of 43270 m^2 with approximate dimensions of 180 m W-E and 350 m N-S. The vertical extent of the model is of approximatively 27 m, from surface down to the end of the second sand layer. The vertical extent varies with the height of the phreatic aquifer. The distance between nodes is 1 m (Figure 5.2). There are, in total, 50552 nodes and 100194 elements.

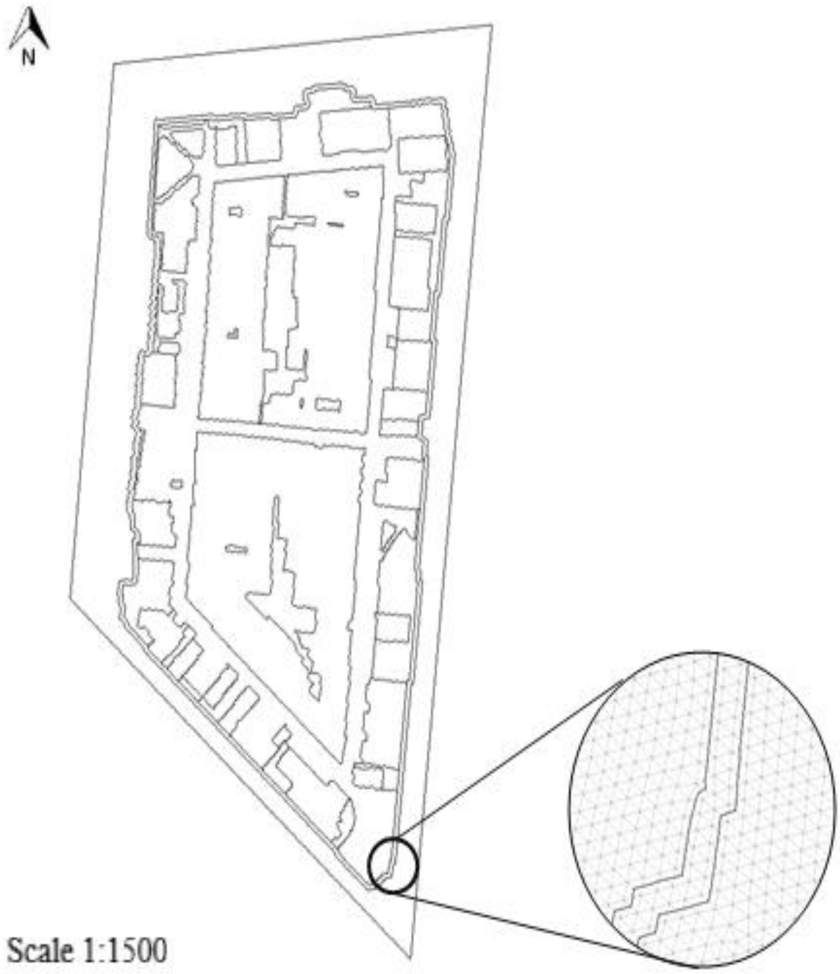


Figure 5.2 Model area and structure.

The model area of the island was divided based on the permeability of the cover surface: building, pavement, garden (Figure 5.3). The quay of the island was included in the model by using a different hydraulic conductivity value than for the rest of the island. Even though during field observations different materials were observed for the quay (wood, concrete, bricks, metal etc), the same K was assigned for the entire construction. As seen in Figure 5.4, the structure does not fully penetrate the phreatic aquifer, but it is considered when calculating the total transmissivity. The sewer system was added as a river, so it can simulate both infiltration and exfiltration. Its total surface is of 624 m^2. The boundary of the modelled area is the middle of the surrounding canal.

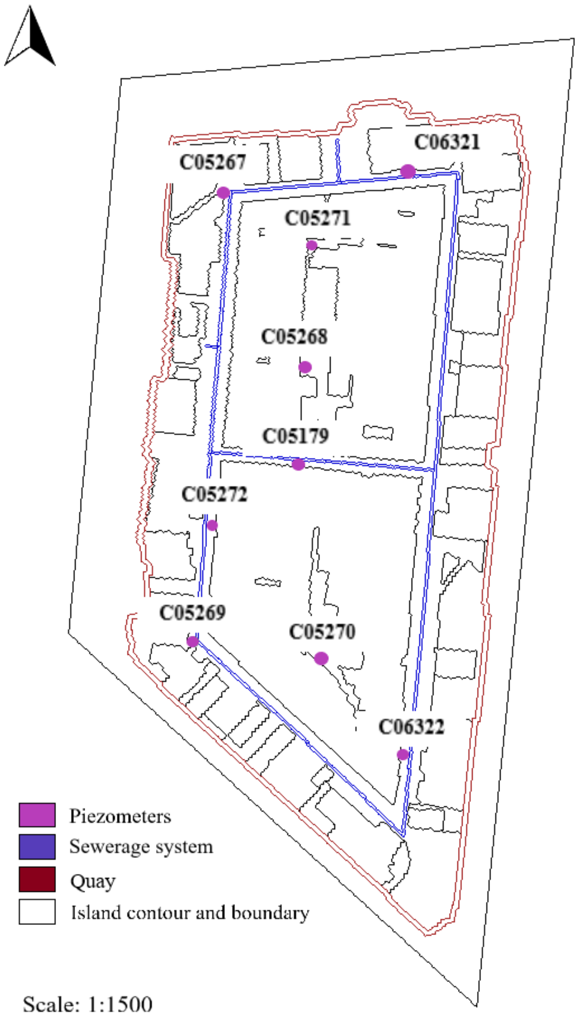


Figure 5.3 Elements of the groundwater model.

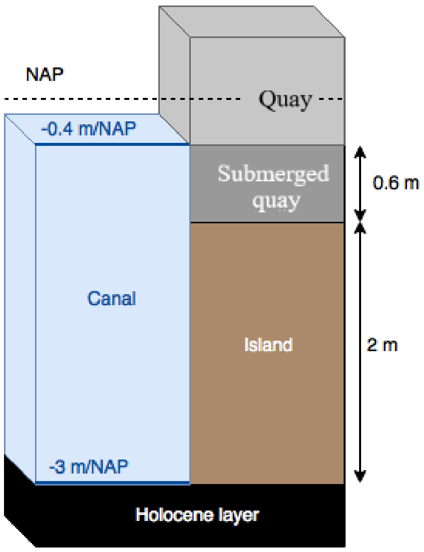


Figure 5.4 The modelled version of the bank protection.

5.2.3. Boundary conditions

The fixed boundary conditions of the phreatic aquifer are the different transmissivity for the quay, which creates resistance for the groundwater flow (0.45 m^2/d) and a fixed head for the canal (-0.4 m). In the first aquifer there is a fixed head of -1.9 m in the southern part of the island, based on the isohypse maps (§ 3.2.2**)**. This gives the approximate flow direction for the first aquifer. The second aquifer has a fixed head of -1.92 m as boundary condition.

The value for infiltration through the pavement is a fixed value obtained through literature and TSA, method that is explained in Annex I**.** The recharge values for the gardens were calculated as net precipitation, which is obtained by subtracting the Makkink evaporation (times a factor) from rainfall, for the two-year study period. The infiltration rates for the stationary model were:

vi. 0 m/d for the built areas;
vii. 0.00053 m/d for the paved areas;
viii. 0.0012, 0.00127, 0.00107 m/d for the garden areas.

The distinction between gardens was made after field inspection (Annex F). The inner gardens turned out to be partially paved. However, the paved area was not connected to the sewerage system, therefore, runoff is assumed to be redirected to the unpaved part of the garden. This results in more input. Moreover, the inner gardens are confined within tall buildings, creating thus a dark and moist space, which might reduce evaporation (personal communication, Jacqueline Flink, December 2017). The following factors for evaporation were used for the pervious areas:

- 0.73 for the northern garden (G1);
- 0.84 for the southern garden (G2);
- 0.77 for the other gardens (G).

5.2.4. Calibration

For the stationary state, the model was manually and automatically calibrated.

At first, the manual calibration was done by changing the input parameters to minimize the RSS (§ 5.1.3.) for each run. The RSS was calculated by comparing the measured groundwater levels, averaged over the study period, with the modelled heads, for the 9 piezometers on the island.

This process helped with better understanding the model and what are the most sensitive parameters. It also helps identifying reasonable starting values for the automatic calibration. It was initially done for one parameter and then for more (Higgins, 2017).

The automatic calibration was done in MicroFEM with the Optimization Tool.

5.2.5. Sensitivity analysis

The sensitivity analysis provides an overview on the importance of each of the varying parameters of the model. The parameters that most influence the modelled heads are the ones used in the calibration of the model.

Since the current research is aimed at observing infiltration and groundwater flow into the phreatic aquifer, the parameters used for sensitivity analysis are the ones of the unconfined aquifer (hydraulic conductivity for various surfaces). However, the model is also very sensitive to the resistance of the confining Holocene layer and to the amount of groundwater recharge.

The sensitivity analysis was done by starting with a basic model, with approximate values suggested by Waternet experts, and by increasing one parameter's value with +10 %, while all the other parameters were kept at their initial values. This helped understanding the impact of each one of them and helped reaching an optimum during calibration process.

Even though groundwater recharge has the most impact on the model, the model was not calibrated on that. However, during the calibration process, it was observed that the initial recharge values could not explain the groundwater levels in the gardens. Filed inspection provided information on the state of the gardens and which led to using the reported recharge amounts (§ 5.2.3).

5.3. Sewage modelling

5.3.1. Aim of sewerage system modelling

The purpose of sewage system modelling is to analyse the wet and dry weather flow on Prinseneiland and to obtain an overview of the frequency and volumes of the combined sewer overflow (CSO) for the study period.

The modelling was conducted in InfoWorks ICM v7.05 (Integrated Catchment Modelling) (Innovyze, 2015). This is a hydrologic and hydraulic modelling software that solves the Saint Venant equations to determine flows, velocities and water levels in hydraulic objects (Rubinato, et al., 2013). For this model, 1D simulations were run.

5.3.2. Model

The model covers Prinseneiland. The model network was built by Koen Tromp (Waternet) and includes two catchments: the storm water catchment and the foul water catchment.

The island is divided into subcatchments with one corresponding manhole each (Annex A). They will capture the generated DWF corresponding to the population of the subcatchments and the runoff from the contributing surfaces. The overflows are discharged to Realengracht, north of island. The outfall has a fixed water level of NAP -0.40 m.

Model elements:

- **Existing sewer –** only the main sewer network is included in the model. Elevations and hydraulic parameters were imported from the existing model of Amsterdam. The houses are connected to the assigned manhole; in reality, the connections are made at the pipe (§ 3.3.2).
- **Areas –** delimited in ArcMap, based on existing maps, field trips and Google Earth inspection. In each subcatchment, the pervious areas and the impermeable areas that are not connected to the sewer system are marked as "unconnected".
- **Pump –** the pump is a Variable Frequency Drive Pump, meaning it will increase its pump rate gradually. It is connected to an outfall through the pressure pipe. A RTC (Real Time Control) was programmed by Juan Bonillo Martinez (Waternet) to replicate the pump's behaviour, based on its "ON" and "OFF" levels (§ 3.3.2.) and construction settings. This will impact the results, as the modelled pump has only 3 stages (off, on, and one step increase in rpms), while the real pump is regulated infinitely. The head discharge table of the pump, both from the manufacturer and the one in InfoWorks, and the script for the pump's RTC can be found in Annex G.
- **Overflow –** the overflow structure was designed as a Standard Weir (a thin plate weir that extends to the full extent of the channel (3.7 m) (Figure 5.5). The crest's extent is the equivalent of the overflow's circumference. The crest level is 0.58 and the free discharge coefficient of 0.46.

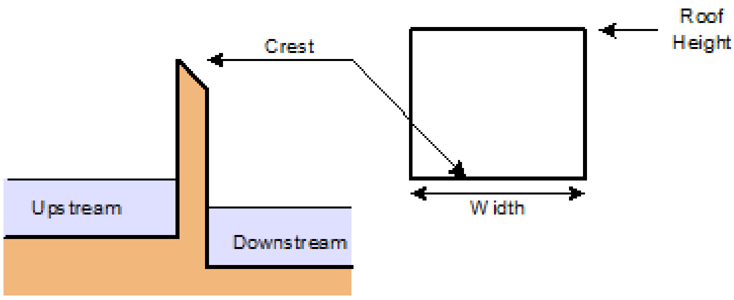


Figure 5.5 Weir representation in InfoWorks (Source: InfoWorks).

InfoWorks (Innovyze, 2015) indicates that overflow is calculated based on the following weir formulas:

- For free discharge (the usual situation, is when the downstream water levels are below crest level), the model uses an equation based on the Kindsvater and Carter equation:

$$Q_0 = C_d \times \sqrt{g} \times B \times D_u^{3/2} \quad (5.14)$$

where Q_o is the free outfall discharge, C_d is the discharge coefficient, g is the acceleration due to gravity, B is the width of the weir and D_u is the upstream depth with respect to the crest.

- For the drowned discharge, when the downstream water levels are higher than the top of the weir (which never happens, since the canal water levels do not go significantly higher than NAP -0.4 m), the equation used is the following:

$$Q_0 = C_d \times \sqrt{g} \times B \times D_u \times (D_u - D_d)^{1/2} \qquad (5.15)$$

where D_d is the downstream depth with respect to the crest.

Input data:

The input data are the rainfall from Papaverweg and the drinking water measurements.

- A daily consumption pattern was created based on the MDW. It was assumed that there were no losses. The daily average is of 78.43 m^3. Separate weekday and weekend patterns were not possible to add to the model, but an artificial pattern to match weekend and weekday behaviour was created based on overall daily averages.
- Rainfall from Papaverweg was added at 5-minute time step. Since this rain gauge is 1.7 km away from the island, this might cause differences in results during very local showers.
- Evaporation – Makkink evaporation from Schiphol airport, on daily time step.
- WWF is generated by InfoWorks through runoff surfaces. In the current model, five types of surfaces are used: “Flat closed pavement”, “flat open pavement”, “sloped roof”, “flat roof”, and “flat unpaved” (Figure 3.8 § 3.3.2). Predefined parameters corresponding to each surface will influence the response time and the amount of losses (Table 5.2).

Table 5.2 Values used in the simulations (Rioned, 2004, p.54). The used values in the model are written in bold letters.

Surface no	**Type surface**	**Type of runoff**	**Runoff delay (min-1)**	**Initial loss (mm)**	**Infiltration capacity (mm.h-1)**		**Time factor (h-1)**	
					max.	**min.**	**decrease**	**recovery**
1	closed	sloped	0.5	0.0				
2	**pavement**	**flat**	0.2	0.5				
3		flat extended	0.1	1.0				
4	open	sloped	0.5	0.0	2.0	0.5	3.0	0.1
5	**pavement**	**flat**	**0.2**	**0.5**	**2.0**	**0.5**	**3.0**	**0.1**
6		flat extended	0.1	1.0	2.0	0.5	3.0	0.1
7	**roof**	**sloped**	**0.5**	**0.0**				
8		**flat**	**0.2**	**2.0**				
9		flat extended	0.1	4.0				
10	**unpaved**	sloped	0.5	2.0	5.0	1.0	3.0	0.1
11		**flat**	**0.2**	**4.0**	**5.0**	**1.0**	**3.0**	**0.1**
12		flat extended	0.1	6.0	5.0	1.0	3.0	0.1

The parameter input values presented in Table 5.2 for the different subcatchments were extracted from the Leidraad Riolering guideline (Rioned, 2004, p.54). These values were calibrated for The Netherlands, as result of extensive measurements. These values have been chosen so that there is never an underestimation of the amount of runoff, for the safety of the sewerage design (personal communication, Koen Tromp, February 2018). As an example, the runoff coefficient from all impervious and semi-pervious surfaces is set as 1. These values differ from the ones used for the groundwater modelling, but mainly because the models serve different purposes.

5.3.3. Calibration

Calibration of a sewerage system model is done by adjusting parameters of the model until the desired output data is generated. The accuracy of the model is given by the difference between the measurements and the simulated output, and is expressed as a percentage. A difference of 10% is typically the case. The calibration can be done by minimizing a series of objective functions, among which is the Difference in Total Volume (5.16) (Dickinson, 2017).

$$Minimize \left(\sum_{i=1}^{N} Pobs_i - \sum_{i=1}^{N} Psim_i\right) \quad (5.16)$$

Initially, the model was calibrated on the DWF. Secondly, the model containing the wet weather flow was calibrated based on the total volumes measured at the pump for the two years.

5.3.4. Sensitivity analysis

The sensitivity of the following parameters was determined: the initial loss per type of surface, the runoff coefficient and the discharge coefficient. The results will be compared with the measurements at the pump. The overflows will be compared with the TSA overflow results.

CHAPTER 6

Results

6.1. Time series analysis

A TSA using a linear regression model was undertaken to predict how runoff, waste water and groundwater drainage in the sewerage system contribute to the total amount of sewage water but also to determine the extent to which groundwater levels fluctuations can be explained by rainfall and evaporation only.

6.1.1. Statistical analysis of the groundwater levels

The model was applied for the period 05.12.2013 until 11.04.2017, as groundwater level measurements were not available until the end of the study period at the time the analysis was undertaken. The last 4 months of the time series are predictions. The model was run for piezometers with data logger measurements (Figure 6.1). An overview of the results is presented in Table 6.1 and Table 6.2.

Table 6.1 Time constant for the smoothing part of the model (in days).

	T_{precip}	T_{evap}	T_{canal}
C05270A	31.40	150.30	18.40
C06322A	13.30	85.70	3.60
C05179A	34.50	200.00	0.10
C05267A	31.90	62.60	8.80
C05268A	34.00	117.10	0.10
C05269A	24.00	160.10	0.10
C06321A	11.80	77.50	0.10

The time constants might be interpreted as the lag between the explanatory series (precipitation, evaporation and canal level) and the response in groundwater levels. However, further interpretation is still in development (personal communication, Theo Janse).

A short sensitivity analysis of the T_{precip} and T_{evap} was done for piezometer C05270. With an increase of 10% in the RMSE, in either direction, it results in: T_{precip} a range between 26 and 38 days, while for T_{evap} a range between 130 and 170 days.

Table 6.2 Parameter values of the regression model for the groundwater levels.

	c*	b_{nslg}	b_{evap}	b_{surfw}	RMSE (m)	R^2
	Regression coefficients				Model fit	
C05179A	-0.08	0.09	-0.07	0.21	0.03	0.85
C05267A	-0.23	0.08	-0.04	-1.13	0.04	0.75
C05268A	0.19	0.20	-0.21	-0.08	0.06	0.91
C05269A	-0.09	0.09	-0.14	0.26	0.04	0.83
C05270A	0.25	0.13	-0.26	3.66	0.05	0.90
C06322A	-0.36	0.03	-0.04	1.31	0.02	0.87
C06321A	-0.37	0.03	-0.04	0.34	0.02	0.82

* signifies drainage level.

It can be observed that most of the models have a good explained variance, meaning that the groundwater levels can be explained through evaporation and precipitation. The highest explained variance is for piezometers C05268A and C05270A, which are placed in unpaved areas, while the lowest explained variance is for piezometer C05267A, which is placed in the upper western side of the island.

Based on the standard statistics of the Regression Analysis, and corrected for autocorrelation of residuals, a test could be performed on the significance of the regression parameters. The test results are given in Table 6.3. These results offer an overview for what piezometers there might be an influence for surface water (C06322 gives a significant correlation with the surface water levels, piezometer which is located very close to the surface water).

Table 6.3 Relative standard deviations of the regression parameters (green: significant, yellow: just significant, red: not significant, at 95% significance level), as computed by Theo Janse.

	Sbnslg	Sbevap	Ssurfw
C05270A	9.10%	6.70%	31.00%
C06322A	2.50%	2.50%	5.30%
C05179A	7.90%	13.20%	21.20%
C05267A	9.50%	17.50%	59.80%
C05268A	5.50%	6.90%	541.60%
C05269A	9.50%	10.20%	143.30%
C06321A	3.60%	4.40%	20.60%

A comparison between the TSA results and Menyanthes results for linear modelling is presented in Table 6.4.

Table 6.4 Menyanthes results for drainage levels and explained variance.

	Drainage level Menyanthes	RMSE (m)	R^2
C05179A	-0.3	0.03	0.86
C05267A	-8.8	0.06	0.49
C05270A	-0.2	0.08	0.79
C06322A	-0.5	0.03	0.82

Based on the poor explained variance of C05267, it might be concluded that the groundwater levels might be influenced by other factors, apart from rainfall and evaporation. It might be influenced by an abstraction or by interactions with the sewer system (personal communication Bierkens, November 2017 and Jacqueline Flink, 2018).

