Ref. Ares(2019)5013996 - 31/07/2019

AquaNES

Demonstrating Synergies in Combined Natural and Engineered Processes for Water Treatment Systems

Deliverable D2.2

High flow pre-treatment and infiltration system for aquifer storage and recovery with storm water runoff

COLOPHON

Project

Title:	AquaNES Demonstrating synergies in combined natural and engineered processes for watertreatment systems
Call identifier:	H2020-WATER-2015-two-stage;
Topic:	WATER-1B-2015 Demonstration/pilot activities
Funding scheme:	Innovation Action (IA)
Start date:	01.06.2016
Duration:	36 months

Document information

Deliverable no. :	D2.2
Work package:	WP2: Managed aquifer recharge & soil aquifer treatment for water storage and quality improvement
Title:	High flow pre-treatment and infiltration system for aquifer storage and recovery with storm water runoff
Lead Beneficiary:	KWR
Authors:	Luuk de Waal, Steven Ros, Dr. Koen Zuurbier , Dr. Marcel Paalman, Dr. Martin van der Schans (all KWR) Davey Smet, Toine van Aard (all HydroBusiness)
Contact for queries	Marcel Paalman KWR Watercycle Research Institute Groningenhaven 7, 3433PE Nieuwegein, The Netherlands E marcel.paalman@kwrwater.nl
Dissemination level:	Public
Due date	30.11.2018 (M30)
Final version:	25.05.2019 (M36)

Disclaimer:

This publication reflects only the authors' views and the Executive Agency for Small and Medium-sized Enterprises (EASME) is not liable for any use that may be made of the information contained therein.

The AquaNES project has received funding from the European Union's Horizon 2020 research and innovation programme under grant agreement no. 689450

Table of contents

List of figures

List of tables

List of abbreviations

AOP	Advanced oxidation process
ASR	Aquifer storage and recovery
ATP	Adenosine Tri-Phosphate
BAC	Biologically activated carbon
BAU	Business as usual: reactive operational reservoir management
BWRO	Brackish water reverse osmosis
CFU	Colony forming unit
CIP	Cleaning in place
cNES	Combined natural and engineered treatment system
COD	Chemical oxygen demand
CW	Constructed wetland
DOC	Dissolved organic carbon
E.C.	Electrical conductivity
EPA	Environmental Protection Agency
FC	Forecasting based: proactive operational reservoir management using now-casts
LOQ	Limit of quantification
MAR	Managed aquifer recharge
MF	Microfiltration
MFI	Modified fouling index
MS	Milestone
NTU	Nephelometric turbidity unit
OM	Organic matter
OMP	Organic micro-pollutants
OWCD	Orange County Water District
RO	Reverse Osmosis
RSF	Rapid sand filter / rapid sand filtration
SAT	Soil Aquifer Treatment
SDI	Silt density index
SF	Sand filter
SSF	Slow sand filter / slow sand filtration
TOC	Total organic carbon
TRL	Technology readiness levels
TSS	Total suspended solids
UF	Ultrafiltration

Executive Summary

Purpose of the study

Retention capacity is needed to manage storm water events and to provide water for the growing world population. The subsurface can act as a significant reservoir to temporarily store water surpluses and to keep the water available for later use through the principle of managed aquifer recharge (MAR). When local soil composition allows it, water storage in aquifers using infiltration wells - called aquifer storage and recovery (ASR) – can be a spatially efficient form of retention. The design of the capacity and dimensions of the pre-treatment required for such systems depend on the availability and characteristics of the infiltration water. A commonly used technique for pre-treatment of rainwater runoff preceding infiltration wells is slow sand filtration (SSF), which is robust to remove suspended solids and decrease available nutrients for biological growth from the infiltration water. The main draw-back however is that due to the irregular nature and intensity of rainwater events, large capacity sand-filters need to be placed with a large spatial footprint. In many circumstances this surface area is not available. This necessitates the development of a high flow rate pre-treatment and infiltration system with a limited spatial footprint.

Complementary to a high flow (in)filtration system, maximized use of any available aboveground volume for storm water retention can help significantly to decrease the required capacity of the pre-treatment and infiltration step. Seasonal mismatch between the dynamic water demand and supply, and a combination of both aboveground and belowground retention of freshwater using ASR, requires optimization of the operational reservoir management. The example of the Glasparel Waddinxveen site (high-tech horticulture relying fully on stored rainwater) was used for this optimisation. Usually, a reactive management approach is applied in such examples to control the aboveground reservoir/retention level using fixed target levels based on the reservoir storage capacity available to retain storm water and the amount of water available to supply to crops. With heavy rain events this leads to unwanted overflows to the surface system and to pluvial flooding. An alternative proactive reservoir management approach is here considered based on weather forecasts to steer the basin level based on predicted heavy rain events or dry spells. Together with a maximum pre-treatment and infiltration capacity, this approach may contribute significantly to limiting or preventing overflow events from occurring. With the help of a water balance based tool, the impact of a change in the reservoir management type from reactive to proactive is studied. This should lead to an optimization of the infiltration and recovery regime for ASR.

Approach

Based on a desk study on high capacity filtration systems which are currently available on the market, an innovative pre-treatment system was successively tested in lab, pilot and full-scale environments. The system was based on fine disc filtration and was self-cleaning using an air-water mixture. The tests provided insights into the suitability of the high-flow filtration system for pre-treatment during aquifer storage and recovery (ASR) applications.

Additionally, a program was designed to study and control the complex interplay between rain-fed aboveground reservoirs and coupled ASR-systems, based on the setting of Glasparel Waddinxveen. With this program, the design of both the reservoir and the ASR-system can be optimized, while it can also facilitate use of weather forecasts to better operate both elements.

Major findings

Tests of this filter in the KWR laboratory (STAGE I) showed that stable operation on relatively clean pond water can be achieved using the smallest mesh size (5 µm) of the Galileo L filter currently available. A model infiltration well which was fed with Galileo L filter 5µm treating KWR pond water showed a smaller infiltration capacity decrease compared to the model infiltration well which was fed with untreated KWR pond water, but still showed rapid clogging. The reduction of the most relevant clogging indicator (MFI) was measurable, but insufficient for optimal infiltration using wells.

During the pilot test at 'de Proefhal' in Kamerik (STAGE II), the Galileo L filter showed to be able to achieve stable operation for around 900 hours on surface water rich in organics (total organic carbon = 32 mg/L), which is known to have a high biological clogging potential. This test gave a strong indication that the Galileo L filter had a positive influence on the particle fouling potential of the RO membrane feed water.

In the field ASR test facility in Ovezande, particle counting measurements were performed and showed that a high particle load on the Galileo L filter results in a decrease of roughly 50-60% of all particles present, irrespective of whether their size is above or below the applied filter mesh size (5 µm). This observation can be explained by so-called cake layer build-up. Particles larger than the filter mesh size form a layer on the filter mesh. The thickness of this layer grows with time. As the layer thickens, more and more fine-grained particles are retained and fill pores between the larger particles. In this way, a dense layer develops around the filter, which can retain a fraction of particles smaller than the original filter mesh size. As the pressure drop over the filter increases during the development of this cake layer, regular removal of the cake layer is required. After each flush, particles smaller than the filter mesh size will be able to pass through, until the cake layer is (again) dense enough to (partially) retain them. The results indicate that still too many particles were transported towards the infiltration well. Together with the high growth potential in the feed water, this resulted in the observation of well clogging as evidenced by a 50% decrease in well capacity after 4 months, which is a poor performance compared to the slow sand filtration.

In 2019, the same set-up was operated again for 3 month, treating and injecting 4 000 m^3 of surface water. This time, 1 micron cartridge filters were used for extra treatment after the Galileo L. Later UV-disinfection and chlorination were stepwise added. The results show that finer filtration (until the cartridge filters showed a breakthrough after 2-4 days of operation) can be successful in preventing clogging. However, also chlorination led to prevention of well clogging. UV point disinfection showed no improvement in the prevention of well clogging.

Overall, the Galileo L is a very effective polishing step for storm water treatment and can keep the worst particle loads out of the infiltration wells. However, the removal of particles is too limited to completely prevent well clogging. This was underlined by the decrease in specific capacity observed at the ASR well in Ovezande.

In parallel to the high flow (in)filtration system tests, a water balance study was carried out for the Glasparel+ case in Waddinxveen. The model was used to compare a business as usual (BAU) reactive reservoir management approach with a forecast based proactive reservoir management approach. The aim of the proactive approach is to obtain significantly more reduction in overflow, making optimal use of the benefits of a high flow-rate pre-treatment train for infiltrated water.

It was found that both the overflow and shortages are expected to decrease by applying a proactive management approach, even with constant over- or underestimation of real precipitation data. However, a certain minimum reservoir size is required (here: >30 to 40 mm of rainfall) to benefit from a proactive approach. The proactive reservoir management approach did not result in a decrease in the use of ASR. More water was stored belowground, which partly had to be recovered in a later stage. Regarding the necessity of well regeneration, it is advised to regenerate well systems at a relatively early stage to prevent water shortages (here: when capacity is below 80% of the initial capacity) if reactive reservoir management is applied: infiltration capacity is critical. With proactive management the risk is lower, but it is still advised to maintain the pumping well rate higher than the peak demand. The storm water harvesting model can now be applied in early stage decision making to estimate required reservoir size in combination with ASR pumping capacity.

1 About this document

The aim of this document is to report about the design, implementation and optimization of a full scale demonstration system composed of a high velocity infiltration filter.

This report consists of three parts which are linked to the optimization of large-volume infiltration of runoff water for aquifer storage and recovery (ASR) system.

In Chapter 2, the results are presented of a program coupling above ground reservoir and ASR systems allowing to link the infiltration strategy to forecasted weather conditions.

Chapter 3 consists of a desk study performed on different fast pre-treatment concepts for storm water ASR infiltration , as currently available on the market.

In Chapter 4 and 5, the performance of a compact, self-cleaning rapid filtration system is described.

The results described in these chapters have not been published in a paper or scientific magazine.

2 Proactive operational reservoir management using belowground storage and recovery of freshwater

2.1 Introduction

In horticultural areas, water demands are high while freshwater availability fluctuates depending on both quality and quantity of river discharge and on precipitation. Because of strict water quality requirements for various crop types, freshwater availability often solely depends on storm water. Fresh storm water is collected via greenhouse roofs and stored in aboveground reservoirs, to be used in times of freshwater demand. Often, large reservoirs up to several thousand m^3/ha are implemented to collect and store sufficient water. However, these are expensive and require a lot of space aboveground, which could otherwise be used for growing crops or for other purposes. Moreover, the quality of water stored aboveground may deteriorate as a result of changing temperatures, and annual differences in precipitation may result in water shortages in various years. To prevent shortages, brackish water reverse osmosis (BWRO) (Stuyfzand & Raat, 2010) is used as cost-effective desalination technique to increase freshwater availability and to minimize the need for large aboveground reservoirs. Yet, it results in overuse of groundwater in coastal areas such as The Netherlands and subsequently in salinization. On the other hand, while there are periodic water shortages, there are also significant amounts of overflow events which result in losses of valuable freshwater for the horticulturists and may cause flooding problems in the region.

Both water shortages and overflow can be limited or prevented by water storage belowground using wells in aquifers during times of water surplus, to be recovered in times of high demand. This principle of aquifer storage and recovery (ASR) - a type of managed aquifer recharge (MAR) - is applied at various locations worldwide (Pyne, 2005).

The amount of water that is collected and stored using ASR depends amongst others on the reservoir management. Often, the operational strategy is to infiltrate and recover water based on the reservoir filling level at the time (target recovery level somewhere between 40 and 80%). This principle of reactive storage and recovery of freshwater works well if the inflow into the reservoir by precipitation is slow enough to prevent short-term overflow. However, this tactic may still lead to unwanted overflow in case of more extreme weather events or in needless infiltration when dry spells follow precipitation events.

In this study, a proactive approach of (remote control) reservoir management with ASR systems is presented and compared with the reactive approach. The principle of this type of management relies on now-casts of water demand and precipitation predictions. The aim is to make decisions on whether to infiltrate or recover freshwater (or do nothing) based on predicted reservoir levels for a predefined number of days ahead (in this study: 5 days ahead). A modelling approach has been followed, for which a water balance tool was constructed. The Waddinxveen location Glasparel+ was used to study the water balance impacts.

A main task has been the selection of appropriate proxy parameters for well clogging including trigger values for intervention. For this, the actual infiltration capacity of the ASR well at a fixed infiltration pressure (a proxy for the specific discharge) was selected. The same proxies have been chosen in earlier chapters as a means to discuss the rate of well clogging. Trigger values for intervention are based on the (relative) well capacity at which the owner(s) should intervene to prevent significant overflows or water shortages. This trigger point for intervention - defined as percentage of initial specific well capacity - is case dependent and cannot be directly extrapolated from the Waddinxveen case to other locations.

2.1.1 Drivers and pressures for proactive operational reservoir management

Drivers for optimization of operational reservoir management:

- Availability of high-quality water for production at low costs (economy): Make more efficient use of available freshwater from precipitation and rivers (‘water in the circular economy’).
- Climate change: longer periods of drought and also more periods with extreme rainfall resulting in (pluvial) flooding. Water supply with the lowest carbon-footprint is required (limit pumping and reduce need for desalination).

Pressures for optimization of operational reservoir management:

- Pluvial flooding is a major concern in urban areas and low-lying (polder) regions in general.
- Groundwater resources become depleted by overexploitation; in coastal regions this results in saltwater intrusion.
- Use of reverse osmosis on itself is not sustainable and the disposal of membrane concentrate without compensation is not allowed by the Prevent and Limit principle of the EU Water Framework Directive.

2.1.2 Potential benefits of proactive operational management

- Prevent or limit overflow events: start early with infiltration given the fact that infiltration is often slower than precipitation. That is, optimize the use of available well capacity.
- Related: improve groundwater quality by infiltrating the maximum volume of freshwater surpluses
- Limit unnecessary usage of pumps (lower carbon-footprint).
- Opportunity to increase crop production per surface area: keep reservoirs relatively small while not necessarily missing out on freshwater.

2.1.3 Aims of the study

This modelling study has the following aims:

- Limit overflow
- Reduce water shortages in dry periods if well capacities become less than the water demand.
- Increase freshwater storage by collecting additional surpluses from storm water events.

  Reduce pumping costs:

- reduce energy use and potential well clogging because of unneeded dynamic infiltration and recovery between aboveground reservoir and aquifer
- Provide design principles for future projects: identify required reservoir size and well capacity. A next step would be to weigh capital costs against overflow prevention.
- Study when to ultimately regenerate a well system (trigger point, defined as percentage of initial specific well capacity).

2.1.4 Research questions

1. What is the difference in expected (total) overflow between the reactive and proactive management approach if real ASR capacities are used?
2. What is the effect of proactive reservoir management on the required freshwater recovery efficiency from the subsurface (ratio of volume out to volume in)?
3. What is the potential influence of changing the well capacity by -50 to +50% on the expected overflow and water shortage?
4. What is the trigger point for well remediation to prevent significant increases in overflow or water shortage? How sensitive are the reservoir management approaches to changes in well capacity?
5. When is a relative increase in ASR pumping capacity more effective than a relative increase in reservoir size as a measure to reduce overflow (events), and vice versa?
6. Which (type of) datasets are available for weather forecasting? How to deal with uncertainties in predictions of precipitation?

2.1.5 Approach

- For this study, earlier assumptions made regarding the surface areas, reservoir sizes, water demands and ASR capacities are maintained (K.G. Zuurbier & Ros, 2017).
- A water balance tool for the region was built using the programming language python and adjusted to allow for input of precipitation forecast data.
- Daily and hourly datasets from meteorological station 'De Bilt' (KNMI) were used (1 January 1987 – 31 December 2017): precipitation and reservoir evaporation.
- Water balance calculations as input for both the reactive and proactive reservoir management approach are on an hourly basis.
- The impact of well clogging on overflow and shortage has been studied for both the proactive and reactive approach by varying the ASR pumping well capacity in steps of 10 % between 50 and 150% of the original well capacities (22 scenarios in total).
- Subsequently, the number of scenario runs was intensified to a total of 840; by varying the relative well capacity and reservoir size for both the proactive and reactive approach from 0 to 200% and from 10 to 200%, respectively. All other working conditions were kept equal. The aim was to find out in what spectrum of well capacity and reservoir size the proactive case would be most favourable.
- Weather forecast data from (The Weather Underground) have been retrieved (29 August 2018 - 12 November 2018) including 10-day now-casts of precipitation. These were used to see if ('random') overestimation or underestimation of weather forecast brings additional risks to overflow, water shortages or unnecessary ASR use.
- Because of the limited amount of real now-casts, additional sensitivity analyses were applied on the 31-year KNMI dataset. This was done by including a constant over- or underestimation of up to 50% of the real rainfall in steps of 10 % (11 scenarios). If predictions would be wrong in reality, is it then better to overestimate the precipitation or to underestimate it?

2.2 Demonstration case description

2.2.1 Study area

The Waddinxveen location Glasparel+ is a new horticultural/business park in the Netherlands, located approximately 10-15 km Northeast of Rotterdam. In the period 2016-2020 an area of 186 ha will be developed. The location is situated in one of the typical deep Dutch polders with soil surface level at 5 m below seawater level. Approximately 89 ha is destined to greenhouses (horticulture), 40 ha to logistic activities (e.g. distribution centre of Lidl Supermarket) and to an agribusiness park.

The water ambition for this area is:

- To be self-sufficient in the use of fresh water.
- Avoid groundwater salinization.
- High quality of irrigation water for horticulture (high quality: 0.5 mmol Na/L)
- Limiting spatial and visual impact of water storage
- Prevent pluvial flooding

In the Waddinxveen case (see Figure 2-1 and Figure 2-2) the ambition is to limit use of brackish/saline groundwater and desalinization by BWRO, as infiltration of the residual membrane concentrate in the subsurface may lead to salinization of the groundwater. To assure enough freshwater for the horticultural companies, the storm water which falls on roofs of the agro-business park and the logistic centre will be captured also and will be infiltrated into the subsurface. Precipitation falling on the roofs of the greenhouses, business park and logistic centre is collected into a canal (capacity 16,000 m^3, covered with foil). To prevent well clogging and prevent pollution of groundwater the collected storm water is filtrated by sand filtration or an alternative filtration (this study) and is infiltrated by using at least seven ASR wells.

Late construction of the water system at the Glasparel+ site did not allow ASR use before the Autumn of 2018. Therefore, testing of the rapid filtration system was executed at other field-sites.

The surface areas for collection of storm water are presented in Figure 2-3. Storm water collected on greenhouse roofs is stored directly in the horticulturists' reservoirs and storm water falling on neighbouring businesses' roofs is led to and temporarily stored in rainwater canals, located central in the area, before it is infiltrated in the subsurface (aquifer). The canals are kept at a very low level to prevent microbiological growth in stagnant water. Surpluses of water in the reservoirs and from the canals are stored in the aquifers for later use. A schematic overview of the system is presented in Figure 2-4. The horticulturists present in the area are Royal Peppers, Porta Nova, and G2 (yet unsold terrain, of which the water demand per hectare is assumed equal to Royal Peppers'). Porta Nova cultivates Roses and Royal Peppers cultivates Paprika.

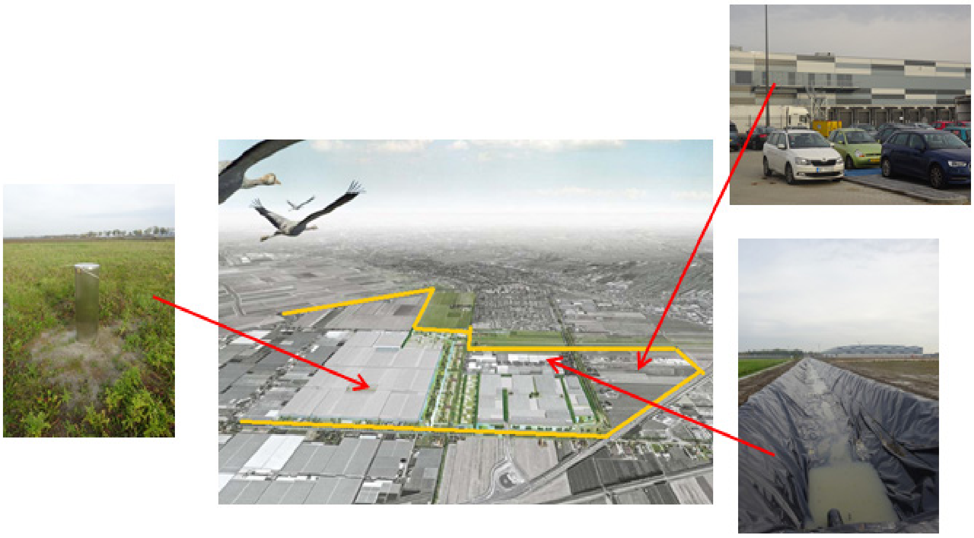


Figure 2-1: **Impression of Glasparel+ location (Waddinxveen). Photos taken on November 4 2016 (left: groundwater monitoring well; right top: distribution centre of Lidl; right below: rainwater canal for storage and transport of roof water from the Lidl distribution centre**

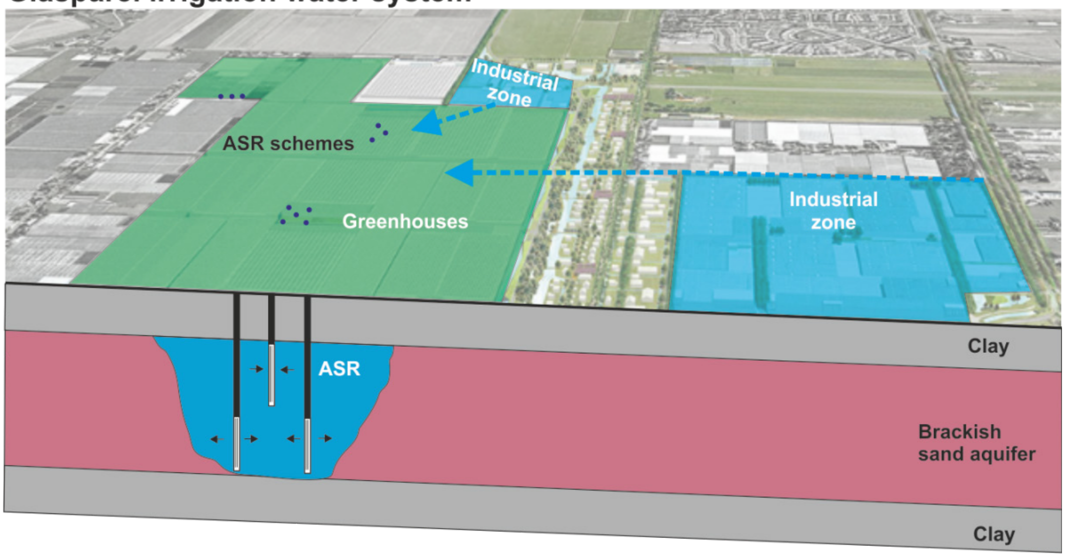


Figure 2-2: **Water system Glasparel+: an example of storm water harvesting, storage, and usage in modern greenhouse areas**

2.2.2 Model working conditions and assumptions

- Daily and hourly dataset from meteorological station 'De Bilt' (KNMI) to obtain input for precipitation, glass retention and reservoir evaporation.
- Model calculation uses hourly time steps from 1 January 1987 – 31 December 2017 (31 years) with 5 days extension for predictions of virtual reservoir levels 5 days ahead (proactive approach).
- Climate data from weather station 'De Bilt' is directly applicable for Waddinxveen.
- Predictions of meteorological data are assumed 100% correct (based on 31-year weather data at meteorological station 'De Bilt': January 1987 – December 2017). Neglecting this uncertainty in the data has an advantageous effect on the outcome of using 'predictions'

(proactive) instead of controlled reservoir levels (reactive) for starting infiltration or recovery of freshwater. This has been justified for this study assuming the occurrence of dry periods and extreme weather events can be reasonably well predicted 95% of times.

- Seasonal dependent reservoir target levels for infiltration and recovery (October – February) to limit overflows during the winter period.
- One millimetre of daily retention of rainfall falling on greenhouse roofs or business terrain roofs is evaporated from 1 May – 30 September, this water will not reach the reservoirs/canals.
- Initial owners' reservoir filling level is at 60% of their maximum; the rainwater canals are empty.
- Storm water collected in the rainwater canals is divided amongst the owners in relation to the ASR pumping capacity per owner.
- There is no quality limitation (modelled) on the amount that can be recovered by ASR, only a capacity limitation. It is assumed that desalination is used in addition to ASR if the quality is inadequate by slight salinization.
- Evaporation of water stored in rainwater canals (only) according to hourly (and daily) evaporation from KNMI dataset 'De Bilt'.
- No evaporation of rainfall from 1 October – 30 April.
- Evaporation in reservoirs of greenhouse owners has been neglected because the reservoirs are generally covered.

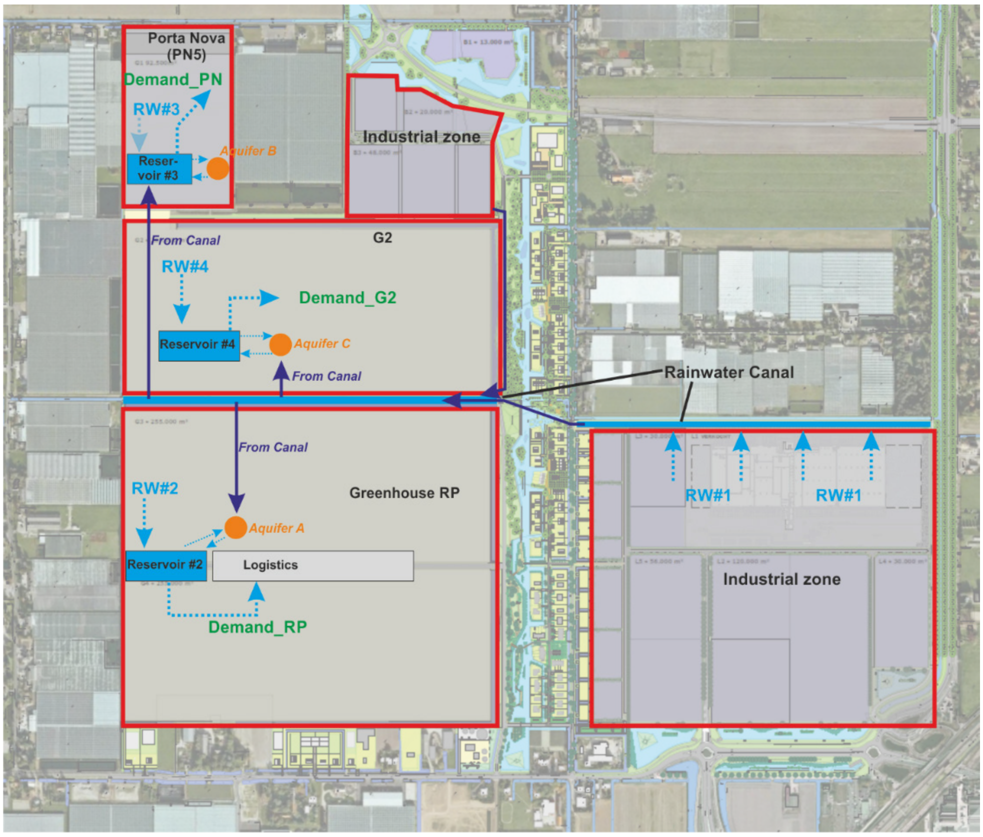


Figure 2-3: Top view of the Glasparel+ water system including water fluxes

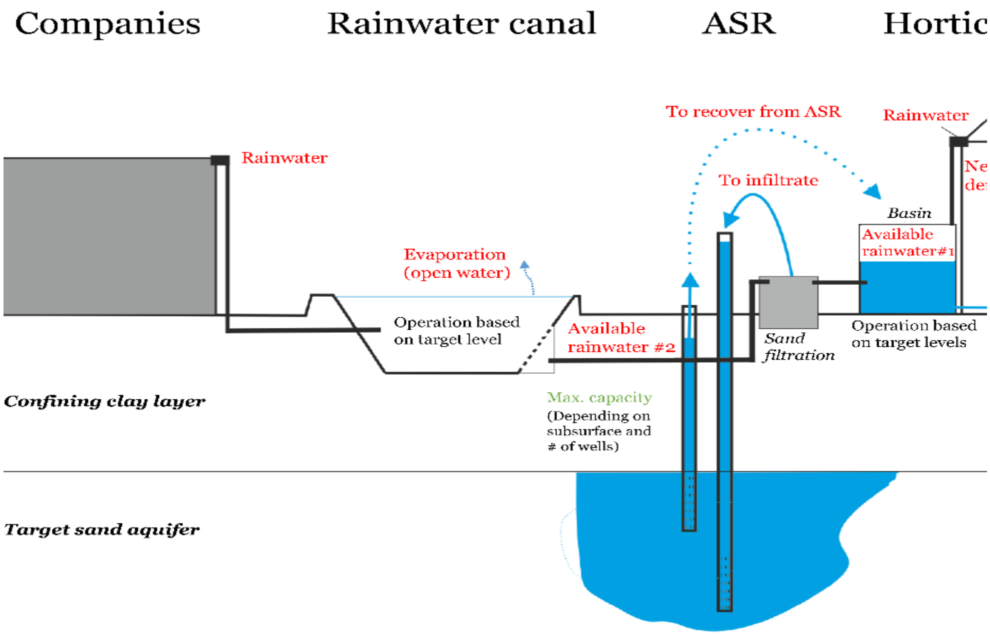


Figure 2-4: Schematic overview of the Glasparel+ water system components including water fluxes

2.2.3 Surface areas and reservoirs Glasparel+

The total surface areas connected to the Glasparel+ terrain are given in Table 2-1. Porta Nova develops his ground together with 'Sjaloom', located outside of the Glasparel+ terrain. The reservoir has been assigned to the study area proportionally.

The buffer capacity from the rainwater canals has been assigned to the parcels available at Glasparel+ assuming water rights are directly proportional to their parcel sizes (Table 2-2).

Table 2-1: Roof areas and buffer volume for storm water collection, and for calculation of water demand.

Surface areas		Unit	Buffer	Unit	Remarks
1. Business terrain					
Roof area	260,000	m^2			
Storm water canals	16,000	m^2	16,000	m^3	Distributable over end-users' ASR systems
Total	276,000	m^2			
2. Royal Peppers					
Office spaces	16,500	m^2			
Reservoir	8,750	m^2	26,250	m^3	With overflow to ASR
Greenhouse	450,000	m^2			Primary water demand area

Surface areas		Unit	Buffer	Unit	Remarks
Total	475,250	m²			
3. Porta Nova (excl. Sjaloom)					
Office spaces	0	m²			
Reservoir	1,875	m²	5,625	m³	With overflow to ASR
Greenhouse (Glasparel part)	75,000	m²			Primary water demand area
Total	76,875	m²			
4. G2					
Office spaces	30,000	m²			Assumption
Reservoir	5,500	m²	16,500	m³	Reservoir = average Royal Peppers / Porta Nova
Greenhouse	244,500	m²			Primary water demand area
Total	280,000	m²			
Total collection area	1,108,125	m²			
Total buffer volume			64,375	m³	

Table 2-2: Buffer capacity available to Royal Peppers (RP), Porta Nova (PN), and G2. The rainwater canals are connected to each owners' ASR system. Water is infiltrated before it is supplied to the owners

Buffer capacity	RP	PN	G2	Storm water canals	Unit
Reservoir size	583	750	675	192.3	m3/ha

2.2.4 Water demand

The water demand has been requested from all (potential) buyers. This concerns the total water demand to be provided by the storm water catchment system (Table 2-3). The given water demand for Porta Nova (PN) is relatively high for the crop Roses, which is because the owner uses artificial light to increase production. The demand for Paprika cultivation (Royal Peppers: RP) matches literature values (Voogt, 2008). The demand of G2 has been chosen equal to RP, with a net cultivation area of 90% the total greenhouse glass area.

Table 2-3: Water demand Royal Peppers (RP), Porta Nova (PN), and G2 (up for sale)

Period	Water demand RP	Water demand PN	Water demand G2	Unit

January	0.2	3.2	0.2	L/m2/day
February	0.6	3.2	0.6	L/m2/day
March	1.5	4.4	1.5	L/m2/day
April	3.3	4.4	3.3	L/m2/day
May	4.1	4.4	4.1	L/m2/day
June	4.8	4.4	4.8	L/m2/day
July	5.1	4.4	5.1	L/m2/day
August	4.1	4.4	4.1	L/m2/day
September	2.3	4.4	2.3	L/m2/day
October	0.9	3.2	0.9	L/m2/day
November	0.6	3.2	0.6	L/m2/day
December	0.3	3.2	0.3	L/m2/day
Total	8,449	14,204	8,449	m3/ha/year

2.2.5 ASR system capacities

For the infiltration and recovery capacity of the well systems, assumptions were based on observations from well drillings and on the number of ASR wells considered. The recovery capacity (Table 2-4) is higher than the presumed peak annual water demand.

Table 2-4: ASR capacities per greenhouse owner and number of ASR wells required

ASR system	No. of ASR wells	Infiltration capacity	Recovery capacity
	(-)	(m^3/h)	(m^3/h)
Royal Peppers	5	100	100
Porta Nova	2	40	30
G2	3	60	50
Total	10	200	180

2.2.6 ASR (reservoir) management approach

The amount of freshwater which is infiltrated by the greenhouse owners in the area is based on the ASR capacities and on decisions on reservoir fill target levels for when to infiltrate and recover freshwater. The operational target levels used in this study are the same for the reactive (current) and proactive (proposed) approach, see Table 2-5. With the reactive approach only the actual ('real') water level is considered, whereas with the proactive approach actions are based on predicted reservoir levels 120h (5 days) later, calculated (and reset) daily at 6:00h.

This predicted (virtual) reservoir level can in principle be higher than 100% or lower than 0%. The virtual level initially only considers increases and decreases in reservoir level based on the amount of precipitation and water use of the greenhouse owners, respectively. During each hourly time interval the level is then corrected for overflow events (lower level) or shortage events (higher level), (still) occurring during the 120h period. The virtual level is also corrected for any ASR operation taking place during the 120h forecasting period, see Table 2-6.

Table 2-5: Reservoir management action via ASR, if applicable

ASR action	Storm water canals	Owners' reservoirs (March – September)	Owners' reservoirs (October – February)
Start infiltration	10%	>80%	>50%
Stop infiltration	0%	<75%	<45%
Start recovery	N/A	<35%	<35%
Stop recovery	N/A	>40%	>40%

N/A: Not applicable

Table 2-6: Virtual reservoir fill correction in any calculation time step as a result of a ('real time') action.

Action	Virtual reservoir level
Overflow	Decrease
Water shortage	Increase
ASR infiltration	Decrease
ASR recovery	Increase

2.2.6.1 Example case: overflow events in a summer period

To give a practical (simplified) example of the difference in operational reservoir management a case is presented of a horticulturist with a summer (peak) demand of 4 mm/ha/day and ASR capacity of 5 mm/ha/day (Table 2-7). A 10-day forecast is available indicating two dry days followed by three days with an increasing amount of rainfall and a dry period thereafter (Table 2-8). At day one the owner's reservoir is at 80%. With the current reactive management approach the (automated) ASR system starts infiltrating water when the actual reservoir level exceeds the target level of 80%. With the pro-active management approach the ASR system starts infiltrating water when the predicted reservoir level exceeds the target level. Without any ASR operation the overflow event totals 20 mm.

Table 2-7: Boundary conditions of the example case per ha of agricultural area. A high reservoir fill (80%) with approaching rain events during a summer period

Parameter	Amount	Unit
Water demand	4	mm/ha/day
ASR capacity (IN = OUT)	5	mm/ha/day
Initial reservoir level	80	mm/ha
Maximum reservoir level	100	mm/ha

Table 2-8: Example case: high precipitation events during a summer period

Day	Precipitation (mm)
1	0
2	0
3	10
4	20
5	30
6 - 10	0

2.2.6.2 Reactive management approach: example case

With the reactive management approach (business as usual: "BAU") the actual reservoir level remains below 80% until after 72 hours (Figure 2-5). It takes until day 4 before the reservoir level exceeds 80%. It is assumed that the ASR can infiltrate water for approximately 24 hours before overflow occurs. With the reactive management approach the overflow event is still relatively high (15 mm). The level in the reservoir is lowered too late.

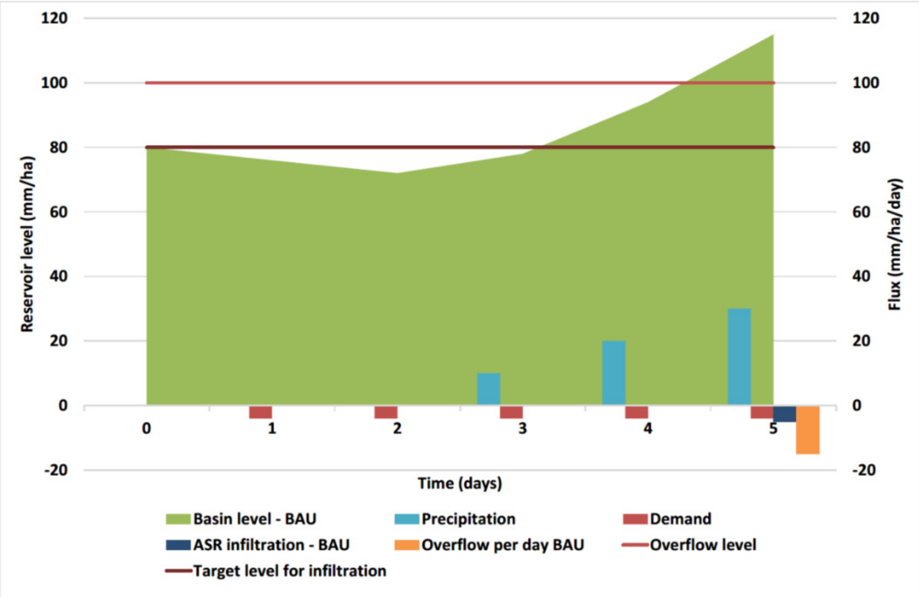


Figure 2-5: Reactive management approach: ASR infiltration only when reservoir level exceeds target level for infiltration. The overflow event is only slightly reduced. BAU = business as usual (standard, current way of operation)

2.2.6.3 Proactive management approach: example case

With the proactive management approach (forecast based: "FC") ASR infiltration already starts at day one (Figure 2-6), as by doing nothing the reservoir level would exceed the target level for infiltration after 5 days (that is, the "predicted reservoir level"). As a result, the actual reservoir level remains below 80% until after 96 hours. During the total rain event the actual reservoir level remains below its maximum. Thus, with the proactive management approach there is no overflow. The level was lowered sufficiently before the rainfall event occurred.

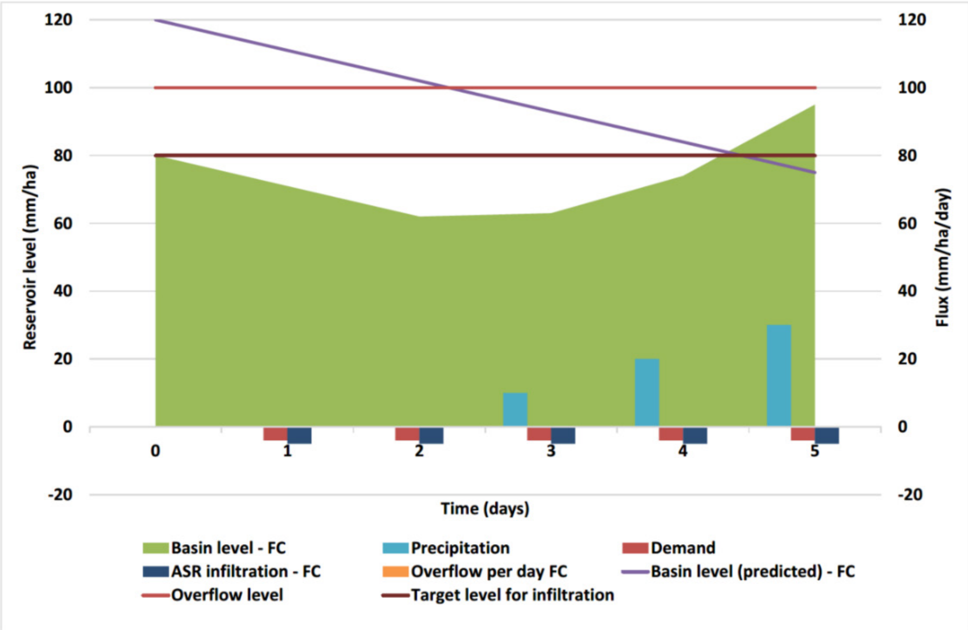


Figure 2-6: Proactive management approach: ASR infiltration when predicted reservoir level exceeds target level for infiltration. The actual reservoir level is lowered before the precipitation is actually occurring. FC = forecasting based (new way of operating based on predictions of rainfall and water use)

2.3 Results - water balance simulation

2.3.1 Potential of storm water harvesting system

For the Glasparel+ area, the average water fluxes have been calculated to get an understanding of the multi-year capacity of the system to collect sufficient fresh storm water for use in times of demand (Table 2-9). The fluxes from precipitation are equal for both types of operational reservoir management: reactive ('Business as usual': BAU) or proactive ('Forecasting based': FC). What stands out is that only 629 thousand m^3 of ca. 915 thousand m^3 collected storm water needs to be supplied to fulfil the water demand, i.e. 68.7%. The commercial terrains included in the water balance are with 24% of the total influxes a significant contributor to the overall water balance. Besides losses from horticultural demand there are some losses that cannot be prevented (assumptions of evaporation and retention: 3.2%) and losses by overflow which can be prevented or at least significantly limited. The overall overflow with forecasting (FC) is reduced by 17.8 thousand m^3 per year (a reduction of 58.5%). This volume is transferred to a net groundwater infiltration.

Table 2-9: Total water balance Glasparel+ in average annual fluxes in and out of the system (January 1987 – December 2017). The effluxes of the system are partly influenced by the type of reservoir management (BAU: reactive; FC: proactive).

