Effects of reducing extraneous water on the performance of wastewater treatment plants

MSc Thesis Civil Engineering, T.U.Delft:
Watermanagement

T. Visser

Photo on cover made by Erik Laurentzen

Effects of reducing extraneous water on the performance of wastewater treatment plants

by

T. Visser

in partial fulfilment of the requirements for the degree of
Master of Science
in Civil Engineering at the Delft University of Technology,

to be defended publicly on:
9th May 13.30

Student ID: 4024214

Graduation committee:

prof. dr. ir. J van Lier	T. U. Delft, Sanitary Engineering
dr. ir. H. Spanjers	T. U. Delft, Sanitary Engineering
dr. ir. J. Langeveld	T. U. Delft, Sanitary Engineering
dr. E. Mostert	T. U. Delft, Water management
ir. R. van der Veen	Waterboard Scheldestromen

Abstract

Besides wastewater and rainwater, the sewers in the Netherlands also transport a lot of water that is not supposed to be in the sewer. This water is called extraneous water and is the result of unwanted discharges in the sewer system. This could be groundwater infiltrating leaky sewer pipes, or pumped from construction sites directly into the sewer. Or it could be surface water flowing in from wrong illicit connections. In the Netherlands, all this extra water account for a quarter of all the influent that enters the treatment plants.

This thesis studies the effect of a reduction of extraneous water in the influent on the workings of the treatment plant. For this two cases are studied: the treatment plant of Dokhaven in Rotterdam and Willem Annapolder in Kapelle. Dokhaven is a high loaded treatment plant with an A-B configuration that treats the wastewater from the center of Rotterdam in which it is located. Willem Annapolder is a low loaded treatment plant with biological phosphor removal and pre-denitrification and treats the wastewater from different cities and municipalities in its surroundings connected by long pressure mains. Both cases are known to receive a lot of extraneous water in their influent.

Two models were made for each case study: one that simulates the dynamics of the influent concentrations and another that describes the water treatment processes in the treatment plant. After calibration, the models of the treatment plant were run with a different influent in which the extraneous water was reduced.

Previous studies have indicated that a reduction in extraneous water leads only to small changes in the effluent concentration, but due to the reduced flow leads to significant reduction in the effluent load. In the case of Dokhaven the models show similar results. In the case of Willem Annapolder, the effects of the pressure mains was also taken into account. This caused the resulting effluent ammonia concentration to increase when extraneous water was reduced. The reduction in clean extraneous water increases the dry weather concentration of the pollutants in the pressure mains. When a rain event occurs, this water with high concentration is pushed towards the treatment plant with increased flow, which causes increased peak loads at the start of every heavy rain event.
Although the overall nitrogen load on the effluent was also decreased at Willem Annapolder when the extraneous water was reduced, the pressure mains resulted in a much lower decrease than in the case of Dokhaven. This thesis thus shows that it is important to take into account the sewer system when evaluating the effects of reducing extraneous water.

Samenvatting

Het rioolstelsel vervoert afval- en regenwater in een stedelijke omgeving. In het geval van een gemengd systeem worden beide vervoerd naar een rioolwater zuiveringsinstallatie; in het geval van een gescheiden systeem gaat alleen het afvalwater naar de zuivering.
Naast het water dat het riool behoort te vervoeren is er ook heel veel water in het riool dat er niet in hoort: het zogenaamde rioolvreemd water or parasitaire water. In Nederland is dit gemiddeld iets meer dan een kwart van al het water dat bij de zuivering aankomt. Rioolvreemd water komt voort uit ongewenste afvoer naar het riool. Dit kan grondwater zijn via scheuren in de leidingen, of hemel- of oppervlaktewater via foutieve aansluitingen. Wat rioolvreemd water precies is, hangt af van het type rioolstelsel; regenwater mag worden vervoerd door een gemengd stelsel, maar hoort niet thuis in een vuilwaterriool van een gescheiden stelsel.
Rioolvreemd water krijgt steeds meer aandacht in de wereld van de gemeenten en waterschappen. Dit is een gevolg van de nieuwe uitdagingen die het rioolstelsel en de daaraan verbonden zuiveringen te wachten staan, zoals meer zware regenbuien en hogere eisen voor de effluent kwaliteit voor de zuivering. Als dit extra water niet meer hoeft te worden vervoerd is er meer capaciteit voor het opvangen van hevige buien, wat leidt tot minder riool overstorten. Als dit extra water, en vaak schoon, water niet gezuiverd hoeft te worden zijn de kosten lager.