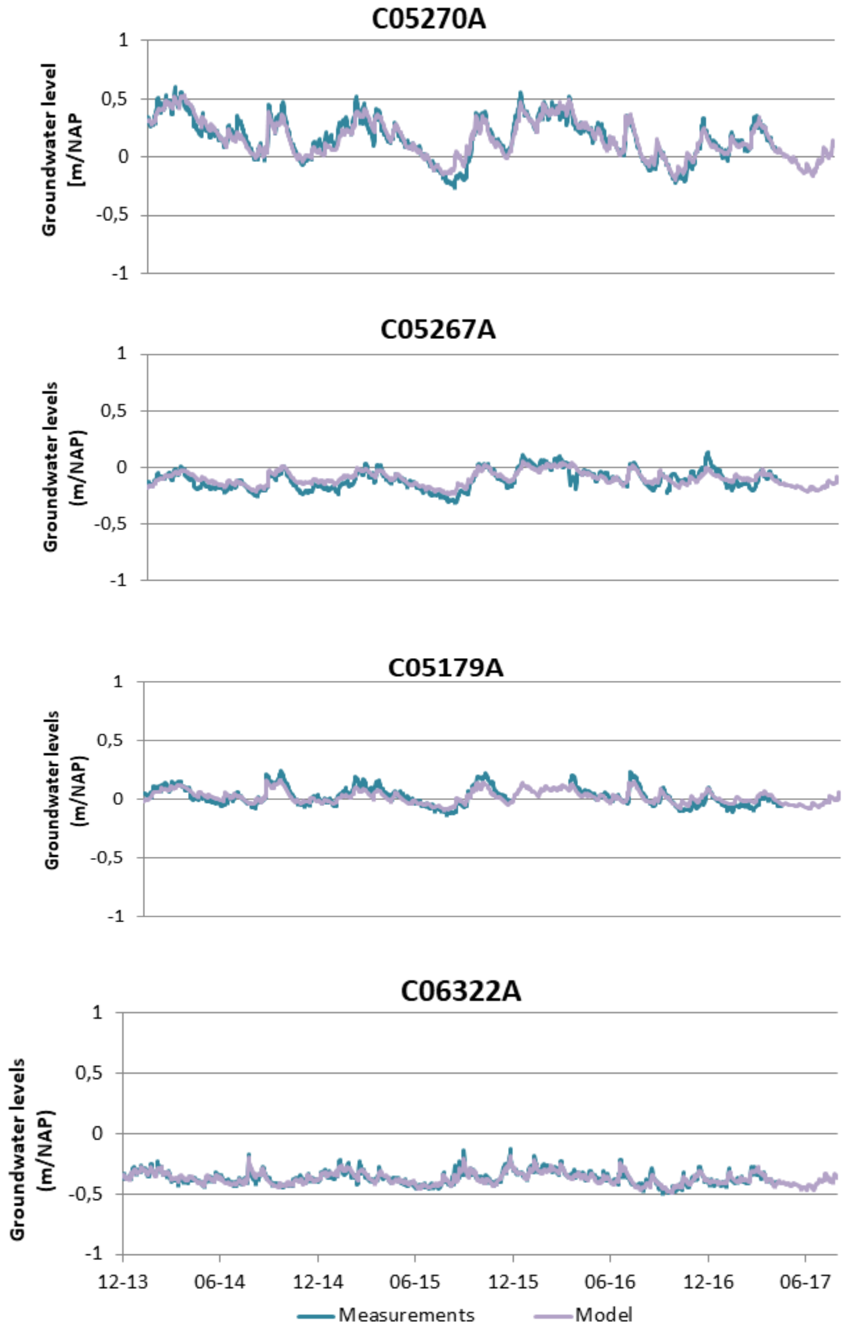


Figure 6.1 Modelled groundwater levels compared to the measured groundwater levels for the period 05.12.2013 – 03.08.2017.

6.1.2. Statistical analysis of the sewer flow

The model was run on both hourly and daily steps. While both the daily and hourly models yield the similar results, the daily model has a better fit, as there are no hourly variations. The models show the proportion in which DWF, runoff and groundwater drainage into the sewerage contribute to the total amount of sewage water (Table 6.5).

Table 6.5 Regression model results, with piezometer C05270 as input for the groundwater component.

	Hourly model	Daily model
% DWF	65%	65%
% Runoff	26.90%	27.60%
% Groundwater drainage	8.10%	7.30%
% of total runoff that reaches the sewerage system	67.40%	69.90%

The DWF was estimated to be approximatively 77 m^3/d. The groundwater draining into the sewerage is roughly the amount of water that cannot be explained through either DWF or precipitation. It is estimated to be 8.85 – 9.75 m^3/d. This is a similar result to the one obtained by Jan Willem Voort (Waternet) through DWAAS Analysis (Voorhoeve & van de Kerk, 2003). He estimated for year 2016 a rate of 77.6 m^3/d DWF and a groundwater drainage rate into the sewer system of 10.7 m^3/d (Voort, 2018). An extract of the model can be seen in Figure 6.2.

The values from runoff obtained through the daily model were further used in calculating infiltration through pavement for the groundwater model. A description of that can be seen in Annex I.

The values obtained in Solver for the parameters are shown in Table 6.6.

Table 6.6 Parameters of linear regression model of sewage flow.

	Tx rain	Tx dw	B coeff	C_{gw}
Hourly	1.77	0.75	12.94	3.03
Daily	0.49	0.28	13.31	65.96

The time constants provide information on the time necessary for the input (rainfall, DWF) to reach the pump. The average values are small, suggesting that the response time and draining time of the system are small. This corresponds with reality, since the surface area of the island is small and 64.3% of the active area consists of roofs, meaning that the generated runoff will travel fast via the roof drain pipe.

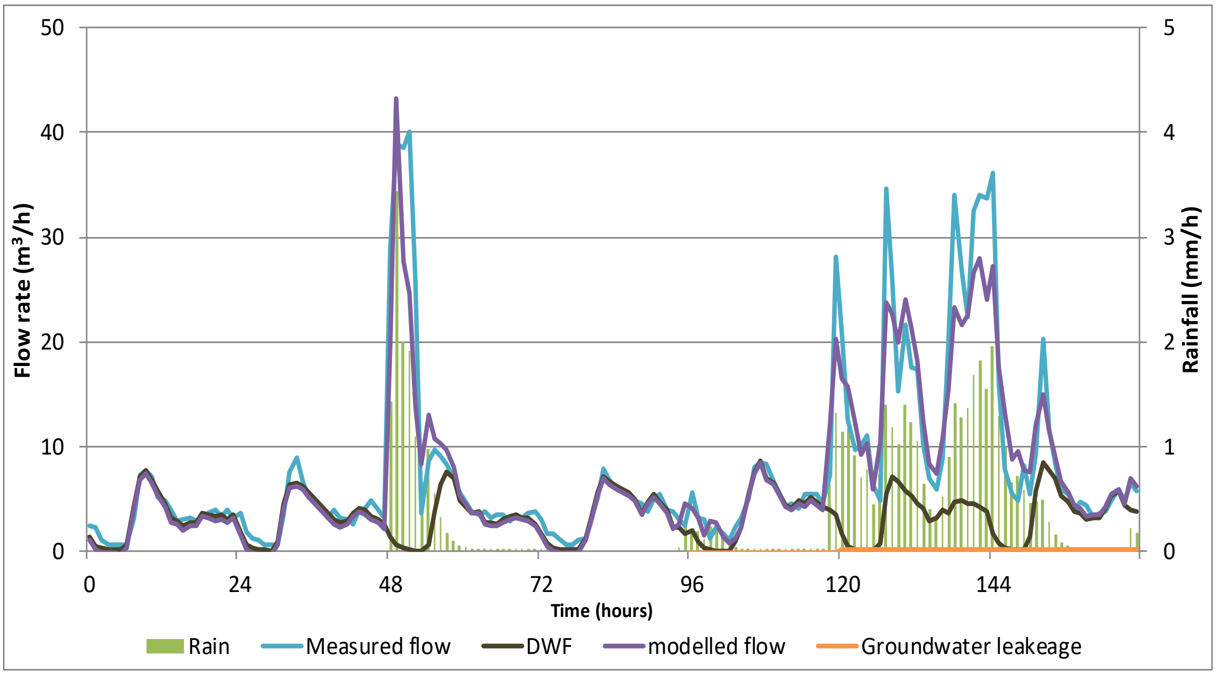


Figure 6.2 TSA Model and its components vs. measurements for the period 12.08.2015 – 18.08.2015.

Overflow results

The hourly model can give an estimate of the overflow events. It "simulated" a total of 25 events in two years, with a total amount of 4460 m^3 (Figure 6.3 and Table 6.7). This approximated value is 4 times the value calculated by InfoWorks (§ 6.3.2.). There are two reasons for the overestimation. Firstly, the model does not consider the storage available in the sewerage system. Secondly, the hourly model creates the overflow events by calculating with hourly averaged pump rates and hourly totals for rainfall. Therefore, whenever the pump is exceeding its capacity, the current exceedance summed with a possible previous exceedance (m^3/h), is counted as an overflow, resulting in enormous amounts of overflow. However, CSOs require a more detailed time-discretisation (minute).

Table 6.7 Overflow events generated by TSA. The marked events are the events obtained also in InfoWorks.

No of event	Date	Time of the overflow	Duration overflow event [h timesteps]	Max Intensity [mm/h]	Total amount rain [mm]	Total Volume Overflow [m³]
1	14-8-2015	01:00	1	6.1	12.8	1.67
2	25-8-2015	03:00	1	5		0.83
3	**26-8-2015**	**20:00**	**5**	**10**	**18.2**	**195.93**
4	31-8-2015	07:00	1	5.2		0.23
5	4-9-2015	15:00	2	7	30.7	5.75
6	14-9-2015	17:00	1	5.7	10.1	2.00
7	15-9-2015	19:00	2	6.6	13.5	2.94
8	**17-9-2015**	**12:00**	**4**	**7.4**	**20.2**	**86.77**
9	7-1-2016	17:00	3	8.2	16.9	41.96
10	18-5-2016	19:00	2	7.3	10.8	9.07

No of event	Date	Time of the overflow	Duration overflow event [h timesteps]	Max Intensity [mm/h]	Total amount rain [mm]	Total Volume Overflow [m^3]
11	**30-5-2016**	**21:00**	**4**	**15.1**	**16.2**	**181.56**
12	**20-6-2016**	**16:00**	**7**	**10.5**	**31.1**	**219.15**
13	**23-6-2016**	**03:00**	**13**	**21.1**	**50.6**	**1975.18**
14	28-6-2016	20:00	1	7.3	7.5	3.349
15	30-6-2016	16:00	2	8.8	11.1	20.23
16	9-8-2016	08:00	1	5	11.3	2.93
17	**19-8-2016**	**21:00**	**7**	**10.3**	**27.5**	**407.78**
18	**21-8-2016**	**09:00**	**9**	**16.8**	**31.8**	**764.27**
19	18-10-2016	15:00	1	7.8	9.9	7.36
20	**19-10-2016**	**06:00**	**5**	**8.9**	**26.5**	**151.91**
21	10-11-2016	17:00	2	7.8	12.2	6.56
22	**9-6-2017**	**09:00**	**4**	**7.3**	**18.3**	**84.23**
23	30-6-2017	13:00	1	6.3		0.18
24	12-7-2017	10:00	1			1.97
25	**29-7-2017**	**21:00**	**6**	**9.5**	**23.7**	**285.81**

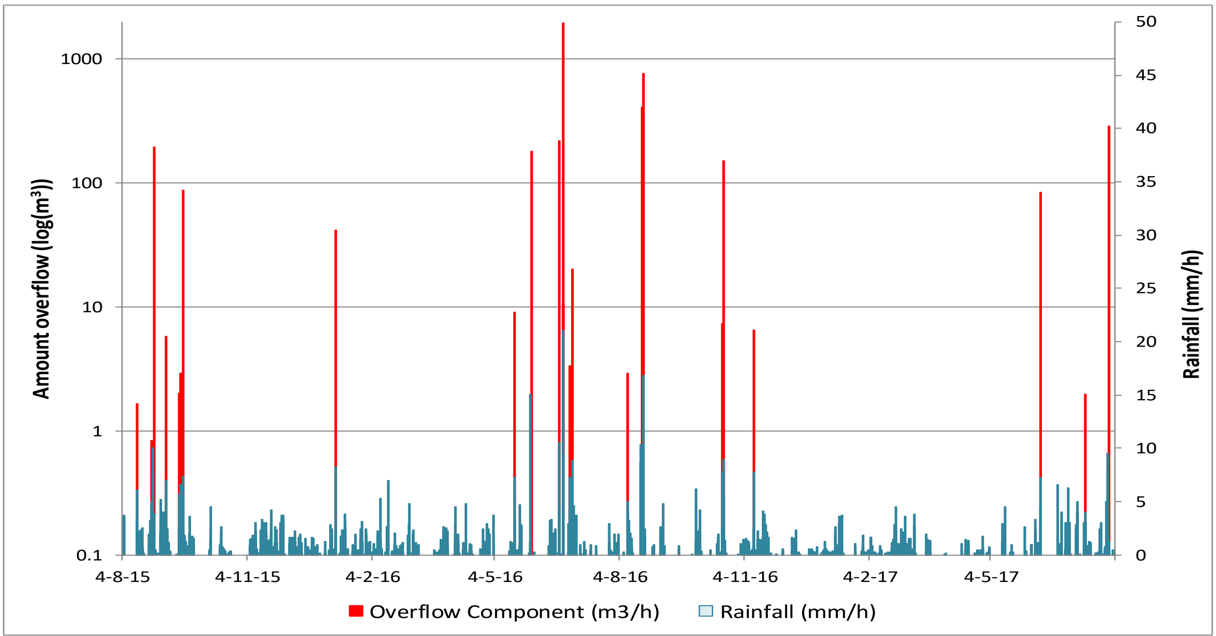


Figure 6.3 CSO events generated by the TSA model.

6.1.3. Uncertainty analysis

The limitations to the regression model for the sewage flows are the following:

- there is a correlation between the estimated groundwater leakage into the system and runoff;

- the results depend on the piezometer chosen to account for the groundwater leakage. Choosing piezometer C05179, for example, will give differences between 5-11% for the flows on hourly steps;
- there is an autocorrelation in the residuals of the model, therefore if corrected for this error, the standard error might be larger than the result given by the Regression Analysis (Theo Janse, personal communication, 22.02.2018).

The hourly model has a R^2 of 0.84 and the daily 0.97. This is due to the absence of hourly variations of the data, and because there is no limitation to the pump capacity in the daily model (the pump has a maximum capacity of 43.2 m^3/h, which appears only in the hourly model). The results of the Regression Analysis for the two models are reported in Table 6.8. The residuals show low autocorrelation; therefore, no correction is done for the effective number of residuals (personal communication, Theo Janse, March 2018).

Table 6.8 Results of Regression Analysis for the sewage flow.

	Observations	Standard error	RMSE
Hourly	17544	2.91	2.47
Daily	731	21.52	23.18

HOURLY MODEL					
	Regression Coefficient	**Standard Error (absolute)**	**Standard Error (%)**	**Lower 95%**	**Upper 95%**
Runoff	0.8149	0.0045	0.55%	0.8062	0.8237
DWF	1.0084	0.0065	0.64%	0.9957	1.0212
GWI	0.8989	0.0427	4.75%	0.8152	0.9827
DAILY MODEL					
Runoff	0.9526	0.0134	1.40%	0.9264	0.9789
DWF	1.0984	0.0139	1.26%	1.0711	1.1256
GWI	0.2949	0.0804	27.25%	0.1372	0.4527

A short sensitivity analysis on the time constants for the hourly model. With an increase of 1% in the RMSE, in either direction, it results in: T_{rain} a range between 1.4 and 2.2 hours, while for T_{dw} a range between 0.4 and 1.2 hours.

6.2. Groundwater modelling

6.2.1. Sensitivity analysis results

Table 6.9 shows the impact on the averaged heads and on the RSS of a parameter increase by 10%. The sensitivity analysis was conducted by increasing one parameter at the time with 10%, while the others were kept at their original, uncalibrated values.

Table 6.9 Impact of uncalibrated parameters change on the sum of squares.

Parameter	Scenario	RSS	% Change
Initial model	-	0.448	-
Hydraulic conductivity outer edge gardens (K_g)	10%	0.416	-7.23
Hydraulic conductivity inner gardens (K_{g1})	10%	0.440	-1.73
Hydraulic conductivity pavement (K_p)	10%	0.415	-7.28
Hydraulic conductivity building (K_b)	10%	0.377	-15.86
Transmissivity quay (T_k)	10%	0.422	-5.77
Resistance of the first confining layer (C_2)	10%	0.603	34.6
Groundwater recharge through pavement (PPN_p)	10%	0.565	26.22

The model is highly sensitive to the resistance of the confining Holocene layer and least sensitive for the hydraulic conductivity changes in the inner gardens. The most sensitive, however, it is to the amount of recharge, due to the very low hydraulic conductivity of the top layer. The model is not so sensitive to the hydraulic conductivity of the quay, as the structure does not completely penetrate the aquifer.

6.2.2. Model calibration results

The model calibration was done in two stages.

To begin with, the sensitivity analysis was undertaken and based on the results presented in the previous subchapter (§ 6.2.1), the manual calibration was conducted for one parameter at the time. Then the calibration was undertaken in MicroFEM with one and multiple parameters at the same time. The final values resulted from the calibration process are shown in Table 6.10. Graphs showing the parameter optimization graphs and the autocorrelation matrix between the parameters can be seen in Annex H.

Table 6.10 Final parameters of the stationary model.

Parameter	Unit	Parameter	Unit
K_p	0.1045 m/d	$\mathbf{K_{g1}}$	0.044 m/d
K_b	0.0958 m/d	$\mathbf{K_{g2}}$	0.0264 m/d
T_q	0.5952 m^2/d	$\mathbf{C_2}$	15265 d
K_g	0.295 m/d		

During the optimization process it was observed that only two K values were not sufficient (for the island and the quay) to properly calibrate the model. A distinction between the surfaces was made taking into consideration the possible anthropogenic influences on the groundwater flow, as mentioned in § 5.2.3.

Lastly, drainage by the sewerage system was not integrated in the model anymore since the rate of groundwater drainage to the sewer obtained through TSA (0.41 m^3/h = 9.84 m^3/d) was almost the same as the input (10.99 m^3/d), which is unrealistic, given the high groundwater levels on the island. The model's final RSS is 0.073.

Figure 6.4 shows the differences between the measured and the predicted groundwater levels. A red disk means the predicted values are higher than the average and a blue disk means that the predicted values are lower.

The model tends to underestimate the heads due to the overestimation at piezometer C06322. The levels of that piezometer are the lowest and least responsive on the island. One of the many explanations for that could be an abstraction, as the buildings on the right side have deep basements, or it could be a very sandy area, therefore the transmissivity is very high. An abstraction of 0.4 m^3/d can explain the groundwater levels there.

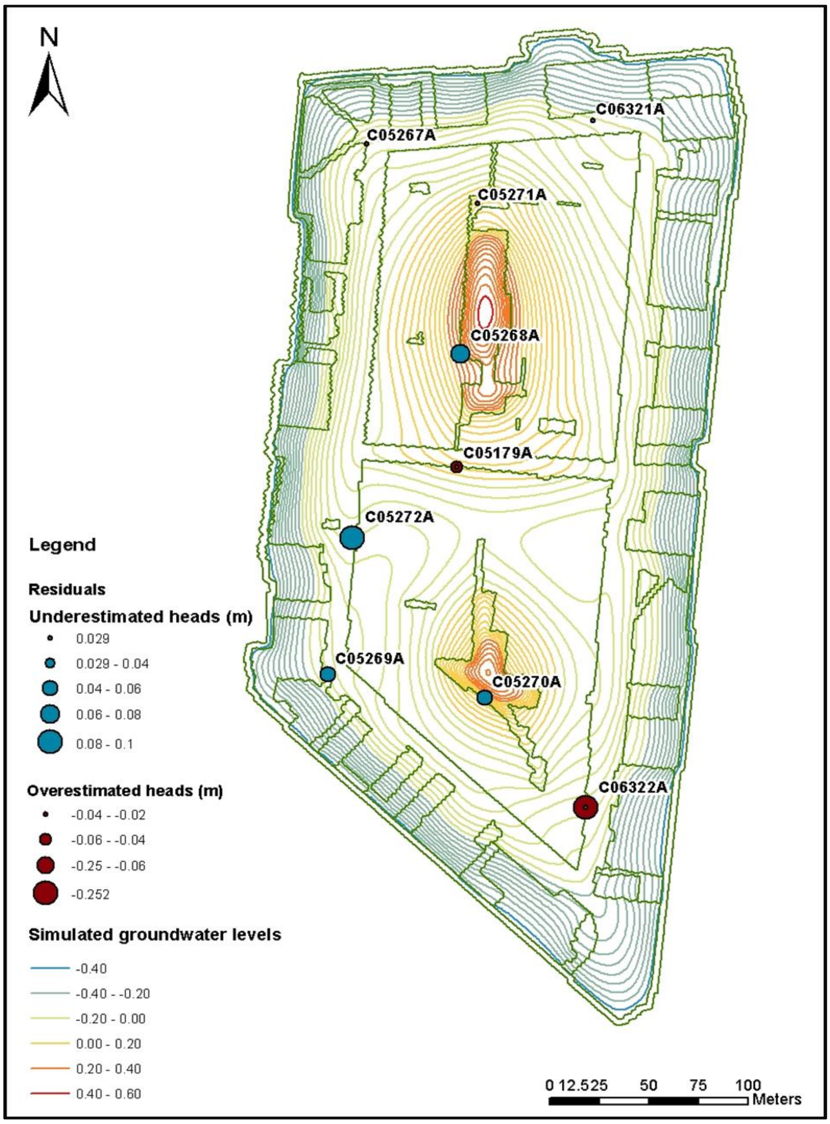


Figure 6.4 Figure with observed and calculated heads per piezometer.

6.2.3. Stationary state water balance

The total recharge into model is 10.99 m^3/d, for the two years. 51.5% of total recharge is infiltrating through the gardens, and 48.5% is infiltrating through the pavement. The paved area of the island (8184 m²) is almost twice as big as the pervious area (4717 m²). Evaporation losses and initial losses were deducted from the total, based on the formula used from literature review, as explained in Annex I.

The summary of the water balance results can be seen in Table 6.11. A graphical representation of the balance is shown in Figure 6.5.

Table 6.11 Summary of MicroFEM stationary model. The results are in m³/d, for a total area of 30866 m².