Fluxes IN	Total in	Part	Fluxes OUT: BAU	Total out	Part	Fluxes OUT: FC	Total out	Part
	(x 1000 m3)	(%)		(x 1000 m3)	(%)		(x 1000 m3)	(%)
Precipitation in reservoirs	27.4	3.0	Water demand horticulturists	629.0	68.7	Water demand horticulturists	629.0	68.7
Precipitation glass & other roofs	666.7	72.8	Overflow owners' reservoirs	25.0	2.7	Overflow owners' reservoirs	4.3	0.5
Precipitation commercial terrains	221.2	24.2	Overflow storm water canals	5.4	0.6	Overflow storm water canals	8.3	0.9
			Evaporation and retention	29.6	3.2	Evaporation and retention	29.6	3.2
			Net infiltration groundwater	226.2	24.7	Net infiltration groundwater	244.1	26.7
Total in	915.3		Total out - BAU	915.3		Total out - FC	915.5	

2.3.2 Precipitation and demand

The mismatch in collective precipitation (with and without rainwater canals) and water demand at the owner's reservoirs is given in Table 2-10. The relative monthly contribution of each flux and the average net surplus are visualized in Figure 2-7.

- Average precipitation in owners' reservoirs is nearly equal to annual demand.
- Storm water collected and diverted to storm water canals is a prerequisite for sufficient supply, to overcome losses in the subsurface by seasonal belowground storage (estimated recovery efficiency is ~70%).

Table 2-10: Combined averaged monthly and annual net precipitation and demand (Royal Peppers, Porta Nova, G2), and the net precipitation collected in the (storm water) canals in cubic metres and millimetres.

Total model period	Precipitation collected in owner's reservoir (m^3)	Seasonal Precipitation collected in canals (m^3)	Precipitation collected (mm)	Water demand (m^3)	Water demand (mm)
January	57809	17587	69.5	10571	19.5
February	51919	15553	62.4	16701	25.3
March	49426	14737	59.4	38272	52.2
April	34965	10253	42.0	70790	86.6
May	41495	14730	49.9	88651	106.1
June	47635	17299	57.2	98917	116.7
July	65333	23349	78.5	108027	126.8
August	57365	20276	68.9	90697	108.3
September	55517	20047	66.7	52038	66.6
October	66463	20702	79.9	24135	34.0
November	66057	21040	79.4	17731	26.9
December	65955	20309	79.3	12496	21.5
Total	659939	215883	793.1	629028	790.5

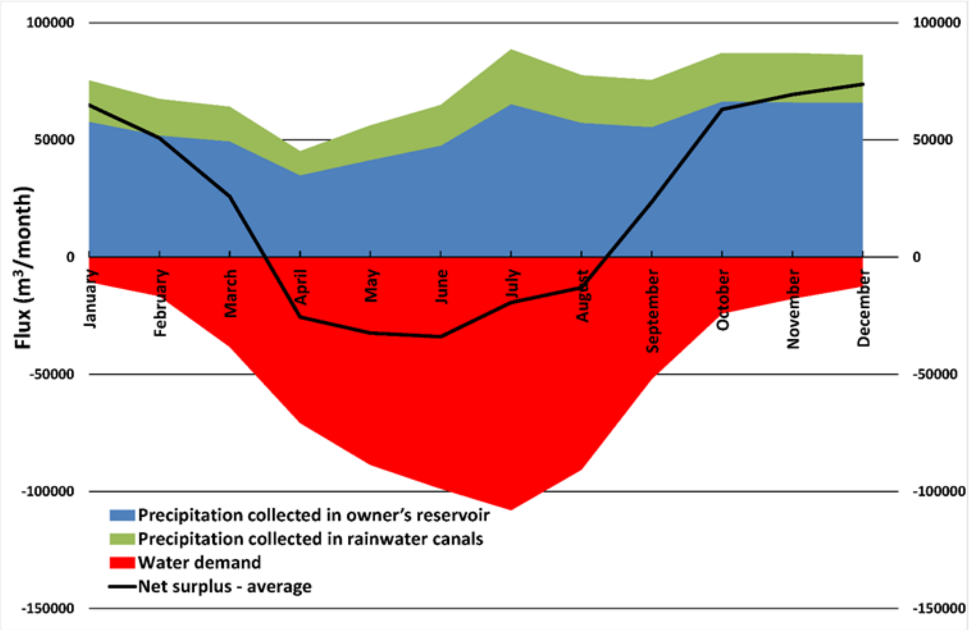


Figure 2-7: Average storm water supply, led either directly to the owners' reservoirs or to the storm water canals, compared to the owners' average freshwater demand. The average net monthly surplus indicates relatively wet (winter) and dry (summer) periods.

2.3.3 Influence of using forecasts on prevention of overflow

With the proactive management approach an overall decrease in expected monthly and annual overflow was reached (Table 2-11 and Figure 2-8). On average, the reduction in overflow was 17.8 thousand m^3 per year (a reduction of 58.5%), which for the total Glasparel+ area equals a 16 mm difference.

Table 2-11: Average annual overflow at Glasparel+ with reactive (business as usual: BAU) and proactive reservoir management (Forecast based: FC) given in cubic metres and in millimetres.

Total model period	Overflow BAU (m^3)	Overflow BAU (mm)	Overflow FC (m^3)	Overflow FC (mm)
January	1871.8	1.7	677.8	0.6
February	1004.2	0.9	329.4	0.3
March	2657.3	2.4	495.0	0.4
April	56.6	0.1	0.0	0.0
May	354.5	0.3	14.8	0.0
June	1790.8	1.6	678.0	0.6
July	4013.1	3.6	2070.1	1.9
August	2666.7	2.4	642.9	0.6
September	4512.8	4.1	1306.0	1.2
October	4887.2	4.4	2276.9	2.1
November	4272.2	3.9	3176.0	2.9
December	2348.9	2.1	969.6	0.9
Total	30436.3	27.5	12636.4	11.4

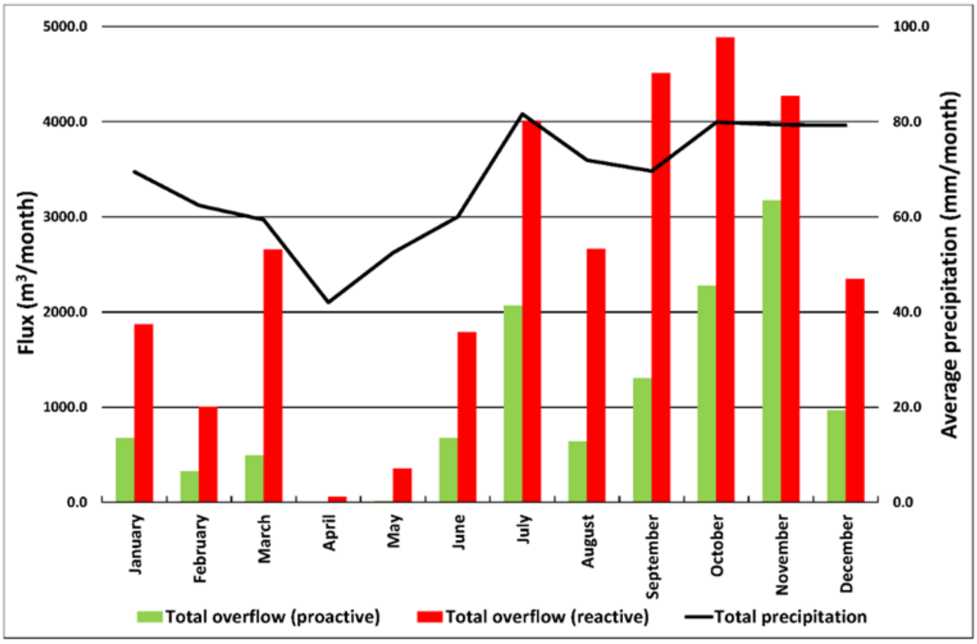


Figure 2-8: Average monthly overflow with the reactive and proactive management approach (in m³/month), and the average monthly precipitation (in mm).

2.3.3.1 Influence of changing the well capacity on cumulative overflow

- With forecasting, total overflow is expected to decrease by 58.5% in the simulated period.
- The effectiveness of the (proactive) forecasting system is much higher if the well capacity is adequate to reduce or increase reservoir levels significantly within a time period of a few days: 71.5% reduction of overflow with 150% well capacity (FC vs BAU), but only 20.7% less overflow with 50% well capacity (FC vs BAU).
- The currently chosen well capacities can be considered economically optimal: the overflow is limited to a few percent. A larger-and-larger increase would be required to further reduce overflow.

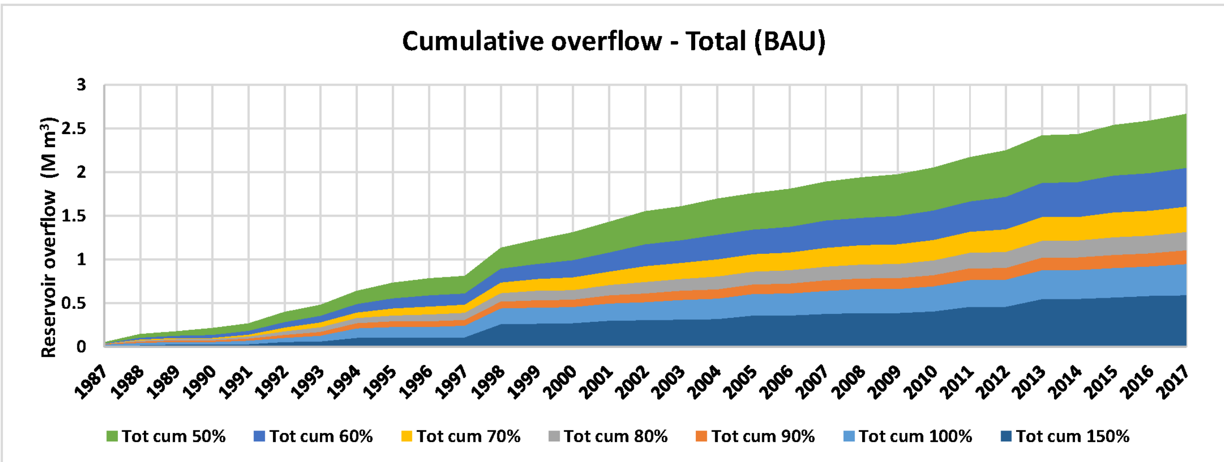


Figure 2-9: Cumulative overflow in case of business as usual (BAU) from January 1987 – December 2017. Well capacities are varied from 50% - 150% of the original (real) well capacities

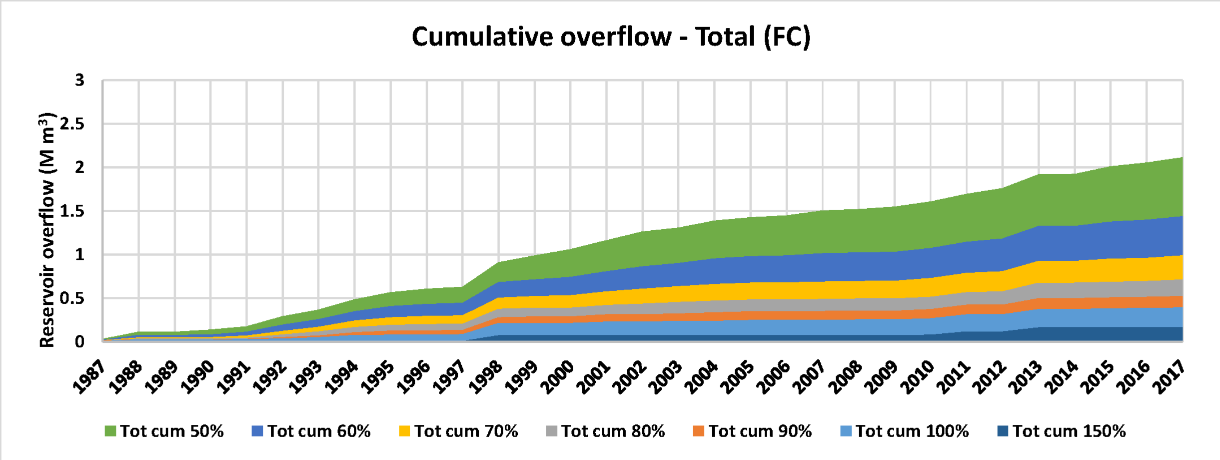


Figure 2-10: Cumulative overflow considering forecasts (FC) during operation from January 1987 – December 2017. Well capacities are varied from 50% - 150% of the original (real) well capacities

2.3.3.2 *Forecasting and size of overflow events (total: greenhouse reservoir and rainwater canals)*

- Overflow events become less significant (up to 18 mm reduction in daily overflow)
- The number of overflow events is reduced by 90 from 223 to 133 (-40%).

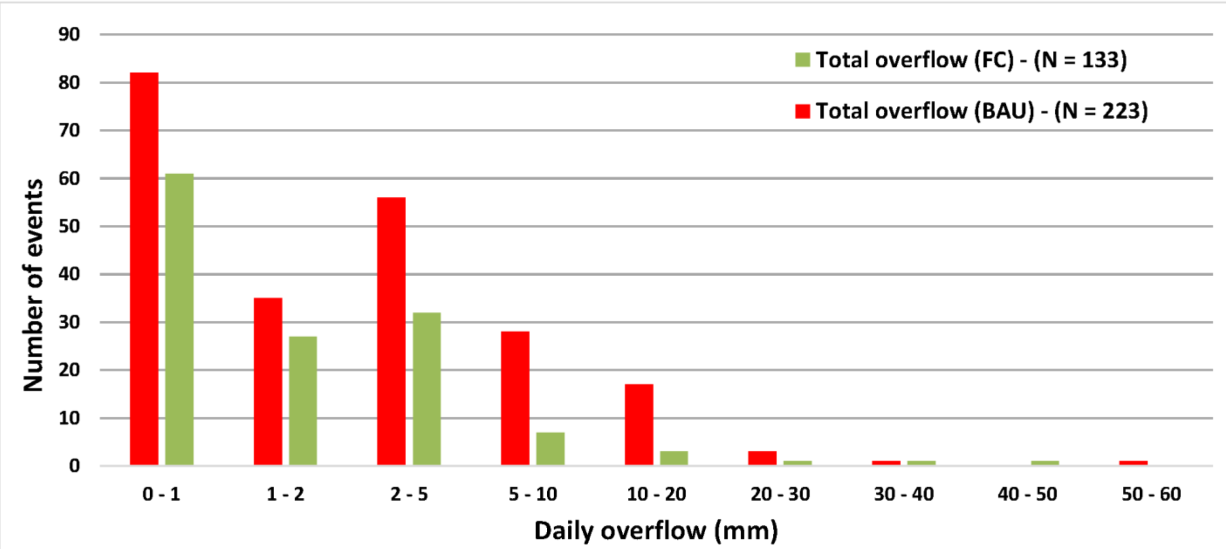


Figure 2-11: Number of overflow events (N) and their size, averaged over the total collection area from January 1987 – December 2017, for the business as usual case (BAU) and with forecasting (FC)

2.3.4 Freshwater shortage and ASR at different well capacities

The freshwater shortage is defined as the deficit in volume of freshwater required for assumed optimal growth for the time of the year. The water shortage has different underlying reasons for the reactive (BAU) and proactive (FC) scenarios.

- Reactive (BAU): shortages are expected to occur in dry periods because of limited well capacity (capacity < peak demand) with 70% or less than original well capacities (Figure 2-12).

Remediation in the Glasparel+ case is advised when reaching 80% of the initial well capacity (or earlier).

– Proactive (FC): shortages are expected to occur only after a more firm reduction of well capacity, to 60% or less than original well capacities (Figure 2-13). This is because a proactively operated system will keep the reservoirs at higher levels days earlier when droughts are forecasted, for the system will start recovering freshwater from the subsurface when lower reservoir levels are forecasted (instead of observed).

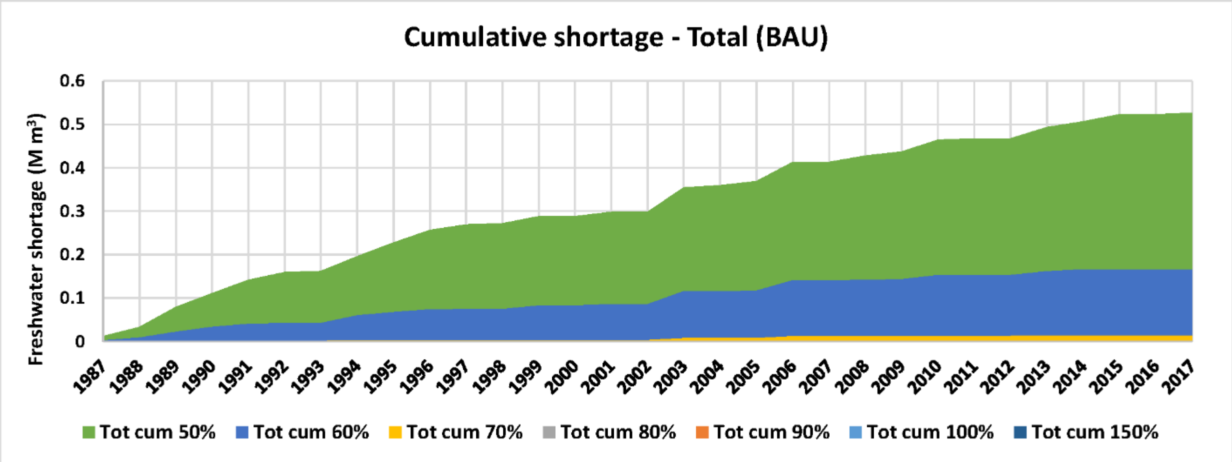


Figure 2-12: Cumulative freshwater shortage from January 1987 to December 2017, for the business as usual case (BAU)

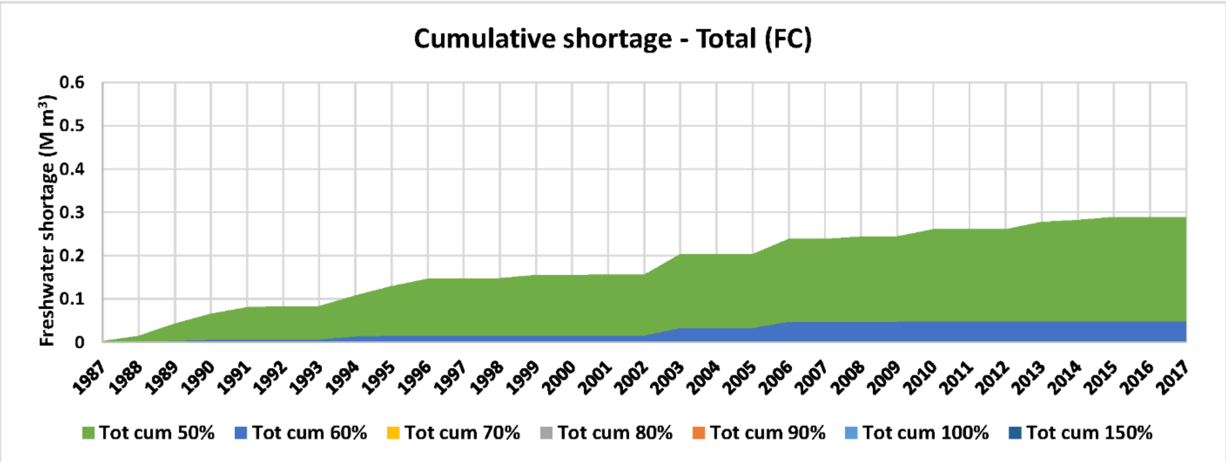


Figure 2-13: Cumulative freshwater shortage from January 1987 to December 2017, for the forecasting case (FC)

2.3.5 ASR use and recovery efficiency

– Necessary recovery efficiency (RE) (near 100% is required) without the rainwater canals too marginal to guarantee good quality of recovered water. Reverse osmosis (BWRO) will be necessary in most of the years to guarantee water quality.

- With the rainwater canals the recovery efficiency (maximum RE < 70%) is expected to guarantee sufficient water quality of recovered water; necessity of desalination by for example reverse osmosis is limited to (part of) every few years.
- Operating the ASR system(s) proactively reduces the annual recovery efficiency required to provide sufficient freshwater to the horticulturists by only a few percent.

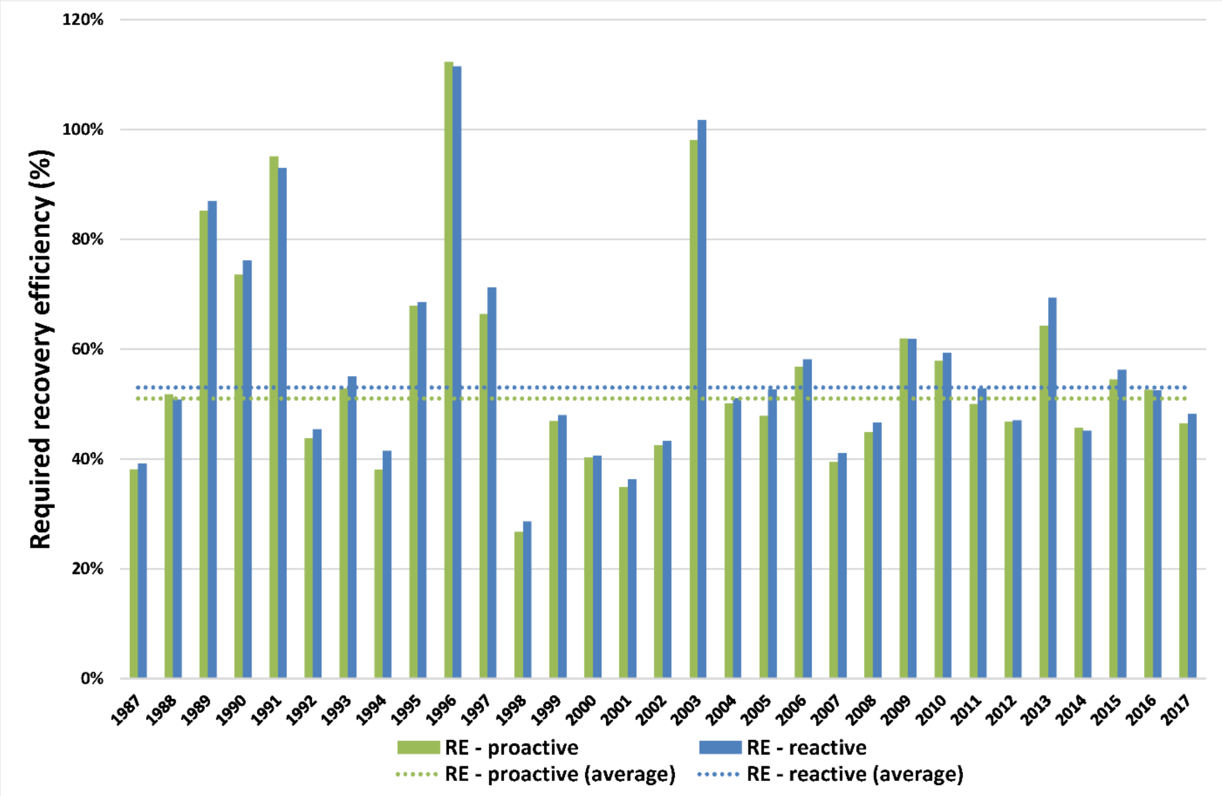


Figure 2-14: Required recovery efficiency with the proactive approach and the reactive (BAU) approach. The average over the total model period is indicated with dotted lines.

2.3.6 Supply of freshwater demand

With both reactive and proactive reservoir management water shortage can be prevented as long as the well capacity is maintained and the required quality limit can be met. This is due to the high net availability of storm water with respect to the water demand. In this section, it is analysed which part of the freshwater demand can come from direct supply of storm water from the reservoirs (without aquifer storage), which part of the demand is to be directly recovered from the aquifer (so, without desalination, based on a ASR recovery efficiency of 70% yearly (Paalman et al., 2012)), and which part of the freshwater demand is to be supplied using ASR in combination with desalination using reverse osmosis.

The type of freshwater supply does not differ significantly between the reactive BAU approach and the proactive FC approach. Differences in supply between both approaches are small (within a few percent) (Figure 2-15). Freshwater supply from (direct) ASR recovery is relatively large in the proactive approach (ca. +3,000 m^3/year) because more water is infiltrated and recovered to anticipate on forecasted rain and droughts (to prevent overflow and shortages). The aim to reduce infiltration and recovery and to lower the energy demand is therefore not met with the chosen operational setup and reservoir set points. However, the additionally harvested and stored storm water - by preventing overflow - resulted in an overall decrease in the required use of desalination (ca. -700 m^3/year), which is a benefit from an energy perspective.

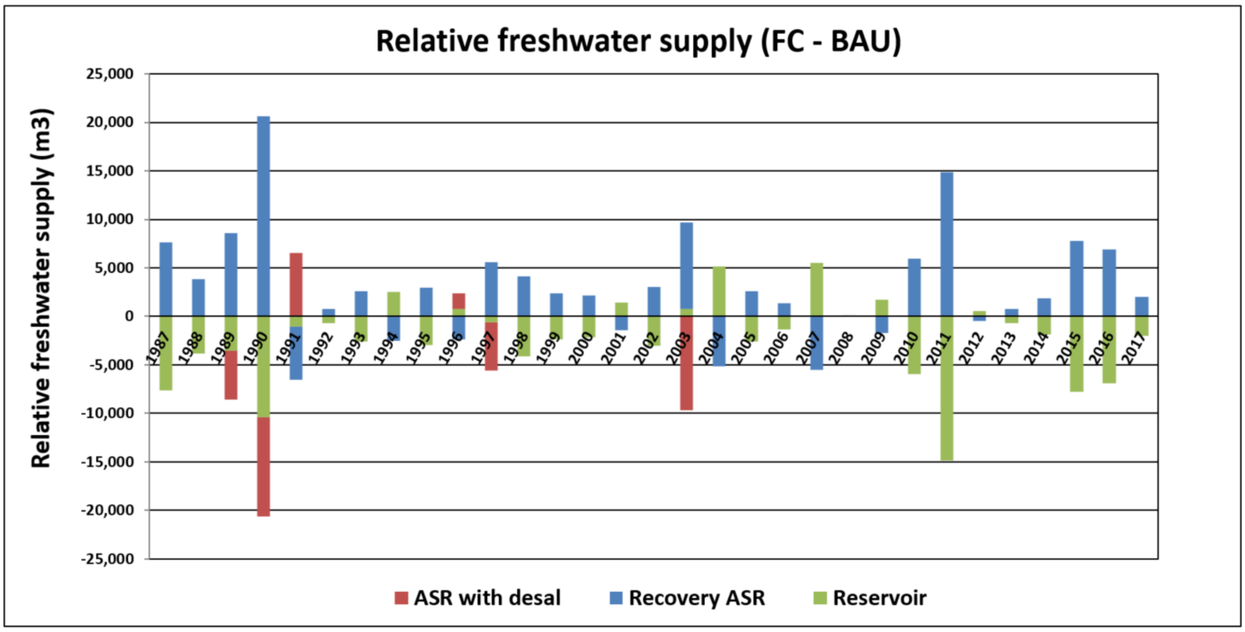


Figure 2-15: Difference in the type of freshwater supply source (FC – BAU: proactive minus reactive): precipitation without infiltration ("Reservoir"), directly usable after recovery ("Recovery ASR") or after recovery with desalination ("ASR with desal")

2.3.7 Sensitivity analysis: well capacity and reservoir size

A total of 840 scenarios was run, varying both the well capacity and reservoir size of the system by steps of 10% for both types of reservoir management between zero and 200%. The aim hereof is to evaluate the boundary conditions for proactive reservoir management in the Glasparel+ case and to identify the sensitivity to cumulative overflow (from reservoirs plus rainwater canals) in both situations. Such a set of scenario runs can be used later to provide design principles for future projects in addition to cost evaluation of varying the well capacity and reservoir size of a system.

The total overflow under various design conditions is shown for both the reactive management approach (Figure 8-5) and the proactive management approach (Figure 8-6) as cumulative overflow in 31 years. In Figure 2-16, the impact of the proactive operation on overflow under different design choices is visualized relative to the reactive approach. The reactive management approach can be considered a safe approach to limit overflow with a relatively small reservoir size and large well capacity. In this case, corrections of the reservoir level can quickly be made and there is insufficient storage volume to actually be proactive. Contrarily, the reservoir may even be kept too high sometimes by the proactive method resulting in (additional) overflow during peak rainfall events.

With a slightly below average to above average reservoir size (range: 30 – 150 mm: or 50 – 200% in the Glasparel+ case) the proactive reservoir management approach is significantly more effective than the reactive approach. The gain hereof increases with increasing well capacity as quicker responses can be achieved after receiving precipitation forecasts. As the reservoir size further increases, reactive reservoir management becomes almost as effective as reactive management since more extreme weather events are needed to cause overflows.

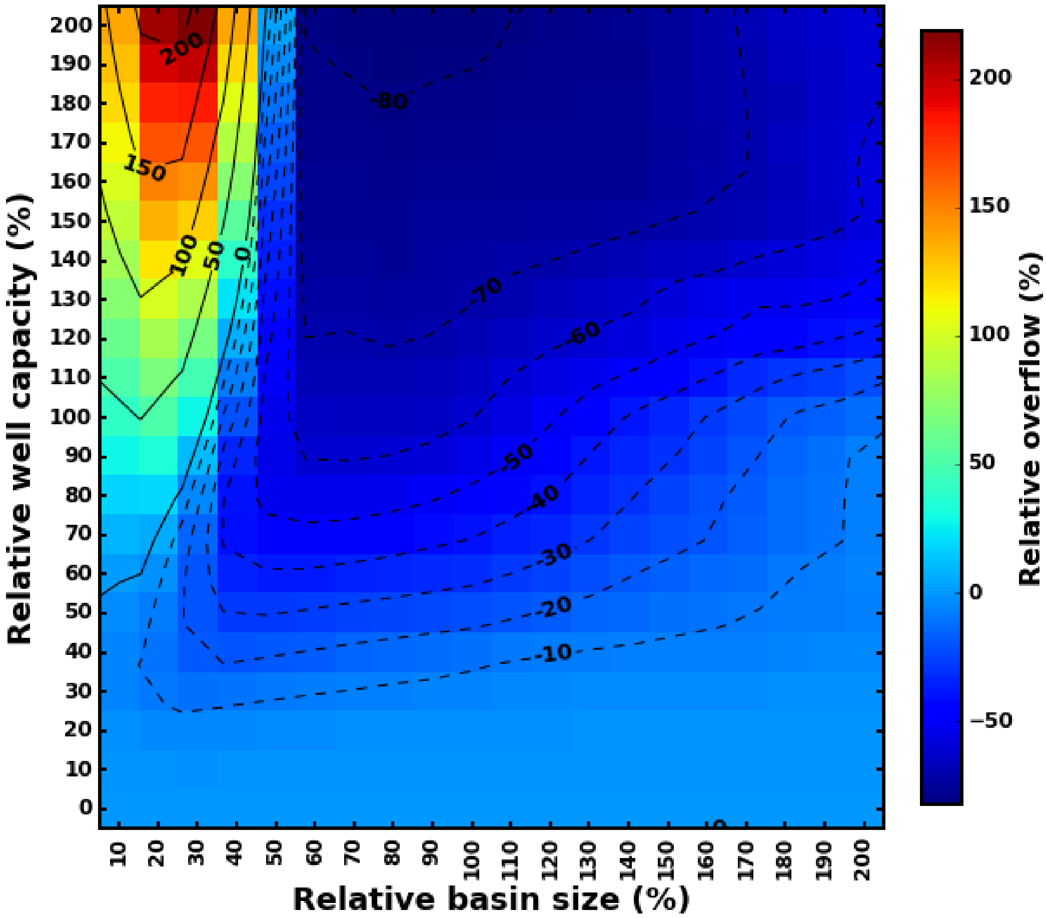


Figure 2-16: Relative increase (+) or decrease (-) in overflow with proactive instead of reactive reservoir management as a result of changing either/or the well capacity and reservoir size of the system relative to the real situation (at 100%).

2.3.8 Proactive reservoir management and uncertainty in weather forecasts

Using weather forecasts in operational reservoir management comes with a certain degree of uncertainty even as predictions have improved over last decades. For example, there can be the problem of interpolation of predicted precipitation data in space and in time. The uncertainty in time can be dealt with partly by also considering 5-day predictions from previous days and to apply a weighted average. The most recent prediction then should still have the largest weight assuming that the uncertainty decreases closer to the moment of truth.

The risks involved with misjudging forecasts will be higher if the connected reservoir is sensitive to changes (limited storage capacity per ha of collection area). Risks are highest when high precipitation events are expected which ultimately do not occur. In such case, an unnecessary amount of storm water is stored belowground, which in absence of the event may still be required in the near-future. The actual risk has to be studied on-site.

Weather forecast data from (The Weather Underground) have only recently been retrieved (29 August 2018 - 12 November 2018). Also, it has been a relatively dry period lately. Therefore, a first test of the water balance model using these daily precipitation data for predictions (until 12 November 2018) did not result in any overflows, neither with reactive management nor with proactive management, assuming an initial high reservoir level of 80%.

An alternative to verify the legitimacy of using weather forecast data was chosen, which is to simulate the effect of a constant 50% over- or underestimation in actual measured precipitation data (January 1987 – December 2017). It was shown that over the course of 31 years the overflow in the proactive approach is especially sensitive to underestimation of precipitation (by > 20%) occurring in the near-future (Figure 17) and not so much to overestimation of precipitation. If preventing overflow is important, it can thus be a safe choice to purposely overestimate the predicted rainfall by some degree. Depending on the degree of (constant) over- or underestimation of the precipitation, a reduction of 24.8% (50% underestimation), 53.9% (50% overestimation), and at most 58.5% (correct predictions) was reached with the current situation for the Glasparel+ case.

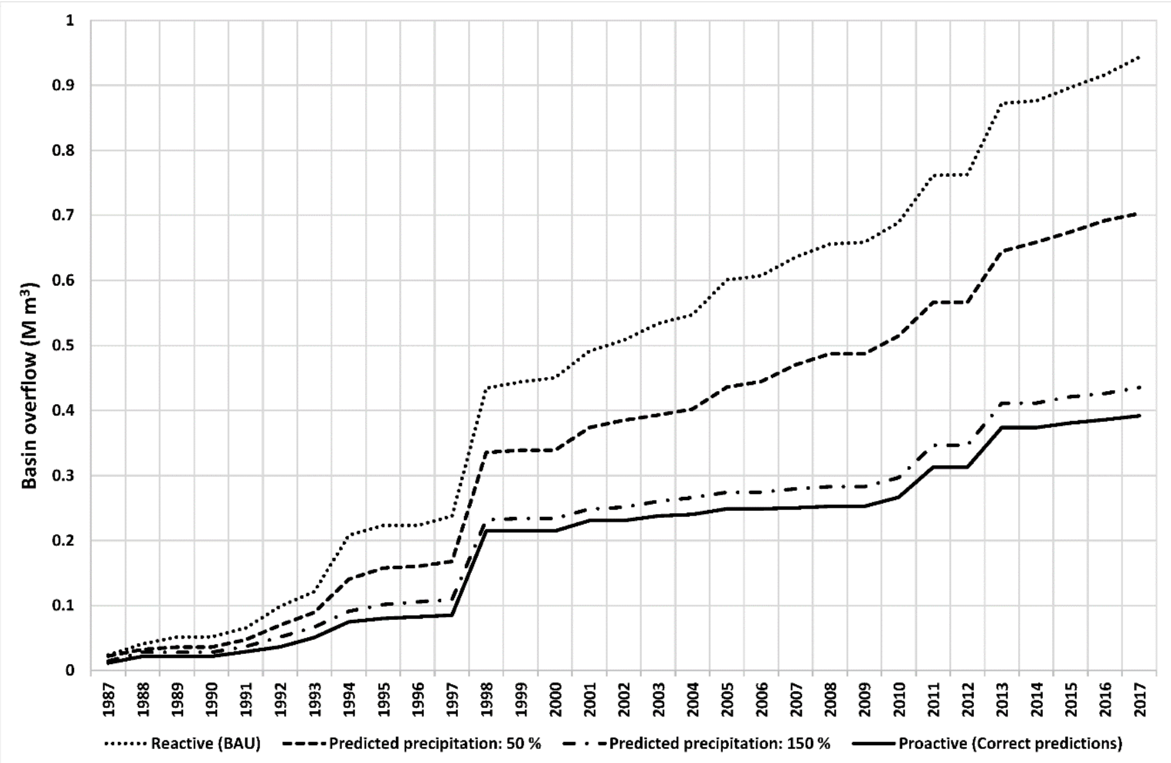


Figure 17: Cumulative overflow assuming either 100% correct predictions, or with an over- or underestimation by 50%; all compared to expected overflow with a reactive management approach.

While reducing the overflow on itself is positive as it decreases losses to the surroundings and reduces flooding, often not all the water stored in the subsurface can later be recovered (<80% recovery is common (in brackish regions) and often the recovery efficiency is also <60%). In the optimal case, solely the water which would otherwise overflow to the surface system is stored in the subsurface. However, constant overestimation of the (predicted) precipitation will result in unnecessary storage and recovery (Table 2-12). This brings the required recovery efficiency closer to 100% and leads to increases in energy consumption.

In the Glasparel+ case, there is an additional factor to overflow which has to be highlighted here, i.e. overflow from the rainwater canals to the surface water system. If the predicted reservoir level is high (as is more often the case with overestimation of precipitation) more water than necessary is abstracted from the owners' reservoir. In addition, as a result of preferred infiltration from the owners' reservoirs, the rainwater canals remain relatively full. This ultimately leads to more overflow from these canals even if overflow from the owners' reservoir is expected to be very limited as a result of low reservoir levels. Early intervention is required to prevent unnecessary overflow from the rainwater canals by infiltrating from the canals sooner.

Table 2-12: Sensitivity to overflow and the impact on well operation of applying the proactive approach, assuming either underestimation (-50%) or overestimation (+50%) of the predicted precipitation.

	Reactive	Proactive	Proactive -50%	Proactive +50%	Unit
Overflow	30436.3	12636.4	22677.0	14042.3	m^3/yr
ASR In	548.8	573.4	558.7	645.6	x 1000 m^3/yr
ASR Out	264.6	266.6	257.1	341.0	x 1000 m^3/yr
RE	48.2	46.5	46.0	52.8	%

3 Concepts for fast pre-treatment of ASR infiltration water

3.1 Introduction

3.1.1 Motivation

Horticultural areas, urban areas and industrial areas often suffer from an imbalance between fresh-water availability and demand. Extreme rainfall therefore can lead to flooding, whereas water needs to be supplied or purified during drought. Buffering of significant temporary surpluses of water put a big claim on the increasingly scarce above-ground space. Alternatively water surpluses can be infiltrated, stored and recovered from ground water wells. This technique is known as aquifer storage and recovery (ASR). ASR is already used locally in horticulture areas in order to store more rain and storm water. Interestingly, increasing the capacity and possibilities for spatial planning of ASR increases the potential and leads to prevention from flooding. The challenge is to purify large amounts of water in a short period of time, in such a way that it is suitable for infiltration into the ground. Therefore, the infiltration water needs to be purified from suspended solids and potential contamination.

In urban areas, some experience was gained in disconnecting storm water to deep injection wells with higher peak capacity. Research by STOWA (STOWA, 2016) showed that these systems in practice often suffer from clogging by high concentrations of particles and nutrients in the collected storm water. The particles and nutrients were not removed sufficiently by conventional purification methods in urban drainage systems. Conventional systems which are currently used in ASR (slow sand filtration, relative coarse self-cleaning filters) either have a large footprint or are insufficiently effective. An overview of several alternative 'Best Available Techniques' in the pre-treatment of ASR infiltration water has great value in the implementation of ASR.

3.1.2 Goals and scope

The goal of this chapter is to visualize purification concepts which are promising for rapid pre-treatment of ASR infiltration water in area's with limited available surface space. The scope of this chapter is focussed mainly on gathering available information. No comprehensive evaluation of the different techniques is performed.

3.1.3 Method

This desk study includes a literature review. The gained knowledge is used in order to select the pre-treatment filter which will be used as prototype of a high flow pre-treatment and infiltration system for storm water runoff.

3.1.4 Structure

Paragraph 3.2 of this report outlines the required removal efficiency of the pre-treatment. Paragraph 3.3 gives an overview of the available techniques and their efficiencies and paragraph 3.4 concludes with the most promising technique(s).

3.2 Assessment framework

3.2.1 Provided ASR infiltration water and general load

Available storm water and surface water can be considered firstly as source for water recovery, supplemented with other types of water like reused water and drainage water. It is expected that different water types require different treatment strategies, depending on their composition (Figure 3-1).

Source	load	Required purification-efficiency	Demands composition Infiltration water:
Storm water - Horticulture - Roofs - Roads Surface water Waste water (combined sewer) Ground water (Drainage)	Ss (+M + OMP) Ss + NP + M Ss + NP + M + S + OMP Ss + NP + M + OMP Ss + NP + M + S + OMP Ss (clay) + NP + M (reduced)	Suitable techniques (paragraph 2.3)	– End user (intended use after recovery) – Legislation (groundwater quality, discharge after use) – Prevention of clogging of Wells (well management)

Figure 3-1: Framework for determination of required treatment
(SS = Suspended solids, NP = nutrients, M = metals, S = salt (from roads), OMP = organic micro pollutants (incl. Polycyclic aromatic hydrocarbons)). In horticulture, exceedances of metals and organic micropollutants (pesticides) are measured sporadically. NB: most water types might also be contaminated microbiologically (viruses, bacteria). This, however, is particularly relevant for intended reuse of water after recovery.

3.2.2 Demands on quality of infiltration water

The quality requirements which should be met by the infiltration water are based on the following aspects (see also Table 3-1):

- Intended reuse by the end user: recovered water from underground storage should meet the required quality. The quality of the infiltration water has a great effect on the quality of the recovered water
- Legal requirements: infiltration water should meet the required quality in order to protect ground water (Technical Commission Subsoil, 2009). Legislation is available under the Water Act, in particular based on Drinking water Decree and the Infiltration Decree Soil Protection.
- Prevention of well clogging: an operational requirement from the infiltration system is the prevention or strong restriction of well clogging, in order to maintain well capacity high (explained in more detail in section 3.2.2.1).