Over de precieze effecten van minder rioolvreemd water op de zuivering is minder bekend. Dit is dan ook het vraagstuk van deze thesis: wat zijn de gevolgen van een vermindering in rioolvreemd water op de werking van de rioolwater zuiveringsinstallaties? Eerdere studies geven over het algemeen positieve resultaten. Een studie gedaan door STOWA over de vermindering van hydraulische belasting op een basis zuivering geeft aan dat dit een hoger zuiveringsrendement als gevolg heeft (Korving et al. 2015). De meest recente studie gedaan door Dirckx et al. (2019) bestudeerde verschillende zuiveringen in België om te kijken wat de gevolgen zijn van minder rioolvreemd water in het influent. Deze studie concludeerde voornamelijk in een vermindering in de effluent vracht op het ontvangende water.

Deze thesis onderzoekt de gevolgen van een vermindering van rioolvreemd water op twee specifieke zuiveringsinstallaties in Nederland: de zuivering Dokhaven in Rotterdam en Willem Annapolder in Kapelle. De eerste zuivering is een hoog belaste zuivering ontworpen volgens het A-B systeem. Gelegen in het centrum van Rotterdam zuivert Dokhaven het grootste deel van het afvalwater van deze stad. Willem Annapolder is een laag belaste zuivering met een configuratie voor biologische fosfaat verwijdering. Deze zuivert het afvalwater van verschillende steden en dorpen in de regio, vervoerd door lange persleidingen.

Om de gevolgen van een vermindering in rioolvreemd water te kunnen bestuderen zijn er twee verschillende modellen gemaakt: het eerste model beschrijft de verandering in influent concentraties; het tweede model beschrijft de zuivering die dit influent behandeld.
Voor het model van het influent is gebruik gemaakt van een empirisch model ontwikkeld door van Daal-Rombouts voor haar PhD onderzoek in realtime control in stedelijke afvalwatersystemen. Op basis van de tijdreeks van het influent en de routine metingen van de waterkwaliteit, worden er tijdreeksen gemaakt op uurbasis voor de stoffen ammonium, fosfaat, CZV en onopgeloste bestanddelen. Voor de case Willem Annapolder is hierbij ook het effect van het transport door de persleidingen aan toegevoegd. Hiermee wordt er rekening gehouden in de verschuiving in dynamiek tussen het debiet en de concentratie.
De modellen voor de zuivering zijn gemaakt met het programma BioWin. Hierin worden de processen die het afvalwater behandelen gesimuleerd. Na kalibratie zijn de modellen van de zuivering gebruikt voor het behandelen van een aangepast influent waarin het rioolvreemd water is verminderd.

De resultaten voor Willem Annapolder laten zien dat de vermindering van rioolvreemd water nadelige effecten kan hebben voor een zuivering verbonden aan lange persleidingen. Door de reductie in rioolvreemd water verhoogt de droogweer concentratie in de leidingen. Bij een hevige regenbui wordt dit afvalwater met hogere concentratie in een prop-stroom naar de zuivering geduwd, wat een hoge piekbelasting veroorzaakt. Uit de metingen blijkt dat Willem Annapolder al moeite heeft met het verwerken van het afvalwater na hevige regenbuien. Hier zou het verminderen van rioolvreemd water een nadelig effect hebben omdat dit de piekbelasting verhoogt. De uiteindelijke effluent concentratie en belasting voor ammonium zijn hoger voor het ontvangende oppervlakte water. Alhoewel de uiteindelijk concentraties van de totale stikstof in het effluent hoger worden, heeft de vermindering van het influent debiet wel tot gevolg dat de effluent vracht van stikstof afneemt.