		Inflow	Outflow	In - Out
	Recharge	10.99		10.99
	Sum top systems	10.99		10.99
	Leakage from the overlying aquifer			
1	**Horizontal flow**		7.32	-7.32
	Boundary flow			
	Well flow			
	Total	10.99	10.99	0.00
	Leakage from the overlying aquifer	3.67		3.67
2	**Horizontal flow**	1.10	0.43	0.67
	Boundary flow			
	Well flow			
	Total	4.77	4.63	0.14
	Leakage from the overlying aquifer	4.20		4.20
3	**Horizontal flow**	4.89	7.38	-2.49
	Boundary flow			
	Well flow			
	Total	9.09	7.38	1.71

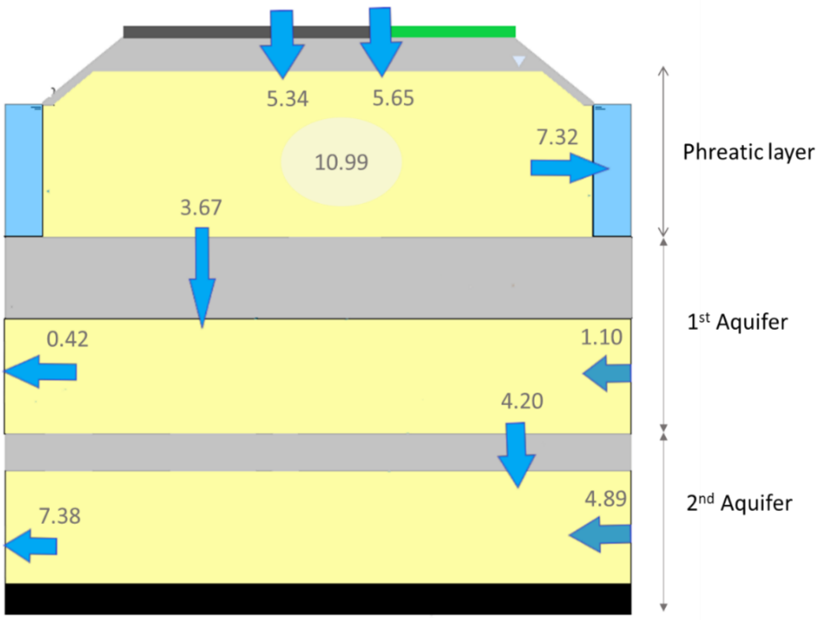


Figure 6.5 Stationary water balance result. Graphical representation.

6.2.4. Transient state results

Transient flow calculations were conducted for 1 year, in the period 01.04.2016 – 01.04.2017. For this, a storativity coefficient was added. The initial value for storativity for the phreatic layer was 0.05 for the island, and lower for the quay area 0.001. 0.001 was the value assigned for the other two aquifers. A second storativity of 0.08 was added for the summer period, when the weather is dryer and evaporation higher. Net precipitation for the gardens was calculated as rainfall minus Makkink evaporation times a factor of 0.5. This yielded better results than the factors used in the stationary model.

The transient model was built on the calibrated model for the stationary model. Therefore, the differences in head that were generated by the stationary model can be observed in the transient model too. The most obvious one is for piezometer C06322.

Despite the initial offset, the piezometers in the gardens (C05268 and C05270) yield better results than the ones placed in paved areas, as seen in Figure 6.6.

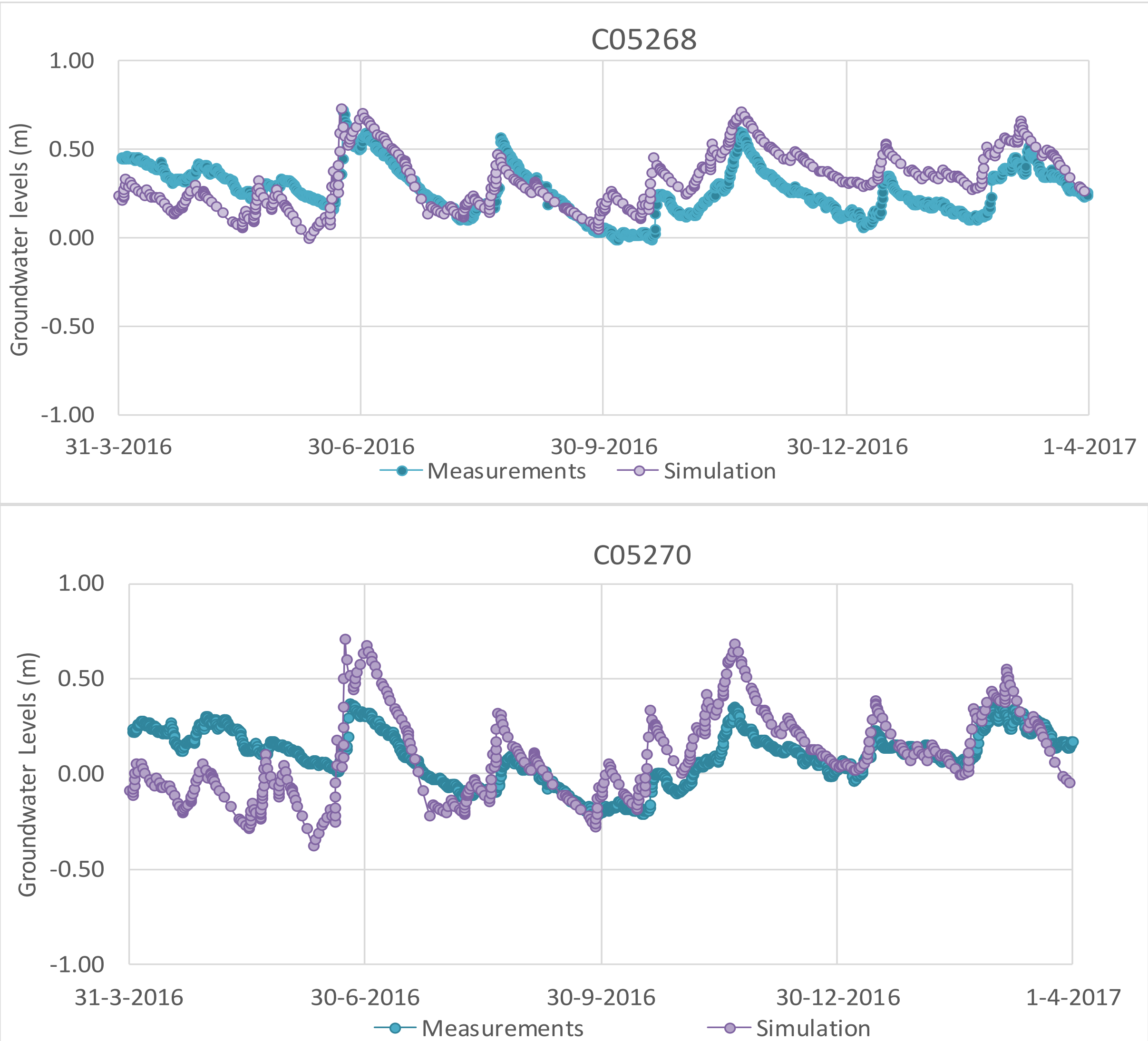


Figure 6.6 Transient calculations results for piezometers situated in gardens.

For piezometers C05269 and C05267, which are placed in paved areas, but very close to the gardens (1 node), the influence of recharge from gardens can be seen in the simulated values (big variations in the groundwater levels), whereas the measured heads seem to be influenced by rainfall and evaporation corresponding to the paved area (Jacqueline Flink, February 2018) (Figure 6.7).

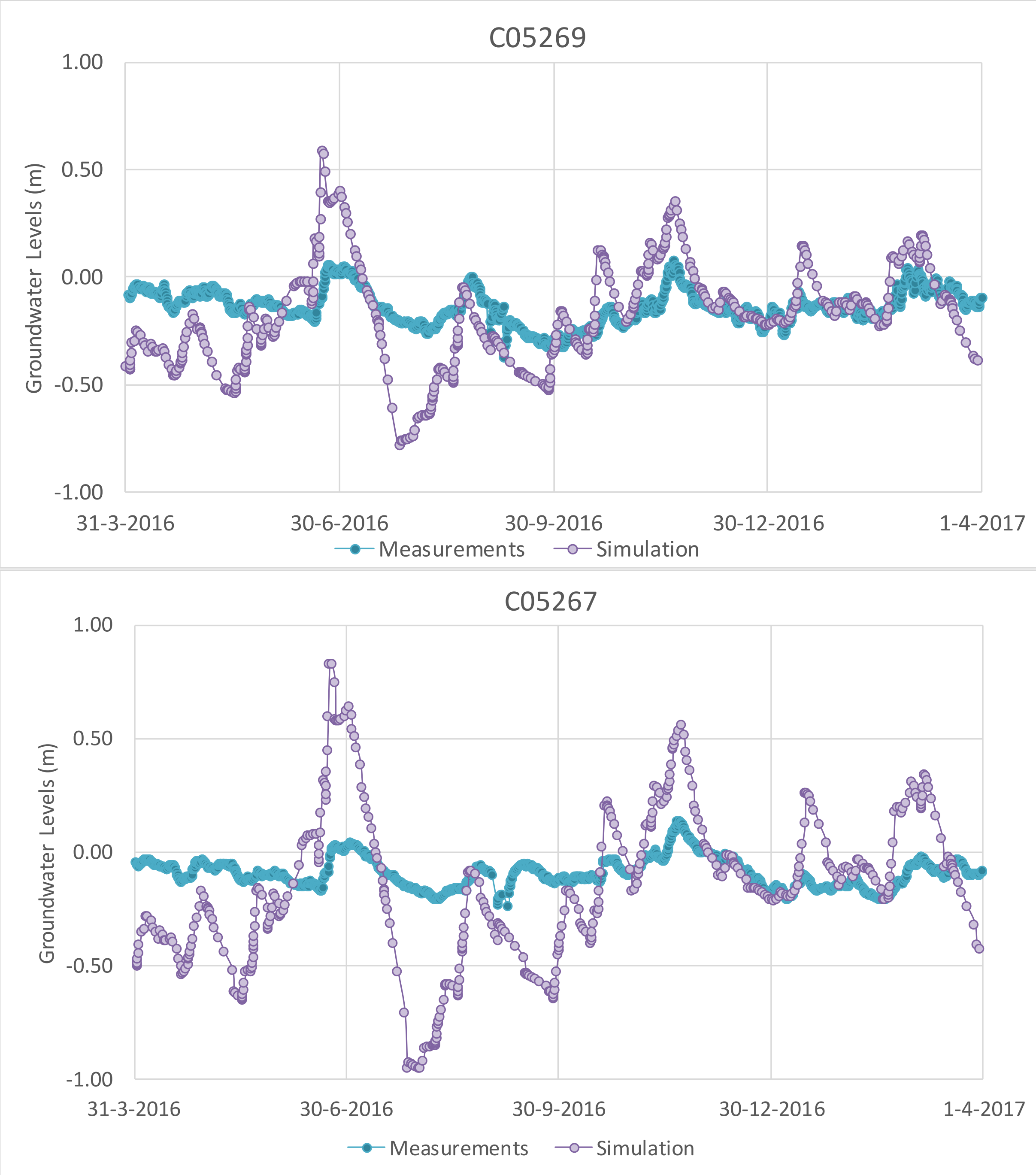


Figure 6.7 Transient calculations results for piezometers situated in paved areas, close to gardens.

Groundwater levels for piezometer C06322 could not be fit for the transient calculations, as the measured values are much lower than the simulated ones (Figure 6.8).

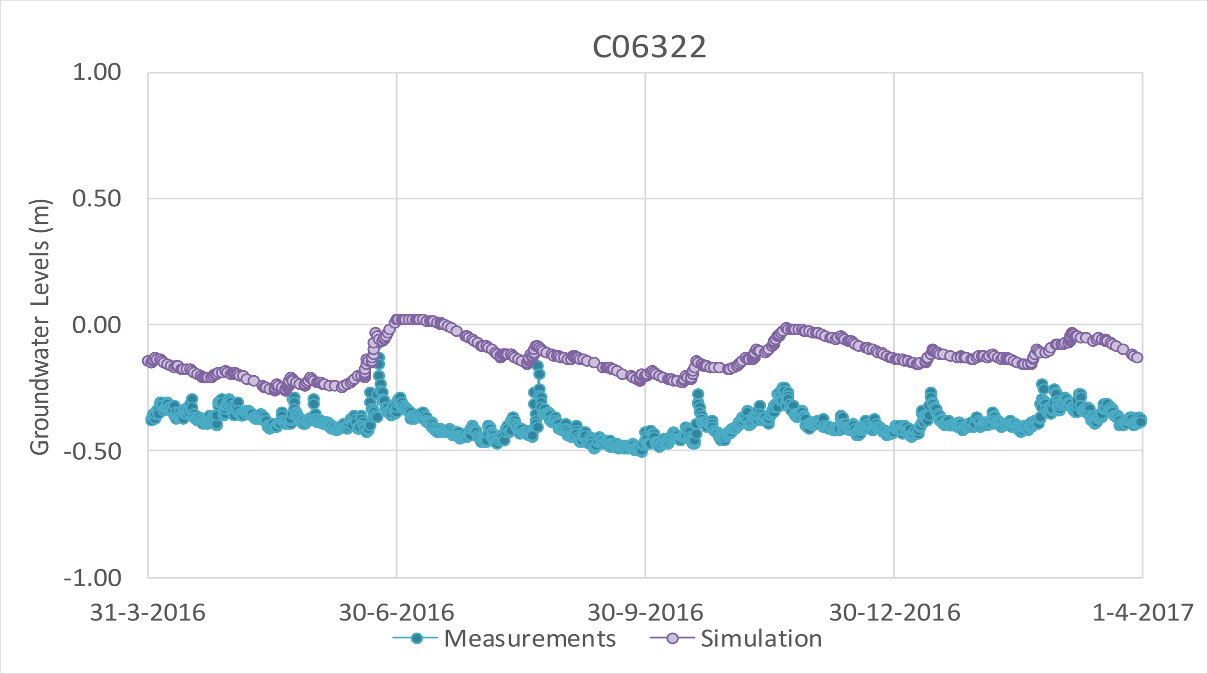
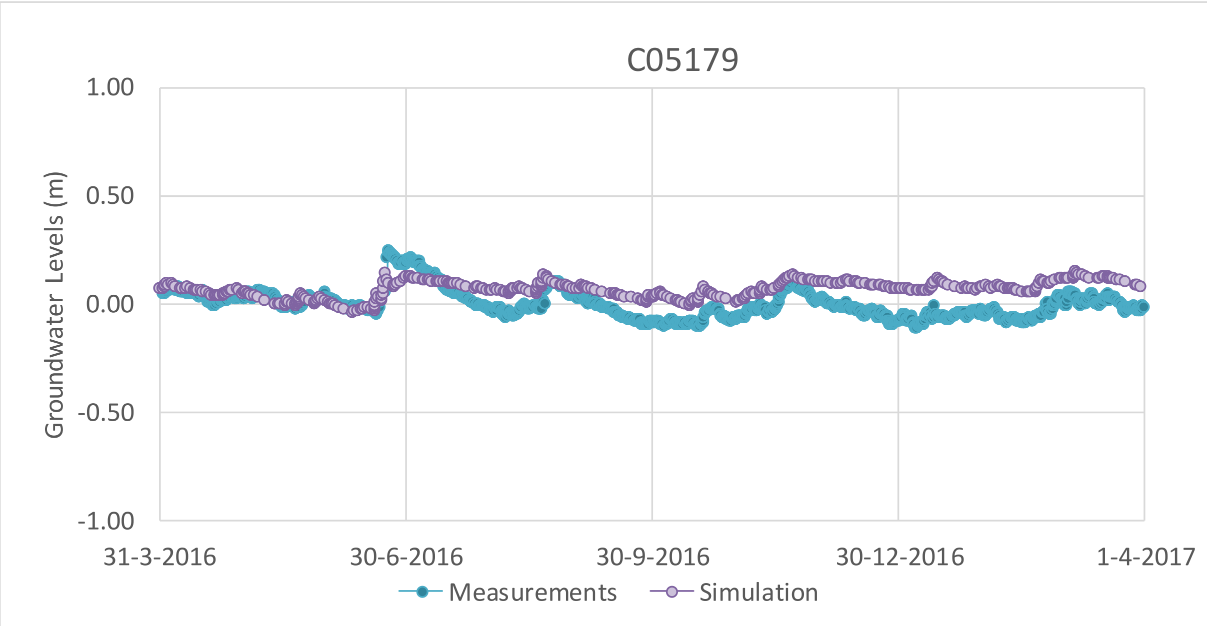


Figure 6.8 Transient calculation results compared to groundwater level measurements, piezometer C06322.

Simulations for piezometers C05179 and C06321 display a relatively good fit, however there are no big variations in the actual groundwater levels. Measurements at piezometer C06321 display what might be an abstraction starting in December 2016, up to March 2017 (Figure 6.9). There are no repairs to the sewerage system or the drinking water network known in that period (Jaqueline Flink, March 2018, personal communication).



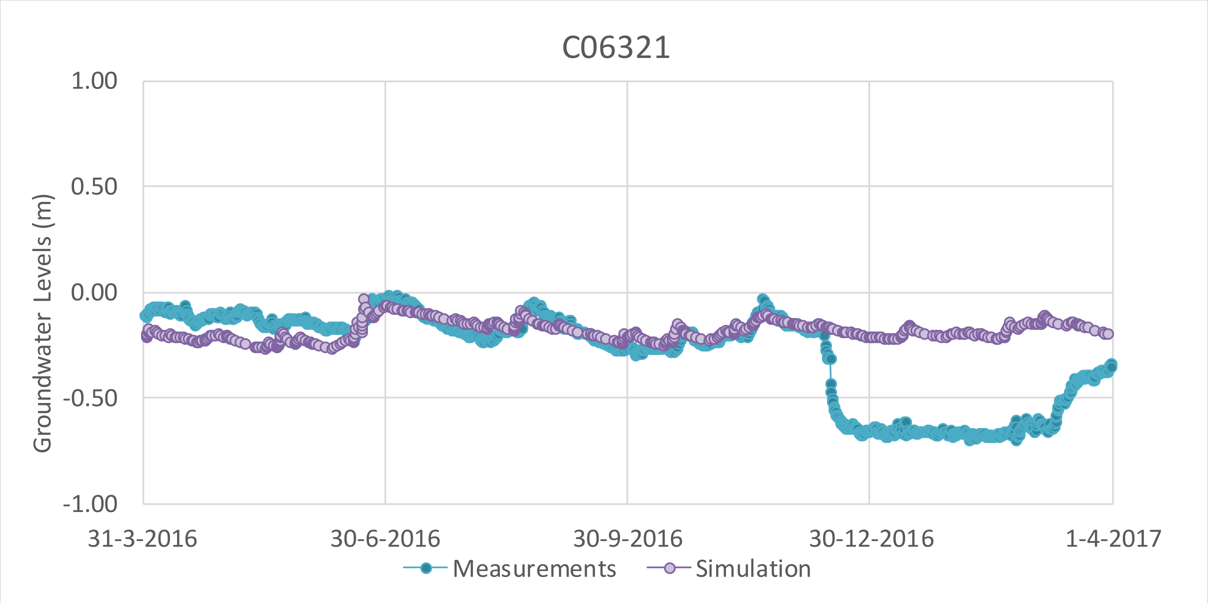


Figure 6.9 Transient calculations results for other piezometers.

6.3. Sewage modelling

6.3.1. Calibration results

Due to the limitations of modelling in InfoWorks and the limits imposed by the pump programming, replication of the exact MDW pattern was not possible. However, the overall totals were simulated with an overall difference of 0.04% from the measured value, which is insignificant. The measured totals for MDW was 57333 m^3, whereas the model generated 57313 m^3. Figure 6.10 shows the differences between the drinking water pattern and the actual DWF measured at the pump. Daily variations are visible. The night flow at the pump of 1 - 0.5 m^3/h can be observed again, compared to the 0 m^3/h drinking water consumption.

Because a pattern based on MDW is compared to the measured values at the pump during dry weather flow, there is a lag between the two. As mentioned in § 3.3.1, this is the time that the MDW spends in the distribution network, before reaching the tap. From the tap to the collection system there is only a couple of minutes delay.

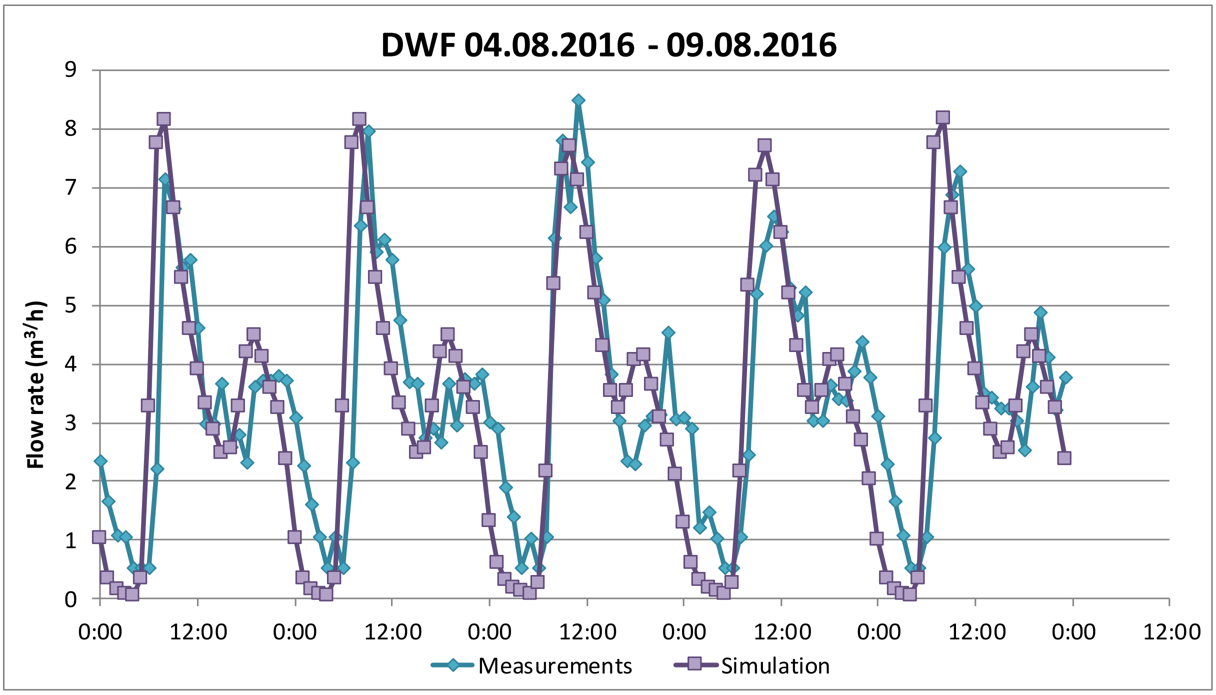


Figure 6.10 DWF during the period 04.08.2016 (Thursday) – 09.08.2016 (Monday).