Table 3-1: Overview of quality requirements. SS = Suspended solids, NP = nutrients, M = metals, S = salt (from roads), OMP = organic micro pollutants, AOC = assimilable organic carbon

Requirements	**Non-permissible (substance) groups**
End user	
Horticulture	SS + M + S + OMP
Agriculture	M
Brewery	M + S + OMP
City (replenishment of ponds, irrigation of sport fields, cleaning)	SS + M + NP
Legislation in NL Water Act: activity is 'Infiltration' (combined with recovery). According to the law, the Infiltration Decree Soil Protection is exclusively applicable on infiltration of surface water, although authorities also apply this Decree on other 'suspected watertypes' in order to meet the required groundwater quality. In particular the obligation for monitoring and the high costs involved have led to the drafting of a practical assessment (STOWA publication number. 35, 2015). In this, the same legislation is leading but recommendations are given for responsible tailor-made work based on risk analysis. Drinking water Decree, chapter 3	SS + M + OMP + NP + S
Prevention of well clogging (general) Suspended solids <0,1 mg/L Turbidity <1 Nephelometric Turbidity Units (NTU) Iron <0.01 mg/L Sodium Adsorption Ratio (SAR) < 6 at E.C. 40-100 mS/m Dissolved Organic Carbon (DOC) < 2 mg/L Assimilable Organic Carbon (AOC) < 10 µg acetate-C/L Modified Fouling Index (MFI*) < 3-5 s/L^2	S + NP + AOC*

3.2.2.1 Prevention of well clogging

In ASR, large water volumes (typically 10 – 100 m^3/hr) per groundwater well are guided through the well screen to the aquifer. Successively, filter slits, a gravel pack and a borehole wall (= transition gravel pack to the formation) are passed. Both at the location of the slits and in the gravel pack filter, clogging of the ASR-well can occur by accumulating particles from the injected water (Figure 3-2, Table 3-2). As a result, the capacity of the well (at the same infiltration pressure) will decrease, and a smaller volume per unit of time can be injected into the aquifer. The decreasing capacity may result in not being able to inject the target volume, causing flooding and / or loss of fresh water. Clogging may also lead to higher energy costs and rupture of the soil, if the maximum tolerable infiltration pressure is exceed.

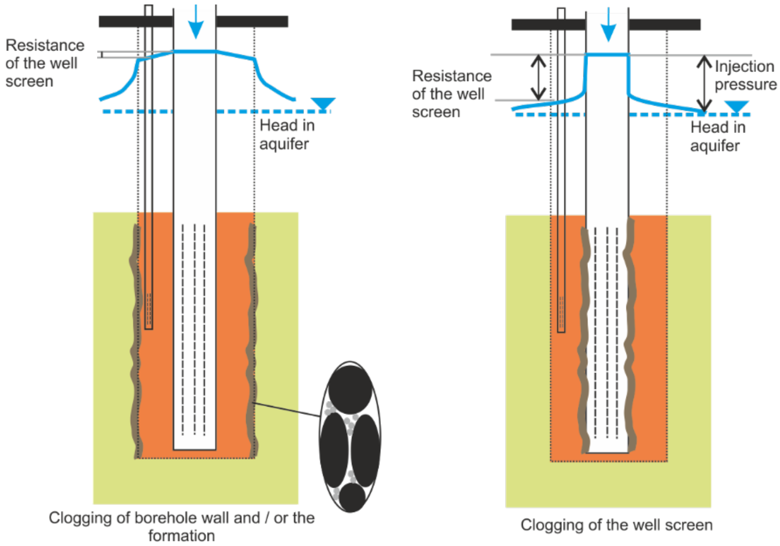


Figure 3-2: Typical well clogging due to particles

Table 3-2: Typology of well clogging and most important processes which cause well clogging during infiltration (Martin, 2013)

Clogging type	Clogging process
Physical	Accumulation of organic and inorganic suspended solids, particularly on the borehole wall. Swelling of clays during infiltration of fresh water into brackish or saline groundwater (eg. Montmorillonite). Dispersion of clays during infiltration of fresh water into brackish or saline groundwater (eg. Montmorillonite), followed by entrapment in small pores.
Mechanical	Transport of air / gas in the infiltration water into the gravel pack
Biological	Growth and accumulation of biofilms

The way in which the wells are operated also affects the clogging process of wells. Regular back flushing of infiltration wells leads to less stringent requirements of the infiltration water quality regarding clogging.

3.2.3 Required pre-treatment

The required treatment for the purpose of reusing infiltration water depends on the quality difference between the water source and the requirements of the infiltration water. Sufficient removal of particles will in all cases be the primary task, followed by removal of heavy metals, nutrients and organic micro pollutants.

In paragraph 3.3, a number of techniques will be evaluated.

3.3 Overview of several ASR pre treatment techniques

3.3.1 Evaluation criteria

In this chapter, a number of pre-treatment techniques is reviewed. In the selection of ASR pre-treatment filtration, the following criteria are discussed:

- Capacity (quantity);
- Treatment efficiency of suspended solids, nutrients (NP), metals (M), salt and organic micro pollutants (OMP);
- Surface requirement;
- Maintenance and precaution (eg. frequency of maintenance);
- Indication of investment costs and operational costs per m^3/hr capacity;
- Energy consumption.

In Annex 8.5, an overview is given of relevant literature for infiltration well pre-treatment technologies.

3.3.2 Slow sand filtration (SSF) combined with rapid sand filtration

The combination of a slow and rapid sand filter technique is considered to be a proven technology based on the experience in infiltration projects for drinking water and irrigation water supply (see Table 3-3). A schematic overview of slow sand filtration in ASR in horticulture is shown in Figure 3-3. Besides, the technology was proven in (large scale) purification of surface water to drinking water (eg. Waternet, Amsterdam). In the latter project, slow sand filtration as final step in the treatment provided the removal of suspended solids, (biodegradable) pollution and nutrients, as well as reduction of pathogen micro-organisms, viruses and protozoa (Hijnen, Schijven, Bonné, Visser, & Medema, 2004; Huisman & Wood, 1974).

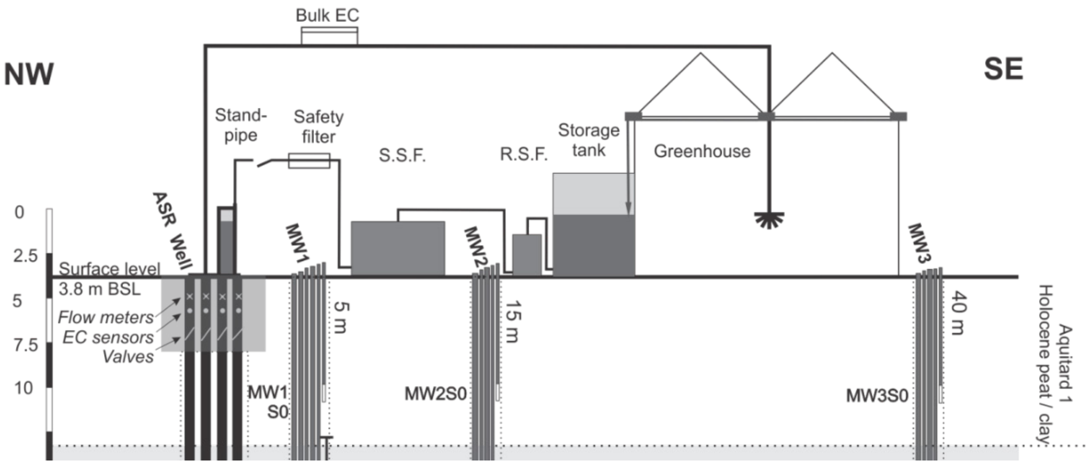


Figure 3-3: Application of slow and rapid sand filtration in ASR in horticulture (The Netherlands), especially for removal of suspended solids (K. G. Zuurbier, Zaadnoordijk, & Stuyfzand, 2014). R.S.F. = rapid sand filter, S.S.F. = slow sand filter)

Table 3-3: Overview of characteristics of SSF with upstream rapid filter. Sources: (Chinu, Johir, Vigneswaran, Shon, & Kandasamy, 2009; Diels, Kramer, Spaans, Roy, & Wouters, 1999; Diels et al., 2003; Hijnen et al., 2004; Logsdon, Kohne, Abel, & LaBonde, 2002)

Aspect	Parameters	Typical values
Capacity		100 to 300 L/m^2
Area requirement		Large (2 m^2 per single m^3/h)
Purification efficiency	SS	Combined with upstream filter system, ('roughing filter') in order to remove bigger particles, an effluent quality of < 5 NTU (ca. 70%) is achievable. Even higher removal efficiencies for SS are feasible.
	Pathogen Micro-Organisms	Depending on the filtration speed (the slower, the better) and grain size (the smaller, the better). For filtration speeds around 0.1 to 0.2 m/hour the removal efficiency is larger than 99.9% for Giardia, >90% for total Coliforms, > 99.99% for Cryptosporidium oocytes (approx. 4 to 5 $\log_{10}$ removal)
	Metals	Zinc, copper, cadmium and lead can be removed by 95% to 99%, depending on their percentage silt-bound, the population of micro-organisms (bio-sorption) and feed water temperature. Iron and manganese can be removed by approx. 60%. Arsenic removal will be lower, between 30% - 40%
	OMP	DOC: approx. 10 to max 25%, AOC-removal is higher: approx. 30% to 40% at drinking water production. Typical AOC at horticulture systems after slow sand filtration reaches approx. 10 µg/L. This leads to limited clogging by biological growth after long infiltration periods (especially in the summer). Many micro pollutants are being degraded microbially. Treatment efficiency is affected by the season (more removal at higher temperatures)
	MFI	2-10 s/l^2
Others	Maintenance and precaution measures	Maintenance at horticultural systems: removal of dirt layer ('schmutzdecke') approx. once per two years. Rapid sand filter backflushes to sewer once the differential pressure increases. Well regeneration in SSF: annual with sodium hypochlorite or hydrogen peroxide (partly preventive). The turbidity of the incoming water must be kept below 10 NTU for optimal operation. A pre-aerated sand filter is necessary for the removal of ammonia by nitrification.
Costs		Investment depends on available surface and space. Maintenance costs are low (removal of contaminated layer, limited monitoring of turbidity in effluent, pH, phosphorus and nitrate measurements)

3.3.3 Self-cleaning filters

Self-cleaning filters form an interesting alternative, because of their limited surface requirements and low maintenance frequency. There is some experience with self-cleaning filters (eg. SAF-filters (Amiad); nominal removal down to 25 micron) as pre-treatment step in subsurface water storage. The description is mostly anecdotic (growers, installers, drilling companies). They suggest that more clogging and rupture occurs at relatively coarse purification. Only at very fine filtration (like with the MT44 filter (Amiad) sufficient pre-treatment seems to occur. From this we deduce, that a finer pre-filtration is necessary and should be mandatory in alternative systems. Two alternatives available on the market are highlighted below.

3.3.3.1 Fuzzy Filter

The Fuzzy Filter is a rapid, self-cleaning filtration method available at Bosman water management B.V. It is used to remove suspended solids from water. The quick filter is formed by pressing Fuzzy balls in between two rosters. The balls are compressed to create a fine, porous medium. Suspended solids can be stopped and captured at high speeds (up to 100 m³/m²/hr) (Visser, 2011). To clean the Fuzzy balls, the roster is moved upwards at one site, after which the filters are back-flushed with influent and air (Figure 3-4)

Filtration

During the filtration cycles, the porosity is arranged with a perforated plate. The influent flows from the bottom to the top through the fuzzy medium. As soon as the filter is saturated with particles the wash cycles is started

Wash cycle

During the wash cycle the perforated plated is moved upwards so that the fuzzy balls can move freely. This enables the balls to be washed with high strength and velocity without losing any medium. For washing, influent water and air are used.

Rinsing

After washing the medium is compressed again by moving the perforated plate. The remaining dirt is discharged and the cycle starts from the beginning.

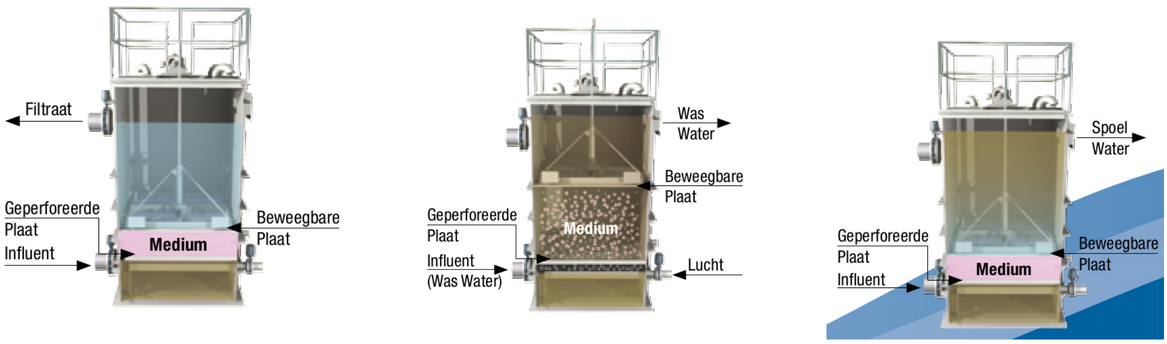


Figure 3-4: Operation of the Fuzzy filter (Bosman water management B.V.)

The Fuzzy filter uses a 30"deep media bed, consisting of individual 1,25"diameter (32 mm) wide, compressible synthetic fibre balls. They obtain high suspended solids removal up to 4 micron (Table 3-4). Because the medium is compressible, the porosity of the filter can be adjusted to multiple filtration needs. Nevertheless, the filters are usually used to replace sand filters so that a higher flux can be obtained (Poff & Wilson, 2010). Chlorine might be added (after 1 – 1.5 years) in order to prevent bacterial growth. Cleaning chemicals might be added to remove build-up fats (Gibbs, 2009).

The Fuzzy filter has been or is recently studied in the following studies:

– Nutrient removal from storm water overflow (Technical University of Berlin, Germany);
– Nutrient removal from effluent communal waste water treatment system (STOWA, the Netherlands) (Visser, 2011);
– Particle removal (Fachhochschule Münster, Germany)

The nutrient removal study of STOWA (Visser, 2011) showed that Fuzzy Filters are more expensive than sand filters in smaller Sewage Treatment Plants (STP). In bigger STPs, the costs of the Fuzzy Filter and sand filter are about the same. The study also showed that the Fuzzy Filter had some cons compared to the sand filter:

– Big increase in hydraulic load of the STP, which might result in a necessary hydraulic expansion of the STP in addition to the filtration step;
– Shorter retention times due to phosphorous, high rinse water production;
– Less practical experience than with sand filter

The Fuzzy filter is modular with a capacity of 5 to 400 m^3/h per module. Bosman Water management B.V has a mobile test system available to test the Fuzzy filter, suitable for a flow of 5 – 20 m^3/h.

Table 3-4: Overview of characteristics of the Fuzzy filter system

Aspect	Parameters	Typical values
Purification efficiency	SS	Removal down to 4 µm, removal achievable to 1 NTU (Gibbs, 2009) or 2NTU when influent water has >8 NTU (Caliskaner, Tchobanoglous, & Carolan, 1999).
	Nitrates	2,5 kg N/m^2 filter per day, 50% removal (Visser, 2011)
	PO_4	7.5 g o-PO_4 /m^2 filter surface, max. concentration for Fuzzy filter is approx. 0.7 mg P/l (Visser, 2011)
	Metals	No specific data (but probably comparable to sand filtration)
	OMP	No data (but probably comparable to sand filtration)
Surface requirements		Limited, modular
Capacity		Comparable to rapid sand filtration, except infiltration rate might be 3-6 times higher; support up to 1230 L/m^2 min (filtration) and *Compression ratio* of 40%. This data makes a backflush flow of 5.4%. (Caliskaner et al., 1999)
Maintenance		Backflush + (chlorine + detergent every 1 to 1.5 year) (Gibbs, 2009)
Other	Chemical dosing	Chemical cleaning (chlorine) and detergent in order to remove biological fouling and fats. Approx. once every 1 – 1.5 year
Costs		No data

3.3.3.2 Galileo L Filter

The Galileo L filter is an automatic ring filter (see Figure 3-5), consisting of plastic rings compressed in a variable holder. The grooved filter rings are compressed after which a flow of water from the outside to the inside is realized. Coarse particles remain on the outside of the filter rings and smaller particles are trapped in between the rings. The filtration potential is comparable to the deep bed filtration in a slow sand filter according to the supplier (Table 3-5). The system uses an automated multi-jet nozzle system, which flushes the debris from the inside to the outside of the filter rings once the pressure difference between the front and backside of the ring filters exceeds a set-point value.

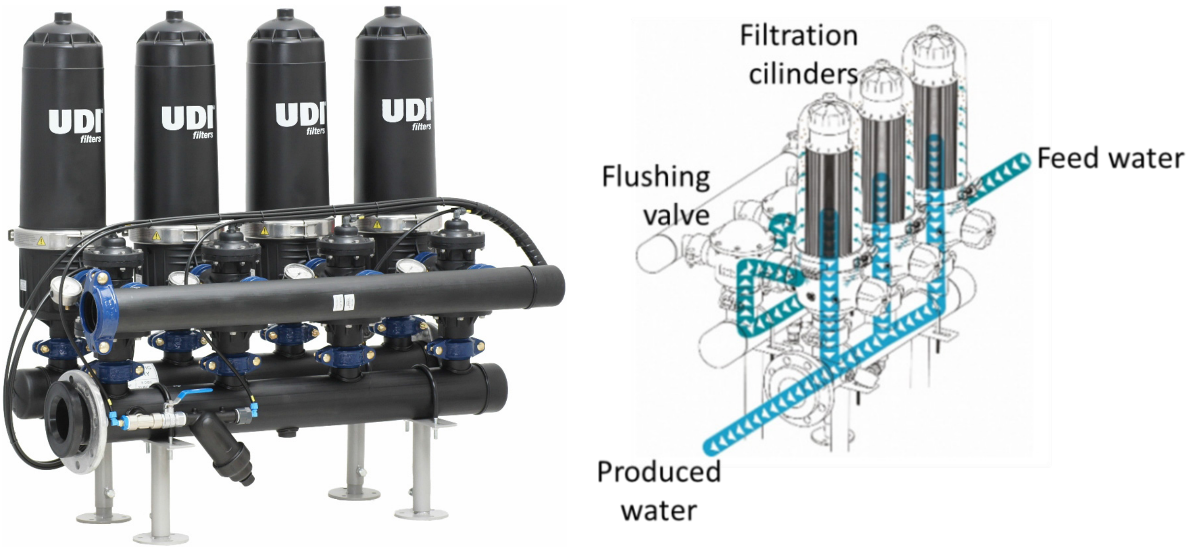


Figure 3-5: UDI Galileo L ring filters (UVAR Holland B.V.)

Technical specifications per module:

– Filter surface.: 1,500 cm²
– Max. pressure: 10 bar
– Min. Rinsing pressure: 2,8 - 3,5 bar *
– Rinse capacity: 9 – 11 m³/hr
– Rinsing water: ± 60 L
– Max. operational temperature: 60°C
– Connection filter: 2"
– pH 4 - 11 resistant
– Filter rings 400 - 200 - 130 - 100 - 50 - 20 micron. 10 - 5 micron available at request

Materials:

– Filter housing: fibreglass reinforced polyamide (option: polypropylene)
– Filter rings: polypropylene
– Manifold: HDPE
– Solenoid: RVS 400, polyamide
– Valve(s): plastic
– Other: polyamide, RVS, NBR

The Galileo L filter system is currently used as pre-treatment for basin water in horticulture. The fine grade (5 micron nominally) is also used as pre-filtration step for RO-systems, disinfection, as preparation for micro- and ultra-filtration, as filtration in cooling water systems, intake water and recirculation systems. There were not yet experiences with the system as pre-filtration in infiltration systems. In relation to its limited surface requirements and probable limited maintenance requirements, this system looks interesting as pre-filtration step in urban areas.

Table 3-5: Overview of characteristics of Galileo L filter system

Aspect	Parameters	Typical values
Capacity		Modular, approx. 5 m^3/h per unit
Area requirement		Limited and modular
Purification efficiency	SS	Removal to 5 µm (nominal).
	Nitrate	No data. No removal expected.
	PO_4	No data. No removal expected.
	Metals	No specific data (probably comparable to sand filters)
	OMP	No specific data. No removal expected.
Maintenance		Every 2-3 weeks: check filter for operation, pressure difference and/or leakages. Yearly: greasing of rubber parts. The ring package must be cleaned manually at least once a year, or more frequently if needed. To remove the growth of algae, plankton etc., the rings can be soaked in a 5% NaOCl solution. All O-rings and rubbers must be greased regularly with silicone grease for optimal operation
Other		-
Costs		No data

3.3.4 Integrated removal techniques in stormwater drain systems

With the arrival of separated storm water drainage systems (draining to surface waters) in urban areas, a number of techniques has been developed to remove suspended solids (and the adhering impurities like metals, PAKs and PO4) and oils. Mostly inline solutions are implemented, based on sedimentation and liquid separation. Examples are Sedipoint® and Sedipipe® (Figure 3-6) of Fraenkische Rohrwerke and the Certaro HDS filter of Wavin (Figure 3-7). In all cases, the sludge needs to be removed via a vacuum truck (approx. once a year). Normative capacities are approximately 50 m^3/hr (Sedipoint, Certaro) and 1,500 m^3/hr (Sedipipe). The sedipipe therefore can treat several hectares of street water.

Based on the available data of these inline solutions (Table 3-6), the application of one of these inline techniques as pre-filtration for ASR seems useful but not sufficiently effective; a downstream step with a fine filter seems to be necessary in order to obtain a sufficient reduction of suspended solids.

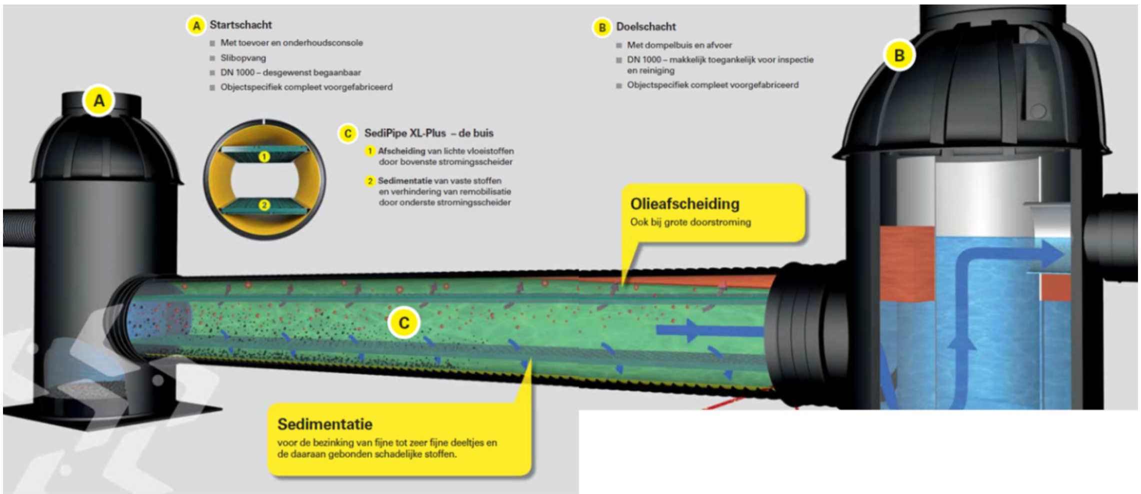


Figure 3-6: Sedipipe (Fraenkische Rohrwerke)

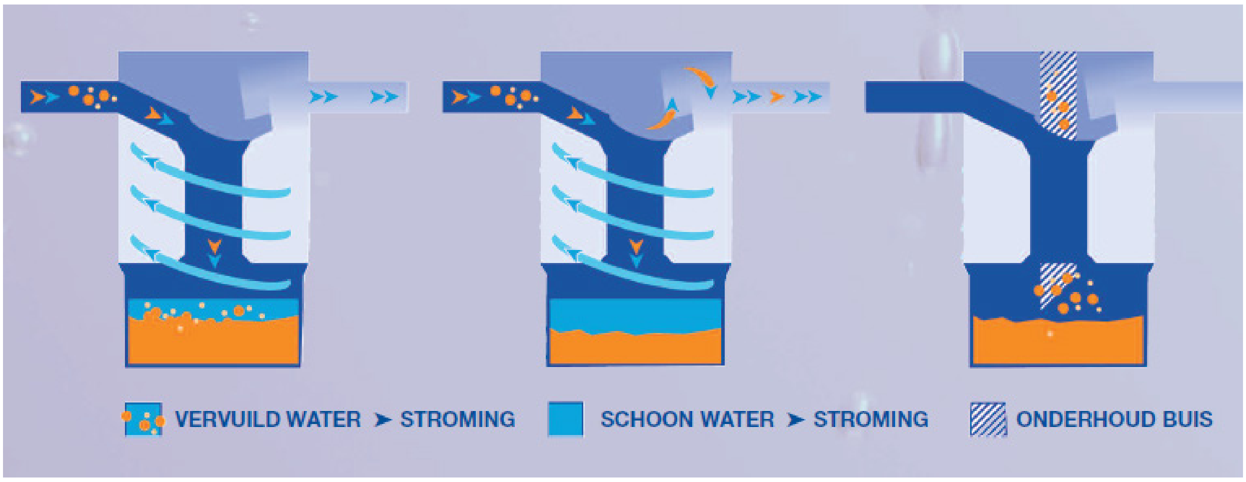


Figure 3-7: Certaro HDS Pro Filter (Wavin)

Table 3-6: Overview of characteristics of in-line stormwater drain treatment

Aspect	Parameters	Typical values
Purification efficiency	SS	Certaro: 80% of fraction >75 micron Sedipipe: 90% of fraction 2 - 60 micron at low flow speeds
	Metals	If adhered to removed fraction
	NP	If adhered to removed fraction
	OMP	If adhered to removed fraction
	S	No removal
Capacity		Sedipoint / Certaro: approx. 50 m³/h per unit Sedipipe: up to 1.500 m³/h per unit
Maintenance		Yearly vacuum action
Other		Studied at TU Delft / Leipzig Universiteit (TAUW)
Costs		Sedipipe 600/24: 15 to 20 k€. Sufficient for treatment of approx. 2.5 hectares storm water

3.3.5 Natural water treatment systems

Especially in densely populated area, waste water treatment plants consist of aerated activated sludge systems, combined with a clarifier and/or membrane bioreactors and sludge treatment. These technologies have a relatively high energy consumption, but on the other hand have small surface requirements. The principles are mainly based on microbiological nutrient removal. Sustainable, maintenance friendly and cheap alternatives are helophyte filters or macrophyte filters, swamp filters, rolling meadows or reed marshes (see Table 3-7 and Table 3-8). Helophytes are two-year old or permanent plants, of which the submerged buds can survive an unfavourable period, like winter. Helophytes are a subclass of macrophytes: aquatic plants which grow in or near by water.

Helophyte filters are typically constructed in a foil or a basin in order to prevent ground water contamination. In the Netherlands, experiments with helophyte filters are mainly focussing on improving the drainage, recreation and water storage (e.g.: Erasmusgracht, Amsterdam (opMAAT, 2005) or landgoed Het Lankheet (Stichting Waterpark, 2018) in the neighbourhood of Haaksbergen), but also small scale systems for industrial or agricultural water treatment processes and communal waste water treatment exist. Abroad, constructed wetlands are mainly used in agricultural applications (M. Scholz, Harrington, & Carroll, 2007) and treatment of waste water from mining industries (Sheoran & Sheoran, 2006). Different types are distinguished (Figure 3-9). The most simple type is the flow field, in which the contaminated water is passed through the plants at the bottom of the swamp after contact with the air. Besides, there are horizontal and vertical flow helophyte filters. At the horizontal flow helophyte filters, the contaminated water drops through the soil along the roots of the helophytes. This type of filter often has a coarse sand or gravel surface.

Limited amounts of contaminated water are often treated in vertical flow helophyte filters. In this configuration, contaminated water runs through plant root zone in approximately 24 hours through the fine sand with which the filter is filled with. Precautions are needed: obstruction of the helophyte filter needs to be avoided by using a pre-filtration step in a sand filter or a cartridge filter. Best results concerning nutrient removal are obtained in a hybrid system, consisting of a horizontal helophyte filter, followed by a vertical one. Such a hybrid filter requires more surface and requires more back flushing with water and/or air in order to remove the sludge cake layer.

An example in the urban domain is currently being developed at Field Factors ((Field Factors, 2018), see Figure 3-8). This pre-treatment still has a firm spatial footprint, but as it is combined with regreening of urban areas, it can often be implemented.

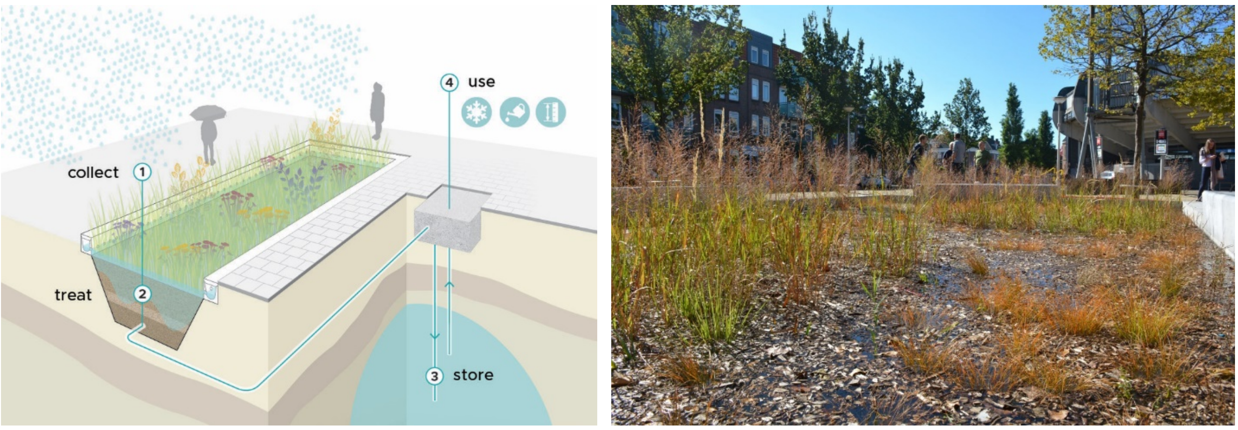
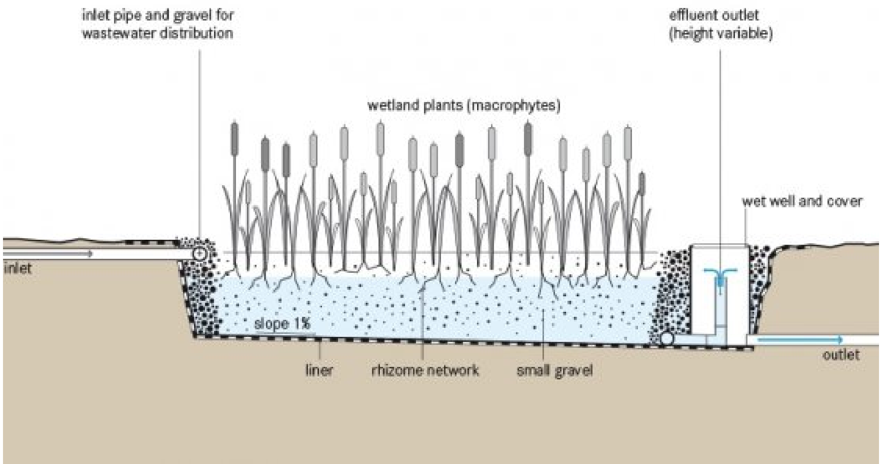
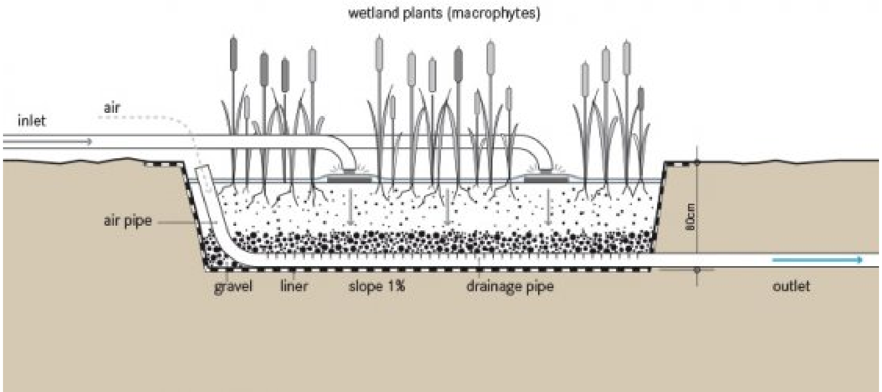


Figure 3-8: Blue Bloqs, currently under development (left) and first pilot at Spangen, Rotterdam



a) Horizontal flow helophyte filter



b) Vertical flow helophyte filter

Figure 3-9: Schematic drawing of a) horizontal and b) vertical flow helophyte filters

Table 3-7: Overview of characteristics of helophyte filters
(Cheng, Grosse, Karrenbrock, & Thoennessen, 2002; Cooper, 1999; Kivaisi, 2001; Lin, Jing, & Lee, 2003; Mulling, van den Boomen, van der Geest, Kappelhof, & Admiraal, 2013; Miklas Scholz & Lee, 2005)

Aspect	Parameters	Typical values
Capacity		Depending on load and available surface
Surface requirements		Big but often able to fit if in a green environment. > 2 m^2 per 1 m^3/h
Purification efficiency	SS	80 - 95 % reduction to <10 mg/L suspended solids, depending on construction and used halophyte (salt-tolerant) species. Filter systems with only 0,2 NTU in the effluent are possible
	Salts	Depending on used, salt-intolerant plant species (halophytes), the salt load can be reduced
	Biol.	Faecal coliforms ≤ 2 to 3 log10-reduction
	Metals	Metals reduction comparable to slow sand filtration, metals are partly stored in the helophyte biomass
	OMP and NP	BOD = 80% to 90%; N-concentration = 15% to 40%; P-concentration = 30% to 45%; Reduction of OMP depends on season, fluctuating from 50% to 90% reduction. Aeration of top layer can improve reduction.
Maintenance		Unknown. Plants should be removed in winter
Costs		Investment depends on available surface and space, maintenance costs are low

Table 3-8: Reduction of OMP by flow of waste water (combination of industrial and communal waste water) over helophyte filter in Spain (Matamoros, García, & Bayona, 2008). n.r.: no reduction observed

	Concentration in waste water	**Removal (%)**	
Medicines	(average ± st.dev) (μg L^{-1})	June 2005	February 2006
Ibuprofen	0.04±0.03	96±2	95±1
Naproxen	0.34±0.06	92±1	52±9
Diclofenac	1.25±0.11	96±1	73±7
Ketoprofen	2.10±0.70	99±1	97±1
Clofibric acid	0.07±0.01	36±3	32±8
Carbamazepine	0.37±0.08	30±10	47±6
Veterinary medicines			
Flunixin	1.06±1.36	n.r.	64±3
Cosmetic products			
Galaxolide	2.86±0.02	85±2	88±1
Tonalide	0.86±0.10	88±2	90±1
Pesticides			
Mecoprop	7.80±3.24	79±2	91±1
MCPA	2.01±1.50	93±1	79±2
terbutylazine	2.30±1.82	1±14	80±1
Hydraulic retention time (hours)	–	720	720

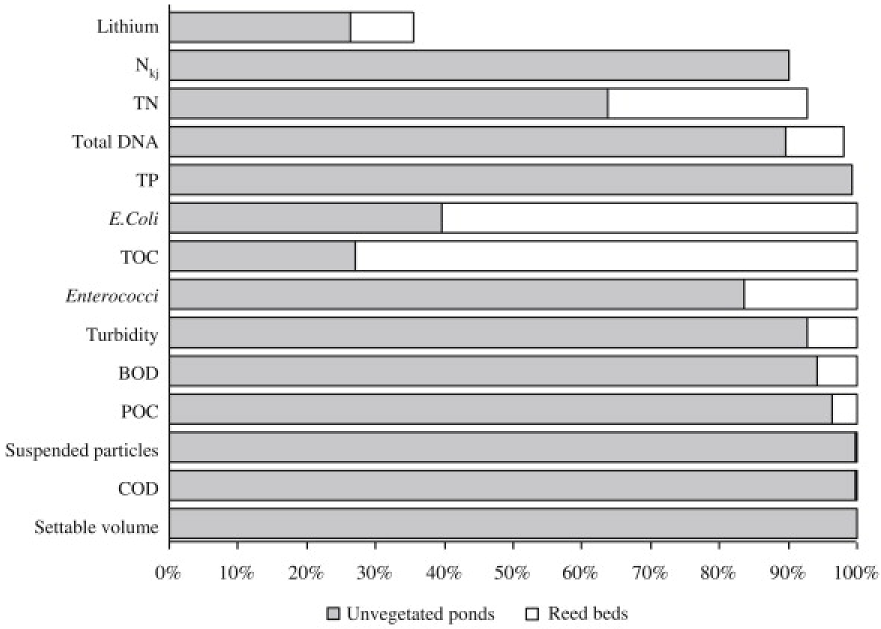


Figure 3-10: Reduction of various parameters in buffer basins without vegetation and downstream (horizontal flow) reed beds after collection of sewage water in Grou, Friesland (the Netherlands) (Mulling et al., 2013). Note: potential impact of aging on removal efficiency not taken into account.

3.3.6 Canal bed filtration

Canal bed filtration is used as drain collectors at PWN and Waternet (Waterleidingduinen, AWD) to (i) reduce the water's clogging potential for injection wells, and (ii) increase capacity without additional surface requirements and without adding additional chemicals in a pre-treatment system.

The filtration is characterized by a relatively high flow rate and short retention time. This system is used at the Freshmaker in Ovezande (Figure 3-11). At the Orange County Water District (OCWD, California, USA) the 'riverbed filtration' technique is used on a large scale in the bed of the Santa Ana River as filtration step prior to infiltration. In the Waterleidingduinen, canal bed filtration has been tested (Figure 3-12). The system consists of a plastic foil with gravel, in which the drains are located. Fine dune sand, in which the filtration takes place, is placed on top of the gravel pack. The presence of the plastic foil under the drain system serves as prevention of the inflow of anoxic groundwater.

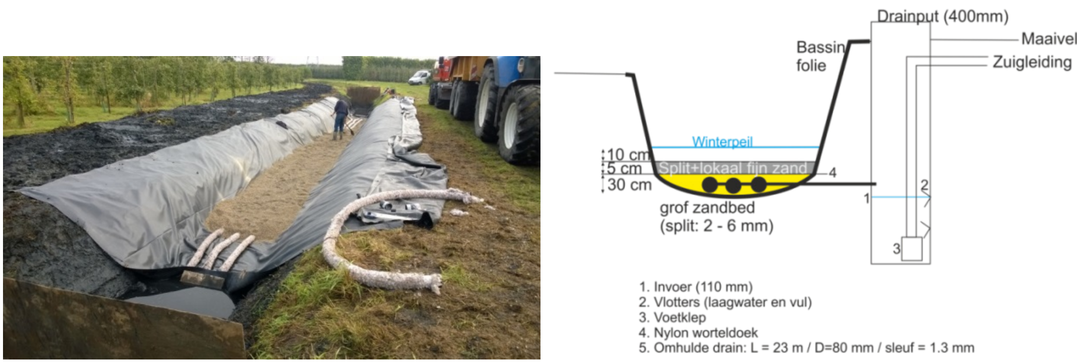


Figure 3-11: Bottom filter at Freshmaker Ovezande (runoff ditch water)

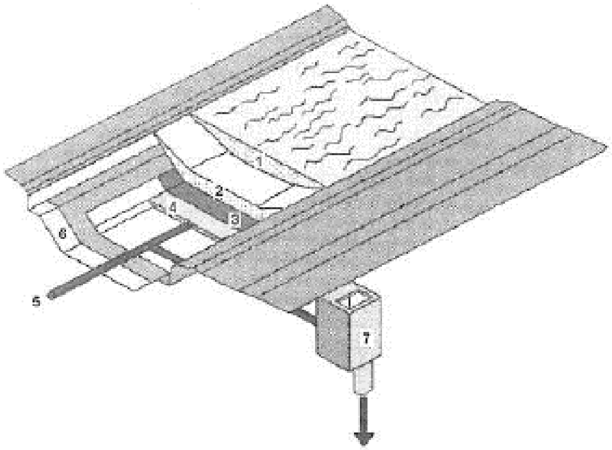


Figure 1. Cross-section of the bottom filter in the supply channel. 1: pre-treated river water, 2: fine aeolian dune sand, 3: nylon fabric, 4: gravel pack, 5: drainage system, 6: impermeable plastic sheet, 7: injection well.

Figure 3-12: Bottom filter of Waterleidingduinen Amsterdam (Van Duivenbode & Olsthoorn, 2002)

The risk with canal bed filtration systems consists of clogging of the water bottom. Frequent removal of the debris layer on the filter was necessary in Ovezande (Figure 3-13) as well as in Waterleidingduinen (AWD). Flotation (regular injection of air bubbles in the sand layer) was tested in Ovezande in 2016, in order to reduce the debris layer continuously from the sand deck. It resulted in higher flow rates. In the Amsterdamse Waterleiding-duinen, an under-water robot was applied to keep the system clean. To our knowledge, that system is no longer in use. The treatment results were good at both locations: the achieved MFI's were between 2 and 5 s/l² (AWD) and approx. 7 s/l² (Ovezande). At OCWD good results were obtained (removal of suspended solids, metals, nitrogen, chlorophyll A and reduced TOC), although also here the filter showed slow clogging.