In het geval van Dokhaven zijn de resultaten meer vergelijkbaar met die van de studie van Dirckx et al. (2019). De ammonium concentratie in het effluent van de zuivering verandert nauwelijks; de beluchting in de B-trap wordt gestuurd op een bepaalde effluent concentratie. De vrachten van ammonium en stikstof nemen wel aanzienlijk af.

Het verminderen van rioolvreemd water heeft niet alleen maar gunstige gevolgen. In de situatie dat het rioolvreemd water veelal uit grondwater bestaat in de stad, kan het weren van de infiltratie leiden tot een ongewenste grondwater spiegel stijging. Deze thesis laat zien dat verminderen van rioolvreemd water ook ongewenste effecten kan hebben op de werking van de zuivering als deze verbonden is aan lange persleidingen: de effluent ammonia concentraties kunnen hoger uitvallen. Wat ook moet meegenomen worden is de criteria waarop een zuivering wordt beoordeeld. Dit gebeurt momenteel op de effluent concentraties, die niet zozeer verbeteren met minder rioolvreemd water. De stikstof vracht op het effluent neemt wel af.

Acknowledgements

This thesis would not have been what it is, if it was not for the help and insights of many people. First of all I ow many thanks to my supervisor Jeroen Langeveld for providing me with this research topic and guiding me along the way. I also thank him for his patience with me, as this has been a very long process. Between the start and the end of this thesis I have moved places four times, met my girlfriend, started working as consultant, stopped working as a consultant and eventually started working at my current job in Almere. Jeroen provided me with new energy and insights to keep the work on this thesis going along the way.

I would like to thank Floor Besten and Willem Verwaal from the waterboard Hollandse Delta for providing me with the necessary information on the workings of the Dokhaven treatment plant and especially Willem for taking the time to welcome me at the plant. In this regard I would also like to thank Jimmy van Opijnen for explaining me the workings of his model of Dokhaven, which I could use to build the model used in this thesis.

At the waterboard Scheldestromen I would like to thank Ron van der Veen, Jo Nieuwlands and Marc Augustijn for providing me with information on and insight in the workings of the plant Willem Annapolder. Though the travel time is long, I always found my trips Middelburg very useful and very much worth the effort.

Many thanks go to Henri Spanjers for helping me with BioWin. Making and calibrating the correct BioWin models for this study has been a very time consuming struggle. Henri has helped me find the necessary solutions when I thought I tried everything.

I would also like to thank Erik Mostert for his willingness to join my commission in the last minute, and providing new insights for the discussion. And I would like to thank professor Jules van Lier for his insights at the start and end of the process that helped to form this thesis.

I ow everything to my, probably way too supportive, parents Leo and Dominique, who I can't thank enough for their support and patience with me. Without them I would never have been able to finish this study, which I might have started as derailed thought from a chemo-brain, but am very happy to have done. And last but not least I would like to thank Iris, without whom the last couple of years would have been much less enjoyable.

Contents

Abstract v

Samenvatting vii

1. Introduction 2

2. Case studies 7
    2.1. Willem Annapolder 9
    2.2. Dokhaven
    2.3. Data used

3. Method 12
    3.1.Influent 13
        3.1.1. Influent generator 13
            3.1.1.1. Dry weather flow pattern
            3.1.1.2. Characterisation of rain events
            3.1.1.3. Dilution processes
            3.1.1.4. Pressure mains
            3.1.1.5. Calibration and accuracy
        3.1.2. Influent characterisation 26
            3.1.2.1. Nitrogen
            3.1.2.2. Phosphate
            3.1.2.3. COD
            3.1.2.4. other
        3.1.3. Reject water sludge treatment 29
        3.1.4. Modelling the reduction of extraneous water 31
    3.2.Wastewater treatment plant 33
        3.2.1. Willem Annapolder 33
        3.2.2. Dokhaven 36
    3.3.Sensitivity analysis 44

4. Results 48
    4.1.Willem Annapolder 49
    4.2.Dokhaven 55

5. Discussion 60

6. Conclusion 64

7. References 66

Appendices 69

1 Introduction

The system that takes care of our sewage has evolved to be ever more complicated. From sanitary waste first being disposed into the open water, it was later collected through an underground system of pipes to improve hygiene conditions. When more attention was paid to the environmental consequences we started to treat the wastewater before it was discharged to the environment. Every step in the development solved the problems of the time, after which time and energy became available to look at the next problem.