The total volume estimated at the pump for the two years was 82768 m^3, including evaporation, which is with 6.12% less than the measured value (88167 m^3). This result was deemed as acceptable, since the focus of this part of the study was on the overflow events, which will be discussed in the section below. Since the drinking water is not what causes the differences, and the runoff is already overestimated, the differences might be caused by groundwater infiltration into the system.

A simulation was done trying to replicate groundwater infiltration in one of the conduit sections indicated by Dirksen (2016) to verify this assumption. For the central section of the island, the average conduit height was compared to the groundwater levels from piezometer C05179. If the groundwater level at this piezometer reached 75% of the height of the conduit, then it was indicated as a day with possible infiltration. 29 days were counted. The total amount of groundwater infiltration based on a 9 m^3/d average, was divided into 29 days, resulting in an infiltration rate of 255 m^3/d, which is of course, unrealistic, due to the simulation at only one point. However, the total amount simulated at the pump for the two years is 89918 m^3/d, which is exceeding the measurements by only 2%.

6.3.2. Sensitivity analysis

The model is most sensitive to changes in runoff. Changing the discharge coefficient of the overflow structure makes no difference for either the amount of overflow or for the total amount at the pump. The results are shown in Table 6.12.

In the calculations, the default values from Table 5.2 and a runoff coefficient of 1 were used, since these values are the standard values used within Waternet for sewerage modelling

Table 6.12 Changes in pump totals by changing initial losses or runoff coefficient values.

	(m^3)	% Change
Initial model total volume (no evaporation)	85292	-
Halved initial losses (no evaporation)	85800	-0.60%
Adjusted runoff coefficients (no evaporation) *	83487	2.12%
*2 - 0.9; 5 - -; 7 - 1; 8 - 0.8.		

6.3.3 Overflow results

The model was first run with the predefined showers by Rioned (2004). This was done to observe what type of shower will trigger a CSO. It resulted that an overflow will be triggered for a shower with a return period of 0.5-years, a total of 14.4 mm in 75 minutes and a maximum intensity of 70 l/s.ha. The simulation was run for the night period, when there is little, or no DWF. The generated amount of overflow is 34.91 m^3.

As mentioned in § 4.5.2., the measurements for the status of the overflow proved to be unreliable, therefore the results were compared to the overflow events that were indicated through TSA. The characteristics of the events were summarized in Table 6.13.

Table 6.13 Summary of the calculated overflow characteristics for the research period (04.08.2015 – 03.08.2017).*

No	Start of Spill (date)	Start of Spill (time)	Spill Duration (min)	Peak Flow (m^3/s)	Time of peak (time)	Spill Volume (m^3)
1	26-08-15	21:06	59	0.0773	21:11	91.70
2	17-09-15	13:16	32.7	0.0273	13:26	26.55
3	30-05-16	21:11	19.7	0.1946	21:16	89.71
4	20-06-16	18:31	33.1	0.1100	18:41	91.47
5	23-06-16	07:46	39.3	0.3332	8:06	445.54
6	19-08-16	21:21	79.3	0.1759	22:16	192.50
7	21-08-16	10:11	52.9	0.2320	10:26	218.32
8	19-10-16	06:36	33.8	0.0256	6:56	29.23
9	09-06-17	10:39	7.5	0.0040	10:43	1.12
10	29-07-17	22:36	29.2	0.1354	22:41	109.08

*The reports summarising the overflow events were generated in InfoWorks. The simulations were for 5-minute time steps and the values presented come from a report with the threshold of 0.001 m^3 or 0.001 m^3/s.

For the 2 – year study period, the model generated 10 overflow events (Figure 6.11). In total, there were 6.44 hours of overflow, and a total discharged volume of 1295 m^3. The largest event was on 23.06.2016, when in 39.3 minutes, there were 446 m^3 discharged to the canal. The peak flow was of 0.333 m^3/s.

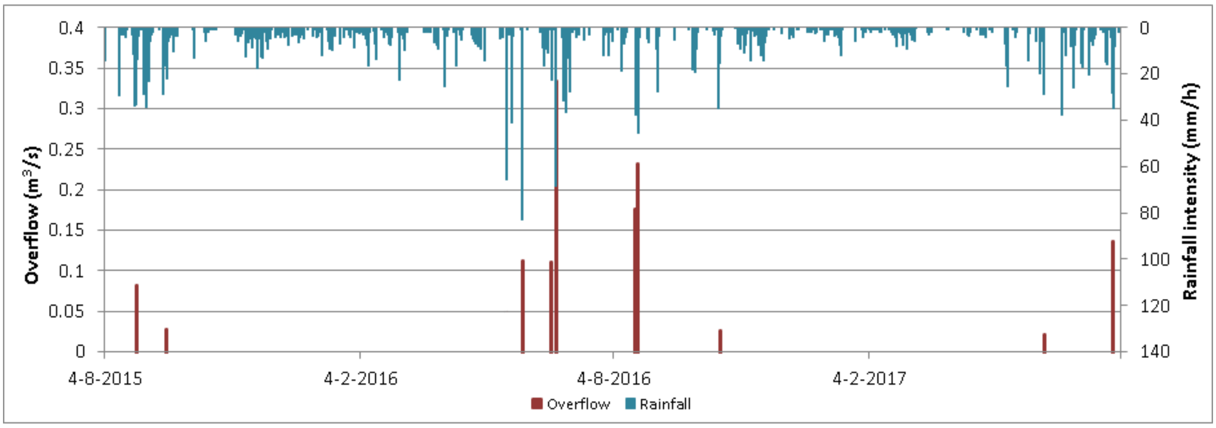


Figure 6.11 The CSO events simulated by InfoWorks.

Figure 6.12 shows the pump flow measurements during separate overflow events compared to the simulated values. To avoid the irregularities caused by the programming of the pump, flows in the conduit preceding the pump were analysed. The reported flows represent hourly averages.

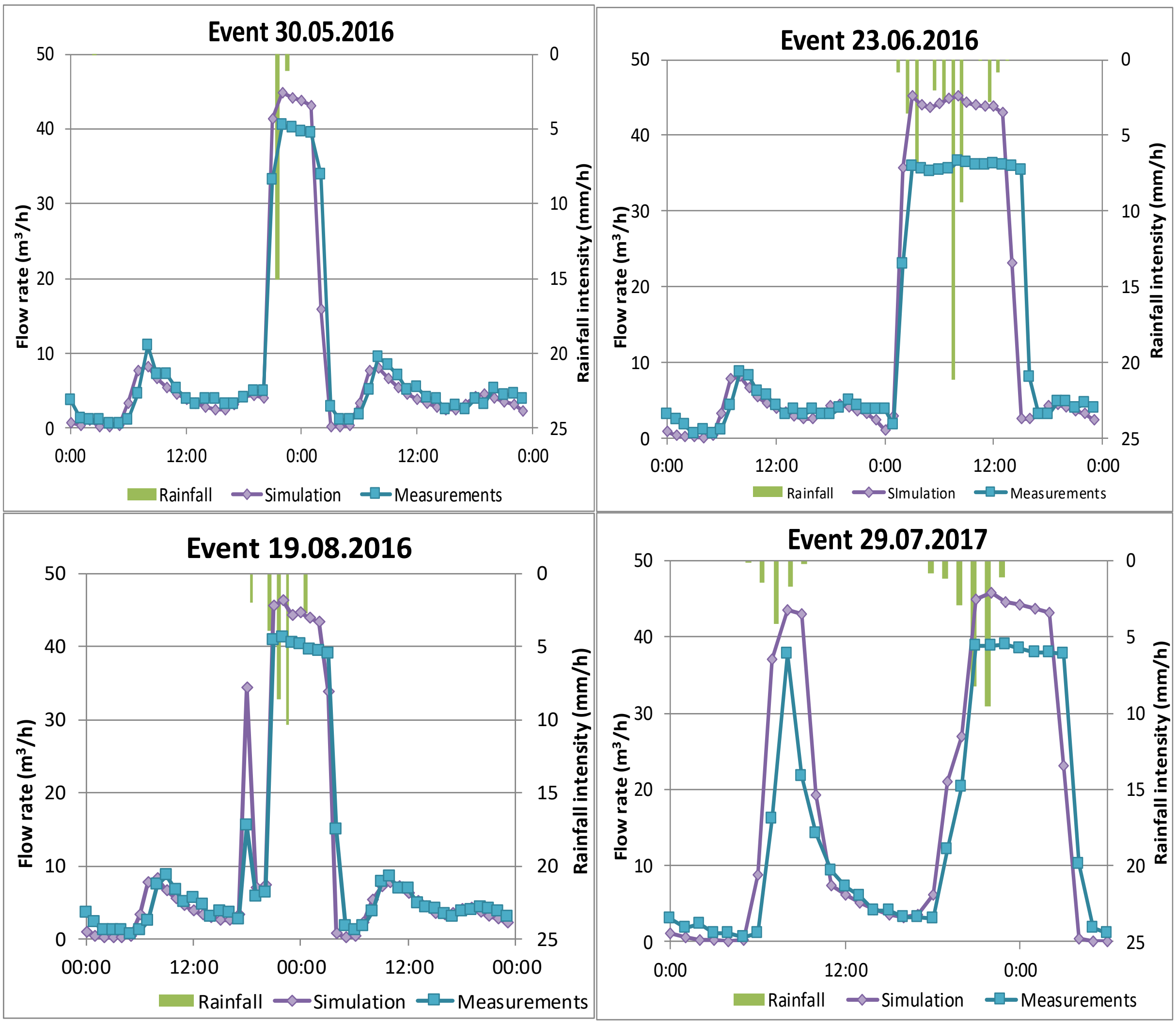


Figure 6.12 Measurement against simulation values for overflow events.

The InfoWorks pump will automatically use its maximum capacity when raining, whereas the real pump rarely reaches its maximum capacity. However, the measured and simulated trends coincide.
Figure 6.13 shows the measured water levels at the pump compared with the simulated ones. 5-minute results were averaged in 1-hour time step. The levels in the simulation fluctuate more than the measured ones. However, they follow the same trend. It can be observed that during an overflow spill, the levels rise to NAP 0.6 m, which is the overflow level. In some of the graphs, due to the averaging of the values, the average value is slightly lower than NAP 0.6 m.

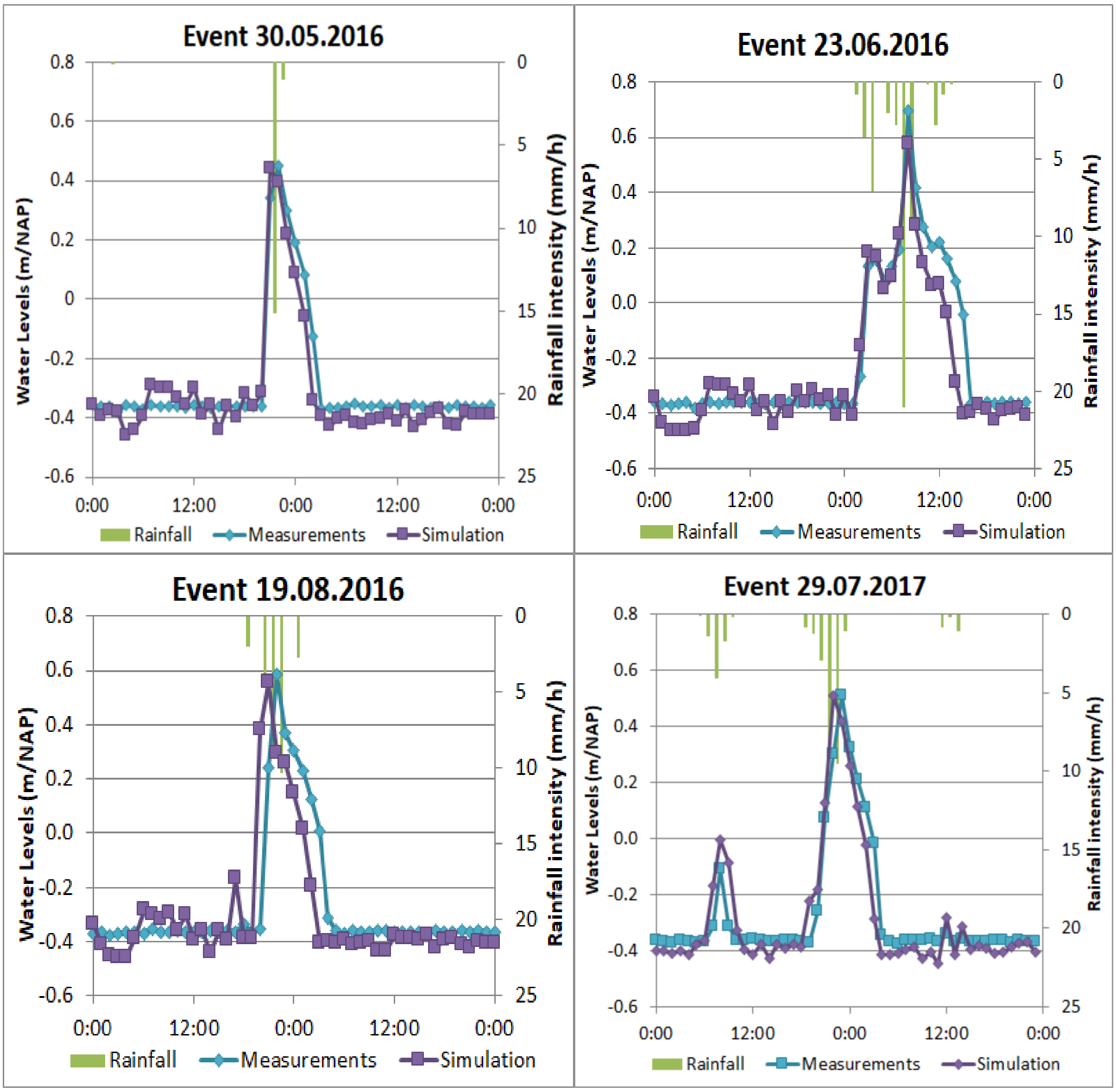


Figure 6.13 Measured levels against simulated levels in the during overflow events.

6.4. Overall water balance

In the following figure the overall water balance is presented (Figure 6.14). The values are presented in m^3/year and they are rounded off to no decimals.

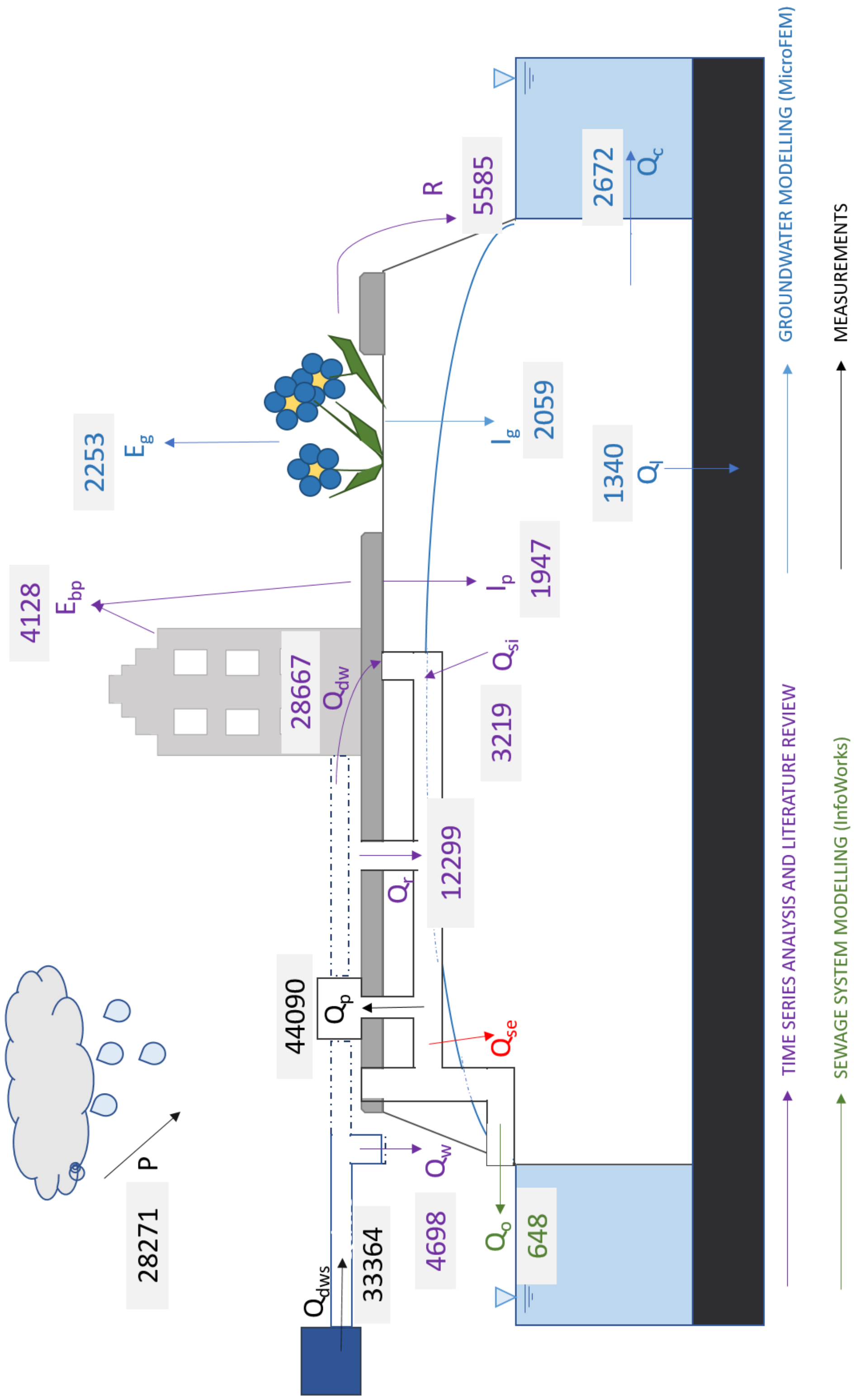


Figure 6.14 The completed water balance.

CHAPTER 7

Discussions

The following chapter discusses the uncertainties and limitations posed by the methods used in this research.

Models are built by assuming that the input data (measurements) are true (Deletic et al., 2012, p.4). However, this assumption does not hold, and all the uncertainties and errors of the input data will propagate through the models as shown in Figure 7.1.

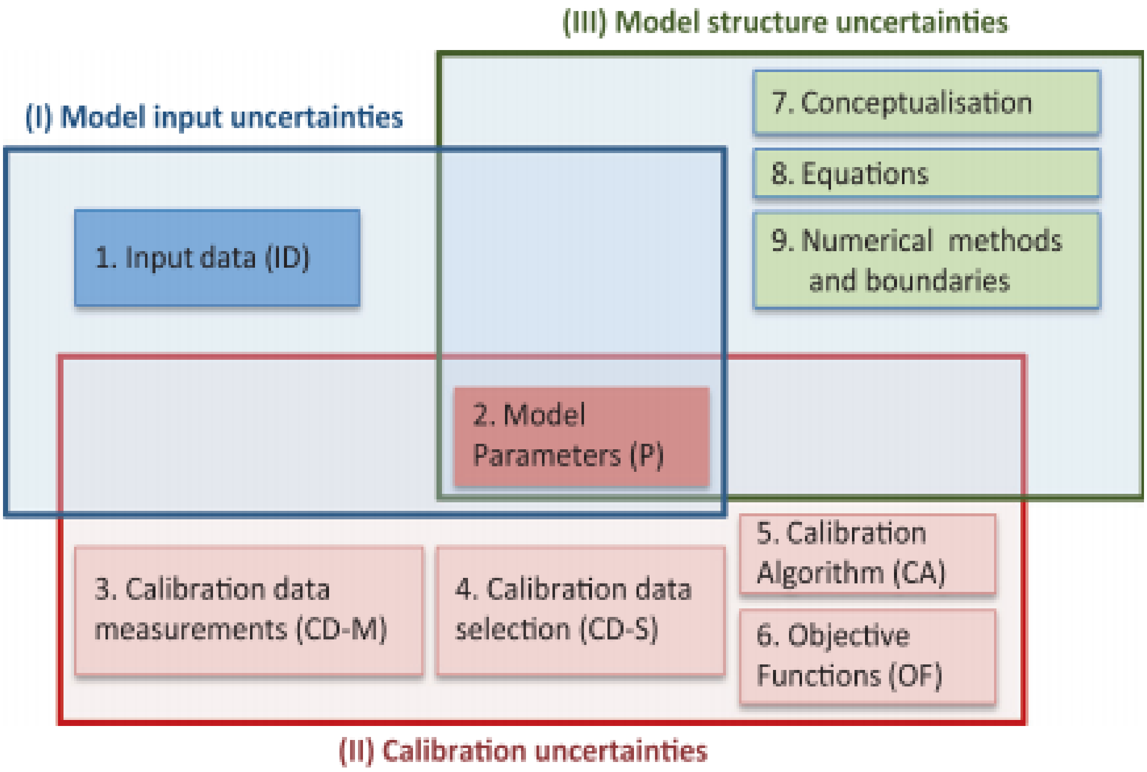


Figure 7.1 The key sources of uncertainties in urban drainage models and the links between them (Deletic et al., 2012, p.5).