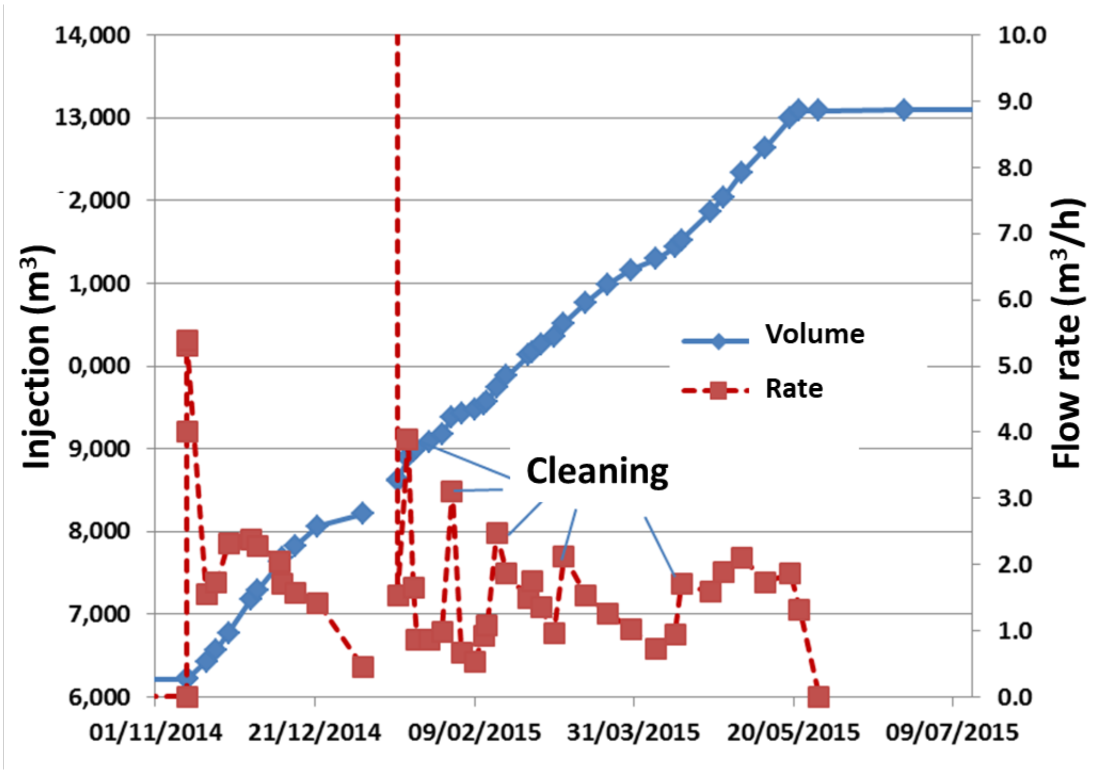


Figure 3-13: Development of the flow rate from canal bed filtration unit in Ovezande in 2014/2015. Injectie = injection, debiet = capacity, schoonvegen toplaag = cleaning.

Table 3-9: Overview of characteristics of canal bed filtration, based on (Stuyfzand et al., 2012)

Aspect	Parameters	Typical values
Capacity		Depending on load and available surface, the capacity can be calculated. Filtration speeds generally comparable to SSF
Surface requirements		Very limited, due to processing in existing bed of canal or river
Purification efficiency	SS	MFI may be reduced to 2-5 s/L^2 (from 10-35 in Waterleidingduinen and to 0.23 NTU (from 0.26 in AWD). Up to 99% reduction of SS (OCWD)
	NO_3	In AWD from 11.3 to 10.7 (mg/l). N-kjeldahl reductie of 99% (OCWD)
	PO_4	No significant reduction (AWD)
	Metals	No significant reduction (AWD), 80-99% reduction (OCWD)
	OMP	No data
Maintenance		Flotation to remove debris layer or yearly cleaning action, for example with mobile submerged robot.
Costs		Costs correspond to the costs for slow sand filtration. This is because the necessary infrastructure is similar. However, money for purchase (or lease) of land can be saved by integration into existing water courses.

3.3.7 Advanced treatment systems

Compared with the more conventional use of self-cleaning filters and slow sand filtration, especially for the removal of suspended matter, advanced solutions are available which have favourable surface requirements and provide a more effective removal of certain substances (Table 3-10). At the same time there are no treatment processes which can simultaneously remove suspended solids, nutrients, OMP, salts, metals and such. Therefore it is necessary to use different technologies. Two possible routes of advanced treatment are considered:

1. Treatment based on Reversed Osmosis (RO, or hyper filtration);
2. Treatment based on adsorption and conversion by advanced oxidation.

For these routes different variations of combined technologies are possible.

In the application of hyper filtration in route 1 it is important to remove as much particles (suspended solids), nutrients and salts beforehand as possible in order to prevent scaling (crystallisation in the membrane module by salts) and biofouling, and to reduce the required maintenance of the membranes. For this purpose ultra-filtration followed by an anion exchange filter can be used. In case only suspended solids needs to be removed a self-cleaning filter can be selected (also see paragraph 3.2) or a cartridge filter. The disadvantage of hyper filtration is the production (and therefore needed treatment or discharge) of a waste flow (brine), consisting of highly concentrated metals, OMP and salts.

The application of advanced oxidation in route 2 is based on conversion followed by adsorption in activated carbon filtration. These techniques have almost no influence on the concentration of salts. The adsorptive material in the ion exchangers (resin) or the activated carbon in the AC filters needs to be regenerated or replaced from time to time, depending on the load of the material. The resin can be regenerated with salt water to flush out the adsorbed nutrients. This generates a salty and nutrient rich waste flow or an effluent with high metals concentration.

In advanced oxidation, ozone or hydrogen peroxide is dosed in combination with UV light. Ozone and the formed oxygen radicals (very reactive oxygen molecules) catalyse the conversion of organic micro pollution. The UV light also provides a log reduction of micro-organisms and viruses by destroying DNA or RNA. Also here a pre-filtration step is necessary to reduce the turbidity of the water and increase the transmission of the UV light. The organic micro pollution is oxidised to metabolites which are largely biologically degradable and collected in the downstream AC filter.

Table 3-10: Global characteristics of different advanced technologies to remove OMP, nutrients and COD. Source: varies studies by KWR

	Purification efficiency					
Technology	**Suspended solids**	**Salts**	**Metals**	**Nutrients and DOC**	**OMP**	**Characteristics and focus points**
Ion-exchange	-	++	++ (cation exchange)	++ (anion exchange)	-	Low investment costs, (relatively) average to high operational costs due to monitoring and regeneration of resin.
Ultra-filtration	++	-	-	-	-	UF membranes can be operated reliably if the membranes are flushed regularly. After some time the membranes will have decreasing capacity (flux) due to pollution and the membranes will have to be replaced in order to maintain the required capacity. The (DOC-rich) concentrate needs to be processed. Operational costs are average and depend on the energy price. Investments costs are average.
Activated carbon	-	-	-	+	++	Depending on the load of the activated carbon, replacement or regeneration is required in order to maintain the adsorption capacity. Activated carbon poorly adsorbs polar compounds.
Reverse osmosis	+	++	++	+	+	Reverse Osmosis is a membrane separation technique which separates pure water from

Technology	Purification efficiency Suspended solids	Salts	Metals	Nutrients and DOC	OMP	Characteristics and focus points
						the feed water. Both mono- and bivalent salts, bacteria and DOC end up in the concentrate stream with a final concentration 2 – 20 times as high as in the feed stream, depending on the feed water characteristics.
Advanced oxidation	-	-	-	+/-	+	Requires continuous dosing of hydrogen peroxide or ozone. High tech equipment is needed and operation is expensive. The advantage is that well designed systems are highly reliable and efficient in conversion of OMP and reduction of micro-organisms.

3.4 Conclusions and recommendations from the desk study

3.4.1 Evaluation and guidelines for the selection of rapid prefiltration techniques

There are different possibilities for rapid pre-filtration prior to ASR. The final choice for pre-filtration depends on the required purification efficiency (depending mainly on the available water source and technology), available space and budget, possible assignments (esthetical reasons) and the available effort in operations and maintenance.

In order to select the best fitting pre-filtration, the following two tables can be used. Table 3-11 shows a qualitative evaluation of the most important characteristics of the different technologies. Table 3-12 compares the way of embedding (natural versus technological) versus the degree of innovation.

Table 3-11: Overview of options for choosing qualitative value estimates for some features of purification technologies

	Treatment efficiency	Surface requirement	Maintenance	Costs
Pre-filter + slow sand filtration	++	-	+	+
Fuzzy filter	++	-	++	+/-
Galileo L filter	+	++	++	+/-
Canal bed filtration	++	+	+/-	++
In-line filtration	--	++	+	++
Pre-filter + helophyte filter (Blue Bloqs)	+/-	-	++	+/-
Advanced treatment	++	++	-	--

Table 3-12: Overview of innovative and proven technologies and their respective environments where these technologies might be feasible

Embedding	Proven technology*	Innovative
Sufficient space with scenic imbedding / social interest	Helophyte filter with pre-filtration or inline filtration	Blue Bloqs
Sufficient space with preference for technological embedding	SSF with pre-filtration step, eventually UV or activated carbon as assurance	-Blue Bloqs with adjustment (UV, Activated carbon) -Canal bed filtration with regular removal of debris layer
Limited space: technological solutions	Advanced treatment*: filtration + hyper filtration, ion exchange, advanced oxidation	Fuzzy filter or Galileo L filter, with downstream advanced treatment step if required

*In certain advanced treatment systems a concentrated waste flow is formed. In hyperfiltration, this concentrate (brine) is produced continuously, in ion exchange the waste flow is formed during regeneration of the resin. The waste flow has to be discharged to the sewer because discharge to surface water / aquifer (mostly) is not permitted

3.4.2 Recommendations pre-treatment storm water to ASR

From Table 3-11 and Table 3-12 different preferences for applied techniques can be derived. In practice, spatial planning and impact key drivers in the area can work in favour of more 'nature-based' solutions such as helophyte filters and canal bed filtration. In the absence of design guidelines it is recommended for the time being to maintain the guidelines (for filtration speed, build-up of the medium and thus the expected treatment efficiency) of SSF.

Technical solutions like backflushing filters can presumably remove sufficient suspended solids. Unfortunately the experience with very fine (<5 micron) filtration of storm water runoff and/or surface water are limited. A combination with a pre-filter (like fast sand filtration) is recommended anyway, in order to reduce the load of suspended solids to the fine filters as much as possible.

In any case: insufficient removal of OMP and/or viruses/bacteria (depending on contemplated water reuse and removal in aquifer) makes a downstream AC filter (OMP) or UV disinfection mandatory.

With the recent emergence of ASR as a solution to bridge seasonal mismatches in freshwater supply and demand and deep infiltration to prevent flooding, several projects have infiltration wells without the ability to backflush ('backwash'). Even with thorough pre-filtration well clogging is a realistic risk as can already be seen in some projects. It is recommended to always include the possibility to backflush an ASR well (and drain the backflush water) as first signs of clogging appear (Pyne, 1995; Van Duivenbode & Olsthoorn, 2002).

4 High-flow pre-treatment of infiltration water

4.1 Introduction

4.1.1 Aims

The aim of this project is to assess the performance of a prototype of a compact, self-cleaning rapid filtration system and develop a set-up for the remote control of such a system within the scope of an aquifer storage and recovery (ASR) system. This system must comply with the current Dutch regulations (see 8.4). Furthermore, it is known from literature that if the system can meet the following criteria, well clogging is not likely to occur (Vries, de la Loma, van der Schans & Zuurbier, 2017):

- Total suspended solids (TSS) < 0.1 mg/L
- Turbidity < 1 Nepholometric Turbidity Units (NTU)
- Total iron < 0.01 mg/L
- Sodium adsorption ratio < 6 at electrical conductivity 40 – 100 mS/m
- Dissolved organic carbon (DOC) < 2 mg/L
- Assimilable organic carbon (AOC) < 10 µg acetate-C/L
- Modified fouling index (MFI) < 3 – 5 s/L^2

4.1.2 Drivers for compact rapid filtration systems

As the world population is steadily growing, demands for water are expected to rise in the future. In order to provide sufficient water, buffer capacity is needed to match supply and demand. Water storage in aquifers can provide this required buffer capacity. A potential water source to buffer is storm water, for instance collected by storm water collection systems, or discharged to surface waters.

Groundwater wells can be interesting for infiltration of storm water, as their spatial impact is limited and they can target favourable (permeable) geological zones (Figure 4-1). One of the challenges is to treat and store large volumes of water in a short time, while preventing well clogging. To prevent flooding and optimize the harvesting of storm water a high rate (in)filtration system has to be developed. For the filtration of storm water collected directly from roofs a simple filter could be sufficient, but for the rapid treatment of large volumes of storm water a space demanding slow sand filtration system is required, resulting in an unacceptable spatial footprint. Moreover, aboveground space is scarce in densely built areas such as cities, so that a filtration system is desired with the following characteristics:

1. with a small spatial footprint
2. allowing high-flow rates to filtrate large volumes of storm water while retaining as much suspended solids as possible
3. without required regular maintenance, like replacement of (cartridge) filters
4. remotely controlled to reduce costs of operation and enable anticipation to (forecasted) weather conditions.

4.1.3 Current reference: no pre-treatment and slow sand filtration

The current standard in the Netherlands is to use (STOWA, 2016):

- virtually no pre-treatment at all: this is often the case in urban areas where infiltration wells are used to dispose of storm water due to a lack of retention or infiltration capacity at surface level. Often however, these wells are quickly clogged and poorly monitored. This is despite existing regulations to protect the groundwater quality;
- rapid sand filtration (RSF) slow sand filtration (SSF). Used for treatment of river water prior to infiltration in the Dune area of The Netherlands (RSF). Commonly used in the greenhouse sector, with the following design characteristics (SSF):

  - Flow velocity <0.5 m/h
  - Grain sizes:
    - 0 – 30 cm: 1.2 – 1.8 mm
    - 30 – 70 cm: 0.8 – 1.25 mm
    - 70 – 130 cm: 0.5 – 1.0 mm

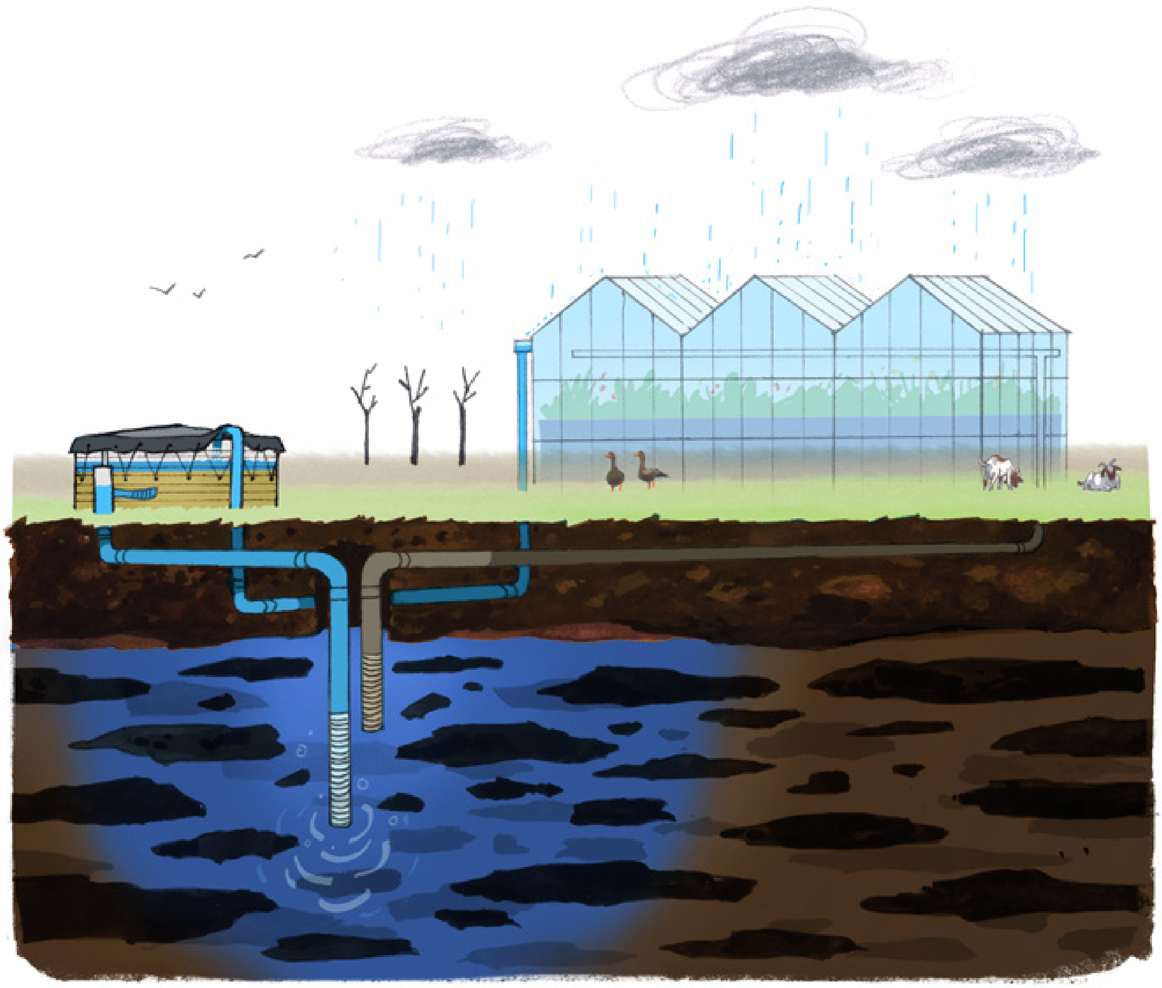


Figure 4-1: Impression ASR using slow sand filtration to treat storm water from greenhouse roofs

4.1.4 Approach

The prototype of a high flow water treatment facility connected to a high flow infiltration system had to feature the following unique properties:

1. capable to handle high rate flows
2. capable to handle a range of water qualities from various sources such as surface water and storm runoff from greenhouses and commercial buildings, and
3. effective in removing at least suspended material in order to prevent clogging of injection wells by particles and contamination of groundwater.

Based on the findings in Chapter 2, the KWR team in collaboration with HYBU composed a prototype of a high flow pre-treatment system composed of the Galileo filter. The Galileo L is a 5-400 micron automatic disc filter. It is aimed to remove coarse (organic) particles on the outside of the disc, while finere particles should be captured between the crossed grooves of the discs. The discs control the extent of filtration; so to adjust the extent of particle removal, different types of disc can be installed.

The perceived advantages of the Galileo L filter are:

1. Limited spatial footprint
2. Minimal loss of water and production during the short (back)flushing
3. No use of chemicals

The following approach (Figure 4-2) was chosen to stepwise test the performance of the Galileo L filter in the AquaNES project. Using this approach, the filtration was gradually brought from controlled lab conditions to (harsh) field conditions. Thereafter, the application of the filtration technology is projected on the Waddinxveen site.

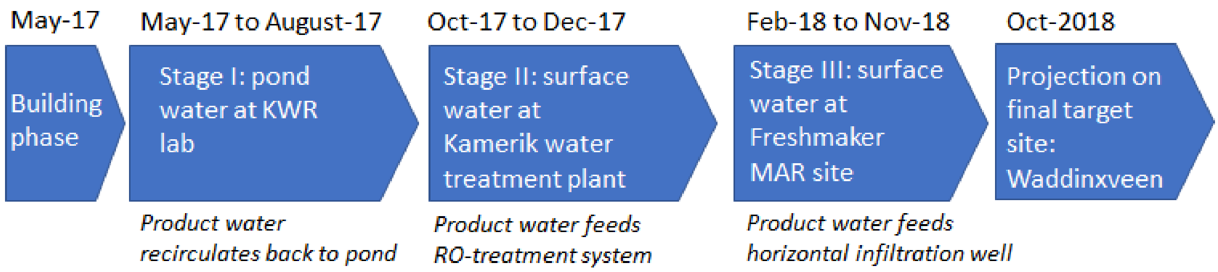


Figure 4-2: Stepwise approach to test the Galileo L filtration system

4.1.5 Innovation exploitation team

Involved organisations:

- KWR Watercycle Research Institute (Research). Design of the total water system, including the Aquifer Storage and Recovery system;
- Hydrobusiness (SME, 'HYBU'). Intended operation of the water system and subsequent owner of the water system;
- UVAR B.V.: supplier of the Galileo treatment system
- Horticulture companies: housing the ASR systems on their plots
- Wayland: Project development and site owner;
- Meeuwse Goes and Fruit Grower Rijk-Boonman: technical support and owners of the Freshmaker system in Ovezande, where the treatment technology is first tested

4.2 Principle of the Galileo L filter

The Galileo L is a 5-400 micron automatic disc filter (see Figure 4-3). The feed water is forced through the filter and then collected as product water. The filter mechanisms itself consists of approximately 150 filter disks squeezed together between a place holder (bottom of filtration cilinder) and a spring (top of filtration cilinder).

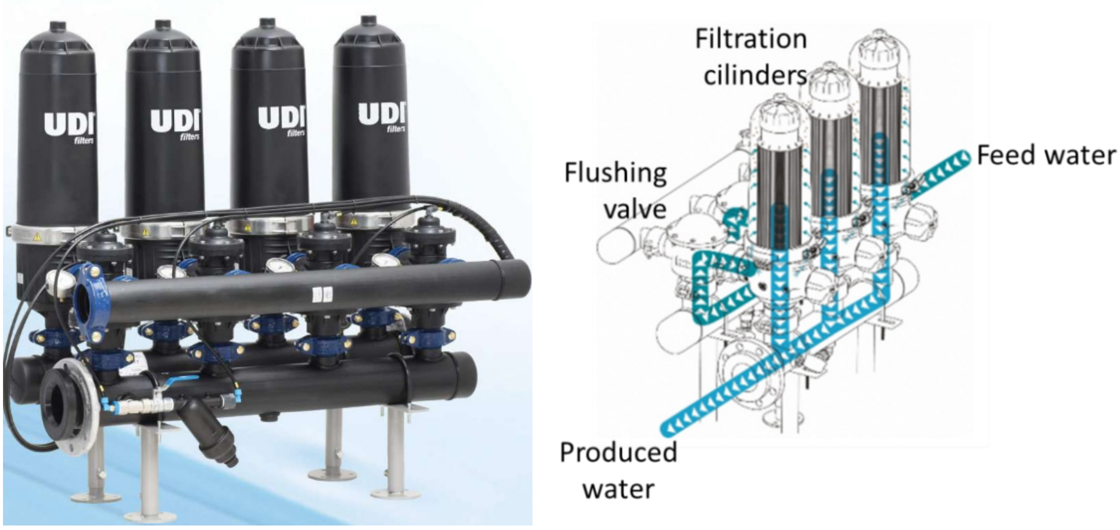


Figure 4-3: Example of Galileo filter set-up (left) and working principle (right)

The grooved plastic filter discs are placed on a holder and compressed during normal operation (see Figure 4-4). This way, the grooves cross and create the filtration mechanism. It is aimed to remove coarse (organic) particles on the outside of the disc, while finere particles should be captured between the crossed grooves of the discs (see Figure 4-5). The discs control the extent of filtration, so to adjust the extent of particle removal, different types of disc can be installed. Feed water is led through the outside of the disc filter under a pressure of 1 to 2 bar. Produced water is transported via the inner part of the filter (the place holder).

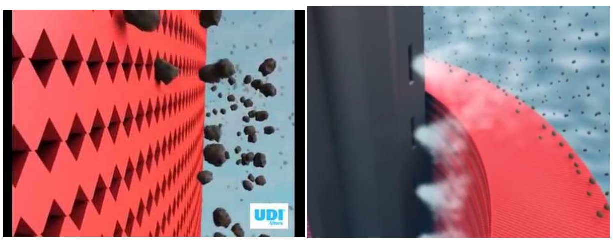


Figure 4-4: Illustration of filtration mechanism (left) and air/water flushing (right) of Galileo filter

Figure 4-5: Compressed 50 micron filter discs on the Galileo L filter

A backflush with a mixture of air (at least 3.5 bar) and water is initiated once a set pressure-difference over the disc filter is exceeded. During the flushing process, the discs are released and the air/water mixture flushes the filter via multi-jet nozzles in the placeholder. The nozzles are placed at an angle, causing the discs to spin around the placeholder. The duration of the flushing process can be adjusted (default: 8 seconds).

The Galileo L filter consists of one filter set with a maximum capacity of around 6 m^3/h and is supplied by UVAR Holland BV (‘s Gravendeel, The Netherlands).

4.3 Description of test locations

The Galileo L filter was tested at three different locations in the Netherlands. Tests were performed at the KWR laboratory in Nieuwegein, at a pilot facility of the drinking water production company Oasen in Kamerik and at an orchard in Ovezande. An overview of these locations is given in Figure 4-6 and detailed location information is given in Table 4-1. Detailed information about the tests performed on each of these locations is given in the following paragraphs.

Figure 4-6: Locations at which the Galileo filter was tested

Table 4-1: **Detailed information of test locations**

Test location	Company / institute	Visiting address	Coordinates
Stage I: KWR laboratory Nieuwegein	KWR watercycle research institute	Groningenhaven 7 3433 PE Nieuwegein Utrecht	B: 52° 1'8.21"N L: 5° 6'30.93"O
Stage II: Pilot facility Kamerik	Pilot facility ‘de Proefhal’ on production location ‘de Hooge Boom’ of Oasen	‘s Gravensloot 36 3471 BP Kamerik Utrecht	B: 52° 5'40.74"N L: 4°51'46.32"O
Stage III: Freshmaker Ovezande	Orchard ‘Rijk-Boonman’	Louisepolderweg 1 4441 SP Ovezande Zeeland	B: 51°26'33.26"N L: 3°48'24.23"O

4.3.1 Stage I: KWR laboratory Nieuwegein

In the first stage, the Galileo L was tested at the KWR laboratory. Here, the performance of the Galileo L was analyzed under controlled conditions, under daily supervision of KWR staff. The targeted water at the KWR lab is the water from the pond next to the laboratory, shown in Figure 4-7. The pond receives overland flow during intense rainfall events and is therefore representative for storm water.

Figure 4-7: Pond at KWR watercycle research institute

As schematically shown in Figure 4-8, the intake of the water was facilitated by a submersible pump in the pond, protected by a screen basket (30 mm spacing), operating with a constant flow rate of approximately 6 m^3/h. The raw intake water was collected in a 600 L buffer tank in the laboratory. This buffer tank was equipped with a high-level overflow which was connected to the pond. This high-level overflow allowed for continious operation of the submersible pump which; 1) stabilized the incoming contaminant load and 2) ensured representative water quality in the buffer tank with respect to pond water throughout the measurement period. From the buffer tank, the water was pumped to a rapid sand filter (RSF) which works as a pre-treatment step for the Galileo L filter. The RSF was backflushed (3 minutes, with water) once exceeding the set maximum pressure difference (0.5 bar). Figure 4-9 shows the main components of the set-up at the KWR laboratory.

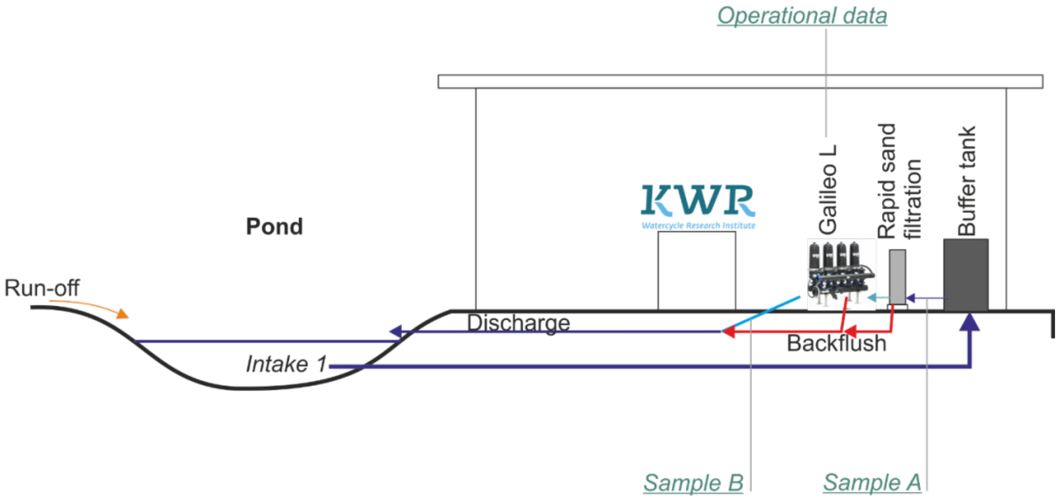


Figure 4-8: Schematic overview of the complete set-up with the Galileo filter

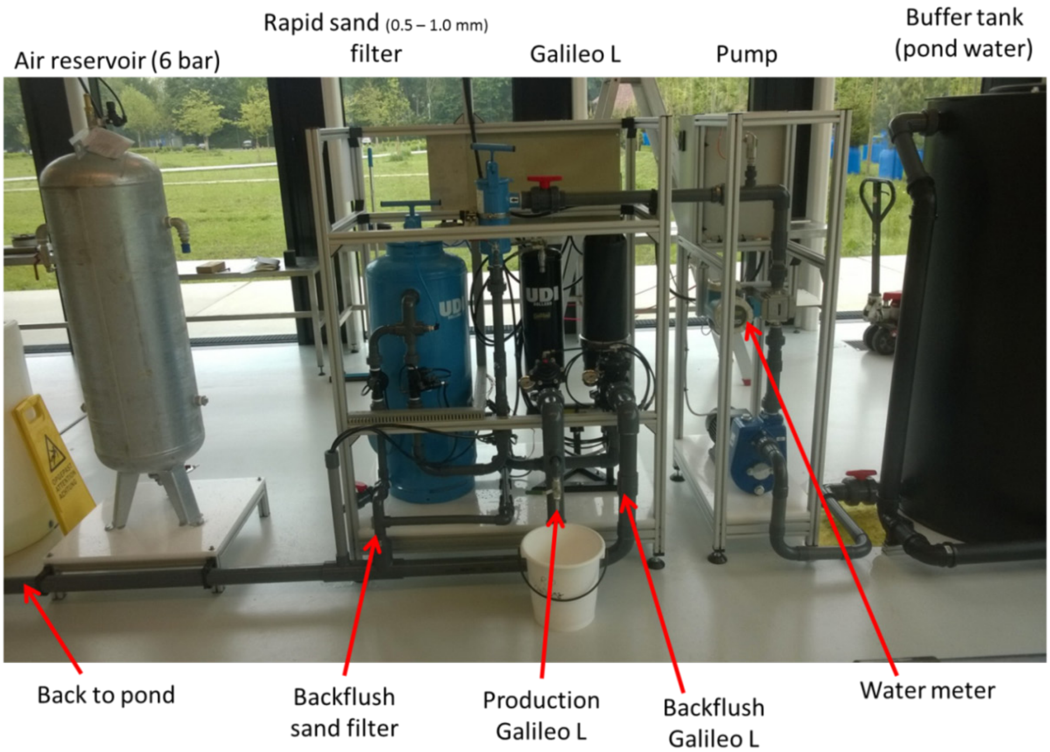


Figure 4-9: Set-up of the filtration experiment in the KWR laboratory

The total operational time of the Galileo L filter trial at KWR laboratory was approximately 2471 hours. In Table 4-2 a detailed overview is provided of these 2471 operational hours, in which each test is linked to a measurement period number. These measurement period numbers will be used in the text below to indicate the corresponding experiments. The variable parameters in each measurement period are disc filter size, capacity, rapid sand filter (RSF) pre-treatment and run time.

Table 4-2: Overview of Galileo L filter configuration during KWR laboratory trial

Measureme	Actual period	Disc filter	Averaged	Pre-	Operational

nt period #	[dd-mm hh:mm 2017]	size [µm]	capacity [m^3/h]	treatment [-]	time [hours]
1	22-5 10:50 - 24-5 16:05	50	5.7	RSF	~53
2a	29-5 9:30 - 6-6 11:58	10	5.0	RSF	~194
2b	8-6 14:05 - 13-6 13:30	10	2.7	RSF	~119
3a	13-6 13:43 - 29-6 10:38	5	1.8	RSF	~381
3b	29-6 10:50 - 10-7 13:07	5	4.1	RSF	~266
3c	10-7 15:54- 11-7 14:07	5	3.5	-	~22
3d	Pond: 11-7 14:46 - 20-7 13:59 Product: 24-7 9:23 - 8-8 13:52	5	2.7	-	Pond water feeding infiltration well: ~200 Product water feeding infiltration well: ~467
3e	10-8 13:32 - 11-9-9:00	5	2.4	-	~763

In the first measurement period, 50 µm filter discs were tested on RSF pre-treated pond water for approximately 53 hours. In order to test to what extent finer particles would clog the disc filters it was decided early on to test the Galileo L using 10-micron disc filters. This second stage was run for approximately 313 hours. In this period, the system had to be shut down shortly once because of a rupture in a coupling piece. This rupture was caused by pressure build-up within the effluent piping during a backflush. At that moment, the effluent pipeline was smothered for sampling purposes. After this incident, the Galileo L filter was re-started at lower capacity (measurement period 2b). In the thirth measurement period, tests were performed with 5 µm filter discs. In period 3a and 3b, the pond water was pre-treated with the RSF before the Galileo L filter. From measurement period 3c – 3e, the RSF was by-passed so that the Galileo L filter was fed directly with pond water. In period 3d both the pond water as well as the filtered pond water (Galileo L filter product) were used to feed scale model infiltration wells.

In order to assess the direct impact of the Galileo L on the water quality for infiltration during managed aquifer recharge (MAR) (with infiltration wells), two infiltration wells were simulated in the lab (Figure 4-10). Therefore, 1-inch HDPE well screens (slot size 0.3 mm) were placed in two 315 mm (transparent) Perspex columns. The columns were backfilled with gravel (1.1 – 1.6 mm) to simulate the gravel pack of an infiltration well. The water is allowed to drain via nozzles at the base of the column.

The elevation of the water in the infiltration wells was recorded every 15 minutes using Solinst pressure transducers and a barometer to correct for air pressure variations. The heads were regularly checked by hand measurements. Because of the low flow and the lack of suitable water meters, the flow was measured manually (using a stopwatch and measure cylinders). Camera inspections were performed to visually assess the condition of the wells.

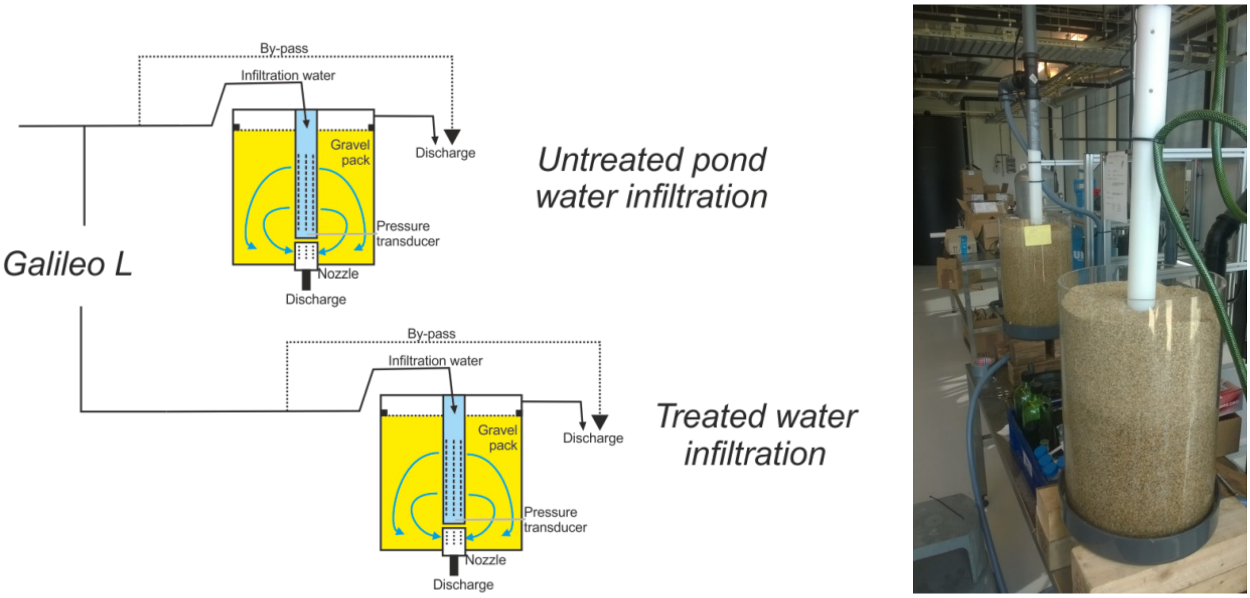


Figure 4-10: Schematic overview of the model infiltration test well (left) and two scale model infiltration wells for evaluation of the clogging potential (right)

4.3.2 Stage II: Pilot facility Kamerik

In the second stage, the Galileo L was tested at 'de Proefhal' on production location 'de Hooge Boom' of drinking water company Oasen. Here, the performance of the Galileo L was analyzed by monitoring the fouling rate of a reverse osmosis (RO) membrane which was fed with Galileo L product water. Since the applied RO membrame consists of tightly packed membrane sheets separated by thin feedspacers (660 µm spacing), it is sensitive to particle fouling (see Figure 4-11), like infiltration wells used during ASR. Therefore, the fouling rate of the RO membrane is a good indicator for clogging of an infiltration well. The RO membrane was fed with 350 liter per hour and produced 60 liter per hour product water (permeate), which are typical values for individual membrane elements in full-scale RO membrane installations. The Galileo L filter was fed by product water of a 25 micron filter, which was in turn fed by the product of a 250 micron filter. The feed water of this 250 micron filter was surface water from 'de Grecht', a small stream situated right next to 'de Proefhal' (see Figure 4-12). Flow velocities in 'de Grecht' are generally low and the main purpose of this stream is to transport water to the Oude Rijn River which discharges into the sea, eventually. During the operational period, which was from 13-11-2017 till 21-12-2017, the Galileo L filter was equipped with 5 µm filter discs. With a regular interval of 10 minutes, or extra when the pressure difference over the filter increased above the setpoint (0.4 bar), the filter was back-flushed. This back-flush consists of 10 liter product water which is flushed together with air at initial six bar pressure from the inside of the sample holder through two bar. On 14-11-2017, ICP-MS analysis of 'de Grecht' water was performed.

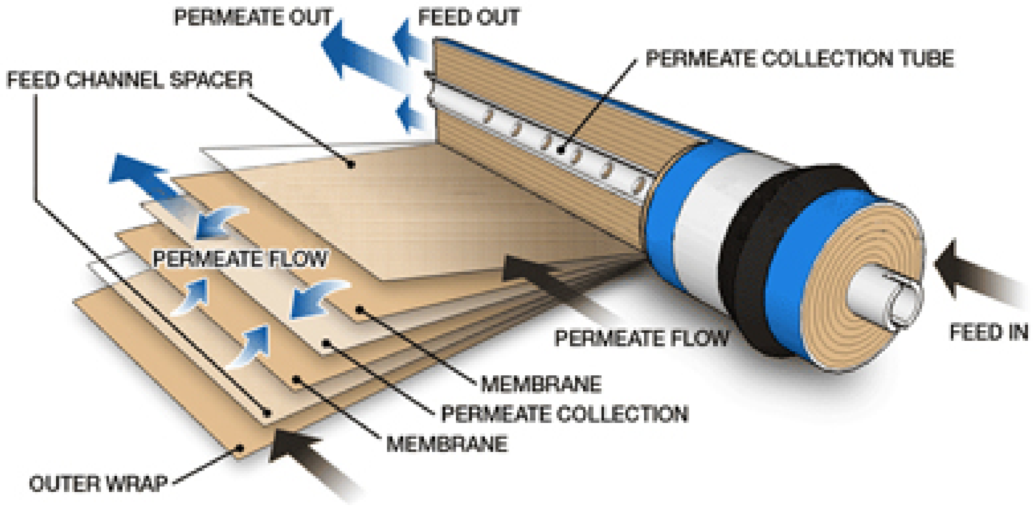


Figure 4-11: Schematic overview of a reverse osmosis membrane

Figure 4-12: Surface water course called 'de Grecht'

4.3.3 Stage III: Testing Galileo filter on field location (Freshmaker Ovezande)

The Galileo filter was finally tested at the special MAR-scheme 'Freshmaker' in the Ovezande area in the southwestern part of the Netherlands (province of Zeeland, Figure 4-13). The MAR location in Ovezande uses Horizontal Direction Drilled Wells (HDDW). In the deeper layer brackish/saline water is abstracted (Figure 4-14). In the shallow layer, fresh water is injected in winter and abstracted in summer. For more information, the reader is referred to (Koen G Zuurbier, Kooiman, Groen, Maas, & Stuyfzand, 2015) and (Koen G. Zuurbier, Raat, Paalman, Oosterhof, & Stuyfzand, 2017).