Deletic (2012, p.5) groups the uncertainties in 3 main categories:

- Model input uncertainties, which includes the input data and parameters;
- Calibration uncertainties, such as deciding on the calibration data or the calibration algorithm;
- Model structure uncertainties, which refer to conceptualisation errors or incorrect boundary conditions.

7.1. Input data

Winter flow correction

An improper correction has influence on what is considered DWF, which will further influence the TSA results (proportion of runoff and groundwater drainage) and the results of InfoWorks. A different runoff result will affect the recharge values for the groundwater model, since the value obtained through TSA is a boundary condition in determining losses of the paved and built areas.

Starting late 2017, Waternet installed a temperature sensor and a flow meter to measure winter flow. The sensor will activate the winter flow only if temperatures will drop under a certain value.

Waste water and drinking water measurement errors

Measurement errors (§ 4.5.2.) will cause further calibration errors into the models. The most important one is probably the one at the pump, where there might be an overestimation of the pumped volume, especially at low volumes and at very high ones, resulting in a large quantity of water that cannot be explained by DWF or runoff, and which is considered groundwater infiltration. While groundwater infiltration is possible in the system, the amount might be overestimated.

7.2. Models conceptualisation and parameters

Model structure and boundary conditions

In the groundwater model there were a series of assumptions that might influence the results: the fully penetrating canal to the Holocene layers, the anisotropy of the anthropogenic layer based on various structures (house basements, sewerage system), the estimated recharge, the fixed transmissivity and depth of the bank protection area, regardless of its real structure.

Sewerage system incorporation in the MicroFEM model

The stationary model could not be calibrated with continuous drainage by the sewerage system. However, on Prinseneiland, infiltration in the sewer is a transient process, which happens only when the recharge will produce high enough groundwater levels to exceed the conduit's level (personal communication, Frank Smits, March 2018). Therefore, simulation of groundwater infiltration into the sewer should be attempted for transient calculations for the suspected areas indicated by Dirksen (2016).

Initial losses and runoff coefficient

The amount of recharge to the phreatic layer was calculated correspondingly to the TSA, literature review and field observations. The chosen initial losses to calculate groundwater recharge for the MicoFEM model are high and do not match with the values used by InfoWorks in the sewerage system model. The higher values were chosen to minimise infiltration, as not much infiltration is believed to happen through old and possibly clogged pavement. Moreover, the two models serve Waternet different purposes: one calculates groundwater fluxes, which requires a precise estimation of evaporation and losses to obtain a good recharge value, whereas the other model is used for conduit dimensioning, and, due to safety reasons, it will always overestimate the input by minimising losses and maximizing the runoff.

Pump characteristics

A closer to reality pump functioning in InfoWorks might yield different overall results for the sewerage modelling. Presently, the pump overestimates volumes during wet periods, since it works at a full capacity of 12 l/s. While comparing the results with the measurements, it can be observed that this does not happen often.

7.3. Calibration

Uncertainties from the input data and parameters will propagate through the calibration processes as well, creating different optima for different combinations of parameters.

TSA sewage flow model

The results obtained in the sewerage flow model are highly dependent on the piezometer data that is used. For the current study piezometer C05270 was used, but using, for example, C05179 changes the results by approximatively 5% for the groundwater infiltration and 10% for runoff percentage, while the drinking water stays the same, 65%. Having a different boundary condition for runoff will result in different percentage of total losses on active area.

Groundwater modelling

The input data that poses most questions is the recharge rate, through both pavement and gardens. Different recharge rates will yield different hydraulic conductivities for the phreatic layer. Therefore, if the recharge was underestimated, it means that the hydraulic conductivity of the island might be higher than modelled. If recharge is higher than estimated, then there is a possibility that infiltration of groundwater into the sewer can be incorporated into the model. The same assumptions are true for the vertical resistance of the Holocene layer.

Moreover, during the automatic optimization process, the model creates an overcompensation to correct for the overestimation of levels of piezometer C06322. Therefore, in general, the stationary model is underestimating the groundwater levels. A new calibration should be undertaken, without piezometer C06322, and observe the results.

7.4. Methodology used in the study

For this study, various methodologies were used to calculate the water balance.

The advantage of this procedure is that a robust view of the water balance and the interconnections between the systems and the modelling techniques can be observed. They give estimates to a wide range of fluxes of a water balance.

There are also disadvantages. For the study to be completed, iterative processes should be undertaken between the stationary groundwater model and the transient one and between the groundwater model and sewage system model to minimise the presented errors as much as possible and to obtain the best results.

CHAPTER 8

Conclusions and recommendations

8.1. Conclusions

A water balance study was conducted for Prinseneiland for the period 04.08.2015 – 03.08.2017. The main objective for this study was to obtain an estimate for groundwater recharge and groundwater drainage through the sewerage system. For this, the research was structured in three interconnected parts, each one of them focusing on different aspects of the urban water cycle, to help answering the sub-questions and main research question.

The first step taken into the research was data review, which answered the first sub-question. Data from the rain gauge on the island was not reliable for the study period, therefore data from another rain gauge was used. However, since the rain gauge was moved to a new location and the batteries of the data transmitters have been changed, they offer better results, comparable with the KNMI database.

Time series analysis

A time series analysis was conducted to analyse sewerage flows, including groundwater drainage into the sewerage system and to analyse groundwater levels fluctuations. The regression model of the sewerage flow showed that 65% of the total waste water is dry weather flow, while only 28% is runoff. The difference is explained as groundwater drainage into the system. The estimated groundwater drainage into the sewer was 9-10 m^3/d, value comparable with an internal DWAAS analysis. The amount of runoff towards the surface water was determined to be 5585 m^3/y, which answers the second sub-question.

The response time of the catchment was estimated to be under 2 hours, which correspond with reality, giving the dimensions and the surface characteristics of the research area. Regarding the groundwater fluctuations, the result of the regression model is that these can be explained by evaporation and precipitation.

Groundwater modelling

Groundwater modelling offered an overview on the flux exchange in the phreatic layer of the island. However, groundwater drainage was not incorporated in the model, as recharge was almost equal to the predicted amount of groundwater drained into the sewer. The observed groundwater levels could not be explained by a system with hardly any groundwater recharge. The total recharge obtained for the island is 11 m^3/d, out of which 48.6% comes from infiltration through the pavement, while the other 51.4% is recharge through gardens. The biggest flux in the phreatic layer is towards the surrounding canals, which is 7.3 m^3/d, while to the underlying aquifer 3.7 m^3/d are leaking. This part of the research partially answers the third sub-question and the main research question.

Transient modelling was also undertaken for the period 01.04.2016 – 01.04.2017. There were two storativity factors used (0.05 for winter and 0.08 for summer). The evaporation factor was determined to be lower than the one used in the stationary model. This gave a good fit for the piezometers placed in the inner gardens. However, for the piezometers located in paved areas but close to the garden areas the fit was low as the simulation seemed to be influenced heavily by the garden conditions.

Sewerage modelling

Having an estimate for overflow was an important part of the research, therefore a sewerage model was built. It resulted that the system will overflow for showers with a return period of 0.5 years, as designed by Rioned (2004). For the 2-year study period, an approximate amount of 648 m^3/y were discharged to the canal, which answers the fourth sub-question. Adding to the model an average 9 m^3/d of groundwater infiltration could explain the total pumped amount of the sewerage system, giving an answer to part of the third sub-question.

Using multiple methods helped answering the individual sub-questions, by obtaining estimates for different water balance fluxes, giving thus a complete overview of the water balance for Prinseneiland.

8.2. Recommendations

Improve data collection on the island

Better data collection could significantly improve the modelling process and the reliability of the results: better overflow sensors, more information on what abstractions happen on the island. Two important steps to measurements improvement were taken by relocating the rain gauge, which gives significantly better results and the installation of the temperature sensor and of a flow meter for the winter flow, which will give more exact values for the extra amount of drinking water used for maintenance.

Improve understanding of time series analysis parameters

While TSA gives valuable information in approximating sewerage flows and their influence, there are still parameters that are hard to explain in the regression models and which need more proper understanding. Do the time constants in the groundwater model have the same meaning as in the sewage flow model? Do they indicate the “time lag” of precipitation/evaporation to groundwater level fluctuations? Can we see this in the groundwater levels? How can we explain the groundwater constant in the sewerage flow? What are the influences of each piezometer in the sewerage model and what do these differences mean?

Moreover, the sewerage model should be run for every piezometer and observe what influence they have on the overall results.

Improve calibration of the groundwater model

Having many unknowns means a lot of assumptions. While the stationary model is relatively correct, it could be improved by taking out of the calibration process piezometer C06322, as the measured heads are much lower than expected in the area, therefore creating a high overestimation in the model. An investigation should be undertaken to determine if there are any abstractions happening or if it is just higher transmissivity in the area and altering the model accordingly. This could minimise the errors for the rest of the heads and give a better start to the stationary model. For the stationary model it would be useful calculating a series of alternative models with an increase/decrease in recharge and observe the optimized transmissivities, thus obtaining an upper and lower estimate of the proper values.

Undertaking iterative steps in between the transient model (calculated for the entire two years) and the stationary one will give better indicates on the evaporation factor needed for determining the recharge. Drainage by the sewerage system could be added to the model for the areas where it is suspected that it happens for the transient state. When it is suspected that the recharge could produce high enough groundwater levels, a well should be incorporated in the calculations.

Simulate groundwater drainage in InfoWorks

A rough simulation was done in InfoWorks to observe if the missing quantities at the pump could be explained by groundwater infiltration. The given groundwater inflow of approximatively 200 m^3/d was of course unrealistic.

Iterative process between the models

Iterative steps should be taken between the InfoWorks model and the groundwater model to obtain a better calibration for both models. This will improve the estimates for runoff and losses (infiltration losses and evaporation).

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Annexes

Annex A: Maps

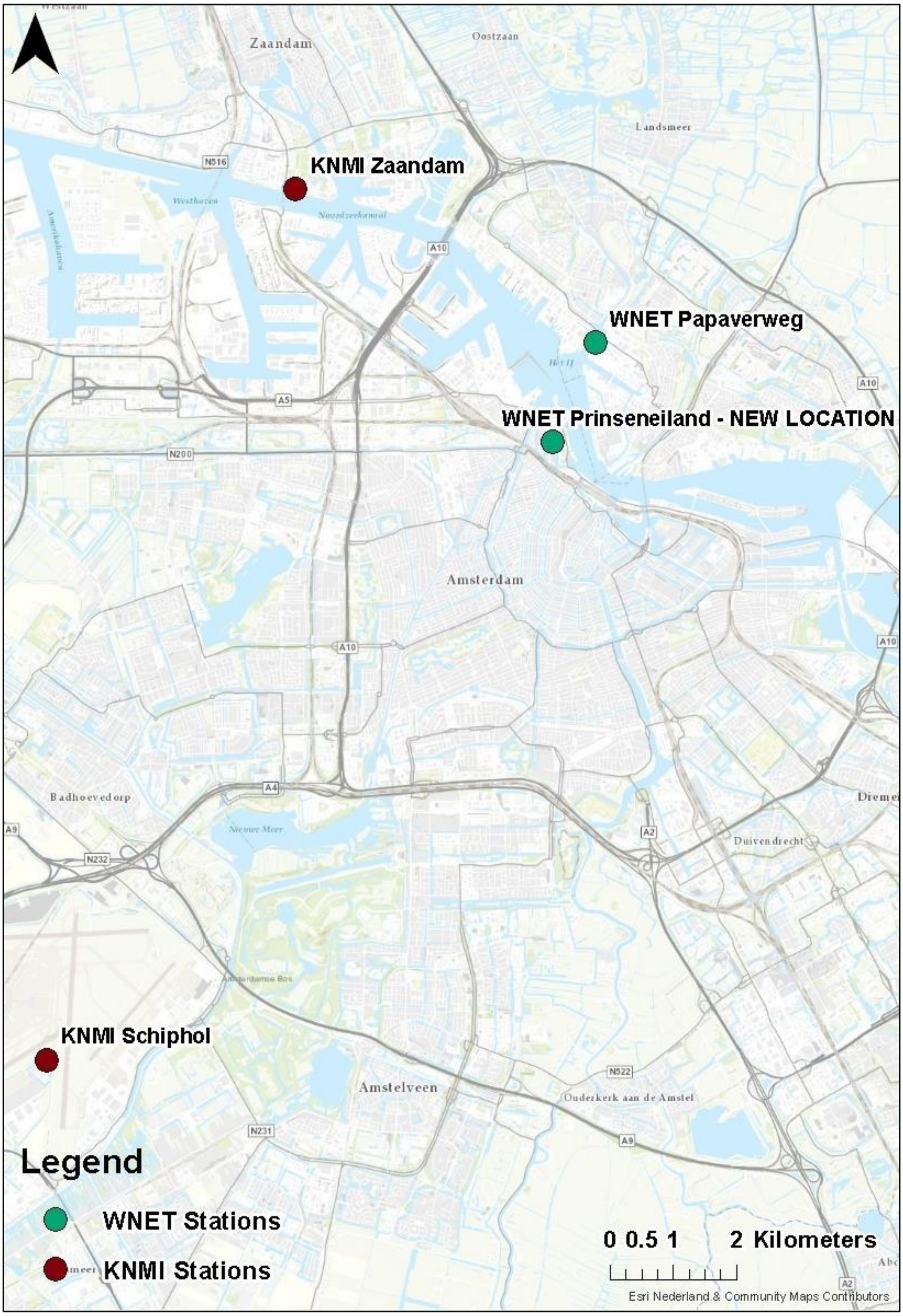


Figure A-1 Location of the weather stations or rain gauges relevant for the study.

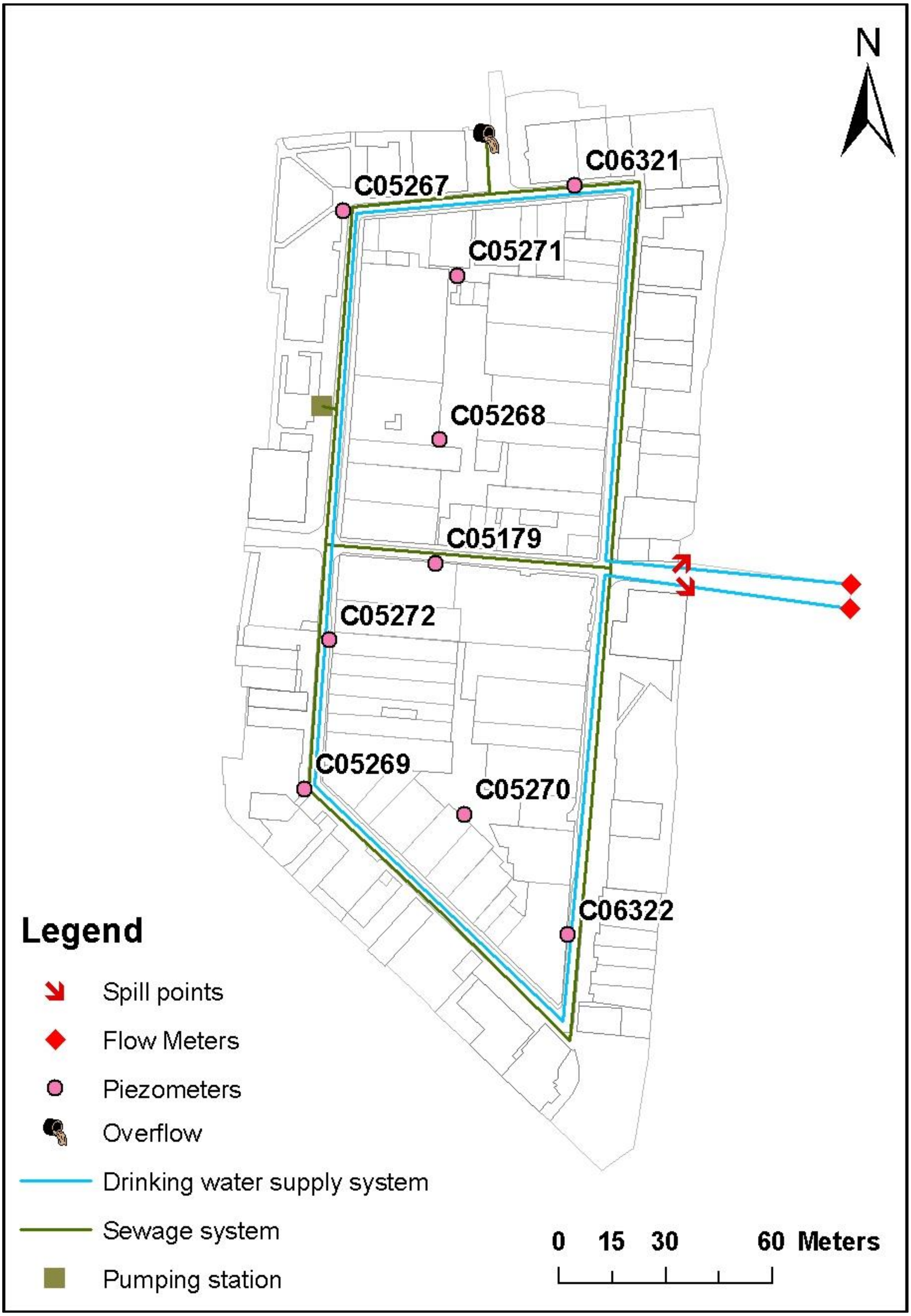


Figure A-2 Elements of the water chain present on the island, including the groundwater measuring points.

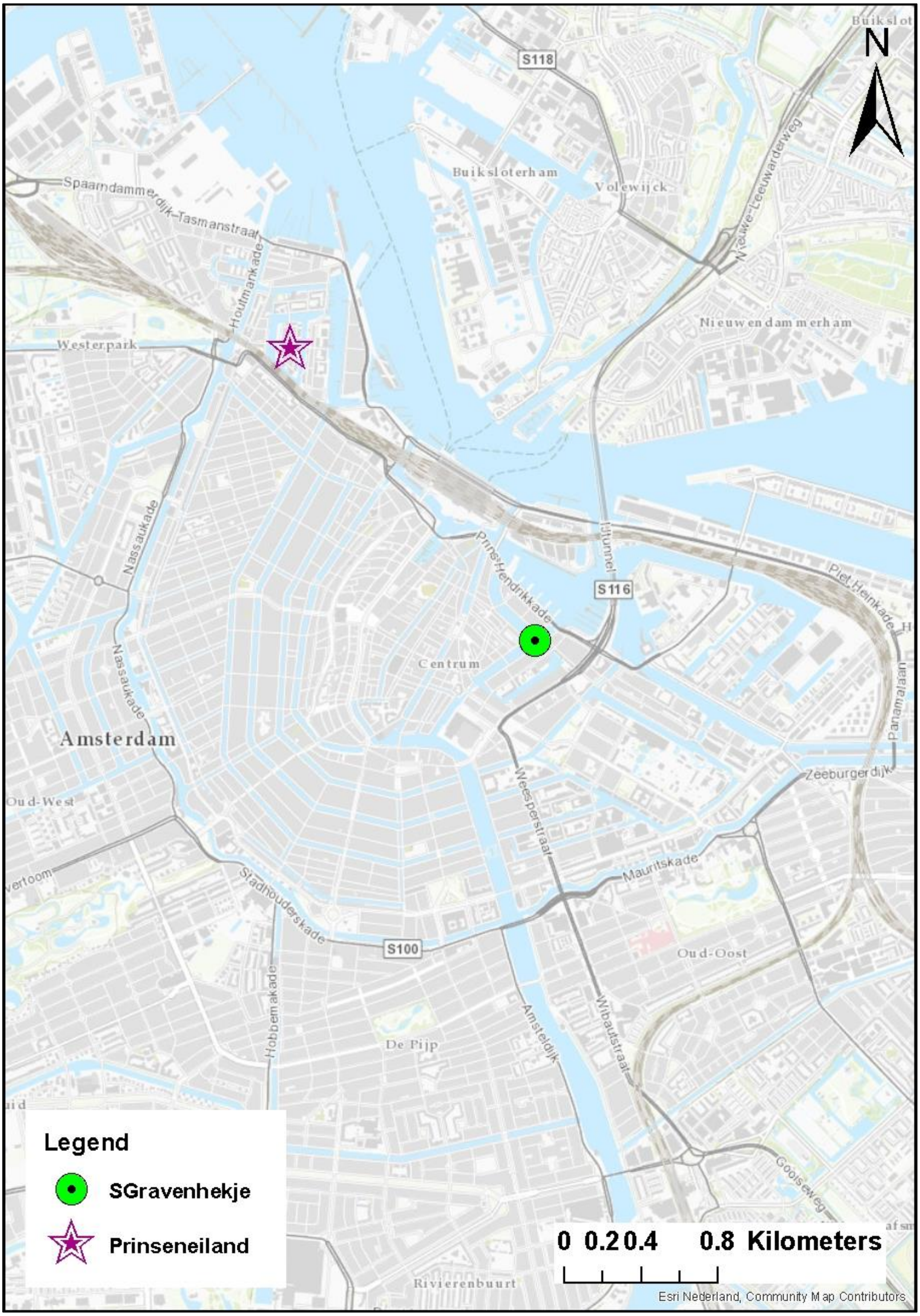


Figure A-3 Location for measuring point of surface water levels.

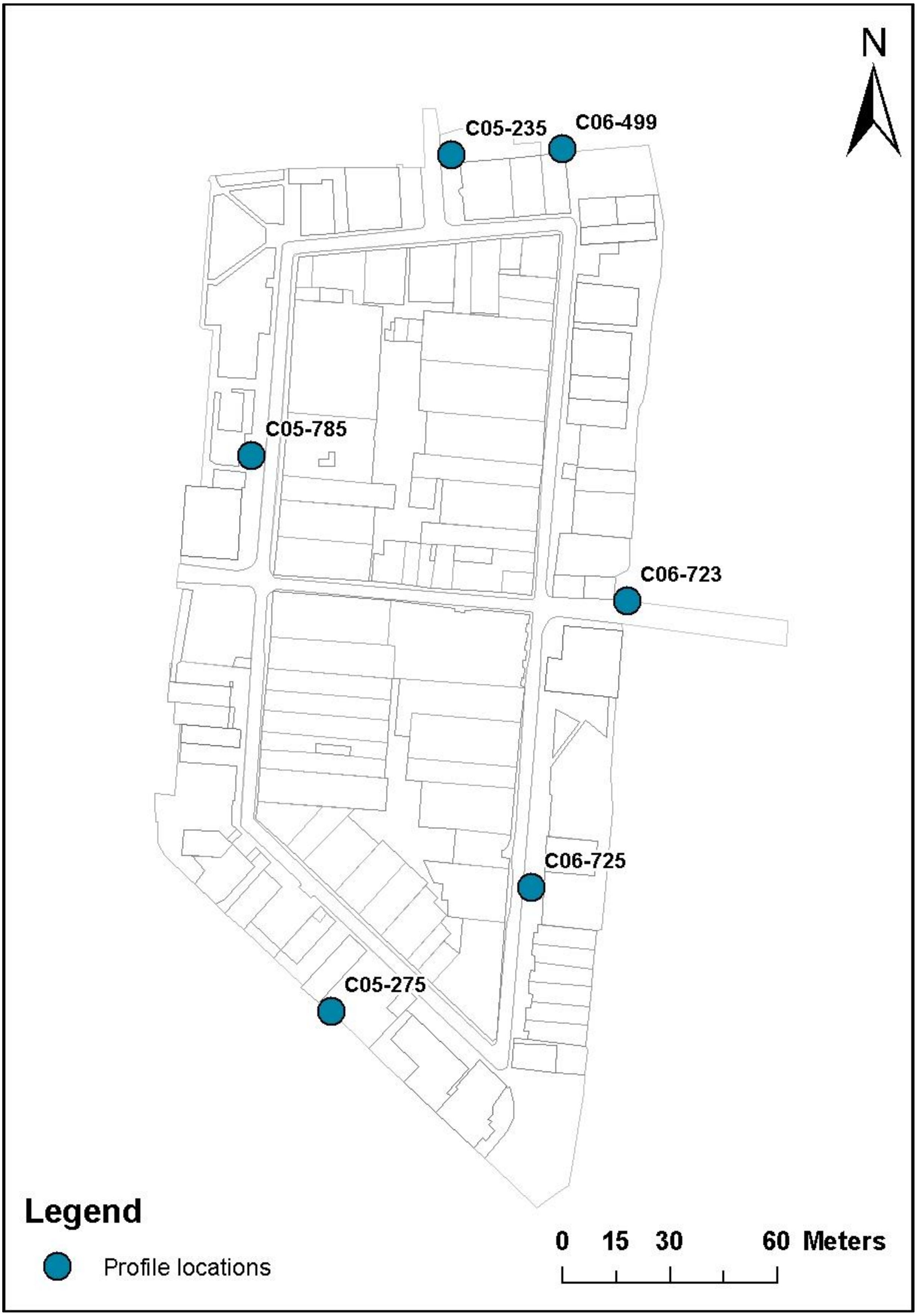


Figure A-4 Location for cone penetration tests or profiles (GeoWeb).

Figure A-5 The division of the island into subcatchments for every manhole (InfoWorks model).

Annex B: Geology and hydrogeology

At about NAP -48 m, lies the Drente Formation, which is made of deposits of clay and silt. This is overlain by Eem Formation. This geological unit consists of approximatively 20 m of marine deposits, mostly sand, clay and shells (Schokker et al., 2015, p.7).

On top of the Eem Formation are the Kreftenheye and Boxtel Formations, respectively. These are thick (~15 m), Late Pleistocene deposits, which contain sand of aeolian and fluvial origins (Schokker et al., 2015, p.7). The second and the first aquifer are in these formations.

The beginning of Holocene is marked by a thin (0.2 – 0.5 m) peat layer, "Basis Veen" or "Base Peat", formed as a result of sea level rise. It is very compacted, due to the weight of the overlying layers, therefore it has very high resistance (Schokker et al., 2015, p.7). On top of it lies a thick clay layer of maritime deposits, which is overlain by another peat layer, "Hollandveen". The first clay layer appears at about 2 – 3.5 m below surface and has maritime origins (Rutten, 2013, p.15). Figure B-1 presents the main geological units at the Prinseneiland location.

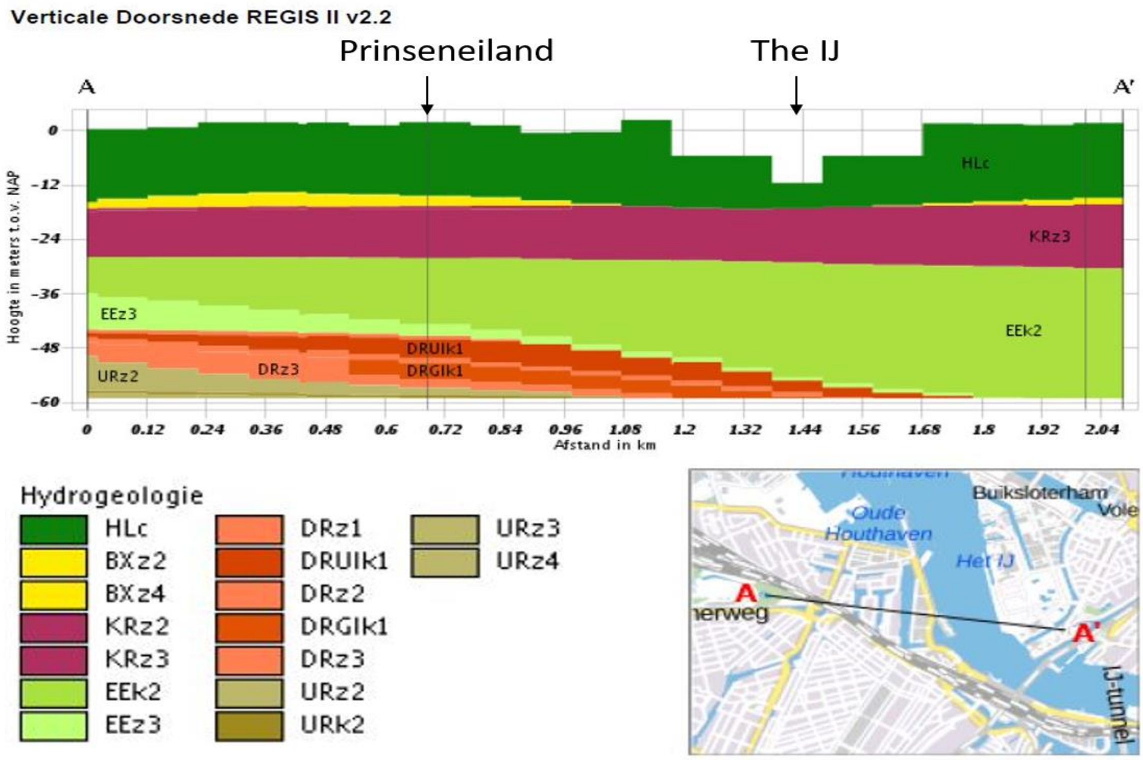


Figure B-1 REGIS II v2.2 Profile. Location of Prinseneiland (Source: dinoloket.nl.).

Legend key. "z" stands for sandy layers, "k" for clay layers (translated from Dutch from: Dinoloket.nl, Modeleenheden REGIS II v2.2).

HL – Holocene Deposits, BX – Boxtel Formation, KR – Kreftenheye, EE – Eem, DR – Drente, UR - Urk

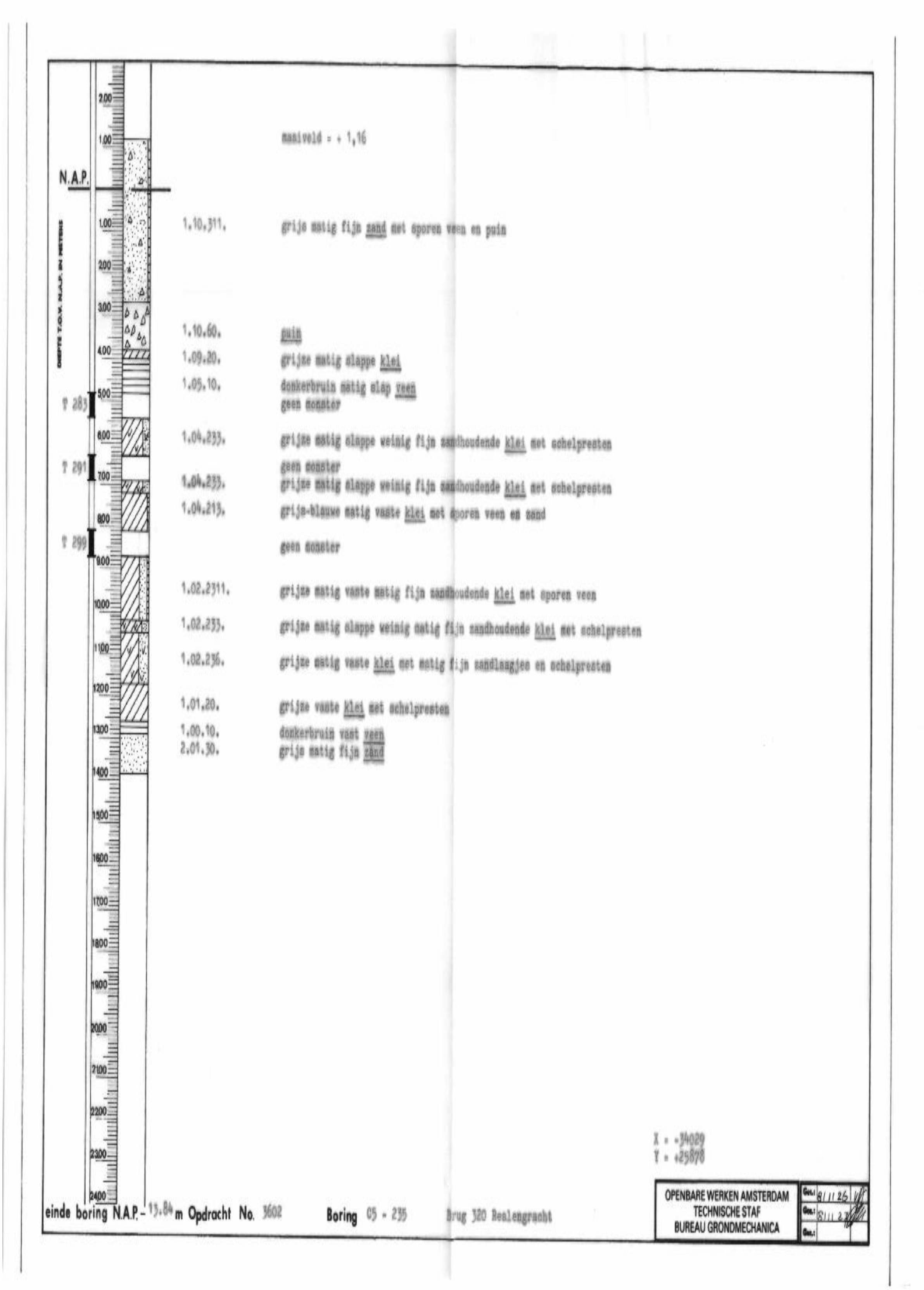


Figure B-2 Profile C05 – 235 (Source: GeoWEB).

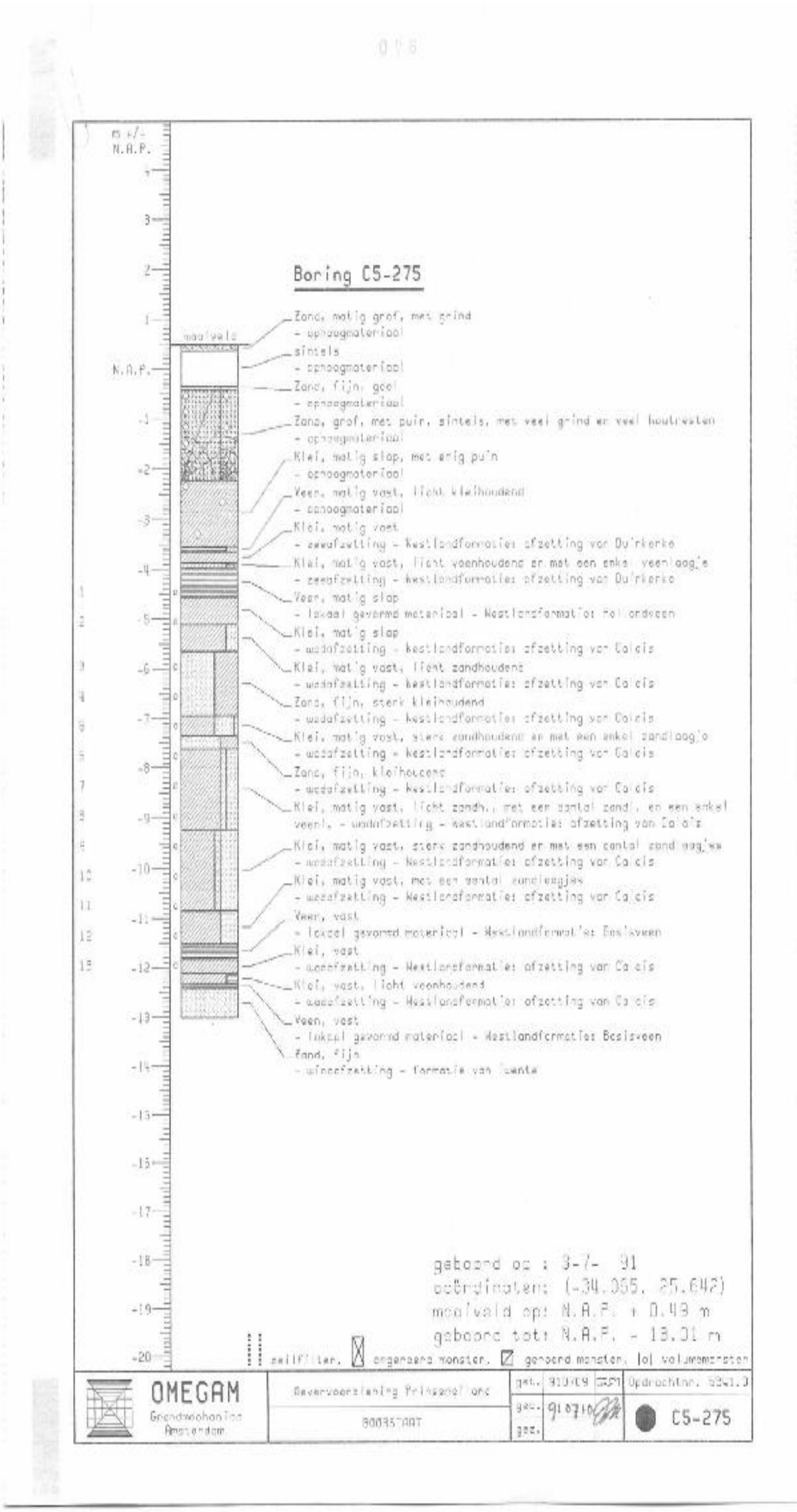


Figure B-3 Profile C05 – 275 (Source: GeoWEB).

Annex C: Piezometer data for the isohypse map

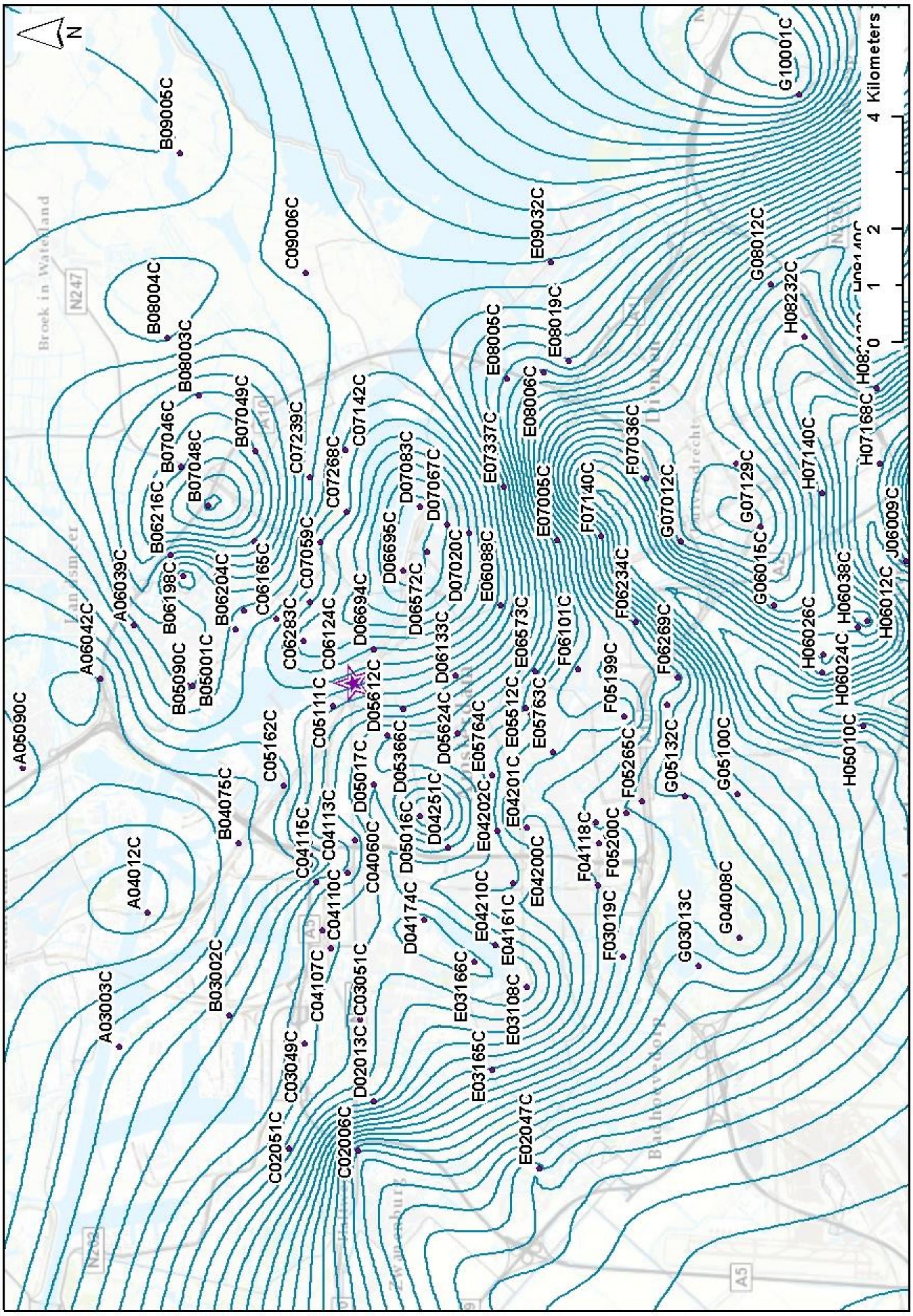


Figure C-1 The piezometers used in creating the isohypse map for the first aquifer.

Table C- 1 Piezometer data used for creating the isohypse map of the first aquifer averaged over 2 years (2015 – 2016).