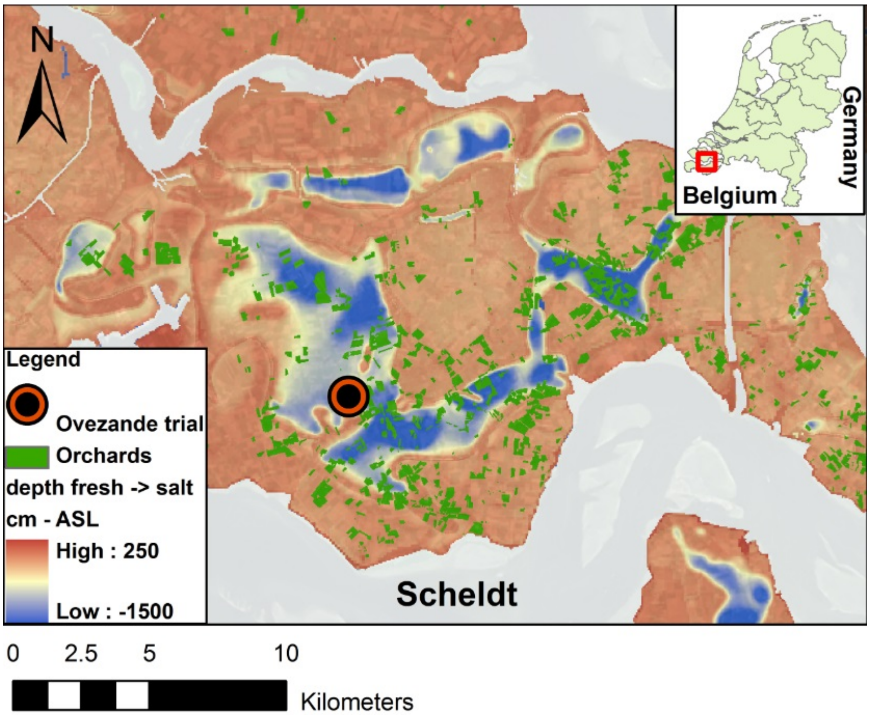


Figure 4-13: Location Freshmaker Ovezande

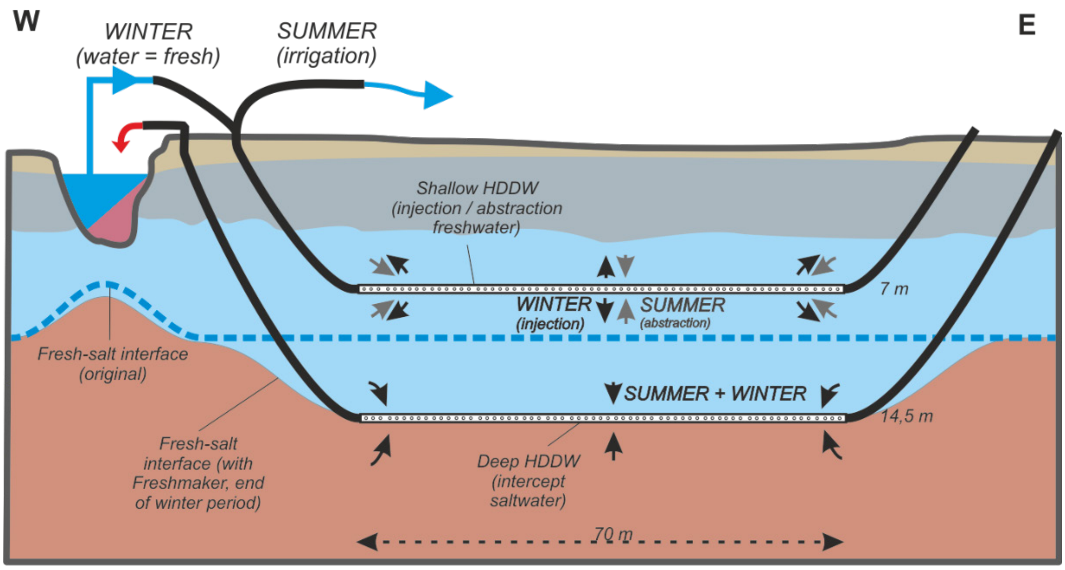


Figure 4-14: Cross section of the Freshmaker in Ovezande

Before, the infiltration water at the Freshmaker site was treated by a large sand filter situated on the river bed of a local creek (river bed filtration, Figure 4-16). Although its spatial claim is negligible as the filter was situated in/below the creek, the regular cleaning of its top layer results in frequent down-time and low capacities. Therefore, this sand filter was by-passed and replaced by the Galileo L filter (Figure 4-15). In order to prevent impact of clogging of the first HDDW used between 2013-2017, a new 80m long horizontal well (125 mm) was installed at 5 m below surface level, at 3m horizontal distance from the earlier HDDW. This was done in August 2017. This well was used for the first time after installing the Galileo L at the site.

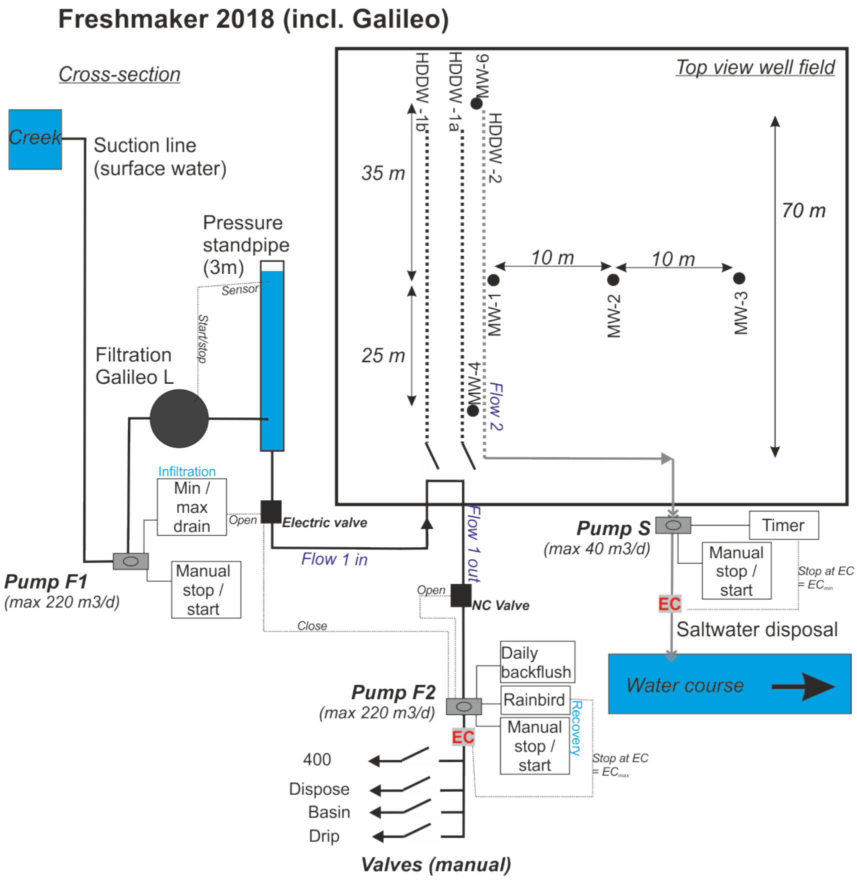


Figure 4-15: Flow scheme at the Freshmaker site after incorporation of the Freshmaker

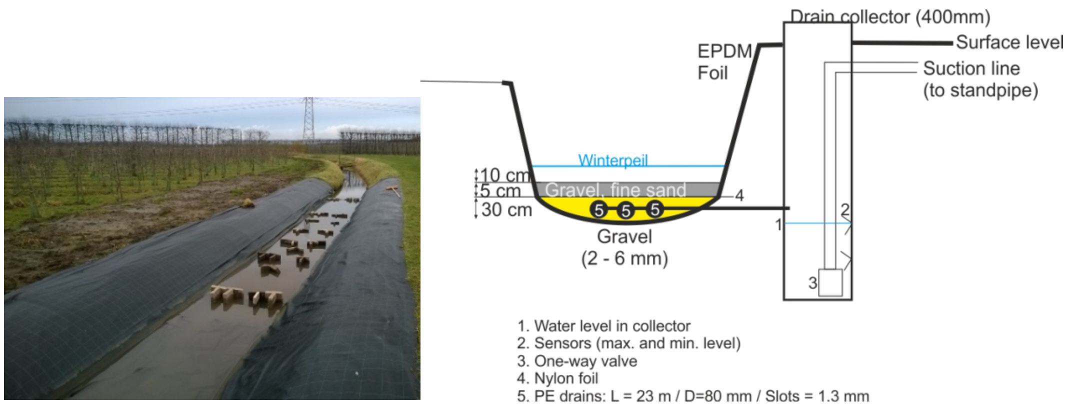


Figure 4-16: Current sand filter ('river bed filtration') at the Freshmaker site

At the Freshmaker site, the conducted research included:

1. Evaluation of the performance of the Galileo L filtration systems (February 2018 – September 2018):
    a. Operational performance (alike Stage I)
    b. Removal of particles based on particle counting and MFI measurements
    c. Chemical analysis

2. Evaluation of well clogging based on:
    a. Monitoring of the injection pressure on the infiltration well with respect to the groundwater levels in the area
    b. Monitoring of the flow rate during injection

4.4 Results & discussion

4.4.1 KWR laboratory test

4.4.1.1 Operational parameters

An overview of the measurement periods of the Galileo L filter (previously described in 4.3.1) with some operational parameters used are given in Table 4-3. Figure 4-17 shows the amount of required backwashes per m^3 of filtered water, to allow direct comparison between the individual measurement periods. This figure shows that both the bigger filter disc sizes (50 µm and 10µm) and rapid sand filtration (RSF) pre-treatment results in low backwashing requirement of the Galileo L filter. Note that during measurement period 3b heavy rainfall caused a steep increase in backwash frequency. This steep increase decreased steadily again after the rainfall event.

Table 4-3: Operational parameters of the Galileo L filter in each measurement period

Measurement period #	Average feed pressure [bar]	Averaged capacity [m^3/h]	Volume treated pond water [m^3]	Pressure difference over filter [bar]	Number of Galileo L backwash cycles[-]
1	1.2	5.7	305	n.a.	4
2a	1.3	5.0	970	0.22	63
2b	1.7	2.7	318	0.20	14
3a	1.8	1.8	689	0.21	37
3b	1.3	4.1	1091	0.31	3182
3c	1.3	3.5	77	0.35	237
3d	1.1	2.7	816	0.37	3976
	1.3	2.7	964	0.40	8488
3e	1.5	2.4	1851	0.44	10332

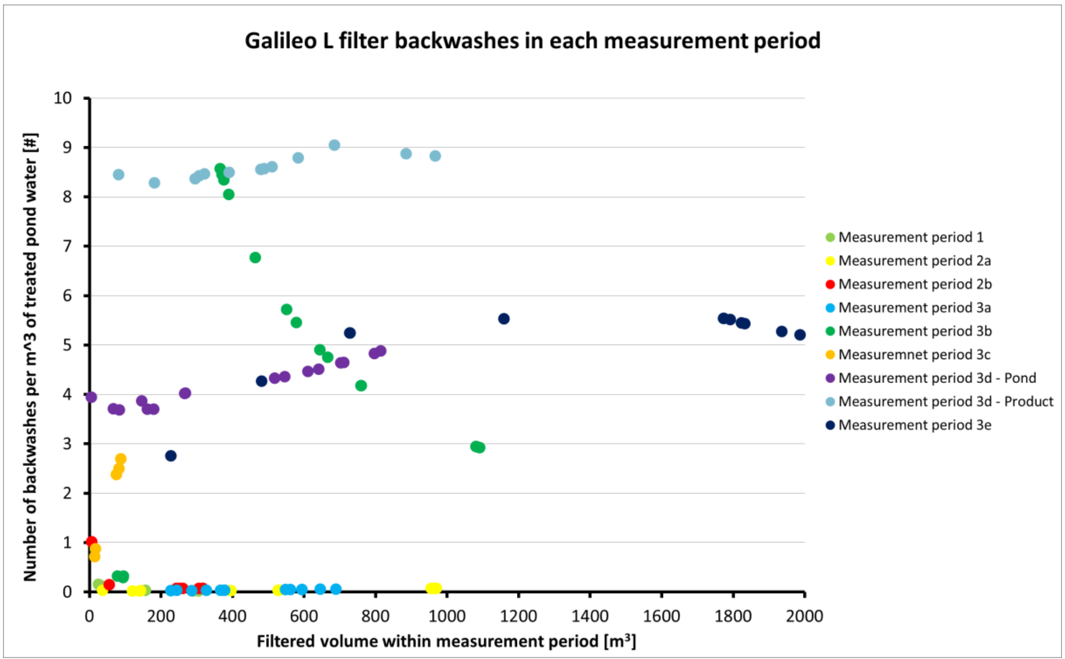


Figure 4-17: Backwash frequency of the Galileo L filter during individual measurement periods

When comparing backwash frequency of the Galileo L filter equipped with 5 µm filter discs with (measurement period 3a) and without (measurement period 3e) rapid sand filtration (RSF) pre-treatment, it is clear more backwashing was required without RSF pre-treatment (see Figure 4-18). As water quality does change with time, it cannot be stated that this decrease in required backwashing is caused by the RSF pre-treatment. However, it is a strong indication that RSF pre-treatment of Galileo L filter feed water does change the water quality in such a way that less backwashing is required. Note that without RSF pre-treatment the Galileo L filter still works, but requires more frequent backwashing (as one would expect).

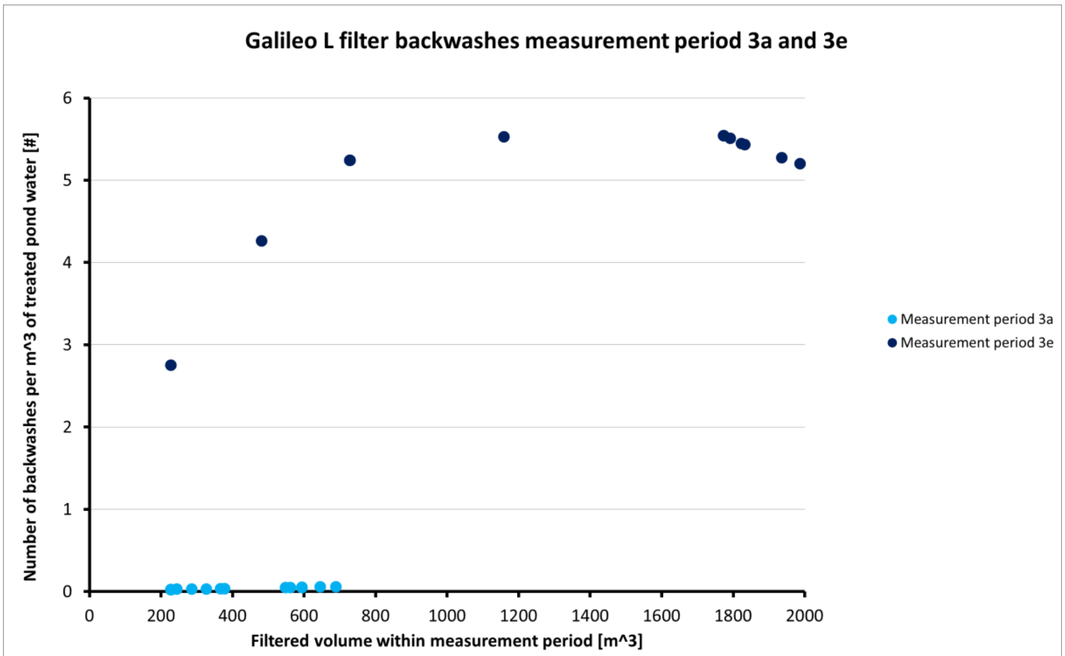


Figure 4-18: Backwash frequency of the Galileo L filter during measurement periods 3a and 3e

4.4.1.2 Characterisation of the feed water

Results of all the ICP-MS analyses of the feed- and product water are given in Table 4-4. As one would expect, the Galileo L filter does not seem to have any influence on the concentrations of the listed elements at all. The smallest mesh size used in the Galileo L filter (5 µm) is approximately a factor 10^9 larger compared to the atomic radius of these elements. However, a biologically active layer can develop on the filter disc rings and, then, adsorb some of these elements. As the concentration of the elements present in the KWR pond do change with time, no conclusion can be drawn with respect to biological uptake of elements from the feedwater.

Table 4-4: Results of ICP-MS analyses of each individual sample (5 rounds: Feed and Product). Reported concentrations are after destruction of the sample with nitric acid. Unfiltered samples.

		31254	31255	32140	32141	33099	33100	35523	35524	36252	36253
		LMC-31256-GW	LMC-31257-GW	LMC-32142-GW	LMC-32143-GW	LMC-33101-GW	LMC-33102-GW	LMC-35525-OW	LMC-35526-OW	LMC-36279-GW	LMC-36280-GW
		Measurement period 1	Measurement period 1	Measurement period 2b	Measurement period 2b	Measurement period 3a	Measurement period 3a	Measurement period 3d	Measurement period 3d	Measurement period 3d	Measurement period 3d
		23-05-2017	23-05-2017	08-06-2017	08-06-2017	21-06-2017	21-06-2017	20-07-2017	20-07-2017	02-08-2017	02-08-2017
		Feed	Product	Feed	Product	Feed	Product	Feed	Product	Feed	Product
Al	µg/l	24	32	< 10	< 10	76	30	< 10	< 10	< 10	20
As	µg/l	< 1.0	< 1.0	2.7	2.8	3.0	2.8	1.6	2.1	1.8	1.4
Ba	µg/l	51	50	56	63	46	51	49	50	53	53
Ca	mg/l	105	105	100	100	100	105	98	97	97	97
Co	µg/l	< 0.40	< 0.40	< 0.40	< 0.40	< 0.40	< 0.40	< 0.40	< 0.40	< 0.40	< 0.40
Cr	µg/l	< 2.0	< 2.0	< 2.0	< 2.0	< 2.0	< 2.0	< 2.0	< 2.0	< 2.0	< 2.0
Fe	mg/l	0.02	0.03	0.04	0.01	0.03	0.04	0.03	0.04	0.04	0.04
K	mg/l	< 2.0	< 2.0	0.82	0.78	0.62	0.64	< 2.0	< 2.0	< 2.0	< 2.0
Mg	mg/l	12	12	11	12	12	12	12	12	12	12
Mn	µg/l	23	22	39	7.3	12	3.2	87	70	85	69
Na	mg/l	16	16	14	15	15	16	17	17	17	17
Ni	µg/l	2.0	2.0	2.1	< 2.0	3.0	2.4	< 2.0	< 2.0	< 2.0	< 2.0
Si	mg/l	< 10	< 10	< 10	< 10	< 10	< 10	< 10	< 10	< 10	< 10
Zn	µg/l	5.0	13	< 2.0	< 2.0	19	31	< 2.0	< 2.0	< 2.0	< 2.0

Dissolved-, total- and assimilable organic carbon (DOC, TOC and AOC, respectively) analysis of the feed- and product water of the Galileo L filter have been performed by the KWR laboratory following standard measurement methods. These parameters are indicators for the clogging potential by organic contaminants. The concentration of dissolved and total organic carbon measured in the KWR pond water is given in Table 4-5. The measured concentrations of assimilable organic carbon (AOC) are given in Table 4-6. These measurements show that DOC, TOC and AOC concentrations do not fluctuate significantly within the test period. This is a strong indication that observed differences in, for instance, backwash frequency of the Galileo L filter in different measurement periods are not a result of differences in biological clogging potential of the pond water.

Table 4-5: Results of DOC and TOC analyses at the KWR-lab

		31254	31255	32140	32141	33099	33100	35523	35524
		LMC-31256-GW	LMC-31257-GW	LMC-32142-GW	LMC-32143-GW	LMC-33101-GW	LMC-33102-GW	LMC-35525-OW	LMC-35526-OW
		Measurement period 1	Measurement period 1	Measurement period 2b	Measurement period 2b	Measurement period 3a	Measurement period 3a	Measurement period 3d	Measurement period 3d
		23-05-2107	23-05-2017	08-06-2017	08-06-2017	21-06-2017	21-06-2017	20-07-2017	20-07-2017
		Feed	Product	Feed	Product	Feed	Product	Feed	Product
DOC	mg C/l	6.5		6.9		7.2		8.1	
TOC	mg C/l	6.9	6.9	7.3	7.1	7.2	7.0	8.0	7.9

Table 4-6: Result of AOC analyses at the KWR-lab

Date	Sample	KWR number	AOC Stem P17		AOC Stem Nox		AOC Total	
			µg Ac-C/l	stdv	µg Ac-C/l	stdv	µg Ac-C/l	stdv
3-7-2017	Feed	34047	67	20	8.5	1.7	76	14
20-7-2017	Feed	35480	60	0.92	11	0.82	72	0.87

4.4.1.3 MFI-SDI

The modified fouling index (MFI) and silt density index (SDI) are both indicators for fouling potential of the feed water. MFI and SDI measurements are generally accepted as indicative for membrane fouling by particulate and/or colloidal matter. In each measurement period, MFI and SDI values were determined once or multiple times. Averaged values of these measurements are shown in Table 4-7. Except for measurement period 1, 2a and 3d, MFI values of the product water of the Galileo L filter are lower compared to untreated feed water in the same period. SDI+ values (when successfully measured) are similar for feed and product water. In the product water, more often SDI5, SDI10 and SDI15 values could be measured, which indicates the product water of the Galileo L filter has less fouling potential compared to the feed water. As all these values are indicative and are not only dependent on feed water quality (but also test filter uniformity, air bubbles, etc), no direct conclusions can be drawn. However, the large part of the measurements performed indicate that the Galileo L filter seems to reduce the fouling potential of the feed water.

Table 4-7: Averaged MFI/SDI measurements in each of the measurement periods

Measure-ment period	Number of measure-ments	Feed [average]					Product [average]				
#	#	MFI	SDI5	SDI10	SDI15	SDI+	MFI	SDI5	SDI10	SDI15	SDI+
1	2	16.0	16.8	n/a	n/a	5.2	18.9	11.3	9.2	n/a	5.1
2a	11	15.1	17.7	n/a	n/a	4.9	21.8	17.3	n/a	n/a	5.3
2b	2	28.1	16.7	n/a	n/a	5.5	10.9	13.7	8.2	5.8	5.0
3a	8	19.6	17.3	n/a	n/a	5.2	16.8	15.6	8.6	4.5	5,0
3b	6	37.8	16.2	9.1	n/a	5.2	13.5	15.6	8.9	5.4	5.0
3c	0	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
3d – pond water	3	43.8	n/a	n/a	n/a	5.5	17.7	17.6	n/a	n/a	5.3
3d – product water	1	47.8	n/a	n/a	n/a	5.6	136.0	n/a	n/a	n/a	n/a
3e	2	49.1	n/a	n/a	n/a	5.6	16.6	n/a	n/a	n/a	4.3

4.4.1.4 Gravel column experiment

The clogging rate of the two model-sized infiltration wells was deducted by comparing the infiltration capacity (in m^3/h) to the head (i.e. pressure) increase (in meters) in each well (Figure 4-19). The feed column experiment was done from 11 July 2017 until 20 July 2017. Because air inflow influenced the clogging rate of the product infiltration well, the experiment of the product column was restarted (see the blue sharp peak in Figure 4-19). Therefore, the maximum infiltration rate of the product column was lower when the experiment was done for the second time (period: 24 July 2017 until 8 August 2017). To better compare the clogging rates of the feed and product column, the values have been normalized to their initial saturated flow rates of 21.4 m^3/h and 8.57 m^3/h, respectively (Figure 4-20). The spikes in the graphs indicate restarts of the experiments. The capacity of the feed column decreased within 3 days to less than 5% of the initial flow, whereas the product column decreased to approximately 30% of the initial capacity. However, this was achieved in the period where the product column received air inflow. Comparing both the feed column and product column over the whole period, it can be concluded that the product water fed column clogged less rapidly compared to the untreated feed water fed column. The product water fed column did decrease in capacity continuously during the measurement period and therefore did not reach a 'stable state' where the capacity remained more or less constant. This indicates that filtering feed water using the Galileo L equipped with 5µm filter discs is not sufficient to completely prevent infiltration well clogging when infiltrating the pond water. When both columns were dismantled, visual camera inspection was performed. Biological growth was found as a potential mechanism for clogging of the wells (Figure 4-21). It was found that this occurred on both the slots of the well screen and in the gravel pack.

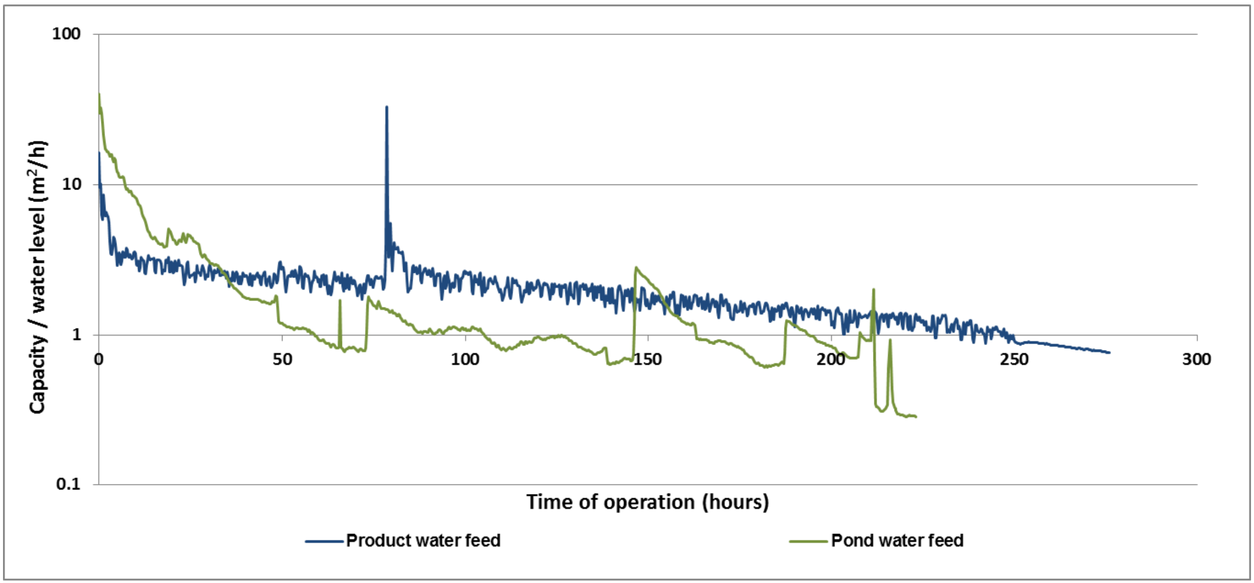


Figure 4-19: **Well capacity divided by the water level (m3/h/m or m2/h) in the Feed and Product columns over time. The Feed column experiment was done from 11 July 2017 until 20 July 2017. the Product column experiment was done from 24 July 2017 until 8 August 2017**

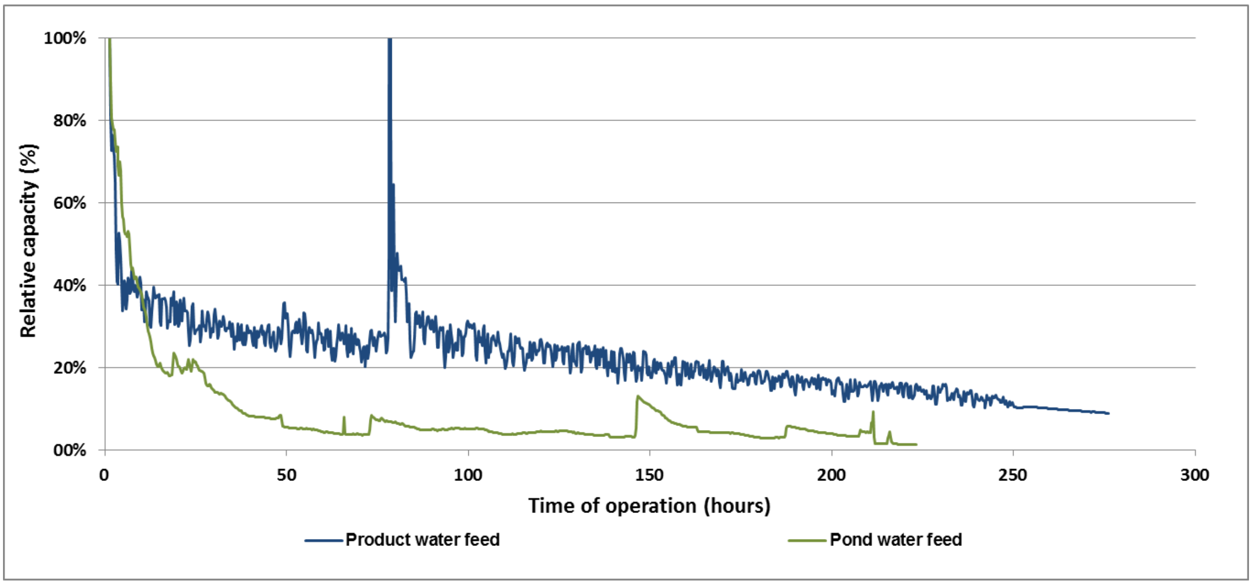


Figure 4-20: **Capacity versus head relative to the initial (saturated) well capacity (in%) for the Feed and Product column. The Feed column experiment was done from 11 July 2017 until 20 July 2017. The Product column experiment was done from 24 July 2017 until 8 August 2017.**

Figure 4-21: **Camera inspection result of the well infiltrating the Galileo product water. before (left) and after (right) removing the well screen.**

4.4.1.5 Implications of Stage I for pre-treatment of infiltration water

The column experiments showed that the model column which was fed by (Product) water treated by the Galileo L (using a 5-micron disc filter) obtained an overall higher relative infiltration capacity and a more gentle decrease in well capacity. The build-up of the head (water level) in the product water fed model column was also much less significant. However, as the decrease did not reach a 'stable state', pre-treatment of KWR pond water with the Galileo L filter did only delay the complete clogging of the model infiltration well. This positive effect is supported by the MFI-SDI measurements, which also indicate a decrease in fouling potential of the Galileo L product water. In addition, the Galileo L filter showed to be able to maintain stable operation when fed with organics-containing feed water, where biological clogging significantly affected the operation of the model infiltration wells. The Galileo L filter does not seem to alter feed water's chemical characteristics (dissolved metals, organics) other than particulate and colloidal matter, as was not expected based on the mesh size of 5 µm. The bigger mesh sized filter discs (50 µm and 10 µm) do not seem to be suitable as pre-treatment for infiltration applications since 1) no increase in backwash frequency of the Galileo L filter was observed when fed with rapid sand filter (RSF) treated KWR pond water and 2) the capacity of the infiltration well decreased continuously when fed with 5µm pre-treated KWR pond water. The steep increase in backwash frequency in measurement period 3b caused by heavy rainfall does indicate the protective function of the Galileo L filter and the ability to deal with high particle load / robustness of the system.

4.4.2 Oasen Kamerik pilot test

4.4.2.1 Operational parameters

Averaged over 912 operational hours, the average capacity of the Galileo L filter during the test period was 2.22 m3 per hour. Figure 4-22 shows the number of back-flushes of the Galileo L filter in the test period at 'de Proefhal' in Kamerik (13-11 2017 till 21-12-2017). As the regular interval for back-flushing was set at ten minutes (see 4.3.2), a minimum of six back-flushes per hour were expected. The calculated number of backflushes per hour was for the complete test period stable at six back-flushes per hour. Therefore, the particle load on the Galileo L filter was not enough to raise the pressure drop over the filter unit above 0.4 bar within 10 minutes, as number of hourly flushes would increase to more than six back-flushes per hour.

During the test period, 5234 back-flushes were performed and 2027 m3 'de Grecht' surface water has been filtered. Based on 10 liter water consumption per backflush performed (Personal communication Sjirk Idzenga – UVAR Holland B.V.), 2.6% (52.34 m3) of the water fed to the filter was disposed of due to the automatic flushing. In practice, this water loss can be further reduced by optimisation of the automatic flushing interval. Note that an increase in filter-specific capacity (maximum is 6 m3 per hour per filter element) can affect the number of required back-flushing as the particle load increases with increasing capacity.

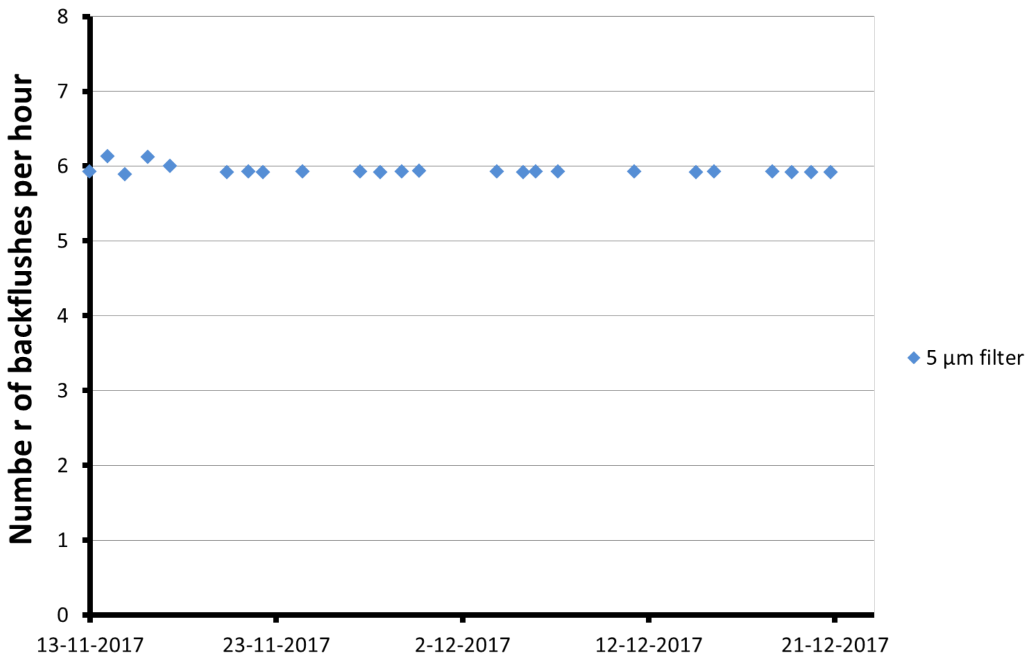


Figure 4-22: **Number of back-flushes of Galileo L filter during test period (13-11 2017 till 21-12-2017)**

4.4.2.2 Characterisation of the feed water

At the start of the operational period of the Galileo L filter (14-11-2017), samples of 'de Grecht' surface water treated by the 250 µm filter were taken for ICP-MS and dissolved- and total organic carbon analysis. The ICP-MS results are shown in Table 4-8. Compared to the KWR pond water (described in 4.4.1.2), total organic carbon (TOC) levels were significantly higher; 31.6 mg/L C. This organic carbon was further characterized by DOC Labor in Germany. All carbon present in the sample (dissolved organic carbon, DOC; 30.89 mg/L) is further specified into two main categories: hydrophobic organic carbon (HOC) and chromomorphic dissolved organic carbon (CDOC) (see 8.2). The hydrophobic organic carbon showed significant removal by the Galileo L filter (see Figure 4-23), for which the authors do not have an explanation. Since HOC is only a small fraction (<3%) of the total amount of organic carbon, this is not expected to have a significant influence on the biological fouling potential of the Galileo L product water.

Table 4-8: Chemical analysis results of surface water 'de Grecht' (after 250 µm filter)

Parameter	Unit	14-11-2017
Aluminium	mg/L Al	0.11
Barium	mg/L Ba	0.05
Bicarbonate	mg/L HCO_3	157
Boron	mg/L B	0.07
Calcium	mg/L Ca	61
Chloride	mg/L Cl	51
Iron	mg/L Fe	0.50
Magnesium	mg/L Mg	14.9
Manganese	mg/L Mn	0.08
Nitrate	mg /L NO_3	3.1
Potassium	mg/L K	15.0
Silicon	mg/L Si	1.42
Sodium	mg/L Na	35
Strontium	mg/L Sr	0.31
Sulphate	mg/L SO_4	74

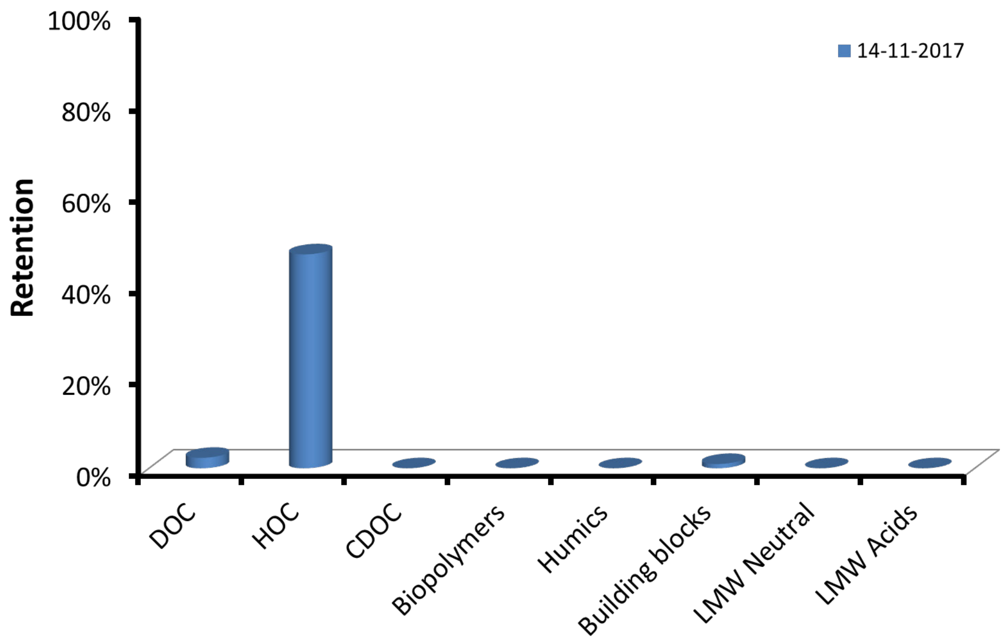


Figure 4-23: Retention of various organic compounds by the Galileo L filter

4.4.2.3 Model reverse osmosis membrane system performance

In Figure 4-24 the normalized pressure drop ($NPD-NPD_0$) in time of a reverse osmosis (RO) membrane fed by Galileo L product water is shown. The black dots show the normalized pressure drop, red dots indicate clean in place (CIP) procedure of the RO membrane. An increase in pressure drop is a direct indicator for fouling of the feed spacer, which separates the membrane sheets. Amongst other types of fouling with different characteristic fouling profiles, particle fouling shows typically a strong and linear increase in pressure drop as the feed spacer becomes increasingly clogged with particles. In the case of the RO membrane which was fed by the Galileo L product water, no such linear and strong increase in pressure drop was observed. After each CIP procedure, the pressure drop restores to the initial value, meaning that all feed spacer fouling was effectively removed. After approximately 16 operational days the pressure drop reached approximately 0.12 bar and the experiment with the RO membrane had to be stopped due to membrane permeability reasons. Because a decrease in membrane permeability is more strongly related to (a combination of) organic fouling, scaling and/or biofouling (but less strongly to particle fouling), this test gave a strong indication that the Galileo L filter had a positive influence on the particle fouling potential of the RO membrane feed water.

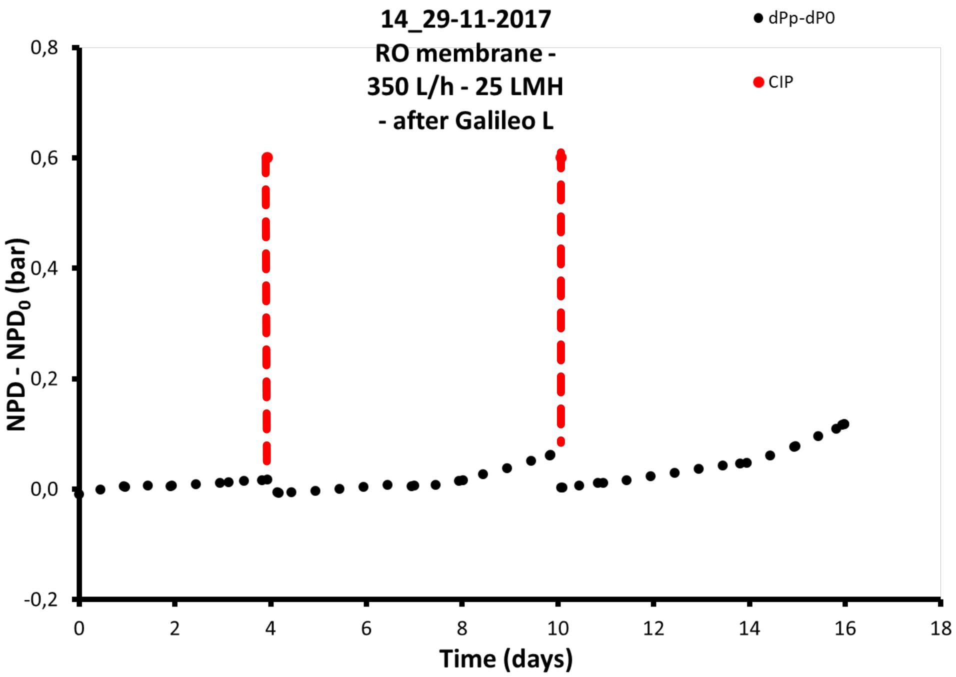


Figure 4-24: Normalized pressure drop of RO membrane fed with Galileo L product water. The two red lines indicate clean in place (CIP) procedures in which the RO membrane is chemically cleaned

4.4.2.4 Implications for pre-treatment of infiltration water

The observation that the RO membrane did not show fouling characteristics which are typical for particle fouling matches with the observations made in the KWR laboratory tests. The fouling that did occur on the RO membrane surface can be attributed to (a combination of) organic fouling, scaling, and/or biofouling. This observation is supported by DOC measurements before and after the Galileo L filter, which show hardly no DOC removal. The type of fouling of the RO membrane surface was not visually inspected but is likely to resemble the (biological/organic) fouling of the model infiltration wells. Since DOC is not removed significantly by the Galileo L filter, the nutrients for biological activity remain in the water, causing organic and biofouling to develop in membrane systems and/or infiltration wells. The Galileo L filter has shown to be able to operate for 38 days with 2.22 m^3 per hour capacity on surface water from 'de Grecht' (as described in 4.4.2.2) which was pre-treated successively by a 250 and a 25 micron self-cleaning filter.

4.4.3 Ovezande field test

4.4.3.1 Flushing, volume treated water & process parameters

In Ovezande, two separate wells are available for runoff fed surface water infiltration. In the period of 1-10-2017 till 23-2-2018, surface water was directly infiltrated into one of the wells. In this period, a total of 1439 m^3 of water was infiltrated. Then, the Galileo L filter was installed and filtered surface water was infiltrated from 23-2-2018 till 7-5-2018 in the other (still unused, so assumed clean) infiltration well. In this period, a total of 5090 m^3 filtered surface water was infiltrated. The Galileo L filter was set at a capacity of 4 m^3/h with a backflush interval of 30 minutes. In case the pressure difference over the filter exceeded 0.5 bar, an extra backflush was performed automatically. On average, the Galileo performed 4,1 back-flushes per hour. This means that the pressure drop reached above 0,5 bar during the 30 minute filtration cycle regularly. The groundwater well was back-flushed once every 6 hours for 15 minutes. The maximum injection pressure was 330 cm water column. Figure 4-25 shows the injection pressure and injection rate in the period of 26-2-2018 till 3-5-2018 for the Galileo L filter treated surface water in the unused infiltration well. Dips in the injection pressure are caused by the regular infiltration well backflush and the dip between 12-3-2018 and 14-3-2018 was caused by clogging of the feed water intake; enlarging the intake filter pore size solved the problem of clogging, while the Galileo L filter stayed operational.