ID	Average	ID	Average	ID	Average	ID	Average	ID	Average
20J00071	-3.58	31F02671	-1.31	C07059C	-1.70	F04133C	-3.05	J10001C	-2.23
20J00101	-2.41	31F02681	-1.11	C07142C	-1.79	F04135C	-2.24	J11014C	-1.52
3.10E+283	-1.02	31F02721	-2.12	C07239C	-2.01	F05195C	-3.00	J11016C	-1.64
31E01631	-1.38	31F02722	-2.47	C07268C	-1.72	F05197C	-0.98	J11020C	-1.43
31E01632	-1.44	31F02723	-2.63	C09006C	-1.98	F05198C	-3.04	J11026C	-1.47
31E01633	-1.38	31F02724	-2.67	D02013C	-2.61	F05199C	-2.75	K05001C	-5.32
31E01661	-1.49	31F02725	-3.66	D04174C	-2.34	F05200C	-3.14	K05002C	-4.74
31E01671	-1.25	A03003C	-1.77	D04251C	-2.39	F05201C	-3.17	K07001C	-3.93
31E01741	-1.29	A04012C	-1.35	D05016C	-1.97	F05241C	-3.27	K08001C	-2.92
31E01742	-1.32	A05090C	-1.75	D05017C	-2.39	F05242C	-3.22	L06010C	-4.74
31E01743	-1.28	A06039C	-2.09	D05366C	-2.12	F05265C	-3.26	M10016C	-1.88
31E01744	-1.27	A06042C	-1.60	D05612C	-2.40	F06101C	-2.93	P04014C	-5.36
31E01745	-1.32	B03002C	-1.99	D05624C	-2.35	F06234C	-3.17	P12005C	-1.17
31E01746	-1.24	B04075C	-1.68	D06133C	-2.08	F06269C	-3.16	P12009C	-1.15
31F00151	-1.32	B05001C	-2.16	D06572C	-1.25	F07036C	-3.40	Q04017C	-5.14
31F01691	-2.10	B05090C	-2.07	D06694C	-1.71	F07089C	-3.95	Q04024C	-5.19
31F01692	-2.69	B06105C	-2.17	D06695C	-1.13	F07140C	-3.41	Q04026C	-5.33
31F01701	-2.05	B06156C	-1.95	D07020C	-1.52	G03013C	-3.47	Q10007C	-3.12
31F01702	-2.33	B06198C	-1.91	D07021C	-1.22	G04008C	-3.21	Q11003C	-1.30
31F01703	-2.32	B06204C	-1.96	D07067C	-1.29	G05100C	-3.55	Q11007C	-3.26
31F01711	-2.35	B06216C	-2.22	D07083C	-1.15	G05132C	-3.26	Q11013C	-1.27
31F01712	-2.99	B07046C	-2.58	E02047C	-4.15	G06015C	-3.42	Q11015C	-3.10
31F01721	-2.43	B07048C	-2.93	E03108C	-2.43	G07012C	-3.22	Q11016C	-2.47
31F01722	-2.82	B07049C	-2.51	E03165C	-3.05	G07084C	-3.14	Q12002C	-0.87
31F01723	-2.83	B07098C	-2.51	E03166C	-2.38	G07129C	-2.99	R10028C	-2.66
31F01731	-2.14	B08003C	-2.42	E04128C	-2.58	G08012C	-3.26	R10029C	-2.30
31F01732	-2.59	B08004C	-1.99	E04161C	-2.63	G09042C	-1.22	R10032C	-2.81
31F01733	-2.60	B09005C	-2.19	E04200C	-3.01	G10001C	-1.68	R11010C	-1.57
31F01734	-2.62	C02006C	-3.68	E04201C	-2.76	H05010C	-4.07	R11013C	-1.47
31F01735	-3.46	C02046C	-2.40	E04202C	-2.67	H06012C	-3.45	R12004C	-1.02
31F01831	-1.03	C02051C	-2.40	E04210C	-2.51	H06024C	-3.81	R12012C	-1.08
31F01832	-1.06	C03049C	-2.26	E05106C	-2.82	H06026C	-3.76	R12017C	-1.02
31F01833	-1.05	C03051C	-2.44	E05512C	-2.59	H06038C	-3.75	S11004C	-1.14
31F01834	-1.09	C04012C	-2.28	E05549C	-2.79	H07140C	-3.67	S11005C	-1.14
31F02291	-1.16	C04060C	-2.15	E05763C	-2.87	H07168C	-4.13	S11006C	-1.03
31F02311	-1.15	C04107C	-2.27	E05764C	-2.65	H08140C	-2.69	S11016C	-1.51
31F02321	-1.13	C04110C	-2.12	E06088C	-1.87	H08213C	-3.46	S11017C	-1.21
31F02521	-1.29	C04111C	-2.12	E06370C	-2.02	H08232C	-3.58	S11018C	-1.21
31F02522	-1.28	C04112C	-1.95	E06573C	-2.53	H11002C	-1.64		
31F02523	-1.31	C04113C	-1.65	E07005C	-2.77	H11014C	-1.57		
31F02601	-1.13	C04115C	-2.17	E07337C	-2.04	J02004C	-4.41		
31F02602	-1.13	C05111C	-1.96	E08005C	-2.45	J02005C	-4.34		
31F02611	-1.14	C05162C	-1.76	E08006C	-2.54	J03002C	-4.28		
31F02612	-1.10	C05177C	-1.53	E08019C	-2.30	J03003C	-4.26		
31F02613	-1.02	C06003C	-1.41	E09032C	-1.90	J05007C	-4.41		
31F02631	-1.19	C06066C	-1.52	F03019C	-3.15	J06009C	-3.90		
31F02641	-2.08	C06124C	-1.75	F04111C	-3.05	J07073C	-1.59		
31F02651	-1.30	C06165C	-1.86	F04115C	-3.14	J08044C	-4.19		
31F02661	-1.23	C06283C	-1.53	F04118C	-3.00	J09003C	-2.49		

Annex D: Rain gauges data correlation

Data was collected for the study period 04.08.2015 – 03.08.2017 for various time steps for all stations in the vicinity, including Radar data and Radar interpolations for sewage catchment (Radar Westerpark). It can be observed from the tables below (Table D-1, D-2) that the gauges on Prinseneiland do not correlate with either of the other stations. This is due to the poor initial placement, which influences the measurements. From August 2017 the gauge was relocated on the adjacent Bickerseiland, which will improve the reliability of the measurements significantly.

Table D-1 Correlation between daily data of various stations and rain gauges near Prinseneiland.

Correl	Prinseneiland 1	Prinseneiland 2	Zaandam	Papaverweg	Schiphol	Radar 98,78	Radar Westerpark
Prinseneiland 1		0.76	0.43	0.14	0.14	0.18	0.15
Prinseneiland 2	0.76		0.42	0.16	0.15	0.17	0.16
Zaandam	0.43	0.42		0.59	0.57	0.60	0.57
Papaverweg	0.14	0.16	0.59		0.93	0.93	0.88
Schiphol	0.14	0.15	0.57	0.93		0.90	0.87
Radar 98,78	0.18	0.17	0.60	0.93	0.90		0.92
Radar Westerpark	0.15	0.16	0.57	0.88	0.87	0.92	

Table D-2 Correlation between hourly data of various stations and rain gauges near Prinseneiland.

Correl	Prinseneiland 1	Prinseneiland 2	Schiphol	Radar 98,78	Radar Westerpark	Papaverweg
Prinseneiland 1		0.79	0.01	0.02	0.01	0.02
Prinseneiland 2	0.79		0.01	0.02	0.02	0.01
Schipol	0.01	0.01		0.77	0.75	0.68
Radar 98,78	0.02	0.02	0.77		0.93	0.85
Radar Westerpark	0.01	0.02	0.75	0.93		0.81
Papaverweg	0.02	0.01	0.68	0.85	0.81	

Further down is presented a series of figures (D-1, D-2) showing the correlation between the Prinseneiland Rain Gauge 1 and the other rain gauges, and the correlation between the same stations after the rain gauge was moved off Prinseneiland and the batteries were changed (August 2017). The correlation improves significantly.

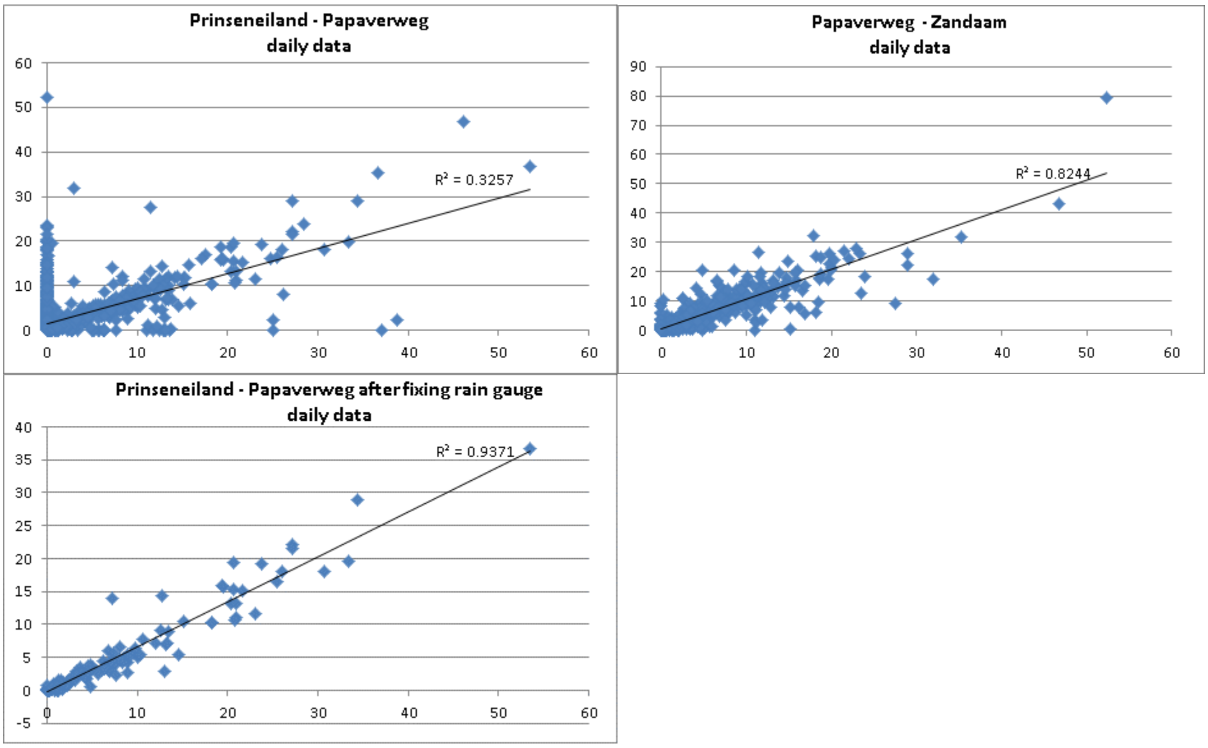


Figure D-1 A representation of the correlation between the rain gauges on Prinseneiland, Papaverweg and Zaandam station for daily data, for the periods 02.12.2013 – 12.03.2018 and 31.08.2017 (Zaandam).

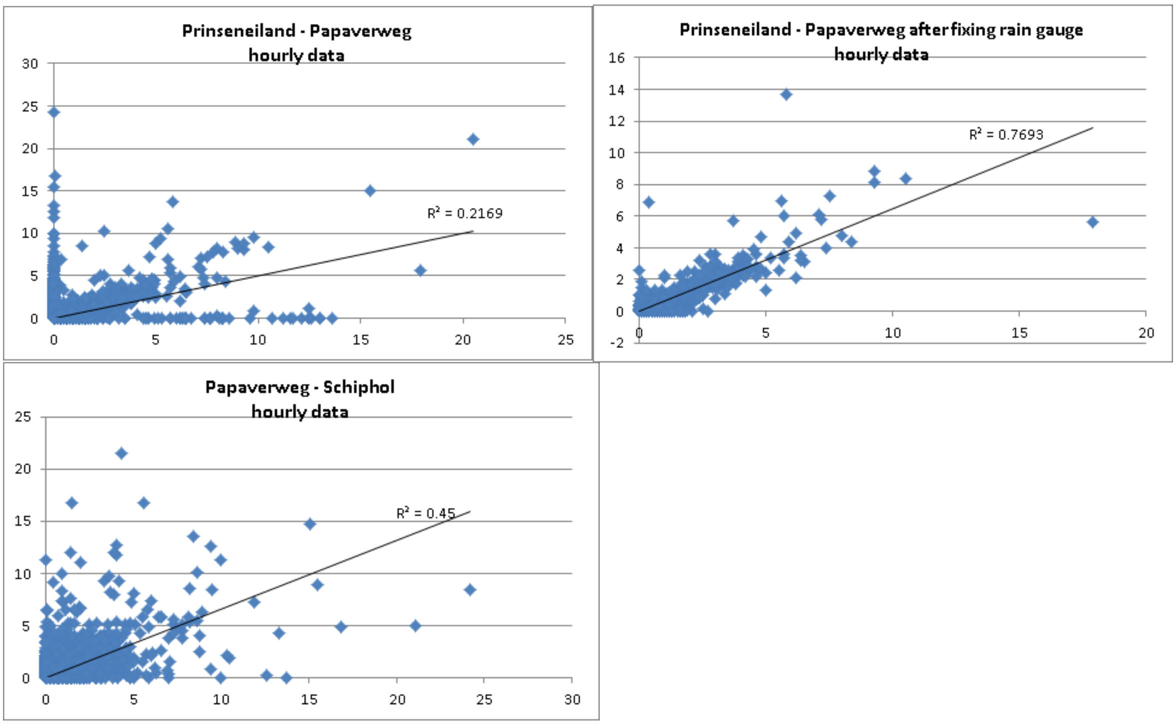


Figure D-2 A representation of the correlation between the rain gauges on Prinseneiland, Papaverweg and Schiphol station for hourly data, for the periods 01.12.2013 – 12.03.2018 and 10.12.2017 (Schiphol).

Annex E: Overflow measurements

The following graphs (E-1, E-2, E-3) will present a couple of individual events that cannot be explained with the data, which led to the conclusion that this data is not reliable.

In Figure E-1, it can be observed that the sensor of the overflow is faulty. The sensor stops working approximatively half way through the study period, later showing constantly the "On" mode.

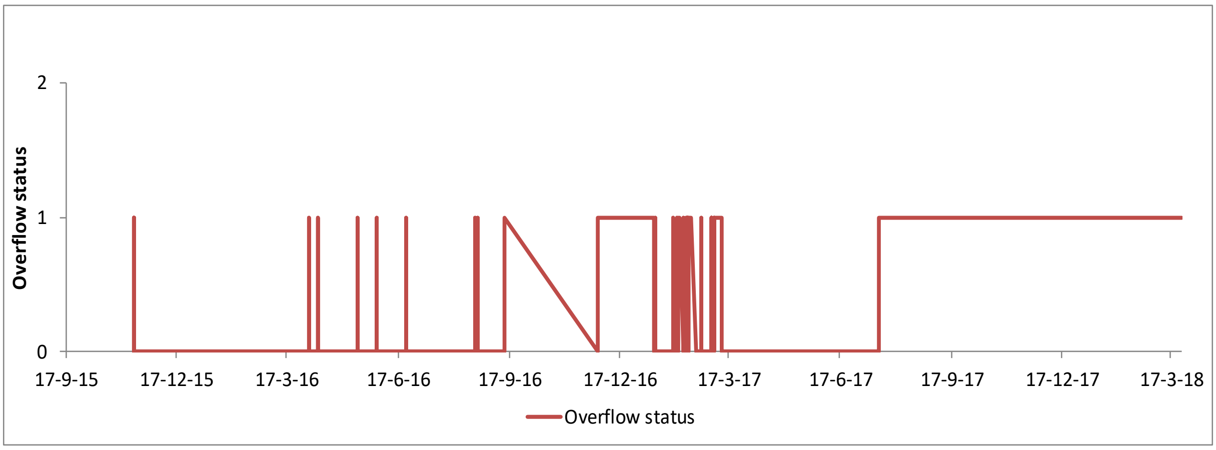


Figure E-1 Extract from FEWS showing in the top graph the "On" and "Off" mode of the sensor.

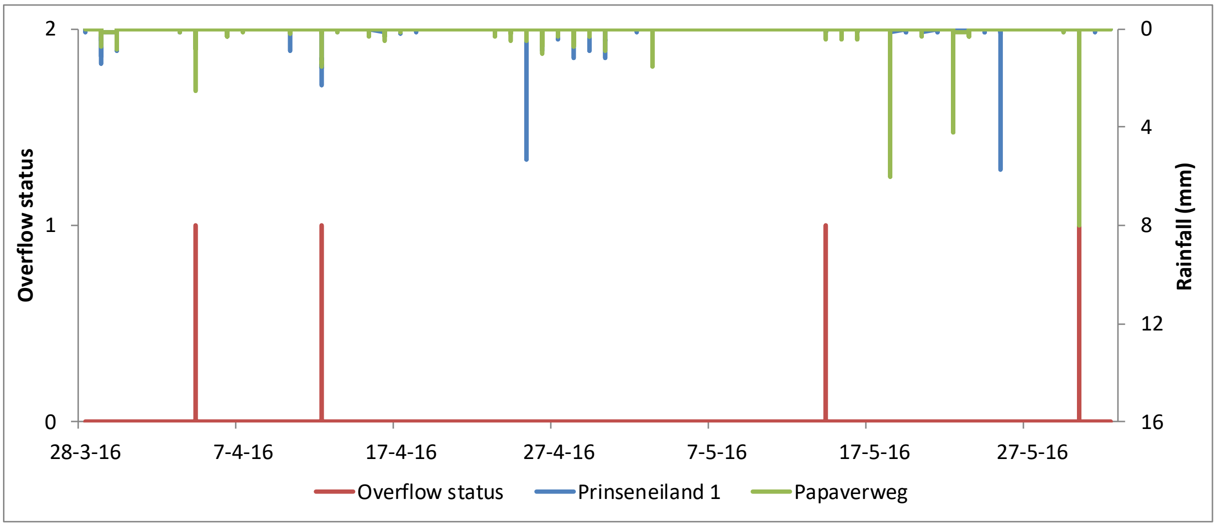


Figure E-2 Shows the overflow being triggered by a small amount of rainfall, and later an overflow event before a high intensity shower. The data was compared to Papaverweg (for accuracy of the data) and with Prinseneiland (for possible local showers).

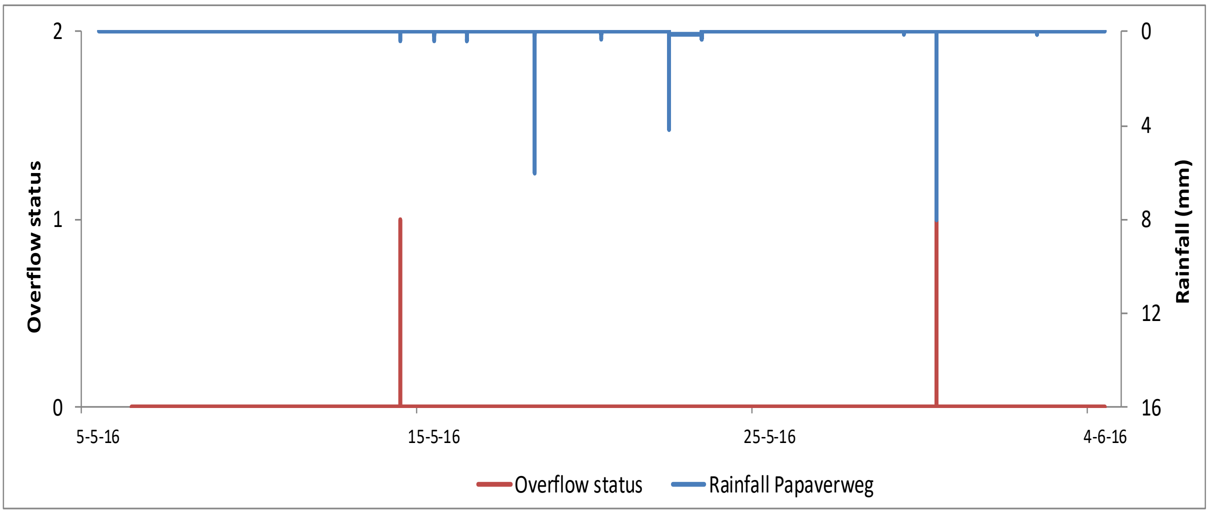


Figure E-3 The figure shows a "measured" overflow event even though there is no rainfall event, followed by 2 high intensity rainfall events that do not produce any overflow event.

Annex F: Images from inner gardens

The following pictures (Figure F-1 a, b, c) were taken during field work in the inner gardens of the island, on 28.11.2017, at midday and afternoon. On the 27th there were 8.3 mm of rain registered at Papaverweg rain gauge.

During the fieldtrip it was observed that the gardens were partially paved and there were no drains from the paved areas towards the sewerage system. The unpaved area was highly vegetated and unkept, very moist. The tall buildings create a dark environment.

Figure F-1 Images a), b), c) showing the highly vegetated and sheltered inner gardens on the island.

Annex G: Pump specifications

Manufacturer specifications for the pump

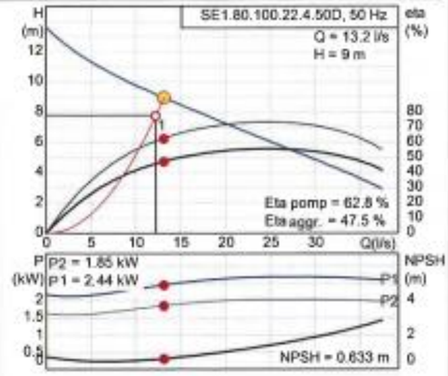
GRUNDFOS

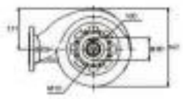
Firma naam: -
Gemaakt door: -
Telefoon: -
Fax: -
Datum: -

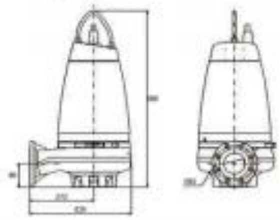
Project: prinseneiland 2
Referentienummer: -

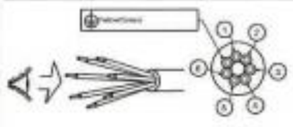
Klant: -
Klantnummer: -
Contact: -

Omschrijving	Specificatie
Productnaam:	SE1.80.100.22.4.50D
Product Nr:	96048021
EAN nummer:	5700395082731
Technisch:	
Max flow:	37.5 l/s
Resultaat in opvoerhoogte van de pomp:	9 m
Max. opvoerhoogte:	13.7 m
Type van de waaier:	Enkel kanaals
Maximum doorlaat vaste delen:	80 mm
primaire asafdichting:	SIC/SIC
Secondaire asafdichting:	CARBON/CERAMICS
Max hydraulisch rendement:	67 %
Keurmerken op typeplaat:	EN12050-1
Materialen:	
Pomphuis:	EN-GJL-200
Waaier:	EN-GJL-200
Installatie:	
Maximum omgevingstemperatuur:	40 °C
Maximale bedrijfsdruk:	6 bar
Flens norm:	DIN
Pomp persaansluiting:	100
Druktrap:	PN 10
Maximum montagediepte:	20 m
Inst. droog/nat:	DRY/SUBMERGED
Installatie	horizontaal of verticaal
Vloeistof:	
Te verpompen medium:	elke visceuze vloeistof
Bereik vloeistoftemperatuur	0 .. 40 °C
Dichtheid:	998.2 kg/m³
Elektrische gegevens:	
Pooltal:	4
Opgenomen vermogen - P1:	2.9 kW
Nominaal vermogen - P2:	2.2 kW
Netfrequentie:	50 Hz
Nominale spanning:	3 x 380-415 V
Spannings tolerantie	+6/-10 %
Aanloopmethode:	direct-on-line
Max starts per uur:	20
Nominaalstroom:	6,0-6,0 A
Stroom bij 3/4 last:	4.8 A
Stroom bij 1/2 last:	4.2 A
Startstroom	32 A
Stroom bij nullast:	3.6 A
Cos phi - vermogensfactor	0,74
Cos phi - p.f. zonder belasting	0,13
Cos phi - p.f. bij 3/4 belasting	0,66
Cos phi - p.f. bij halve belasting	0,53
Nominaal toerental:	1445 omw/min
Startkoppel:	32 Nm
Kipkoppel:	45 Nm
Massa traagheidsmoment:	0.057 kg m²
Motor rendement bij vollast:	76,3 %
Motor rendement bij 3/4 last:	75,2 %
Motor rendement bij halflast:	70,9 %









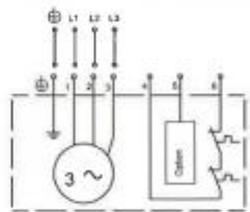


Figure G-1Pump specifications from the manufacturer.

Programmed specifications for the pump

Table G-1 Head discharge table as programmed in InfoWorks.

	Head (m)	**Discharge (m^3/s)**	**Power (kW)**
1	8.2134	0	2.10
2	6.8749	0.0039	2.35
3	5.9623	0.0078	2.44
4	5.1714	0.0117	2.65
5	4.3196	0.0156	2.60
6	3.6504	0.0195	2.65

Table G-2 RTC Script for the Prinseneiland Pump.

Item	**Type**	**Description**
F48001C1.1	VFDPMP	initial = 809.0000rpm , threshold = 10.000rpm, +ve = 1.000rpm/s, -ve = 1.000rpm/s
morethan20	Range	height above datum @ F48001c1 [-0.200m AD, +Inf)
-0.55to.20	Range	height above datum @ F48001c1 [-0.500m AD, -0.200m AD)
swithoff	Range	height above datum @ F48001c1 [-Inf, -0.550m AD)
swithon	Range	height above datum @ F48001c1 [-0.350m AD, +Inf)
swithoff	Rule	Switch off
swithon	Rule	Switch on
-0.55to.20	Rule	Set to 809.000rpm
morethan20	Rule	Set to 1445.000rpm

The script presented in Table G-2 appears in InfoWorks explained as it follows:

“*Unless subsequent rules are true, if the height above datum at node F48001c1 is less than -0.550m AD then F48001C1.1 will be switched off.*

Unless subsequent rules are true, if the height above datum at node F48001c1 is greater than or equal to -0.350m AD then F48001C1.1 will be switched on.

Unless subsequent rules are true, if the height above datum at node F48001c1 is between -0.500m AD and -0.200m AD then F48001C1.1 will be set to 809.000rpm.

If the height above datum at node F48001c1 is greater than or equal to -0.200m AD then F48001C1.1 will be set to 1445.000rpm.”

Annex H: Optimization parameters for the groundwater model

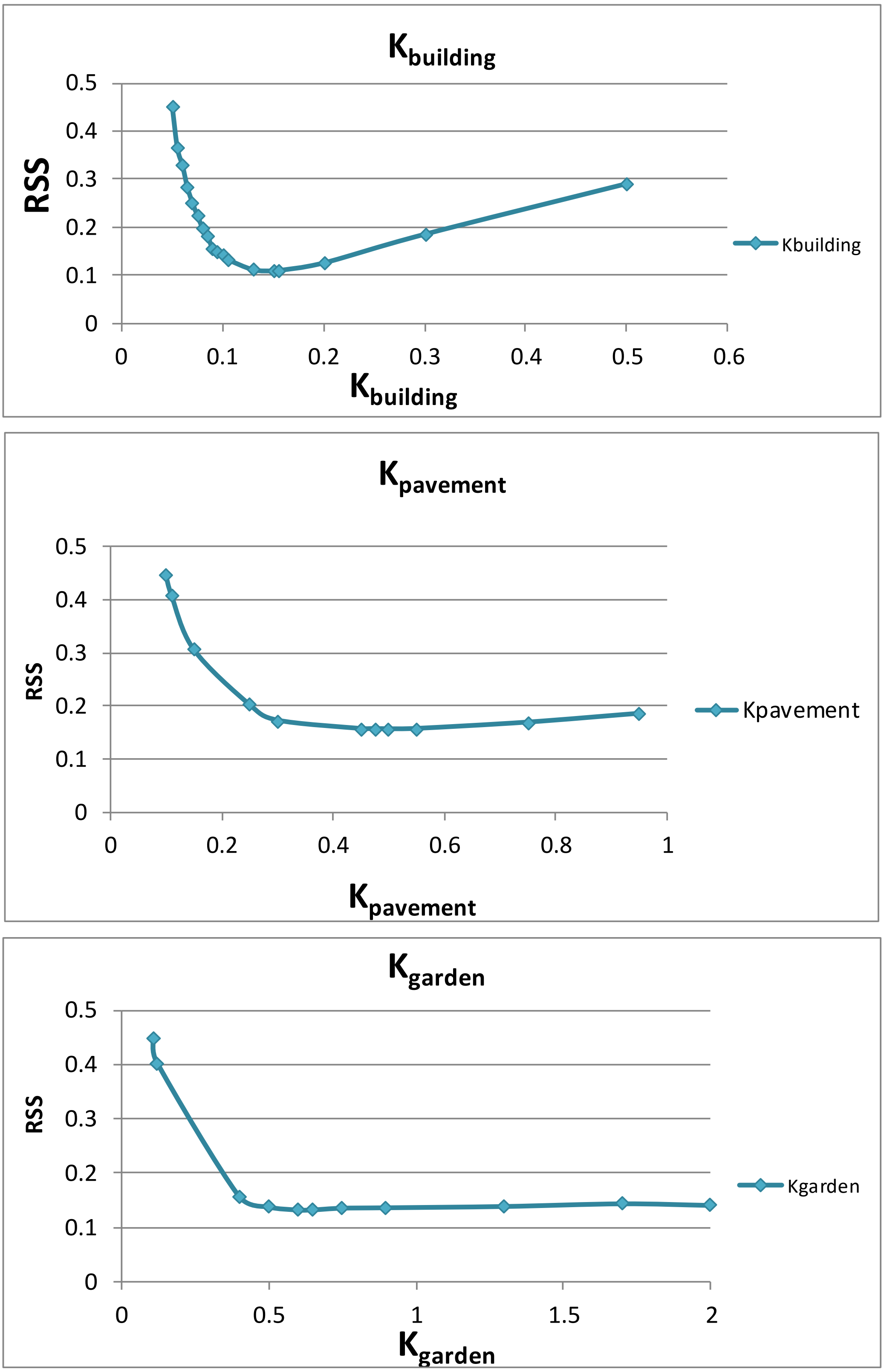


Figure H-1 Manual optimization process for the groundwater model, for the most important parameters.

Table H-1 Autocorrelation Matrix from the automatic optimization process of the stationary model (values in percentages). T1 for the quay area was taken out of the optimization process as the it had a correlation with T1 garden higher than 95%.

Parameter Label						
T1 Building	100					
T1 Garden 1	-84	100				
T1 Garden 2	75	-88	100			
C2 All Nodes	33	-7	-24	100		
T1 Garden	-5	-2	-11	6	100	
T1 Pavement	16	-4	-7	74	36	100

Annex I: Estimation of groundwater recharge through pavement on Prinseneiland

Rainfall contributes to the sewerage system by creating a water layer on an impervious/partially impervious surface, which will then be redirected to the sewerage system. However, not every rainfall event will produce runoff. Runoff will be produced only when the depth of the rainfall will exceed the incipient runoff depth, also known as minimum rainfall depth to compensate vegetation interception and depression losses (Guo, 2006).

The depth of a runoff-producing event is calculated as the difference between the measured rainfall depth and the incipient depth:

$$p_i = P_i - I_s \quad \text{(I-1)}$$

where p_i is the runoff-producing rainfall depth [mm], P_i is the initial rainfall depth [mm] and I_s is the incipient runoff depth [mm].

Further on, a runoff-producing rainfall depth is converted into its runoff depth by:

$$PR = C(P_i - I_s) \quad \text{(I-2)}$$

where PR is the runoff volume in inch or mm per watershed and C is its runoff coefficient (Guo, 2006, p. 391).

This formula is applied for the paved and built area, but with an alteration. It is known, as a boundary condition from the TSA model that 70% of total rainfall on active area becomes runoff. It was also assumed that there is no infiltration through the built area, which yielded a runoff coefficient of 0.782. The runoff coefficient for the paved area is unknown. The total resulted runoff and loss is calculated proportionately to the surface area of each surface cover, resulting in a total amount of loss and a total amount of runoff, within the boundary conditions described above.

Initial loss

Initial losses can be classified, according to Smart and Herbertson (1992, p.121), into wetting losses and depression storages.

The values for incipient losses were retrieved from literature. Guo, 2006, p.390 recommends a Is value of 1 inch (2.54 mm). Falk and Niemczynowicz (1979) argue that the initial loss has a value between 0.1 – 1.1 mm, while Kidd (1978) gives a range between 0.1 – 1.5 mm (Smart, Herbertson, 1992, p.121). Koot, (1977) recommends values of 1.5 mm/event in normal situations and 2.5 mm/event in extreme situations. This parameter varies according to weather conditions and according to the initial state of the pavement. For the current study, higher initial losses were chosen, since the runoff percentage is a fixed one, and infiltration through the old pavement on the island is thought to be very low (Table I-1).

Table I-1 Values for interception losses, as suggested by Jacqueline Flink.

Value (mm)	Surface type
1.5	Built
2.0	Paved

Infiltration

The quantity which is not lost through wetting loss and depression storage is lost through infiltration through the pavement. Its value is reasonably significant on surfaces such as new bricks and tiles, and less significant on old bricks, asphalt and concrete, as the interconnections become clogged over time. The infiltrated amount also depends on the type of brick and the infiltration capacity of the underlying layer. Since there is no precise value for this loss, it is estimated that 15-30% of total rainfall will infiltrate into a brick paved area (Smart, Herbertson, 1992, p.121).

Runoff coefficient

This is a very important parameter in calculating the total runoff of a catchment. It varies form one type of surface to the other, and it could be a source of error. It is a dimensionless coefficient with values from 0 to 1, that indicates what is the proportion of rainfall that turns into runoff. A value of 0 indicates that none of rainfall will turn into runoff, while 1 means that all the fallen rainfall will become runoff. It depends on the permeability of the surface, the slope, the vegetation cover, rainfall intensity and so on (Thomas, 2017, p.1-4).

For this analysis, the runoff coefficient was determined by diving the total amount of rain on the built area by the resulted runoff on the same area, with these conditions: no infiltration assumed, and a loss of 1.5 mm per shower. As mentioned above, the resulted runoff coefficient was of 0.782 for the built area.

Rainfall analysis

To determine the amount of rain that reaches the sewage pump, the rainfall data was divided into separate showers by using an inter-event time. The inter-event time is defined as the minimum time (usually hours) without rain in between showers (Guo, 2006, p.389). This interval depends on the catchment characteristics.

In the present study, the inter-event time is 6 hours, as suggested by Guo (2006). This means that in the seventh hour since the last rain drop occurred, if it rains again, that will be new event (Figure I-1). However, if rain will start again within those 6 hours, it will be considered the same event (Guo, 2006, p. 389).

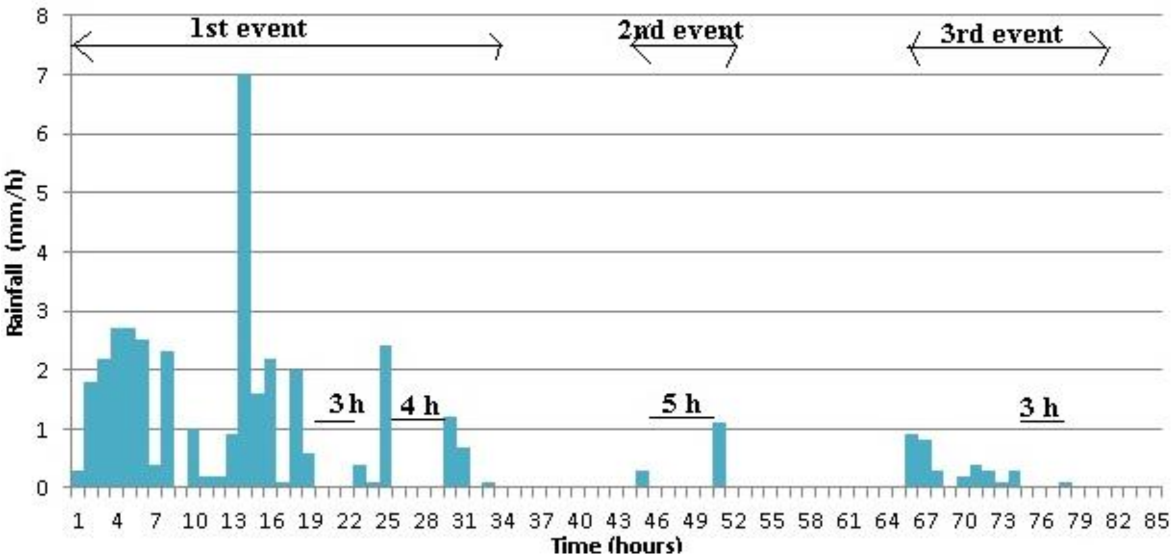


Figure I-1 Event separation of the time series, with an inter-event time of 6 hours (adapted to Papaverweg data from Guo, 2006, p. 389).

Based on these assumptions, the depth of a shower and its frequency can be assessed. The graph below (Figure I-2) shows that most rainfall events have a total depth of up to 5 mm/event, and a wide majority are less than 15 mm/event. Rain events of 30 mm or more were quite rare in the study period, with less than 10 events in total. Within the study period, most of the events happened within less than a day from one another, and in general most of the events happen within 4 days since the last one.

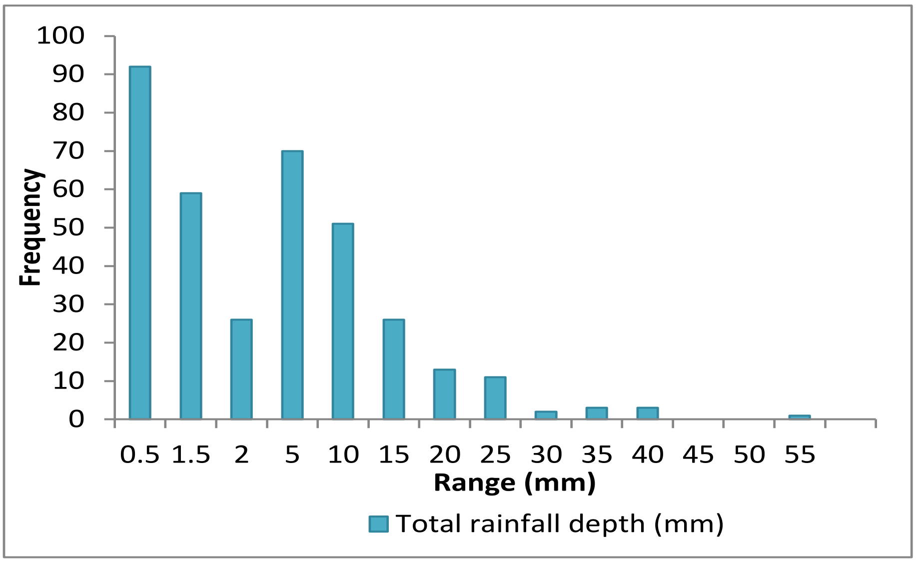


Figure I-2 Histogram representing the frequency of rainfall event's totals.

As shown, many showers are small enough not to produce any runoff. Out of 357 rainfall events within 2 years, based on the definition above, 151 showers are smaller or equal to 1.5 mm, and 26 showers are larger than 1.5 mm but smaller or equal to 2 mm.

Sensitivity analysis

Separating individual events is important in rainfall analysis and especially in urban water modelling (Freni et al., 2010). Therefore, choosing the right interevent time can heavily influence the results.

Decreasing the interevent time results in an increased number of rainfall events, while decreasing their total amount. The initial losses will be applied more often in the analysis, resulting in less infiltration into the ground. The runoff stays the same as it is a boundary condition.

The same analysis was run for an interevent time of 2 hours, while keeping the other parameters the same. Firstly, the runoff coefficient of the built area decreased from 0.78 to 0.74. The infiltration into the paved areas decreased from 1153.2 m^3/year to 294.2 m^3/year, whereas the evaporation from active streets increased from 27.2% of total rainfall to 32.6%. This is because initial losses are being deducted more often from the total amount.

Results

The results of this method are represented in Figure I-3.

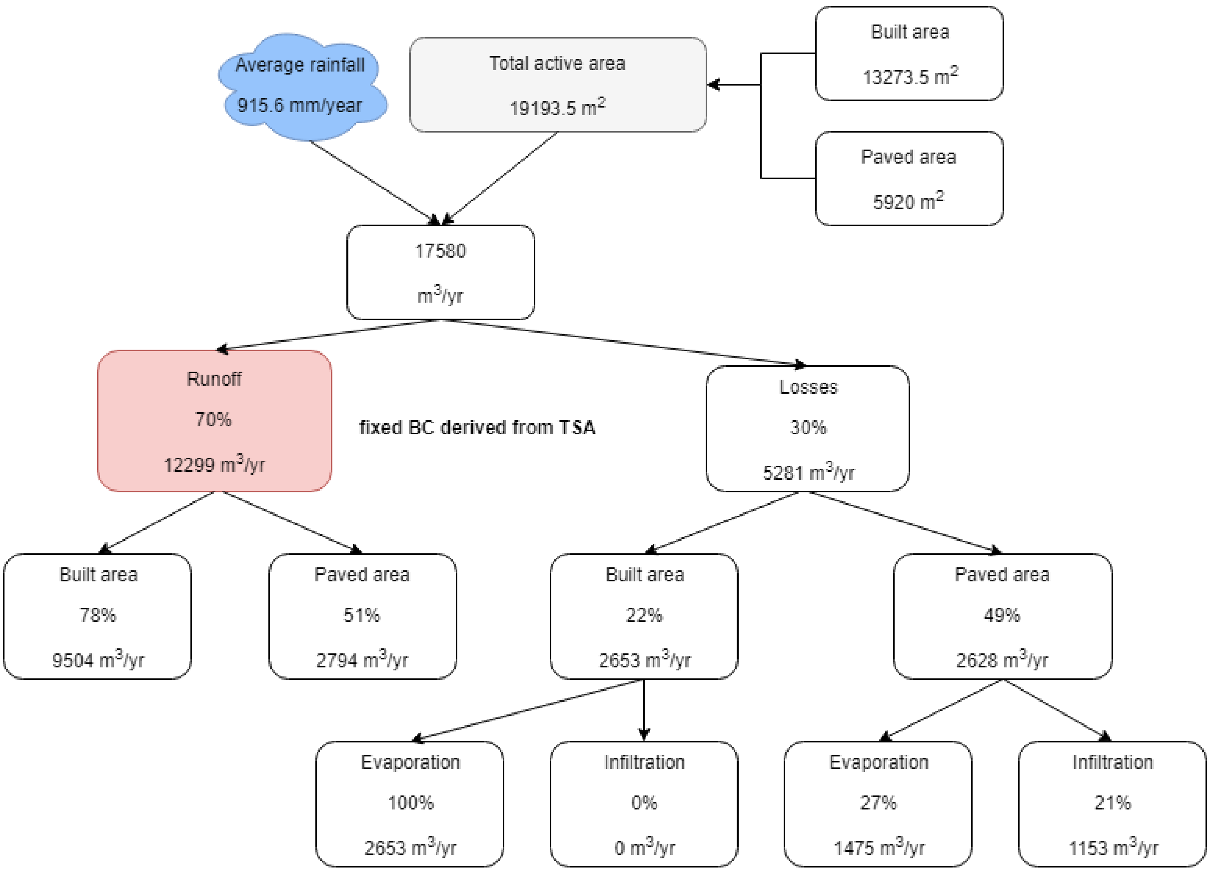


Figure I-3 Amounts of different fluxes on the active area of the island, based on the method mentioned above.

The fluxes are calculated only for the active area. Out of the total amount of rain on the active area, 30% is loss. On the built area there is no infiltration, therefore all the intercepted water by the building will evaporate, which is 21.8% of the total rain on built area. The remaining amount becomes runoff. In the paved area there is more loss due to infiltration. The amount of water that is approximated to infiltrate into the streets is 21.3% of the total amount of rain, which is 0.195 m/day. Only 51.4% of rainfall on the paved area becomes runoff.

These results are however, approximated. Many of the values used in this formula are dependent on other part of the research or are values about which not much is known in general, for example the initial loss.