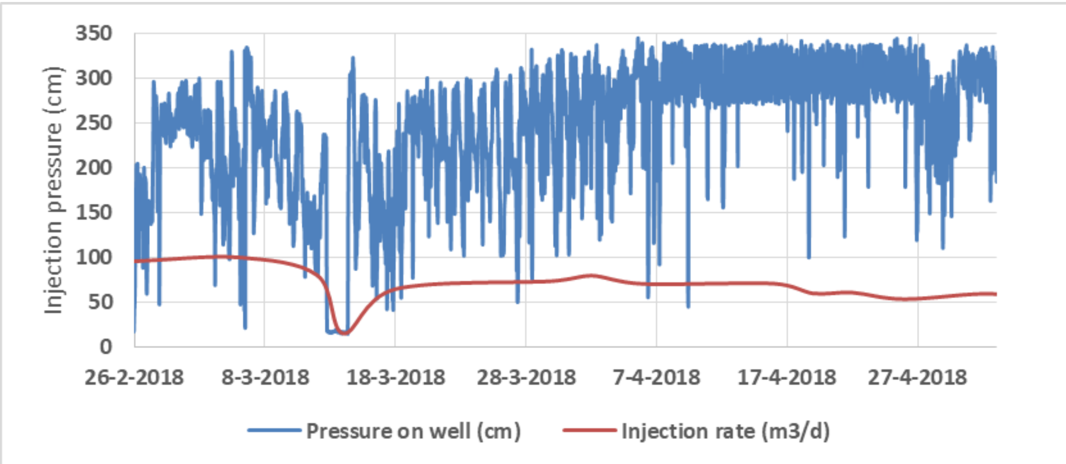


Figure 4-25: Pressure on infiltration well (blue line) and injection rate (red line) in the period 26-2-2018 to 3-5-2018. Sharp variations in injection pressure are caused by regular infiltration well backwash. From 12-3-2018 till 14-3-2018 the installation was stopped due to clogging of the feed water intake

Figure 4-26 shows the specific injection capacity of the same infiltration well in the same period (26-2-2018 till 3-5-2018). Within this period, a general decreasing trend can be observed to roughly 50% of the initial specific injection capacity. This shows that, even though the surface water was treated by the Galileo L filter prior to infiltration, significant well clogging remained.

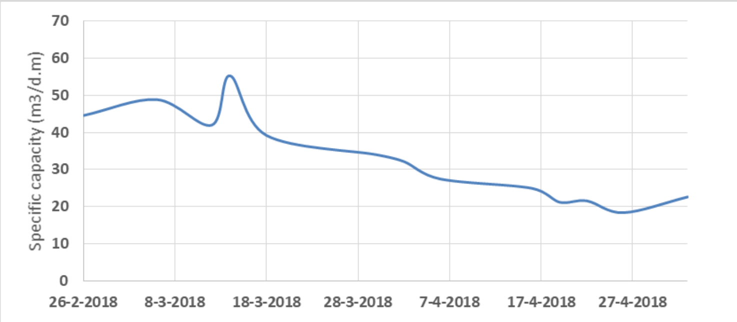


Figure 4-26: Specific injection capacity of the infiltration well fed with Galileo L filter product in the period of 26-2-2018 to 3-5-2018. Not corrected for temperature.

4.4.3.2 Characterisation of the feed water

During the Ovezande field test, water quality of both Galileo L filter feed and effluent (see Table 4-9) was monitored on several occasions. These results show that filtration by the Galileo L filter does not alter the concentration of chloride, dissolved oxygen and/or TOC levels. The total suspended solids concentration decreases with 1 mg/L approximately, indicating some particles are retained by the Galileo L filter.

Table 4-9: Galileo L filter water quality analysis results within the experimental period

Parameter	Analysis result feed	effluent	Unit	Date
Acenaftene	<0.005	<0.005	µg/L	18-4-2018
Antracene	<0.005	<0.005	µg/L	18-4-2018
AOC	87±3.2	n.a.	µg Ac-C/L	14-3-2018
Benzo(a)antracene	<0.005	<0.005	µg/L	18-4-2018
Benzo(a)pyrene	<0.005	<0.005	µg/L	18-4-2018
Benzo(b)fluoranthene	<0.005	<0.005	µg/L	18-4-2018
Benzo(ghi)perylene	<0.005	<0.005	µg/L	18-4-2018
Benzo(k)fluoranthene	<0.005	<0.005	µg/L	18-4-2018
Bicarbonate	530	530	mg/l HCO3	18-4-2018
Chloride	72	72	mg/L Cl	18-4-2018
Chrysene	<0.005	<0.005	µg/L	18-4-2018
Dibenzo(A,h)antracene	<0.005	<0.005	µg/L	18-4-2018
Dissolved oxygen	6.2	8.4	mg/L O2	1-5-2018
	10.2	10.5	mg/L O2	23-5-2018
DOC	8.8	n.a.	mg/L C	18-4-2018
E.C. (20°C)	75	75	mS/m	1-5-2018
	97	97	mS/m	23-5-2018
Fenantrene	0.013	0.019	µg/L	18-4-2018
Fluoranthene	0.0050	0.0089	µg/L	18-4-2018
Fluorene	<0.005	<0.005	µg/L	18-4-2018
Indeno(1,2,3-cd)pyrene	<0.005	<0.005	µg/L	18-4-2018
Naftalene	<0.05	<0.05	µg/L	18-4-2018

Parameter	Analysis result feed	effluent	Unit	Date
pH	7.62	7.65	-	18-4-2018
	7.36	7.42	-	1-5-2018
	8.05	8.07	-	23-5-2018
Pyrene	<0.005	<0.005	µg/L	18-4-2018
Sum PAC's	0.038	0.048	µg/L	18-4-2018
Sum PAC's (Borneff)	0.018	0.021	µg/L	18-4-2018
Sum PAC's (EPA)	0.073	0.083	µg/L	18-4-2018
Temperature	18.5	18.9	°C	18-4-2018
	15.2	15.3	°C	1-5-2018
	17.8	18.1	°C	23-5-2018
TOC	8.4	8.0	mg/L C	18-4-2018
	8.0	7.7	mg/L C	1-5-2018
	12	10	mg/L C	23-5-2018
TSS	12	11	mg/L	1-5-2018
	4.9	3.2	mg/L	23-5-2018

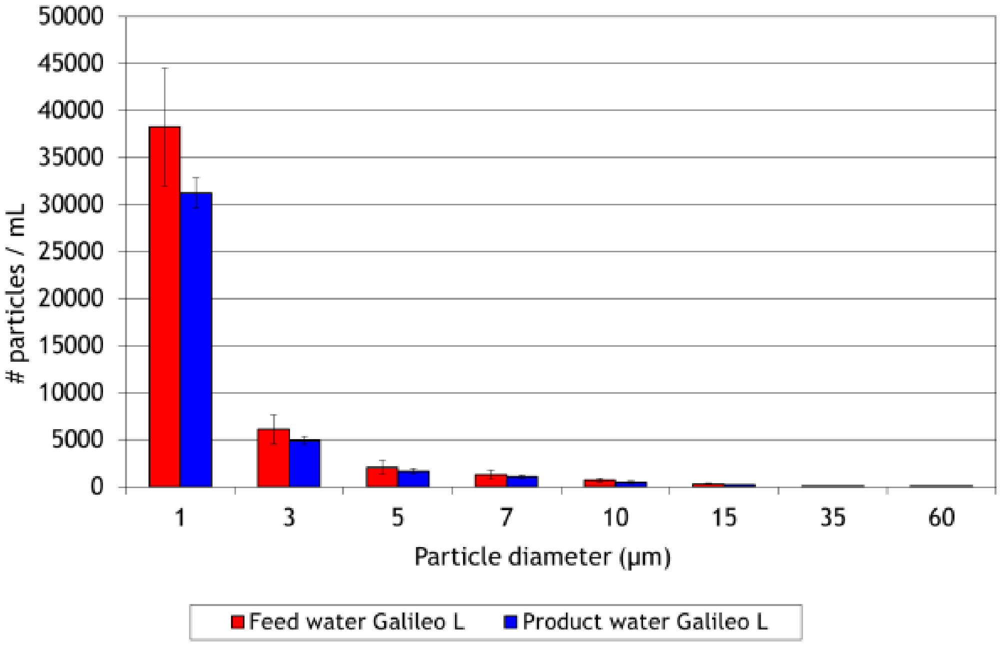
4.4.3.3 Particle counting

4.4.3.4 Low particle load on Galileo L filter

The amount and distribution of particles in the feed (influent) and product water (effluent) of the Galileo L filter was measured (range 1 – 400 µm) during normal operation. This normal operation is described by periods where external influence (e.g. rain and wind causing strong input of particles from surrounding bare lands) is either low or absent. The point where water is collected while normal conditions were applied is shown in Figure 4-27 (picture taken during 14-3-2018 sample campaign).

Figure 4-27: Collection point of water during normal operation conditions

In Figure 4-28, the measured distribution of particles in the feed and product water of the Galileo L filter is shown. The measured total amount of particles in the product water is lower compared to the feed water. However, these results are insignificant when the standard deviation in the measurements are considered. Note that the feed water did not contain many particles in the range of 7 – 400 µm during normal operation conditions; approximately 4% of the total amount of particles. The low load of particles having diameters significantly larger than the mesh size of the Galileo L filter (> 5 µm) could be an explanation for these results. Small amounts of particles bigger than the specified mesh size of a filter can leak into the product water via, for instance, imperfections in the filter material, especially when a cake layer (see Figure 4-30) is not build up due to the low particle load. As the filter rings of the Galileo L filter are pushed apart regularly (with every back-flush) and find a new configuration for the next filtration cycle, permeation of a small amount of big(ger) particle seems feasible.



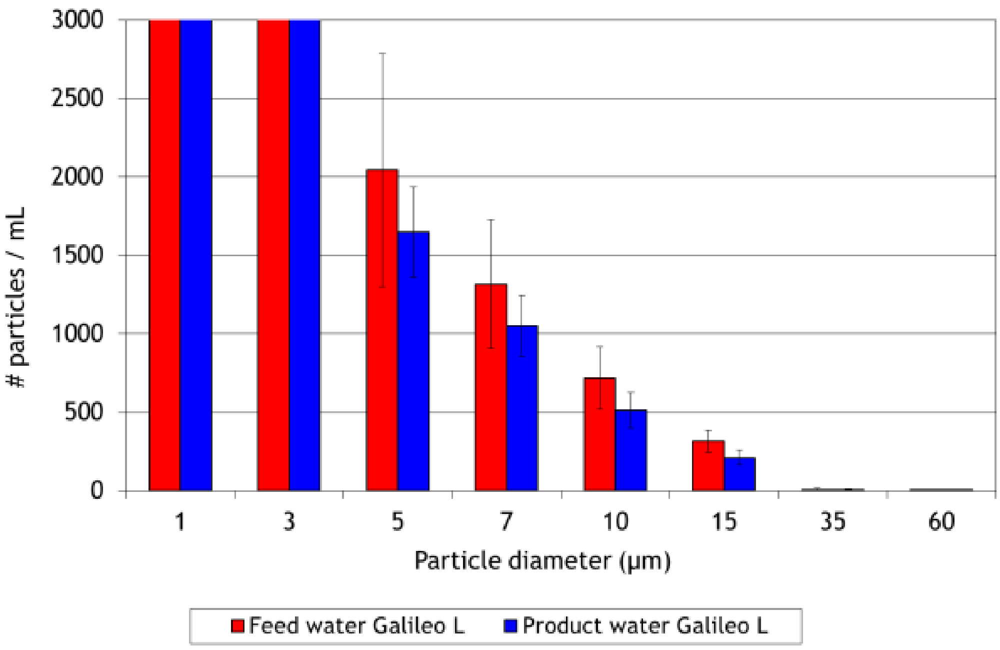


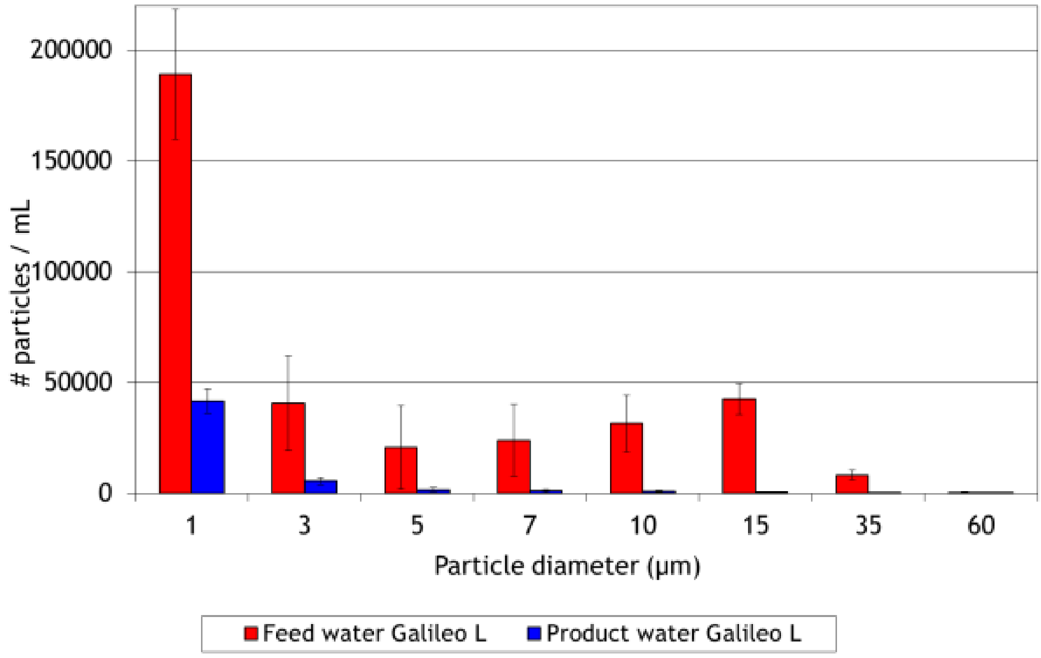
Figure 4-28: Measured distribution of particles in feed (red) and product (blue) water of the Galileo L filter during normal conditions: with a low resolution (top), and in more detail (bottom).

4.4.3.5 High particle load on Galileo L filter

To check the influence of particle load on the Galileo L filter performance, the amount and distribution of particles in the feed (influent) and product water (effluent) of the Galileo L filter were measured (range 1 – 400 µm) during abnormal operation. Abnormal operation conditions are governed by periods with special external influence (e.g. rain and wind) that create a high particle input to the feed water. The point where water is collected during abnormal conditions is shown in Figure 4-29 (picture taken during May 3, 2018 sample campaign). The abnormal conditions were simulated by creating high turbulence around the collection point using a broom, which significantly increased the turbidity and caused lots of particles to come into suspension. Note that during abnormal operation conditions the feed water of the Galileo L filter did contain many particles in the range of 7 – 60 µm; approximately 30% of the total amount of particles. In Figure 4-30 the measured amount of particles in the feed and product water of the Galileo L filter is shown. From the graph it is already clear that the feed contained significant (almost 7 times) more particles compared to the product water in the complete measuring range. This shows that particles are effectively removed when a high load of particles is fed to the Galileo L filter. The effective removal of particles is also confirmed by visual observation; see Figure 4-32.

The largest reduction in absolute numbers are particles in the range of 1 – 3 µm which, at first sight, does not make sense since the Galileo L filter was equipped with filter rings having a 5 µm mesh size. This observation can be explained by formation of a cake layer on the filter rings during abnormal operation conditions (high particle load). A cake layer develops when particles larger than the mesh size of the filter block these channels. This layer then acts as a filter itself by trapping particles smaller and bigger than the filter mesh size in the layer (Figure 4-31). The growth of a cake layer increases the resistance for water passage, thereby raising the pressure drop over the filter. A high particle load will therefore increase the frequency of required back-flushing.

Figure 4-29: Collection point of water during abnormal operation conditions (May 3, 2018)



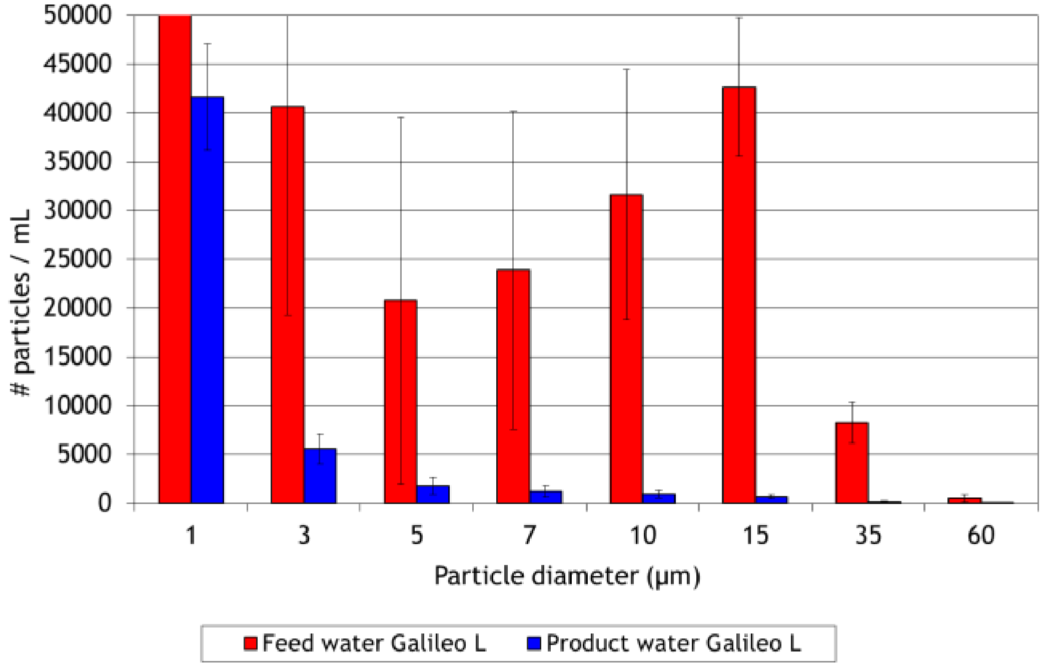


Figure 4-30: Measured amount of particles in the feed and product water of the Galileo L filter during abnormal conditions (May 3, 2018): with a low resolution (top), and in more detail (bottom).

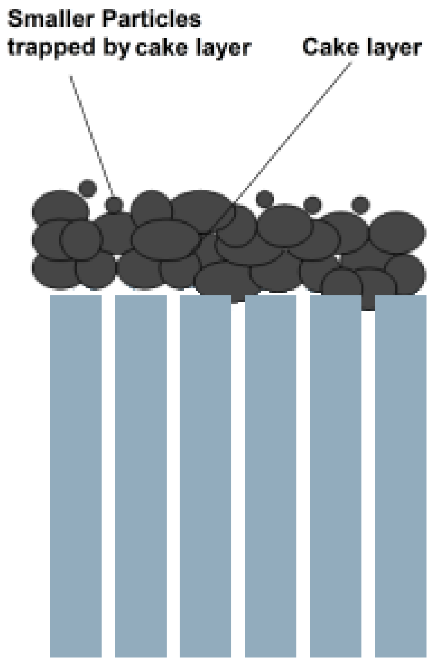


Figure 4-31: Systematic overview of cake layer development. as big particles block the filter channels, particles having a diameter smaller than the filter mesh size can be blocked by these bigger particles forming a dense layer

Figure 4-32: Sample of feed (left) and product (right) taken from the Galileo L filter during abnormal conditions

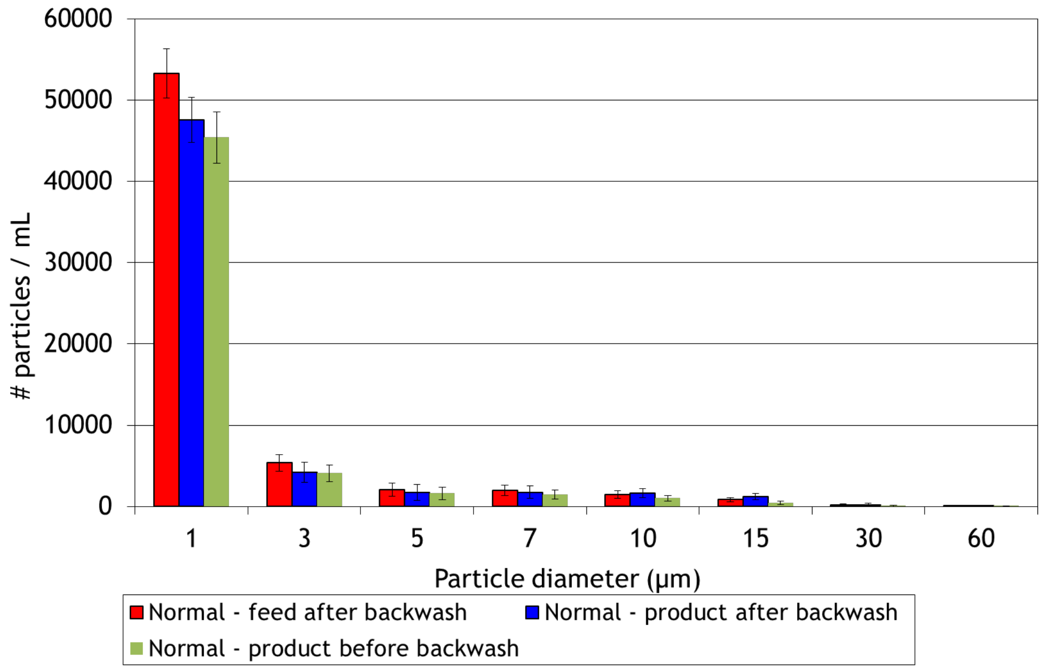
4.4.3.6 Influence of backwash on particle removal

As (re)development of a cake layer is expected to take some time after the backwash of the Galileo L filter, particle size measurements were performed right before and right after a backwash. As the (re)development time of the cake layer is dependent on particle load, these measurements were performed while the particle load was both normal and abnormal (see Figure 4-33 and Figure 4-34, respectively). Since it is practically impossible to measure all these samples during the sample campaign day, the particle stability in time of particle suspension was measured (see 8.1). Even though the total amount of particles decreases in time, the fractions of the sample within a certain size range stay within acceptable limits. This observation is taken into account with interpretation of the data and since removal of size-fractions is the interest in this research (instead of an absolute total number of particles in suspension), all results are used in drawing final conclusions, irrespective of the time between the moment of sampling and analysis.

Even though the measured amount of particles per mL is slightly lower in the product water for both before and after backwash, no significant difference in amount of particles was measured under normal conditions (see Figure 4-35), except for particles sized between 15 µm and 30 µm. In this range, a significant lower amount of particles was measured in the product water before backwash compared to product water after backwash.

Figure 4-33: Galileo L filter inlet during normal conditions (19-6-2018)

Figure 4-34: Galileo L filter inlet during abnormal conditions (19-6-2018)



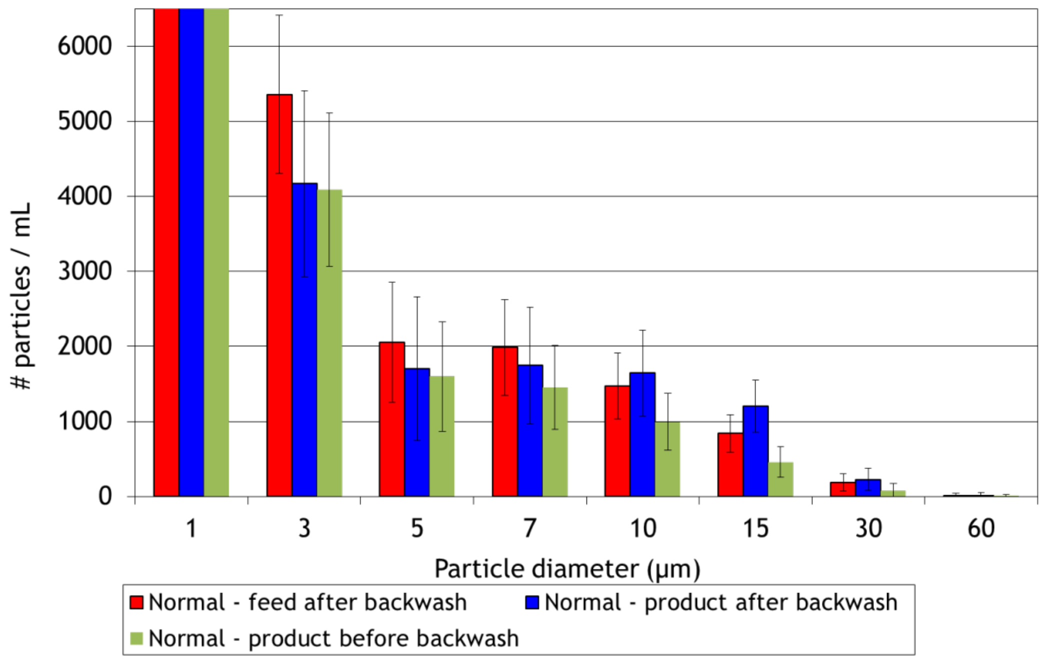


Figure 4-35: Galileo L filter performance during normal conditions: with a low resolution (top), and in more detail (bottom). red = feed water, blue = product water after backwash, green = product water before backwash

During abnormal conditions, measurements before (Figure 4-36) and after (Figure 4-37) backwash were performed as well. As it is difficult to see trends directly from these graphs, since feed water composition is different in both measurements, removal percentages with respect to the corresponding feed composition are shown in Table 4-10.

In Table 4-10 removal rates between 80% – 100% before a backflush and 54% – 68% after a backflush of the Galileo L filter are shown. These values indicate that particle rejection increases with time during a filtration cycle, the cycle being the period between two backflushes. As a cake layer develops over time as well, the (re)formation of a cake layer can explain the observed difference in particle removal percentage.

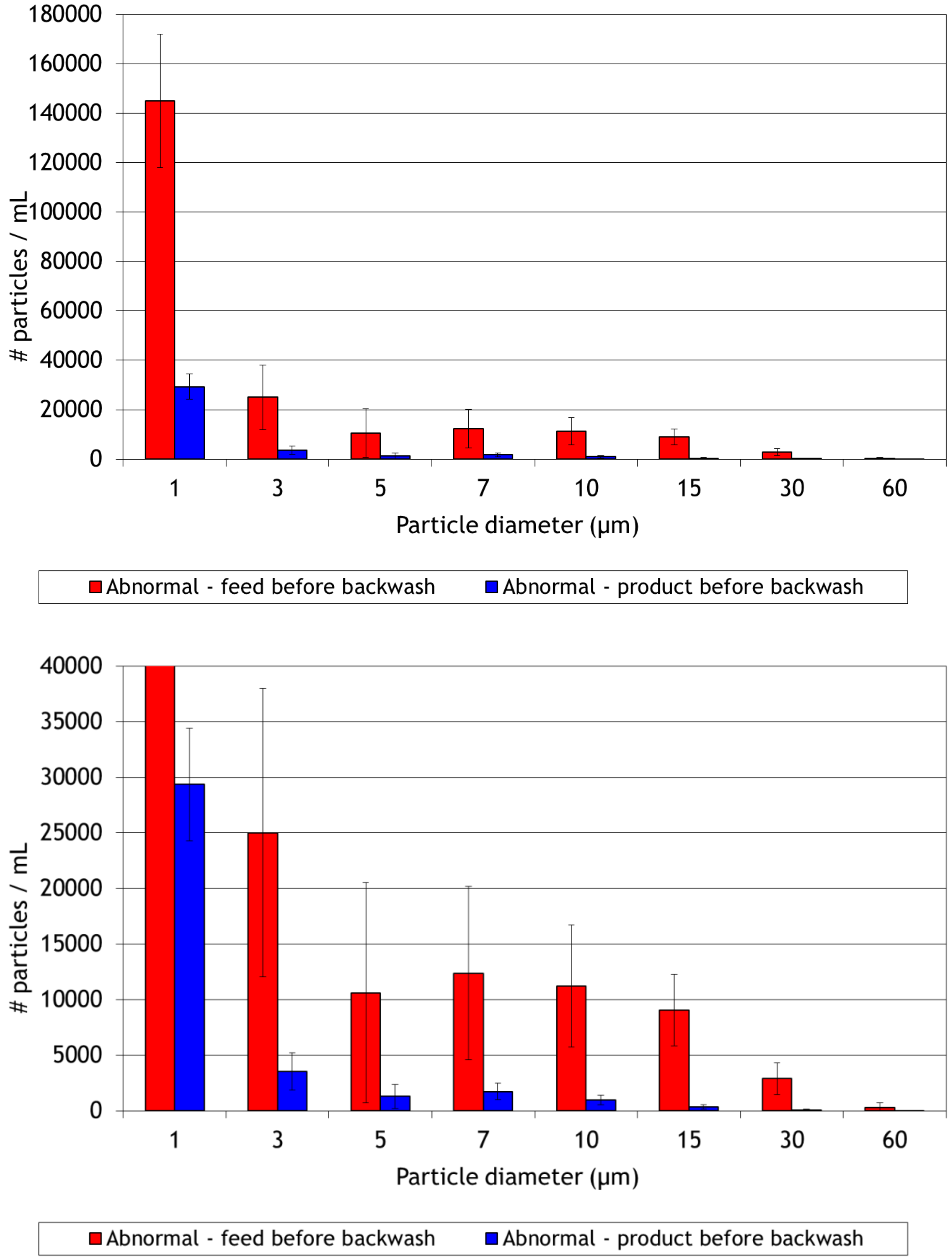
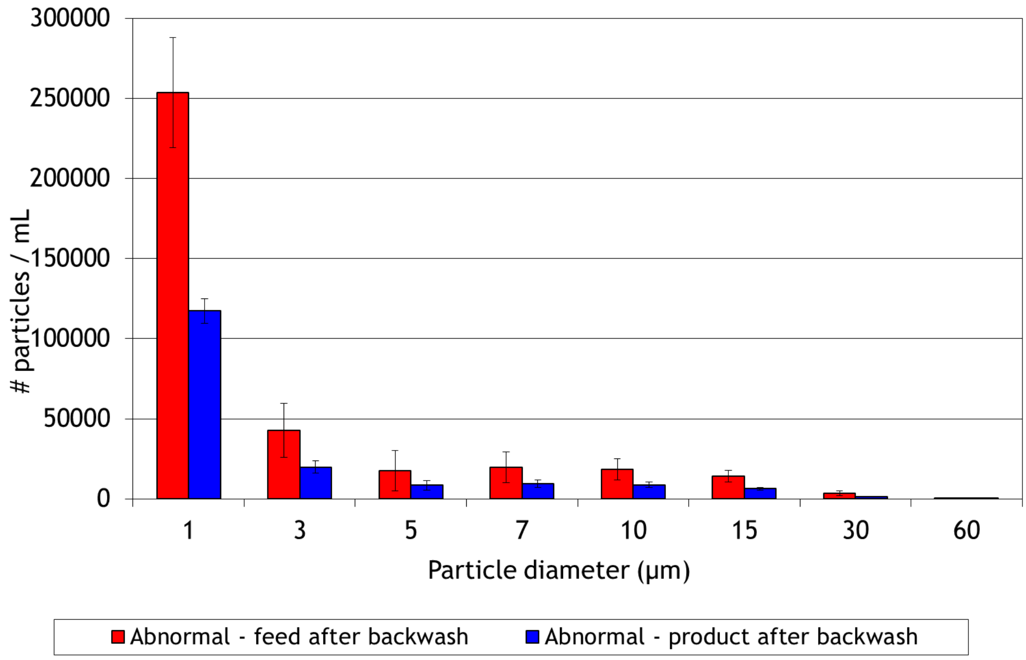


Figure 4-36: Measured amount of particles in the feed and product water of the Galileo L filter during abnormal conditions just before a backwash (19-6-2018): with a low resolution (top), and in more detail (bottom).



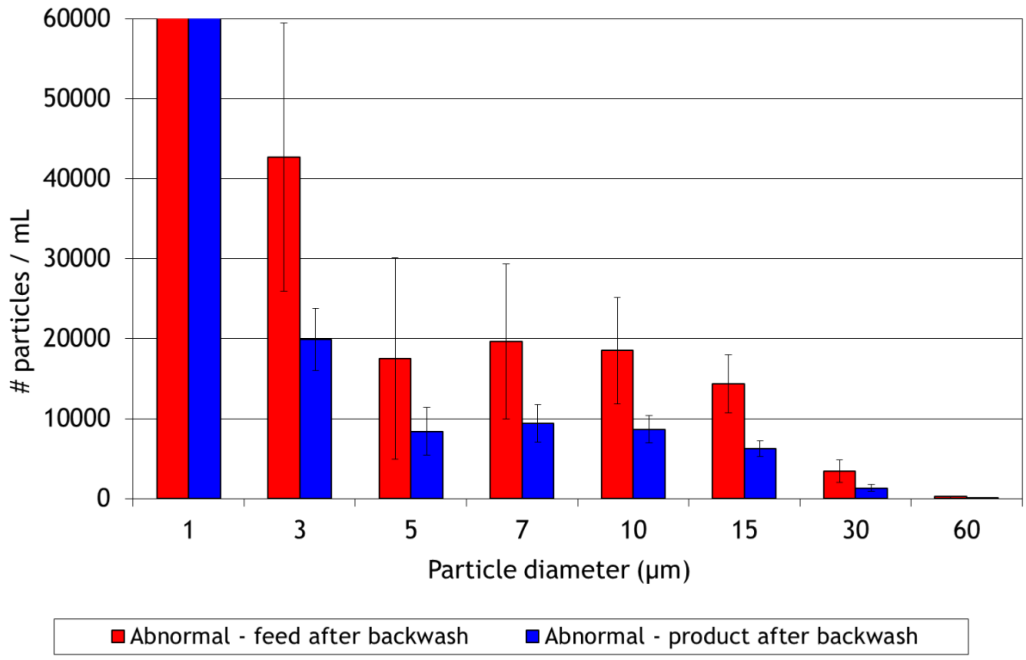


Figure 4-37: Measured amount of particles in the feed and product water of the Galileo L filter during abnormal conditions just after a backwash (19-6-2018): with a low resolution (top), and in more detail (bottom).

Table 4-10: Removal in terms of percentage of particles in feed water before and after a backflush of the Galileo L filter during abnormal conditions

Particle diameter (µm)	**1**	**3**	**5**	**7**	**10**	**15**	**30**	**60**
Abnormal conditions	Removal percentage (%)							
Before backwash	80	86	88	86	91	96	98	100
After backwash	54	53	52	52	53	56	61	68

4.4.3.7 Implications for pre-treatment of infiltration water

The Galileo L filter clearly has an effect on the amount of particles present in the infiltration water. However, especially right after a regular backwash, only a fraction (50%-60%) of the total amount of particles present in the range of 1 – 100 µm is removed during abnormal conditions (see Table 4-10). In periods where particle load is low, the performance of the Galileo L filter is even less. This means that groundwater well clogging by particle fouling is not avoided by the Galileo L filter; the filter will delay the process to some extent. However, since particles larger than the Galileo L filter mesh size are found in the filter effluent, dissolved organic carbon and (a part of) undissolved organic carbon which passes the Galileo L filter (see 4.4.2.2) can still act as a nutrient source causing microbiological well clogging over time. Having said all of the above, the Galileo L filter can be especially useful to prevent direct well clogging after, for instance, a heavy rainfall event or storm where lots of particles are mobilized.

The performance of the Galileo L filter seems to increase with higher particle load, even for particles having a size smaller than the filter mesh size. This effect can be explained using the cake layer concept (see 4.4.3.3). Further optimisation of the Galileo L filter can be the subject of further research, by decreasing the backflush frequency the cake layer build-up can be increased, possibly resulting in increased particle removal by the Galileo L filter. In line with this approach, it can be decided to send the first permeate of the Galileo to a waste stream instead of towards the well, such that only better-treated water is injected. In case the Galileo L filter (pre-)treats the conventionally applied rapid sand filter (RSF) influent, more stable operation and more efficient filter bed replacement is expected.

5 Improving the pre-treatment of ASR infiltration water by enhanced particle removal and desinfection

5.1 Approach

The set-up presented in Chapter 4 with the Galileo L at the Freshmaker in Ovezande was used to further explore the impact of enhanced treatment of the poor quality surface water. Cartridge filters (spun-wound and melt-blown) were used to simulate a(n additional) treatment better capable of removing suspended solids. UV point disinfection using the V140 by Van Remmen UV (The Netherlands) was added to provide better disinfection prior to infiltration.

In the last phase, chlorination of the injection water was applied to prolong the disinfection properties of the water when entering the infiltration well. Na-hypochlorite (13.8%) was dosed with approximately 4.5 ppm. Due to incorrect installation of the dosing unit, the chlorination was initially discontinuous.

Table 5-1: Stages of additional pre-treatment at the Ovezande site.

Week# in 2019	3	4	5	6	7	8	9	10	11
Added pre-treatment									
Spun-wound cartridge filter	■	■	■						
Melt-blown cartridge filter + UV			■	■	■	■			
Melt-blown cartridge filter + UV + hypochlorite (discontinuous)						■			
Melt-blown cartridge filter + UV + hypochlorite							■	■	■

5.2 Pre-treated and subsequently infiltrated volume

Between January and April, almost 4000 m^3 of surface water was taken in and treated in the various stages. The flowrate was mainly controlled by the clogging cartridge filters: fresh cartridge filters showed a capacity of around 4 m^3/h, which could decrease due to clogging to a rate of <0.5 m^3/h. Due to a malfunctioning automated valve, between February 10 and February 17, strong fluctuations were observed (Figure 5-1).

From March 8 onward, a period with frequent rainfall but low intensities, the water appeared to be cleaner and the infiltration rate could remain higher due to limited clogging of the filters. In this period the maximum pressure on the injection well limited the flow rate. Less additional pressure could be applied on this well, due to elevated groundwater levels in this wet period. As a consequence, the Galileo L had to stop operating shortly to lower the injection pressure first.

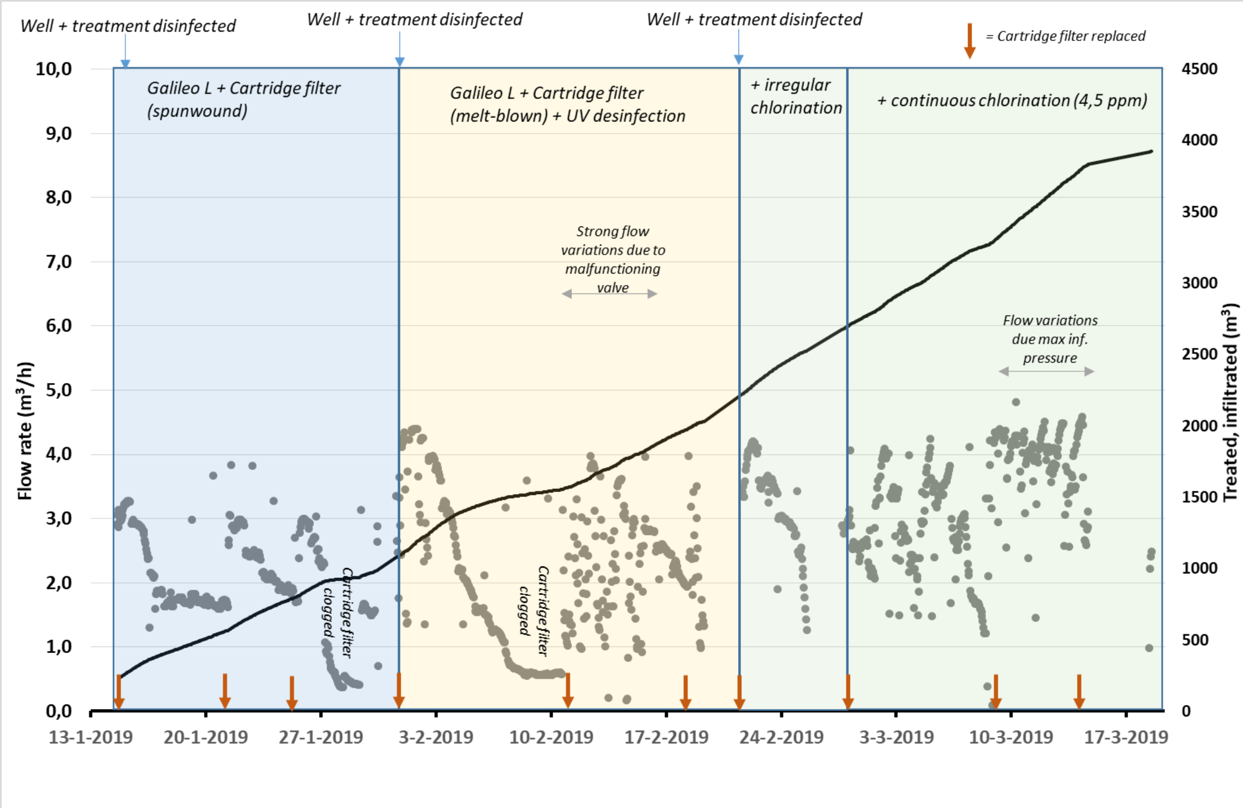


Figure 5-1: Measured flow rate and cumulative infiltration volume during 2019 trial. Black line = treated, infiltrated water. Grey dots refer to infiltration flow rate.

5.3 Pressure on the infiltration well

The pressure on the infiltration well varied throughout the trial as a consequence of flow rate variations, generally following the pattern of the flowrate, with sometimes a minor time lag (Figure 5-2).

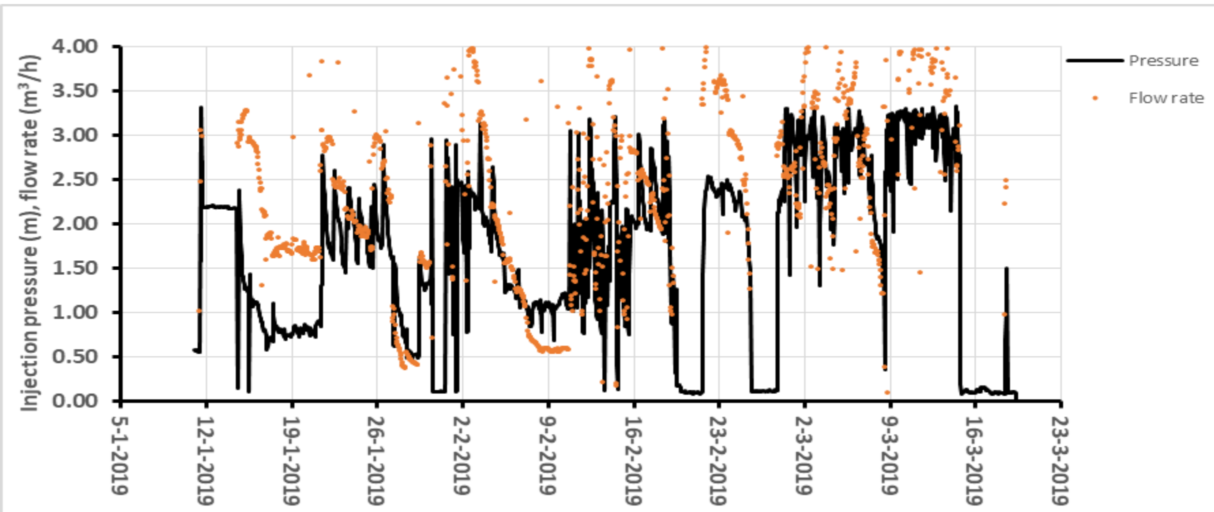


Figure 5-2: Observed pressure on the infiltration well. No correction was made for temperature (varying between 4-10 °C).

5.4 Characterization of the untreated, treated water, and backwash water from the well

5.4.1 Water composition analysis

Based on eight water samples taken during the trial (virtually every week), the water quality before and after the cartridge was assessed. It was found that the cartridge filtration was effective in lowering Fe (slightly), colony count (clearly), and TOC (slightly). As with the particle counting, the results depended heavily on the moment of sampling and the degree of clogging of the cartridge filter at that moment. Given the high levels of DOC and colony count and the known presence of dissolved oxygen and nutrients in the water, a high potential for biological growth can be presumed. Also Fe-concentrations are relatively high, which can result in chemical clogging by Fe-precipitation.

Table 5-2: Average water composition (most relevant parameters) during Ovezande field test (8 analysis)

		Untreated	After cartridge
Fe	µg/L	825	751
Colony count (22 °C)	CFU/mL	87933	5821
DOC	mg/L C	6.7	6.6
TOC	mg/L C	7.8	7.3
Average temperature	˚C	7.1	7.1
Electrical conductivity	mS/cm	1 – 2	1 - 2

5.4.2 Assessment of ATP

The concentration of Adenosine Tri-Phosphate (ATP), a source of energy present in almost all organisms (including bacteria), was analysed in the water treated by the Galileo L filter and after every additional treatment step deployed at the time of sampling (see Table 5-3). The ATP concentration is an indicator for presence of biologically active organisms, as living organisms need to produce ATP to maintain themselves. Cartridge filters seem, irrespective of being fouled or fresh, to slightly decrease the ATP concentration. This can be explained by the fact that except dirt and particulate fouling, also bacteria can be retained. UV treatment seems to have a slightly decreasing effect on measured ATP concentration, except for the sampling at 19-3-2019 which shows a small increase in ATP concentration. This observation can be explained by the formation of most likely calcium-carbonate on the UV lamp, as shown in Figure 5-3. Due to formation of a calcium carbonate scale on the UV lamp via temperature-induced precipitation, the disinfecting capacity of the UV lamp decreased over time. Hypochlorite addition to the cartridge- and UV-treated water did not directly result in a lower ATP concentration, due to the irregular flow of hypochlorite into the system (1 pulse per 50 seconds).

Table 5-3: Sample points and analysis results of ATP concentration measurements

Date	Sample points for ATP concentration measurement [pg/mL]			
	After Galileo	After cartridge filter	After UV	After chlorination
17-1-2019	340	310	-	-
21-1-2019	295	230	-	-
30-1-2019	260	190	145	-
19-2-2019	365	315	280	-
19-3-2019	225	210	215	220

Figure 5-3: Inside the V-140 UV unit. The UV lamp is covered with mineral scaling (probably calcium carbonate)

5.4.3 Results of the BACTcontrol by MicroLAN

The BACTcontrol was installed at the Ovezande pilot in order to monitor the presence of bacteria in the treated infiltration water. The total activity measured by the BACTcontrol is an indicator for biological activity in the water, expressed in pmol/min per 10 mL of sample, is shown in Figure 5-4. Bacteria contain a certain amount of enzyme which, if present, catalyses a light-emitting reaction which is measured and expressed as total activity of the sample. In the period of 7 February to 10 February, the total activity shows a decreasing trend; as the infiltration water was irradiated by the V-140 UV system. Bacteria might have been killed which could explain the decrease in total activity measured. Starting from 11 February up to 19 February, the total activity shows an increasing trend. This might be attributed to the mineral scaling which could have developed in that period on the UV lamp, hindering its performance. In the period of 21 February till 26 February, hypochlorite dosing was applied but due to inconsistent dosing and technical problems (malfunctioning valve on Galileo L filter), total activity measurements did not show any trend. After solving the technical problems, in the period of 6 March up to 15 March, hypochlorite dosing was more or less continuous and total activity seems to drop somewhat compared to the UV-treated period. As the point at which hypochlorite was dosed was within 2 meters distance from the sampling port of the BACTcontrol, one may conclude that the pulsed dose of hypochlorite caused this small decrease over this small physical distance and, therefore, short contact time. The effect of longer contact time of hypochlorite is shown in the period of 26 March to 4 April, in which the infiltration was stopped, but hypochlorite dosing remained. When the infiltration was started (29 March) and again stopped (30 March) and contact time again thus was reduced, measured total activity is comparable to the regular hypochlorite dosing period. This indicates that pulsed hypochlorite dosing may have the desired effect in the infiltration well, where the contact time is larger (and therefore biological activity presumably lower).

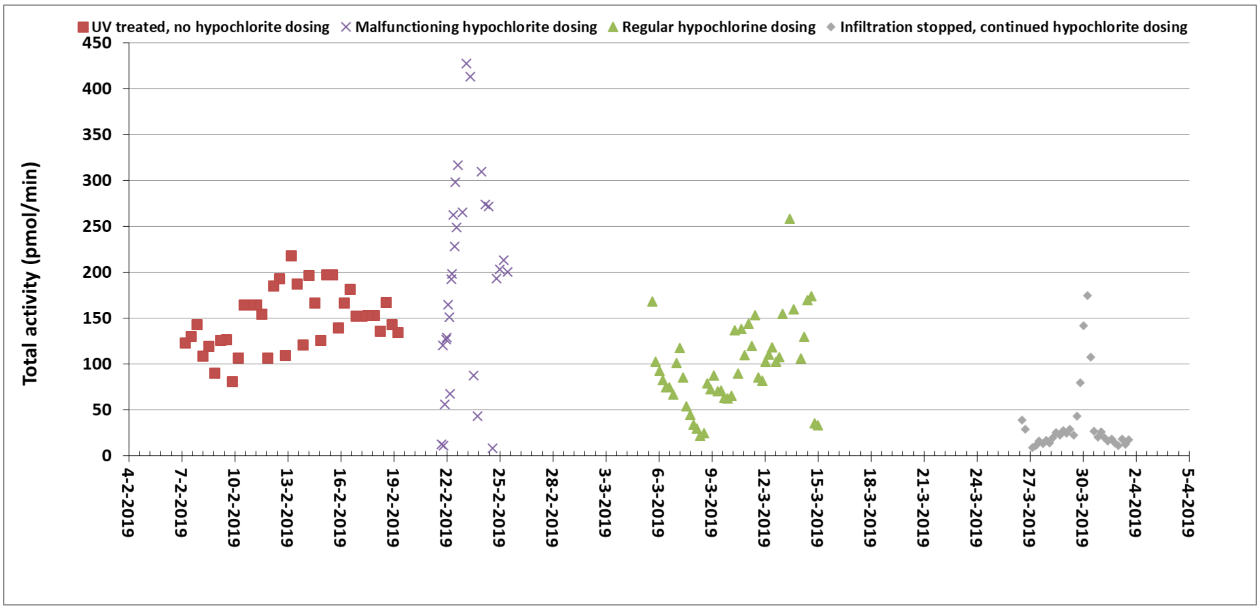


Figure 5-4: Total activity measured in the water right before infiltration in the aquifer. Hypochlorite dosing causes the total activity to drop

5.4.4 Backwash of the infiltration well

The effectivity of backwashing the well was assessed by analysing turbidity and ATP during the back-wash. It was demonstrated that the backwash is successful in removing particles, given the increase in turbidity and ATP during especially the first 10 minutes of the backwash. No more than 15 minutes seems the be required. In practice, a daily backwash of 20 minutes was applied to remove (part of) the particle- and biological fouling developing in the infiltration well. During this daily backwash, water was extracted with 6.1 m^3/h. In Figure 5-5 it is shown that between 3 to 10 minutes after start of the backwash, the largest part of accumulated fouling within the infiltration well is released. Both turbidity and ATP measurement show peaks in backwash water on roughly the same time intervals. This indicates that most of the ATP present in the backwash water is attached to the fouling released by the backwash. Low ATP measurements after 30 minutes of backwash indicate that biological fouling occurs mainly at the infiltration well pipe and does not extend much into the ground around the infiltration well.

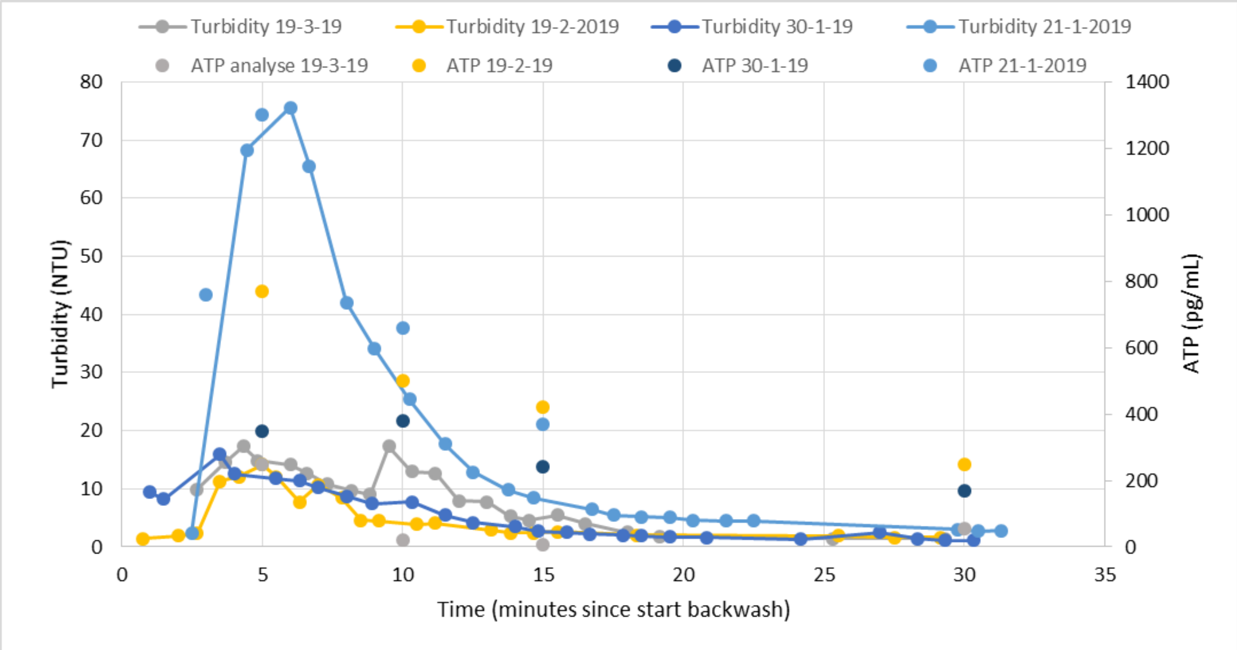


Figure 5-5: Observed turbidity and ATP during backwash of the well

5.5 Analyses of particles in the water

5.5.1 Spun-wound cartridge filters

A fouled filter was analysed on January 17, just before the specific capacity decreased (see Figure 5-6). A small but significant decrease of number of particles ranging from 1µm to 3µm and small decrease in number of particles ranging from 3µm to 60µm in the product of a fouled nominal 1-micron spun-wound cartridge filter compared to the feed of this filter were observed, respectively (Figure 5-7).

Figure 5-6: Fresh (left) and fouled (right) spun-wound 1 micron cartridge filter. Picture taken on 21-1-2019

With an even more extremely fouled spun-wound filter, a large increase was observed of the number of particles ranging from 1µm to 3µm, a small decrease in number of particles ranging from 3µm to 10µm and a small increase in number of particles ranging from 10µm to 60µm in the product of a fouled nominal 1-micron spun-wound cartridge filter compared to the feed of this filter, respectively (Figure 5-8).

With a fresh spun-wound filter, the analyses on January 21 showed large and significant decrease of number of particles ranging from 1 µm to 3 µm and small decrease in number of particles ranging from 3 µm to 60 µm in the product of a fresh nominal 1-micron spun-wound cartridge filter compared to the feed of this filter, respectively (Figure 5-9).

The conclusion is that the filters applied were only functional in the first days after installation, but they had little value once clogging (with subsequent decrease in capacity) was taking place. Moreover, the filter could then transform into a source of particles.

The material caught by the cartridge filter was sampled and analysed using thermographic analysis (TGA) up to 1000 °C at KWR. The results are indicated in Table 5-4 and highlight the origin of the collected material and the composition of the suspend solids in the surface water, which is largely organic matter and siliciclastic material (sand, silt, clay).

Table 5-4: Results of thermographic analysis solid material captured by cartridge filter.

Temperature range	Dry weight	Remark
105 – 550	31.6%	Organic matter
550 - 1000	3.0%	Generally carbonates
Residue	65.4%	Generally silicates

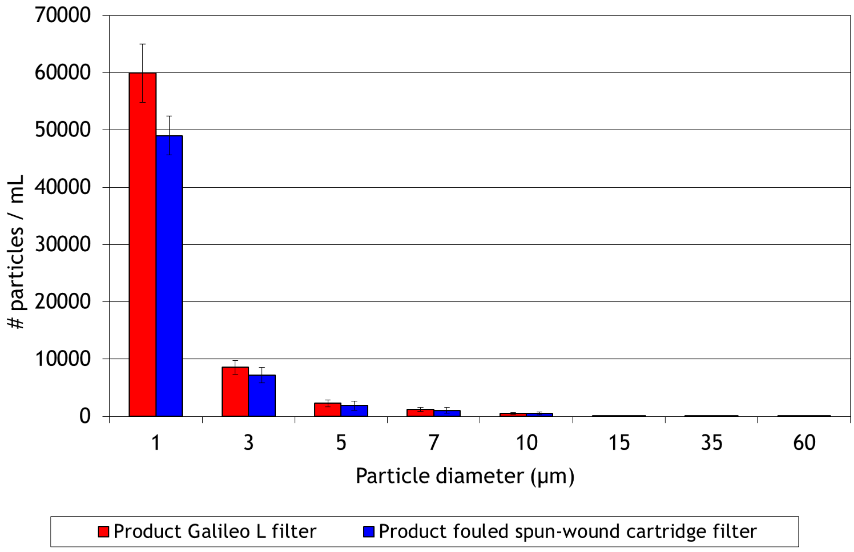


Figure 5-7: Particle counting results of a fouled spun-wound filter on January 17.

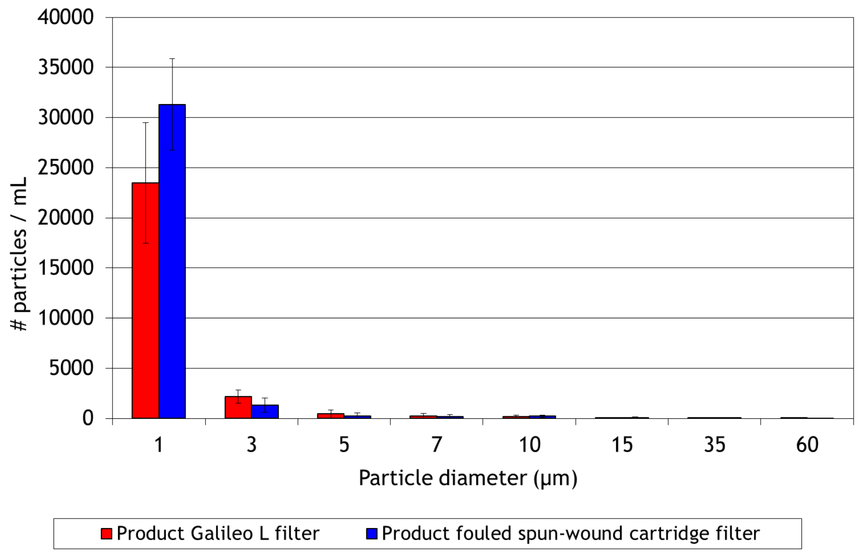


Figure 5-8: Particle counting results of a (very) fouled spun-wound filter on January 21.

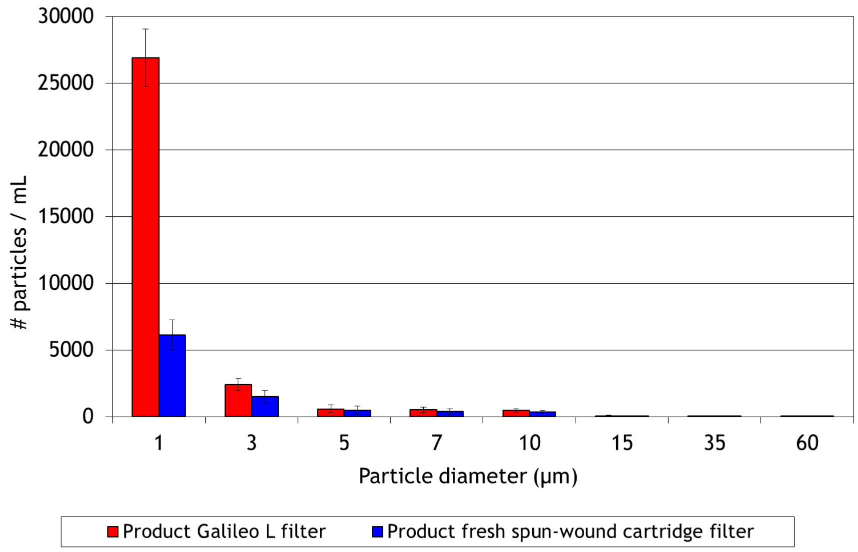


Figure 5-9: Particle counting results of a fresh spun-wound filter on January 21.

5.5.2 Melt-blown cartridge filters

With the fresh melt-blown filter, a large but not significant decrease of the number of particles ranging from 1 µm to 60 µm in the product was found, compared to the feed water (Figure 5-11).

With a fouled melt-blown filter, a small but significant decrease of number of particles ranging from 1 µm to 3 µm was observed. However, an increased numbers of particles ranging from 3 µm to 60 µm was found, indicating that a breakthrough of particles was occurring (Figure 5-12).

Figure 5-10 shows the fouled (left) and fresh (right) meltblown 1 micron melt-blown cartridge filters. The fouled meltblown filter showed a uniform layer of particles over the whole filter surface. The connection point shows no signs of physical misplacement during the operational time of the filter, as the brown fouling layer has build-up uniformly around the connection point. The perforated tube in the middle shows also brown holes, indicating the particles literally broke through the filter material into the permeate tube. As the feed pressure from the Galileo L filter was maximum 1.5 bars (setpoint feed-pump Galileo), the maximum pressure these filters can withstand has not been exceeded during the test. The only reasonable explanation for the observed particle permeation is the high load of particles on a non-absolute filter.

Figure 5-10: Fouled (left) and fresh (right) melt-blown 1 micron cartridge filter. Picture taken on 19-2-2019

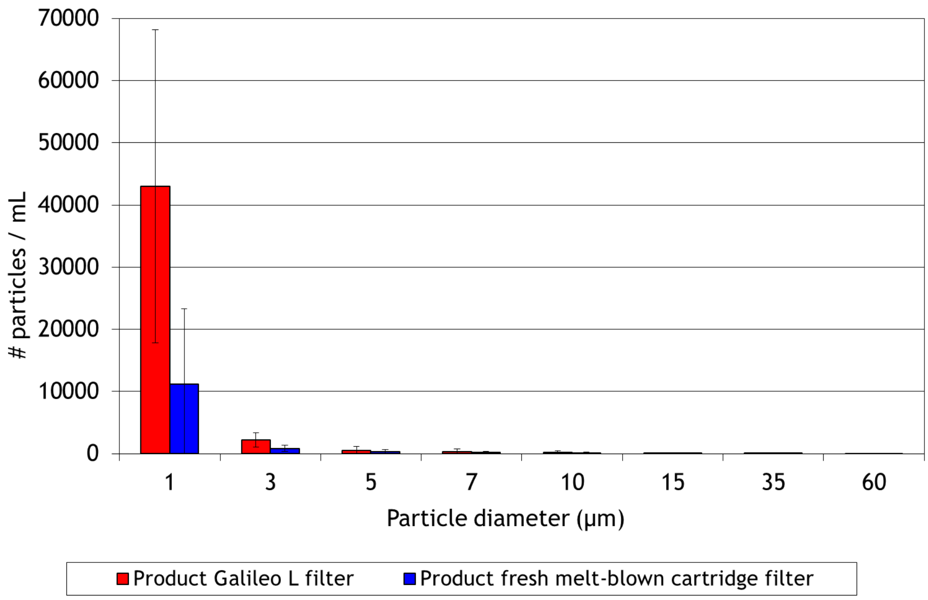


Figure 5-11: Particle counting results of a fresh melt-blown filter on January 30.

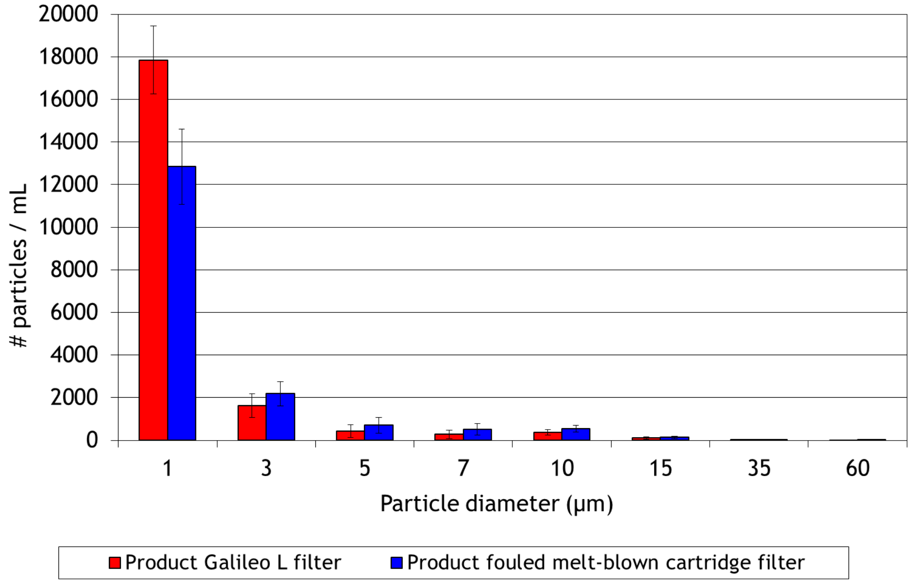


Figure 5-12: Particle counting results of a fouled melt-blown filter on February 19.

5.5.3 Impact of cartridge filters

Both filter types have a clear positive impact, but only during the first days of operation. Once clogged, the filters both allow (accumulated) particles to breakthrough. These particles are transported to the well, were they result in reduced well capacities (Figure 5-13). It is therefore shown that the increased removal of particles has a very positive effect on the well capacity, but that it only provides added value if the high level of particle removal can be continuously maintained. The temporary solution with cartridge filters is unpractical, because this would imply very frequent replacement. There is another type of cartridge filter commercially available, absolute cartridge filters. As opposed to the (in this research used) nominal cartridge filters, absolute cartridge filters pose an absolute physical barrier where particles with sizes above this barrier simply cannot pass. These absolute cartridge filters are available with high surface area and are therefore thought to need less frequent replacement and maintain high flows, while continuously removing all particles larger than the absolute pore size. It is worthwhile to look into these filters in future research, with comparable poor quality feed water.

5.6 Specific capacity of the infiltration well

By dividing the observed infiltration rate by the infiltration pressure, the specific capacity of the infiltration well was determined. This holds the most valuable information with respect to clogging of the infiltration well. Low flow rates (<1.5 m^3/h) were neglected in this analysis because of their low reliability.

The specific capacity was <1 m^3/h per m at the end of the previous infiltration stage (April 2018). Upon recovery of freshwater and an idle period, and treatment with Na-hypochlorite, a high specific capacity of >2.5 m^3/h per m was attained (Figure 5-13). However, upon clogging of the cartridge filter after around 4 days, marked by a reduced flow rate, it was observed that the capacity decreased linearly. Most presumably, breakthrough of particles occurred, resulting in clogging of the pores around the well.

Air entrainment during backwashing of the well led to lowered specific capacities in the second halve of January, which however restored during injection. This complicates interpretation in the period. Most likely, the decrease of the capacity slowly continued, especially once the spun-wound filters were clogged. At the end of the first period, a remaining capacity of around 1.0 – 1.5 m^3/h per m remained.

Cleaning the system with Na-hypochlorite before adding the UV disinfection restored the capacity to 1.8 m^3/h per m, but was followed by a rapid decrease during clogging (and breakthrough) of the (this time melt-blown) cartridge filters. During the rest of this stage with UV, a further lowering to <1.0 m^3/h per m occurred. No positive impact of the UV could be identified.

Upon treatment and disinfection with Na-hypochlorite, the capacity was brought back to 1.7 m^3/h per m, but again a rapid decrease to 1.0 m^3/h per m was observed upon clogging of the cartridge filter and accompanying breakthrough of particles. Chlorination in this first phase was malfunctioning and adjusted within one week. Once continuous chlorination was maintained, a remarkable stabilization of the well capacity was found, with even a seemingly increasing capacity back towards 1.5 m^3/h per m. This shows that chlorination has an appearing positive effect on the well capacity, possibly even after breakthrough of particles.

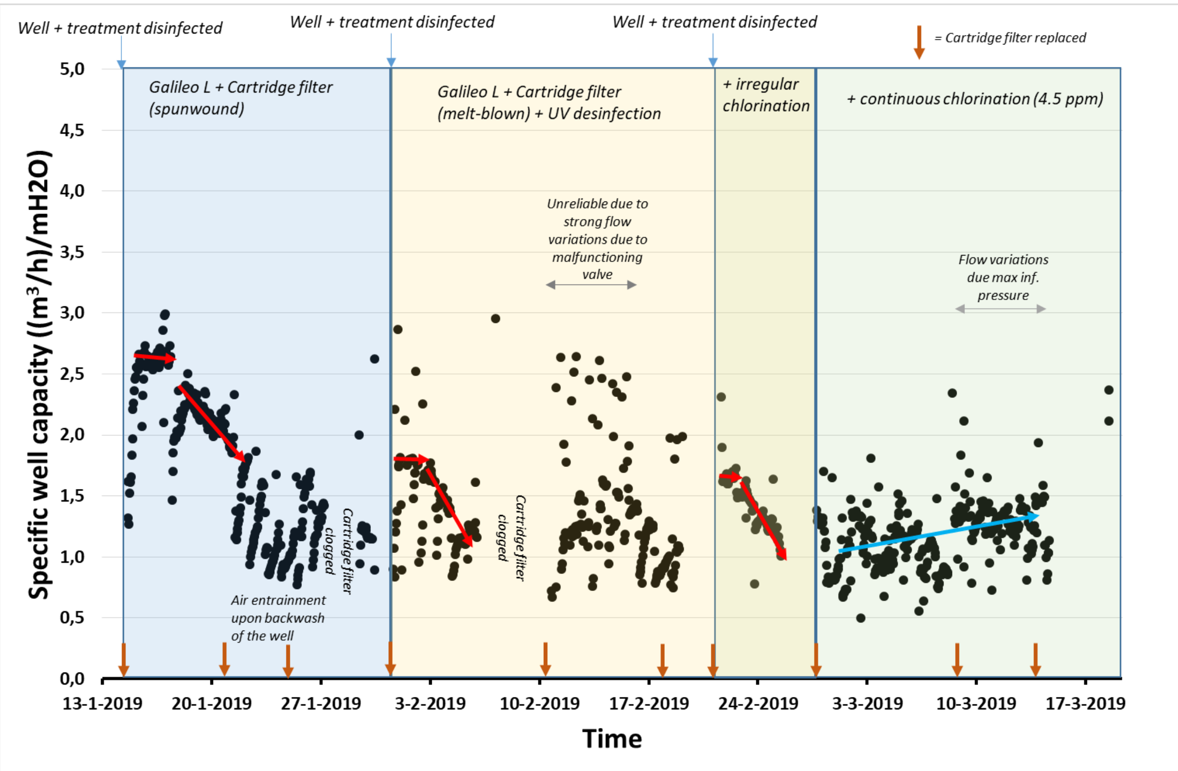


Figure 5-13: Specific capacity of the infiltration well in 2019.

5.7 Conclusions on the 2019 pre-treatment trial in Ovezande.

Based on the results of the extended infiltration tests in Ovezande with enhanced pre-treatment, it can be concluded that:

- The injection water has a high potential for mechanical clogging (once cartridge filters are saturated and show a breakthrough of particles), biological growth and scaling (Fe-oxides);

- With the cartridge filters added, clogging of the injection well was observed once a breakthrough occurred upon clogging of the cartridge filters. It was underlined with particle counting that this breakthrough occurred and the specific well capacity decreased linearly in those phases. Mechanical clogging can be potentially prevented if this fine filtration can be continued without interruption;
- There was no sign of limiting clogging by the UV disinfection. This may also be caused by the overprinted clogging by particles passing the cartridge filter;
- During chlorination of the injection water, clogging did not continue, but a clogging standstill and even an increase in capacity were observed. This suggests that the chlorination can have a positive effect on the prevention of clogging by either (or combined):
    - Limitation of biological growth in the well by disinfection and degradation of organic material
    - Dispersion of the particles, which enables particles to be transported without causing clogging near the well. This may also enhance particle removal during backwashing of the well, but this was not observed.

Chlorination in combination with the Galileo may therefore form a strong combination, which should be further tested in future research. Note that chlorinated organic compounds might be formed during this treatment and future research should further look into the life-time of these compounds in sub-soil environments.

6 Conclusions

6.1 Available technology for treatment of stormwater prior to infiltration

There are different possibilities for rapid pre-filtration prior to ASR. The final choice for pre-filtration depends on the required purification efficiency (depending mainly on the available source of water and filter system), available space and budget, possible assignments (esthetical reasons) and the available effort in operations and maintenance.

In order to select the best fitting pre-filtration, the following two tables with qualitative performance indicators based on available information are used. Table 3-11 shows qualitative evaluation of the most important characteristics of the different technologies. Table 3-12 compares the way of embedding (natural versus technological) versus the degree of innovation.

6.2 High flow infiltration filter

The Galileo L filter was found reliable in operation but was not able to prevent particle- and biological fouling during infiltration with aquifer storage and recovery (ASR) applications. The removal of particles was observed, but led to insignificant reduction of the MFI. The Galileo L filter has the best particle removal performance when the influent water contains a lot of particles, an observation that can be explained by development of a cake layer on the filter mesh. This would also explain the observed poor particle removal performance at low particle load. The passage of relatively large particles also showed that the Galileo L filter is not an absolute filter. As organics, bacteria, and nutrients are not retained by the finest filter mesh available (5 µm), biological fouling was observed and contributed to clogging of (simulated) infiltration wells. In situations where sudden peaks in particle load can occur, the Galileo L filter can help preventing rapid clogging of the down-stream system. To perform a robust pre-treatment step prior to storm water infiltration, a second, finer filtration step is required. In case of a high nutrient and organic carbon content in the storm water, additional steps are also required to prevent biological clogging.

6.3 Climate adaptive storm water harvesting model

Both overflow and shortages in systems combining rainwater harvesting, above-ground reservoirs, and ASR can be decreased by applying a proactive management approach, even with constant over- or underestimation of real precipitation data. The amount of pumping by the ASR system did not change significantly in the Glasparel+ case. Slightly more pumping did take place with the proactive approach to effectively prevent overflows on time. This has a positive effect on the required recovery efficiency. It is advised to remediate well systems at a relatively high level (early) to prevent water shortages (80%) if reactive reservoir management is applied. With proactive management the risk is lower, but it is still advised to maintain the pumping well rate higher than the peak demand. The storm water harvesting model can now be used in decision making processes to estimate the required reservoir size in combination with ASR pumping capacity. A certain minimum reservoir size is required (here: >30 to 40 mm) to benefit from a proactive approach.

6.4 On the rapid (in)filtration of stormwater during ASR

The findings underline the challenge one faces when applying storm water harvesting in combination with aquifer storage and recovery:

- Available storm water is often far from the water quality one desires to safeguard continuous infiltration using wells;
- Available compact treatment technology mainly relies on sedimentation and filtration, but is often not able to sufficiently remove the fines particles and lower the MFI (slow sand filtration excluded). They have potential, however, as a first treatment step to remove the coarser particles before applying a finer filtration step;
- Optimization of available aboveground reservoirs, ASR, and storm water reuse for irrigation is essential due to the complex interplay and the value of choosing the right dimensions in the design phase, maintaining the infiltration capacity, and using weather forecasts to maximize the retention capacity.

6.5 Next Steps

World wide overexploitation of freshwater aquifers result in declining groundwater levels. However, water surplus in periods of heavy rainfall are mostly not utilized to recover these aquifers. One of the major obstacles is the presence of particles in water flows resulting in possible clogging of infiltration wells. The type of 'self cleaning' and robust filtration units as the Galileo filter can contribute to remove the bulk of particles in water flows. In next ASR projects we will consider the Galileo filter as pre filtration step.

7 References

Caliskaner, O., Tchobanoglous, G., & Carolan, A. (1999). High-rate filtration with a synthetic compressible media. *Water environment research, 71*(6), 1171-1177.

Cheng, S., Grosse, W., Karrenbrock, F., & Thoennessen, M. (2002). Efficiency of constructed wetlands in decontamination of water polluted by heavy metals. *Ecological Engineering, 18*(3), 317-325. doi:https://doi.org/10.1016/S0925-8574(01)00091-X

Chinu, K. J., Johir, A. H., Vigneswaran, S., Shon, H. K., & Kandasamy, J. (2009). Biofilter as pretreatment to membrane based desalination: Evaluation in terms of fouling index. *Desalination, 247*(1), 77-84. doi:https://doi.org/10.1016/j.desal.2008.12.014

Cooper, P. (1999). A review of the design and performance of vertical-flow and hybrid reed bed treatment systems. *Water Science and Technology, 40*(3), 1-9. doi:https://doi.org/10.1016/S0273-1223(99)00414-X

Diels, L., Kramer, A., Spaans, P. H., Roy, S. v., & Wouters, H. (1999).

Diels, L., Spaans, P. H., Van Roy, S., Hooyberghs, L., Ryngaert, A., Wouters, H., . . . Tsezos, M. (2003). Heavy metals removal by sand filters inoculated with metal sorbing and precipitating bacteria. *Hydrometallurgy, 71*(1), 235-241. doi:https://doi.org/10.1016/S0304-386X(03)00161-0

Field Factors, x. (2018). Field factors. Retrieved from https://fieldfactors.com/

Gibbs, S. (2009). Smart Management for Public works. *Water & wastes digest*.

Hijnen, W. A. M., Schijven, J. F., Bonné, P., Visser, A., & Medema, G. J. (2004). Elimination of viruses, bacteria and protozoan oocysts by slow sand filtration. *Water Science and Technology, 50*(1), 147-154. doi:10.2166/wst.2004.0044

Huisman, L., & Wood, W. E. (1974). *Slow sand filtration*. Retrieved from

Kivaisi, A. K. (2001). The potential for constructed wetlands for wastewater treatment and reuse in developing countries: a review. *Ecological Engineering, 16*(4), 545-560. doi:https://doi.org/10.1016/S0925-8574(00)00113-0

Lin, Y.-F., Jing, S.-R., & Lee, D.-Y. (2003). The potential use of constructed wetlands in a recirculating aquaculture system for shrimp culture. *Environmental Pollution, 123*(1), 107-113. doi:https://doi.org/10.1016/S0269-7491(02)00338-X

Logsdon, G. S., Kohne, R., Abel, S., & LaBonde, S. (2002). Slow sand filtration for small water systems. *Journal of Environmental Engineering and Science, 1*(5), 339-348. doi:10.1139/s02-025

Martin, R. (2013). Clogging issues associated with managed aquifer recharge methods. *IAH Commission on Managing Aquifer Recharge*.

Matamoros, V., García, J., & Bayona, J. M. (2008). Organic micropollutant removal in a full-scale surface flow constructed wetland fed with secondary effluent. *Water Research, 42*(3), 653-660. doi:https://doi.org/10.1016/j.watres.2007.08.016

Mulling, B. T. M., van den Boomen, R. M., van der Geest, H. G., Kappelhof, J. W. N. M., & Admiraal, W. (2013). Suspended particle and pathogen peak discharge buffering by a surface-flow constructed wetland. *Water Research, 47*(3), 1091-1100. doi:https://doi.org/10.1016/j.watres.2012.11.032

opMAAT. (2005). Helophyte filter on Erasmusgracht, Amsterdam. Retrieved from https://www.urbangreenbluegrids.com/projects/helophyte-filter-on-erasmusgracht-amsterdam/

https://www.urbangreenbluegrids.com/projects/rijkswaterstaat-office-in-terneuzen-the-netherlands/

Paalman, M., Appelman, W., Creusen, R., Stein, N., Raterman, B., & Voogt, W. (2012). *Enlarging selfsufficient horticultural freshwater supply: greenhouse areas Haaglanden (part 1) (in Dutch)*. (KvK105/2013A). Retrieved from Utrecht, The Netherlands:

Poff, J., & Wilson, B. (2010). Flow pacing for optimum results. *Water & wastes digest*.

Pyne, R. D. G. (1995). Groundwater Recharge and Wells: A Guide to Aquifer Storage Recovery.

Pyne, R.D.G., 2005. Aquifer Storage Recovery - A guide to Groundwater Recharge Through Wells. ASR Systems LLC, Gainesville, Florida, USA, 608 pp.

Scholz, M., Harrington, R., & Carroll, P. (2007). The Integrated Constructed Wetlands (ICW) concept. *Wetlands*. doi:https://doi.org/10.1672/0277-5212(2007)27[337:TICWIC]2.0.CO;2

Scholz, M., & Lee, B. h. (2005). Constructed wetlands: a review. *International Journal of Environmental Studies, 62*(4), 421-447. doi:10.1080/00207230500119783

Sheoran, A. S., & Sheoran, V. (2006). Heavy metal removal mechanism of acid mine drainage in wetlands: A critical review. *Minerals Engineering, 19*(2), 105-116. doi:https://doi.org/10.1016/j.mineng.2005.08.006

Stichting Waterpark, x. (2018). Rietfilters produceren ecologisch water. Retrieved from http://www.hetlankheet.nl/index.php?option=com_content&view=article&id=3&Itemid=29

STOWA. (2016). Ondergronds bergen en terugwinnen van water in stedelijk gebied. Retrieved from

Stuyfzand, P.J., P. Nienhuis, A. Anthoniou & K. Zuurbier 2012. Haalbaarheid van ondergrondse berging via A(S/T)R in Hollands kustduinen. KWR-rapport KWR 2012.082, 107p.

Stuyfzand, P. J., & Raat, K. J. (2010). Benefits and hurdles of using brackish groundwater as a drinking water source in the Netherlands. *Hydrogeology Journal*. doi:10.1007/s10040-009-0527-y

Technical Commission Subsoil, T. (2009). *Diepinfiltratie van afvloeiend hemelwater* (TCB A047(2009)). Retrieved from

The Weather Underground, L. W. Retrieved from http://api.wunderground.com/api/be8a560b17cc78f1/forecast10day/q/NL/Bilt.json

Van Duivenbode, x., & Olsthoorn, x. (2002). *A pilot study of deep well recharge by Amerstdam Water Supply*. Paper presented at the Artificial Recharge of Groundwater, Adelaide, South Australia.

Visser, A. (2011). N-en P-verwijdering met fuzzy filtratie op de RWZI Nieuw Vossemeer. *Rapport/STOWA (2011 12) Show all parts in this series.*

Voogt, W. (2008). Waterkwaliteitsnormen glastuinbouw, herziene uitgave (in Dutch).

Vries, D., de la Loma, B., van de Schans, M. L., & Zuurbier, K. G. (2017). *Concepten voor snelle voorzuivering van ASR infiltratiewater*. Retrieved from

Zuurbier, K. G., Kooiman, J. W., Groen, M. M. A., Maas, B., & Stuyfzand, P. J. (2015). Enabling Successful Aquifer Storage and Recovery of Freshwater Using Horizontal Directional Drilled Wells in Coastal Aquifers. doi:10.1061/(ASCE)HE.1943-5584.0000990

Zuurbier, K. G., Raat, K. J., Paalman, M., Oosterhof, A. T., & Stuyfzand, P. J. (2017). How Subsurface Water Technologies (SWT) can Provide Robust, Effective, and Cost-Efficient Solutions for Freshwater Management in Coastal Zones. *Water Resources Management*. doi:10.1007/s11269-016-1294-x

Zuurbier, K. G., & Ros, S. E. M. (2017). *Herziene gietwaterbalans Glasparel+ (in Dutch)*. Retrieved from

Zuurbier, K. G., Zaadnoordijk, W. J., & Stuyfzand, P. J. (2014). How multiple partially penetrating wells improve the freshwater recovery of coastal aquifer storage and recovery (ASR) systems: A field and modeling study. *Journal of Hydrology, 509*, 430-441. doi:10.1016/j.jhydrol.2013.11.057

8 Appendix

8.1 Changes of particle size distribution during sample preservation

In order to verify the results of the particle counting measurements, the stability of the measured particles over time was checked. On 14-3-2018, direct measurements were performed on the samples on-site (Ovezande). These measurements were then repeated after 22, 44, 47 and 116 hours for the feed water (influent) sample and after 23, 45, 48, 117 hours for the product water (effluent) sample. Results for the feed water sample are shown in Figure 8-1 and Figure 8-2. Results for the product water sample are shown in Figure 8-3 and Figure 8-4.

For both the feed and product water samples a decreasing trend in total amount of particles can be observed with increasing sample age. After 24 hours, approximately 80%-90% of the original number of particles are still present in the sample. However, the size distribution relative to the total amount of particles seems stable over time. Note that after homogenizing the sample, a small volume (< 10 mL) was taken using a calibrated pipette and diluted in volumetric flasks; the slight error made in each of these steps can explain the slight variation in relative size distributions over time. Therefore, as the size distribution relative to total number of particles seemed stable, measurement of the sample within 24 hours after sampling is allowed.

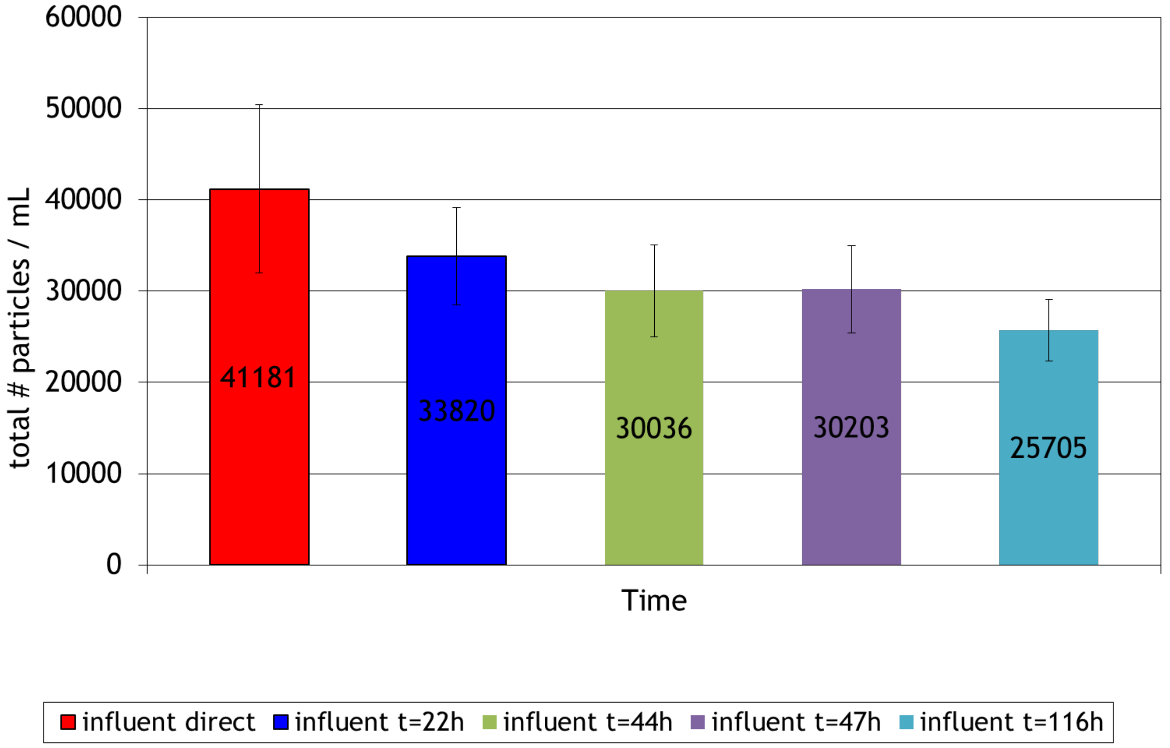


Figure 8-1: Total amount of particles of feed water over time of preservation

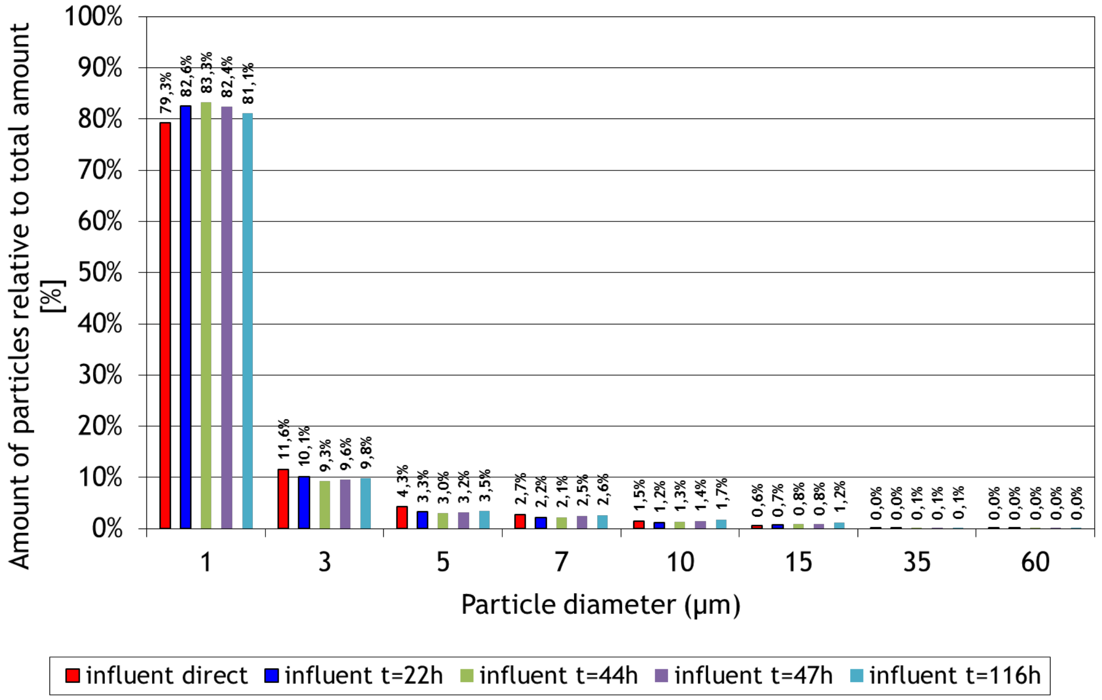


Figure 8-2: **Relative amount of particles with respect to the total amount of particles measured in feed water**

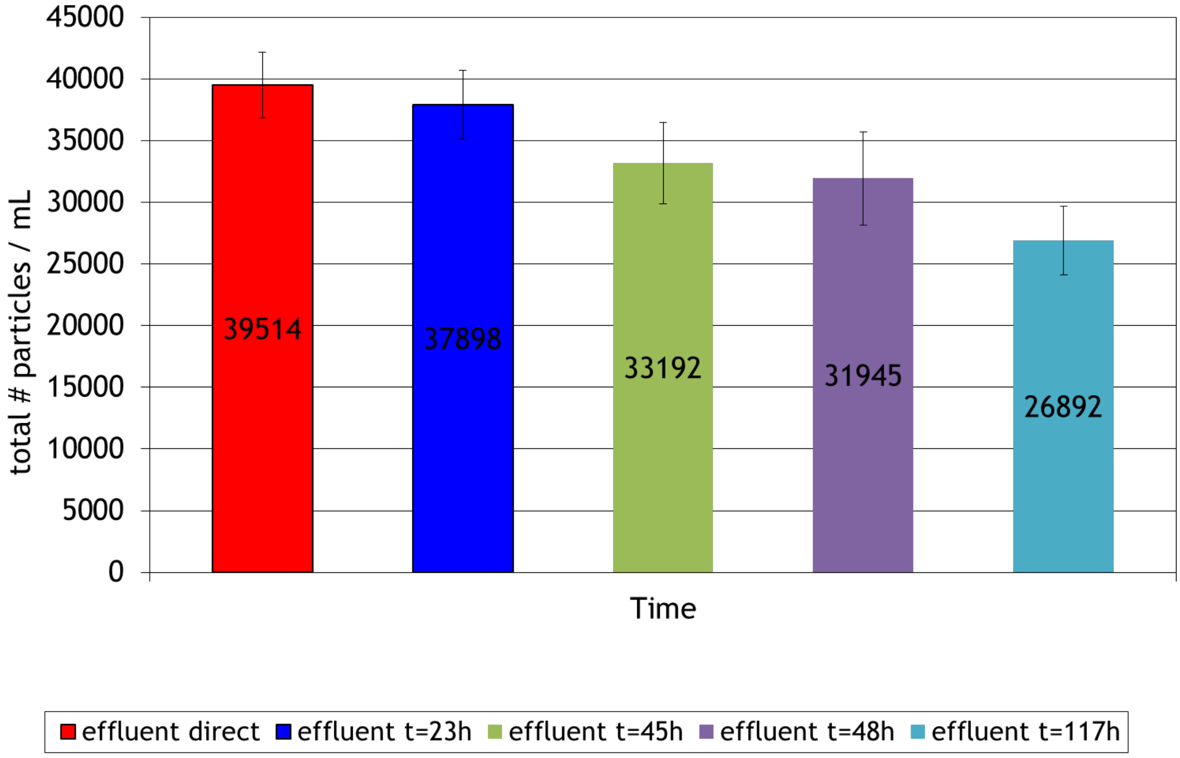


Figure 8-3: **Total amount of particles in product water over time**

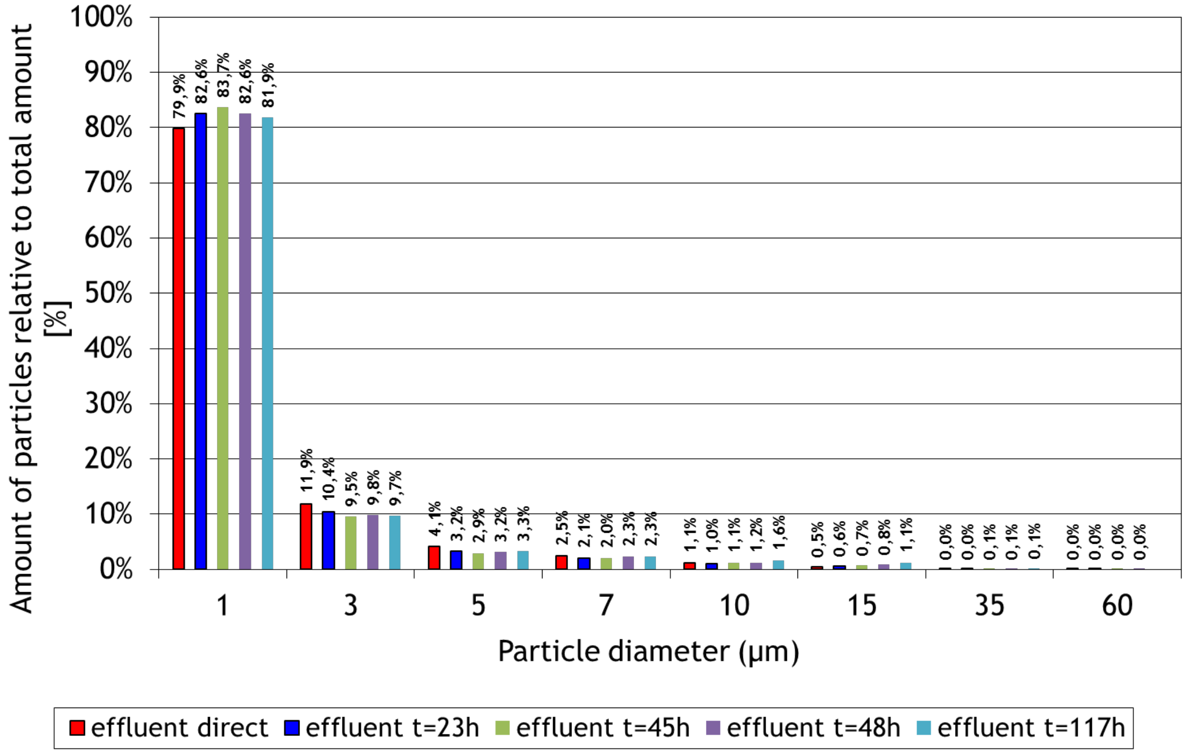


Figure 8-4: Relative amount of particles with respect to the total amount of particles measured in product water

8.2 DOC analysis by DOC-Labor

DOC-Labor Dr. Huber — D-O-C LABOR — www.doc-labor.de

LC-OCD Analyses of 3 Water Samples

Project IDs: yours / ours:	**B17110053DH / kwr_39 (A6020)**		
Contact:	**Wolter.Siegers@ Danny.Harmsen@ Luuk.de.Waal@kwrwater.nl**		
# std.diff / # UPW:	**3 / 0**		
Measuring Conditions:	**System: 031, Column: 52404 , Mobile Phase: Std. x 0.8, Flow rate: 2.0 mL/min**		
Sampling date:	**07 NOV 2017**		
Incoming date:	**16 NOV 2017**		
Measuring date:	**16 NOV 2017**	**data processing:**	**Dipl.-Ing. A. Balz**
Date of Report:	**17 NOV 2017**	**report generation:**	**Dr. S. Huber**

***Disclaimer*: We guarantee the correctness of analytical data according to the actual state or standard of science and technology. All interpretations are based on the assumption that samples are representative for a situation under investigation. We do not take responsibility for any action that is taken on the basis of our reports, irrespective of whether such action has been recommended by us or not. Reports are treated confidentially and are exclusive property of customer. Anonymized data may be used for scientific purposes if no additional agreements are made.**

Technical note: LC-OCD stands for "Liquid Chromatography - Organic Carbon Detection". Separation is based on size-exclusion chromatography (SEC) followed by multidetection with organic carbon (OCD), UV-absorbance at 254 nm (UVD) and organic bound nitrogen (OND). All concentration values refer to mass of organic bound carbon (OC). As a "rule-of-thumb" compound mass is about twice (for acids threefold) the value of OC. Chromatograms are processed on the basis of area integration using the program ChromCALC. In many samples the acid fraction contains low-molecular mass humic acids which are subtracted by ChromRES on the basis of SAC/OC ratio for HS. Thus, despite the visible presence of an acid peak there may no LMW acids be present.

SUMMARIC PARAMETERS:

***DOC (Dissolved OC)*:** Determined in the column bypass after in-line 0.45 µm filtration.

***HOC (Hydrophobic OC)*:** Difference DOC minus CDOC, thus all OC retained on the column is defined as „hydrophobic". This could be natural hydrocarbons or sparingly soluble "humins" of the humic substances family.

INORGANIC COLLOIDS (respond only in UV-Chromatograms): Negatively charged **inorganic** polyelectrolytes, polyhydroxides and oxidhydrates of Fe, Al, S or Si are detected by UV light-scattering (Raleigh-effect).

***CDOC (Chromatographic DOC)*:** This is the OC value obtained by area integration of the total chromatogram. Chromatographic subfractions of CDOC are:

***ROM = Refractory Organic Matter*:**
A: Humics (HS): In LC-OCD measurements there is a tight definition for HS based on retention time, peak shape and SAC. Calibration on the basis of "Suwannee River" Standard IHSS-FA and IHSS-HA. In addition, statistical data are given, like number-averaged molecular mass (Mn) and aromaticity (SAC/OC).
B: Building Blocks (BB): The HS-fraction is accompanied by shoulders, shape, concentration and UV-activity varies. This are sub-units of HS with molecular weights of 300-450 g/mol. Building Blocks are considered to be natural breakdown products of humics. They cannot be removed in flocculation processes.

***BOM = Biogenic Organic Matter*:**
C: Biopolymers (BP): This fraction is very high in molecular weight (20.000 - 2 Mio. g/mol), hydrophilic, not UV-absorbing. BP are typically polysaccharides but may also contain proteinic matter (this is quantified on basis of OND). BP exist only in surface waters.
D: LMW Organic Acids (OA): In this fraction all aliphatic, low-molecular weight (LMW) organic acids co-elute due to an ion chromatographic effect. A small amount of HS may fall into this fraction and is subtracted on the basis of SAC/OC ratios.
E: LMW Neutrals (NEU): Low-molecular weight (LMW weakly or uncharged hydrophilic or slightly hydrophobic ("amphiphilic") compounds appear in this fraction. This includes alcohols, aldehydes, ketones and amino acids. The hydrophobic character increases with retention time, e. g. pentanol appears at 120 min, octanol at 240 min. NEU may be in part refractory.

***SOM = Synthetic Organic Matter*:**
With LC-OCD all water-soluble synthetic organic compounds can be quantified and identified (after comparison with model compound) down to the low ppb-range. However, chromatographic resolution in SEC is moderate (about 15000 theoretical plates/metre). Typical examples for SOM are flocculant polymers, antiscalants, organic additives like amines, resin leaching products like polysulfonic acids (PSS) or trimethyl amine (TMA).

***Inorganic Colloids* (only visible in UV-detection):** Inorganic colloidal or particulate matter eluting slightly before the biopolymer fraction becomes visible by Raleigh light scattering. This material could be iron oxid hydrates or colloidal sulfur.

***SUVA (SAC/DOC)*:** Additional parameter derived from the ratio of DOC and SAC.

1

Discussion

End of report

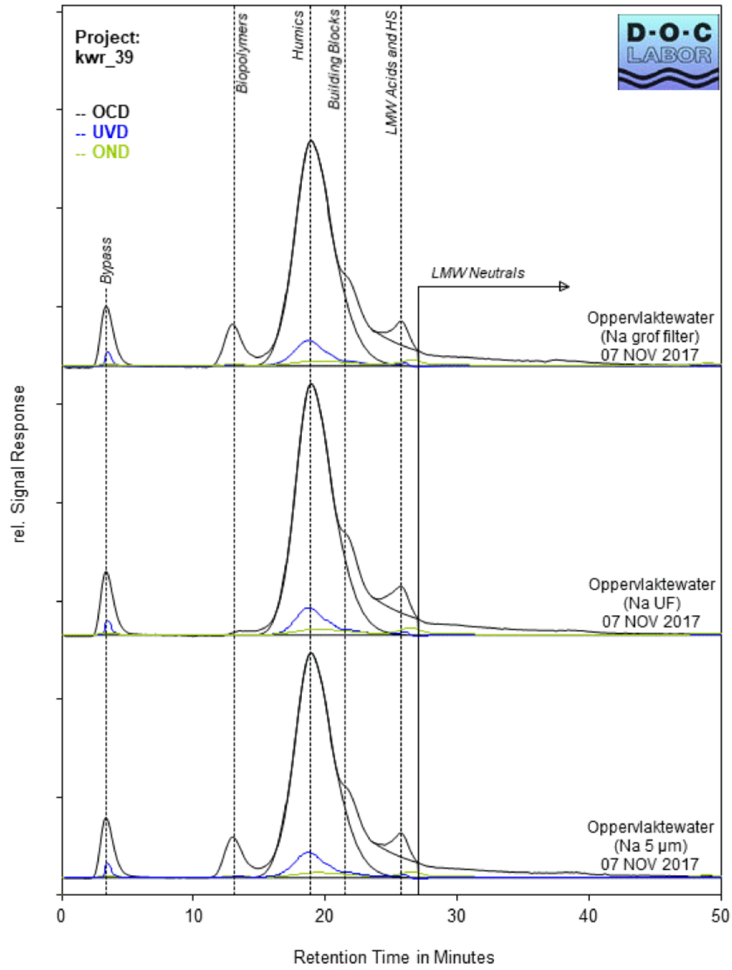


Fig. 1: LC-OCD Chromatograms

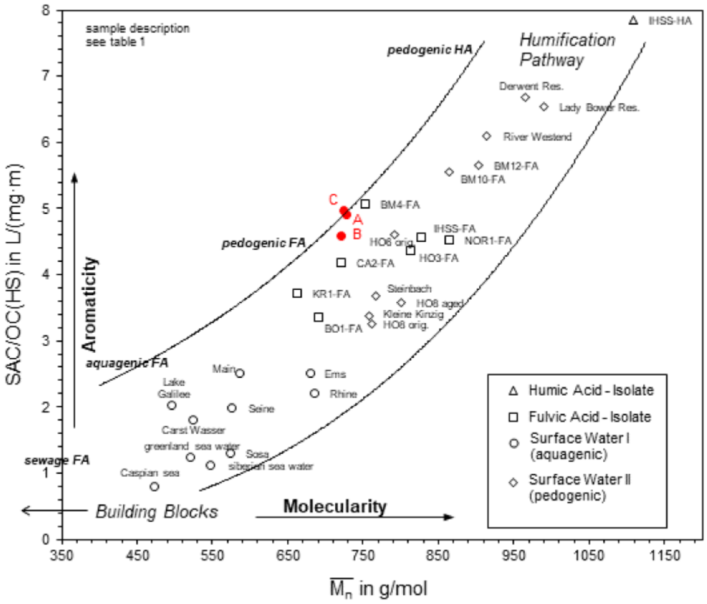


Fig. 2: HS-Diagram

5

Results

LMW = low-molecular weight
DON = Dissolved organic nitrogen
n.q. = not quantifiable (< 1ppb; signal-to-noise ratio)
n.m. = not measured

*:Grey colour in HOC: Significance unclear
**:under the presumption that all org. N in the BIOpolymer fraction originates from proteins
**: pale green: cross sensitivity inferred

Table 1

	DOC Dissolved	HOC* Hydrophob.	CDOC Hydrophil.	BIO-polymers (>>20.000)	DON (Norg)	N/C	% Proteins in BIOpol.**	Humic Subst. (HS) (~1000 (see separate HS-Diagram))
Project: kwr_39	ppb-C	ppb-C	ppb-C	ppb-C	ppb-N	µg/µg	% BIOpol.	ppb-C
	% DOC	% DOC	% DOC	% DOC	--	--	--	% DOC
Oppervlaktewater (Na grof filter)	31613	1709	29904	1954	155	0,08	24	21253
07 NOV 2017	100%	5,4%	94,6%	6,2%	--	--	--	67,2%
Oppervlaktewater (Na UF)	32840	2347	30493	331	41	0,12	37	23082
07 NOV 2017	100%	7,1%	92,9%	1,0%	--	--	--	70,3%
Oppervlaktewater (Na 5 µm)	30891	909	29981	1968	157	0,08	24	21256
07 NOV 2017	100%	2,9%	97,1%	6,4%	--	--	--	68,8%

Approx. Molecular Weights in g/mol:

	HS DON (Norg)	HS N/C	Aromaticity (SUVA-HS)	Mol-Weight (Mn)	Position in HS diagram	Building Blocks (300-500)	LMW Neutrals (<350)	LMW Acids (<350)	Inorg. Colloid. SAC	SUVA (SAC/DOC)
Project: kwr_39	ppb-N	µg/µg	L/(mg*m)	g/mol	--	ppb-C	ppb-C	ppb-C	(m^{-1})	L/(mg*m)
	--	--	--	--	--	% DOC	% DOC	% DOC	--	--
Oppervlaktewater (Na grof filter)	762	0,04	4,96	725	A	3631	2860	206	2,95	5,96
07 NOV 2017	--	--	--	--	--	11,5%	9,0%	0,7%	--	--
Oppervlaktewater (Na UF)	851	0,04	4,59	721	B	3882	2873	326	0,61	5,79
07 NOV 2017	--	--	--	--	--	11,8%	8,7%	1,0%	--	--
Oppervlaktewater (Na 5 µm)	734	0,03	4,90	728	C	3597	2921	241	2,08	6,13
07 NOV 2017	--	--	--	--	--	11,6%	9,5%	0,8%	--	--

8.3 Total cumulative overflow with different types of reservoir management

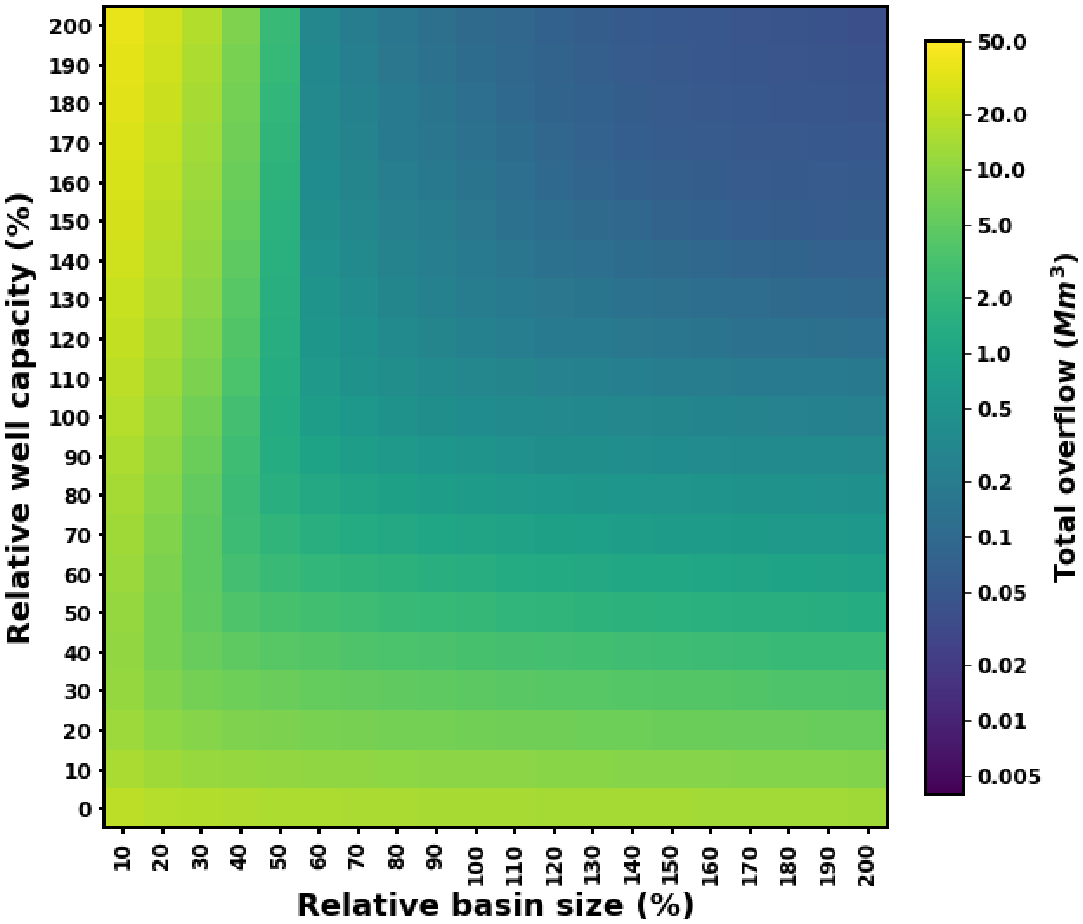


Figure 8-5: Cumulative total overflow from the owners' reservoirs and the storm water canals (in Mm3) with the reactive management approach (BAU) under various design conditions of relative well capacity and reservoir capacity. Model period: January 1987 – December 2017

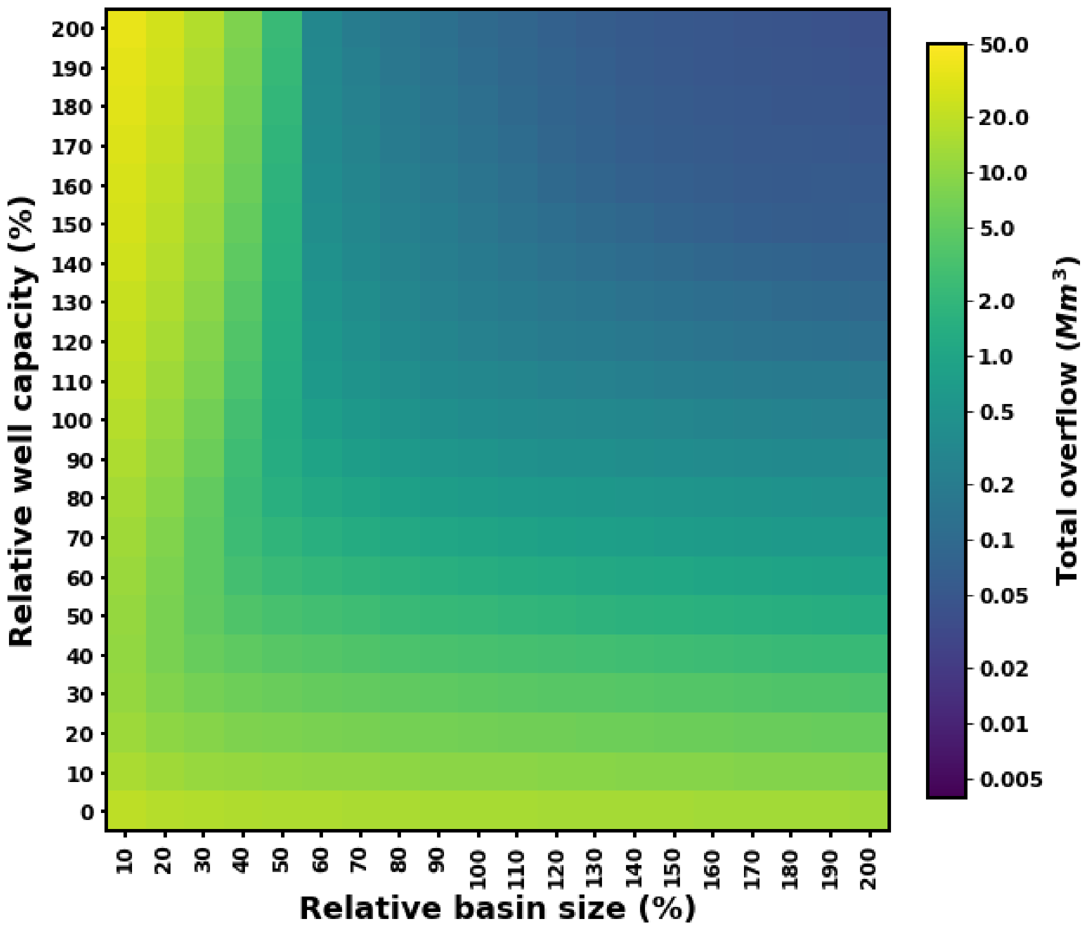


Figure 8-6: **Cumulative total overflow from the owners' reservoirs and the storm water canals (in Mm3) with the proactive management approach under various design conditions of relative well capacity and reservoir capacity. Model period: January 1987 – December 2017**

8.4 Infiltration decree groundwater-well protection (in Dutch)

In vigour as of 22-12-2009

Bijlage 1. (behoort bij artikel 3, eerste lid, van het Infiltratiebesluit bodembescherming)

Toetsingswaarden voor het te infiltreren water

nr.	stof	eenheid	toetsingswaarde (opgelost)[1]
	MACRO PARAMETERS		
1	zuurgraad (pH)	–	–[2]
2	zwev.stof	mg/l	0,5[3]
3	calcium (Ca^{++})	mg/l	–[2]
4	chloride (Cl^{-})	mg/l	200[2,3]
5	waterstofcarbonaat (HCO_3^{-})	mg/l	–[2]
6	natrium (Na^{+})	mg/l	120[2,3]
7	ammonium (NH_4^{+})	mg/l-N	
8	nitraat (NO_3^{-})	mg/l-N	5,6[2,3]
9	totaal-fosfaat (PO_4^{2-}-tot)	mg/l-P	0,4
10	sulfaat (SO_4^{2-})	mg/l	150[2]
11	fluoride (F^{-})	mg/l	1
12	cyaniden totaal (CN (tot))	µg/l	10
	ZWARE METALEN		
13	arseen (As)	µg/l	10
14	barium (Ba)	µg/l	200[3]
15	cadmium (Cd)	µg/l	0,4
16	cobalt (Co)	µg/l	20

l7	chroom (Cr)	µg/l	2
18	koper (Cu)	µg/l	15
19	kwik (Hg)	µg/l	0,05
20	nikkel (Ni)	µg/l	15
2l	lood (Pb)	µg/l	15
22	zink (Zn)	µg/l	65
	BESTRIJDINGSMIDDELEN		
23	som van de bestrijdingsmiddelen	µg/l	0,5[4]
	organochloorbestrijdingsmiddelen		
24	som (org.chl.bestr.mid.)	µg/l	0,1
25	endosulfan	µg/l	0.05
26	α-HCH	µg/l	0.05
27	-HCH (lindaan)	µg/l	0.05
28	DDT (incl.DDD en DDE)	µg/l	0.05
29	dichloorpropeen	µg/l	0.05
30	aldrin	µg/l	0,05
3l	dieldrin	µg/l	0.05
32	endrin	µg/l	0.05
33	heptachloor	µg/l	0.05
34	heptachloorepoxide	µg/l	0.05
35	hexachloorbutadieen	µg/l	0.05
36	hexachloorbenzeen	µg/l	0.05

	organofosforbestrijdingsmiddelen		
37	azinfos-methyl	µg/l	0,1
38	dichloorvos	µg/l	0,1
39	dimethoaat	µg/l	0,1
40	mevinfos	µg/l	0,1
41	parathion	µg/l	0,1
	triazines/triazinonen/aniliden		
42	atrazine	µg/l	0,1
43	simazin	µg/l	0,1
44	metolachloor	µg/l	0,1
	chloorfenoxyherbiciden		
45	2-methyl-4-chloorfenoxy-azijnzuur (MCPA)	µg/l	0,1
46	mecoprop	µg/l	0,1
47	2,4-dichloorfenoxy-azijnzuur (2,4 D)	µg/l	0,1
	ureumherbiciden		
48	chloortoluron	µg/l	0,1
49	isoproturon	µg/l	0,1
50	metoxuron	µg/l	0,1
51	linuron	µg/l	0,1

	chloorfenolen		
52	trichloorfenolen	µg/l	0,1
53	tetrachloorfenol	µg/l	0,1
54	pentachloorfenol	µg/l	0,1
	diversen		
55	dinoseb	µg/l	0,1
56	2,4 dinitrofenol	µg/l	0,1
57	bentazon	µg/l	0,1
	OLIE		
58	minerale olie	µg/l	200
	POLYCYCLISCHE AROMATISCHE KOOLWATER-STOFFEN (PAK's)		
59	naftaleen	µg/l	0,1
60	anthraceen	µg/l	0,02
61	fenanthreen	µg/l	0,02
62	cryseen	µg/l	0,02
63	fluorantheen	µg/l	∑ 0,1
64	benzo(a)anthraceen	µg/l	
65	benzo(k)fluorantheen	µg/l	
66	benzo(a)pyreen	µg/l	
67	benzo(ghi)peryleen	µg/l	
68	indeno(l23cd)pyreen	µg/l	

	GEHALOGENEERDE KOOLWATERSTOFFEN		
69	trichlooretheen	µg/l	0.5
70	tetrachlooretheen	µg/l	0.5
71	trihalomethanen (THM's)	µg/l	2[5]
72	dichloorfenolen	µg/l	0,5
73	adsorbeerbare organische halogeenverbindingen (AOX)	µg/l	30[6]

[1] De toetsingswaarde voor zwevende stof betreft de niet opgeloste hoeveelheid materiaal.

[2] Punt van aandacht bij de vergunningverlening i.v.m. lokale situatie.

[3] In het infiltratiewater mag 70 dagen per jaar een concentratie aanwezig zijn boven de hier genoemde, waarbij de volgende maxima niet overschreden mogen worden: zwevende stof 2 mg/l; CI^- 300 mg/l; Na^+ 180 mg/l en NO_3^{2}- 11,2 mg N/I; Ba 300 µg/l.

[4] Dit betreft de som van de concentraties van de in deze lijst genoemde bestrijdingsmiddelen, waarbij bepalingen waarvan het meetresultaat < detectiegrens is, een meetresultaat O wordt toegekend.

[5] THM te bepalen als som van de concentraties van chloroform, broomdichloormethaan, dibroomchloormethaan en bromoform. Als een transportchloring wordt toegepast, is het toegestane maximum 70 µg/l.

[6] Als een transportchloring wordt toegepast, is het toegestane maximum 100 µg/l.

8.5 Overview of relevant literature

Caliskaner, O., Tchobanoglous, G. and Carolan, A., 1999. High-rate filtration with a synthetic compressible media. Water environment research, 71(6): 1171-1177. Gibbs, S., 2009. Smart Management for Public works, Water & Wastes digest. Poff, J. and Wilson, B., 2010. Flow pacing for optimum results, Water & Wastes Digest. Visser, A., 2011. N-en P-verwijdering met Fuzzy filtratie op de RWZI Nieuw Vossemeer. Rapport/STOWA (2011 12)	Experiences with Fuzzy filter
Cheng, S. et al., 2002. Efficiency of constructed wetlands in decontamination of water polluted by heavy metals. *Ecological Engineering*, 18(3), pp.317–325. Available at: http://www.sciencedirect.com/science/article/pii/S092585740100091X Cooper, P., 1999. A review of the design and performance of vertical-flow and hybrid reed bed treatment systems. *Water Science and Technology*, 40(3), pp.1–9. Diels, L. et al., 1999. Method and plant for purification of metal containing water. Available at: http://www.google.com/patents/EP0952120A1?cl=en. Kivaisi, A.K., 2001. The potential for constructed wetlands for wastewater treatment and reuse in developing countries: a review. *Ecological engineering*, 16(4), pp.545–560. Available at: http://www.sciencedirect.com/science/article/pii/S0925857400001130. Lin, Y.-F., Jing, S.-R. & Lee, D.-Y., 2003. The potential use of constructed wetlands in a recirculating aquaculture system for shrimp culture. *Environmental Pollution*, 123(1), pp.107–113. Mulling, B.T.M. et al., 2013. Suspended particle and pathogen peak discharge buffering by a surface-flow constructed wetland. *Water research*, 47(3), pp.1091–1100. Available at: http://www.sciencedirect.com/science/article/pii/S0043135412008445. Matamoros, V., García, J. & Bayona, J.M., 2008. Organic micropollutant removal in a full-scale surface flow constructed wetland fed with secondary effluent. *Water Research*, 42(3), pp.653–660. Available at: http://www.sciencedirect.com/science/article/pii/S0043135407005465. Scholz, M. et al., 2007. The integrated constructed wetlands (ICW) concept. *Wetlands*, 27(2), pp.337–354. Scholz, M. & Lee, B.-H., 2005. Constructed wetlands: a review.	Experiences with natural water treatment systems

International journal of environmental studies, 62(4), pp.421–447. Sheoran, A.S. & Sheoran, V., 2006. Heavy metal removal mechanism of acid mine drainage in wetlands: a critical review. *Minerals engineering*, 19(2), pp.105–116	
Diels, L. et al., 2003. Heavy metals removal by sand filters inoculated with metal sorbing and precipitating bacteria. *Hydrometallurgy*, 71(1), pp.235–241. Hijnen, W.A.M. et al., 2004. Elimination of viruses, bacteria and protozoan oocysts by slow sand filtration. *Water Science and Technology*, 50(1), pp.147–154. Available at: http://wst.iwaponline.com/content/50/1/147. Huisman, L. & Wood, W.E., 1974. *Slow sand filtration*, World Health Organisation. Logsdon, G.S. et al., 2002. Slow sand filtration for small water systems. *Journal of Environmental Engineering and Science*, 1(1), pp.339–348.	Experience with slow sand filtration (SSF)
Hoogvliet, M., H. van Meerten, M. Paalman, M.L. van der Schans, M. Paalman, R. Stuurman, K. Broks (2016). Ondergronds bergen en terugwinnen van water in stedelijk gebied. STOWA 2016-01 .	Experience with infiltration of storm water runoff in urban areas
Pyne, R.D.G., 2005. Aquifer Storage Recovery - A guide to Groundwater Recharge Through Wells. ASR Systems LLC, Gainesville, Florida, USA, 608 pp.	Main textbook concerning ASR, filled with rich experiences of David Pyne. In particular experience in large-scale projects USA
Russel, M., (ed.), 2013. Clogging issues associated with managed aquifer recharge methods, 212 pp.	Monograph samengesteld door de IAH MAR specialist group en bevat met name praktijkervaringen rondom putverstopping bij MAR.
Schippers, J.C. and Verdouw, J., 1980. The modified fouling index, a method of determining the fouling characteristics of water. Desalination, 32: 137-148. Chinu, K.J. et al., 2009. Biofilter as pretreatment to membrane based desalination: Evaluation in terms of fouling index. *Desalination*, 247(1), pp.77–84. Available at: http://www.sciencedirect.com/science/article/pii/S0011916409004895.	Modified fouling index (MFI)
Stuyfzand, P.J., Nienhuis, P., Antoniou, A. and Zuurbier, K., 2012. Haalbaarheid van ondergrondse berging via A(S/T)R in Holland's kustduinen.	Broad overview of experience with ASR in drinking water and horticulture. Requirements for infiltration water to prevent

	clogging in continuous operated systems Experience for purification by draining into sediment Waternet
Zuurbier, K.G., Zaadnoordijk, W.J. and Stuyfzand, P.J., 2014. How multiple partially penetrating wells improve the freshwater recovery of coastal aquifer storage and recovery (ASR) systems: A field and modeling study. Journal of Hydrology, 509(0): 430-441.	Overview experiences with ASR in coastal areas
TCB, Technische Commissie Bodem (2009). advies Diepinfiltratie van afvloeiend hemelwater. TCB A047(2009).	Environmental risks of deep infiltration of storm water runoff
Van Duivenbode en Olsthoorn (2002), A pilot study of deep-well recharge by Amsterdam Water Supply KIWA Mededeling 79 (1984), Ervaringen met diepinfiltratie.	Experience with deep well infiltration in Amsterdamse Waterleiding duinen.
Zuurbier, K., M.L. van der schans, M. Paalman, P de Putter T. te winkel, J. Velstra G. Oude Essink (2015). Technisch-juridische handreiking risicobeoordeling ‘ondergrondse waterberging.’ STOWA 2015-35a.	Assess environmental risks and other risks to surrounding.
Zuurbier, K.G., Hartog, N. and Stuyfzand, P.J., 2016. Reactive transport impacts on recovered freshwater quality during multiple partially penetrating wells (MPPW-)ASR in a brackish heterogeneous aquifer. Applied Geochemistry, 71: 35-47.	Water quality change due to ASR