The current challenges in the Netherlands are about climate change, resource recovery and micro-pollutants. The sewers have to become more robust and versatile to tackle more extreme weather variabilities and the sewage treatment system has to become even better in removing the smallest pollutants, while at the same time deal with the effects the extreme weather has on the sewers. Because of these new challenges there is a new interest in extraneous water — also called pirate or parasite water or inflow and infiltration (I/I). It is generally accepted that the removal of this extraneous water is good for the performance of the system that collects and treats our sewage and could therefore help to address the current challenges. Less water in the sewer means it has more capacity to transport storm water, and a lower hydraulic load on the treatment plant increases its performance. However, detailed studies on the exact effects of the removal are scarce. This is where present research aims to contribute.

Extraneous water

Infiltration and inflow is defined by Butler and Davies (2000) as unintended discharges into the sewer system. Extraneous or parasite water is thus the name for all the water in the sewer network which is not supposed to be in there. There are different types of sewer systems. The most common are combined systems that transport both rainwater and sewage or separate systems that transport rainwater and sewage in separate pipes. What contributes to extraneous water therefore depends on the kind of sewer system; rainwater is allowed in the combined system, but counts as parasite water in a sanitary sewer.

This water has mainly three possible sources: groundwater, rainwater, and surface water. The parasite water is thus mostly clean water. Basically the sewer pipes act as a drainage for the surplus of surface or groundwater, which can enter the sewer through different pathways. Bennet (1999) differentiated three components of I/I: groundwater infiltration (GWI), rain induced infiltration (RII) and rain-derived inflow (RDI) (Staufer et al 2012, citing Bennet 1999). The groundwater infiltration is a reasonably steady flow and related to the groundwater level at the location of the pipes. Because of this relation it usually shows similar seasonal fluctuations. The rain induced infiltration is similar to the groundwater infiltration — same pathway — but is directly related to the rainfall and therefore has a higher variability. The rain-derived inflow consists of rainfall run-off that flows directly into the sewer through false connections of stormwater drains or manhole openings (Staufer et al 2012). This latter flow is only extraneous water in the case of a separate sewer system, where the rainwater enters the sanitary sewer instead of the storm sewer. Other forms of inflow are also possible: extracted groundwater from construction sites being discharged in the sewer, the inflow of surface water through ill-constructed combined sewer overflows (Weiss et al 2002), or captured water from old culverted streams or connected springs (Broadhead et al. 2013).

The occurrence of inflow and infiltration depends on different factors, which can be categorised in three main categories: natural characteristics —geology and hydrology of the drainage area—, characteristics of the sewer network — method of making pipes, connections and manholes —, and characteristics of the water supply system — the leakage from the drinking water system. Of these three the characteristics of the drainage system are probably most influential (Dimova et al. 2015). In the case of groundwater infiltration for example there is a strong link between the infiltration and the construction date of the sewer pipe (Karpf and Krebs 2011).

Detection

It is not easy to find or quantify the extraneous water in a system, but there is a variety of methods available that try to do just that. These can be divided into methods that use the flow rate or chemical methods. The former looks at the hydrographs—a time record of the quantities— of the incoming water at the treatment plant; the chemical methods analyse the composition of the incoming water to distinguish the different sources (De Bénédittis and Bertrand-Krajewski 2005).

The flow rate methods are similar to methods used in hydrology to look at the behaviour of different elements in a hydrological catchment. The influent of the treatment plant is separated into a sanitary flow — from households and industries —, an extraneous water flow — from surface and groundwater —, and a flow from direct rainfall. As a starting point this method requires long time series of the inflow at the treatment plant. The amount of extraneous water can then be estimated in different ways.

The direct analogy with the base flow of a river is used by Wittenberg and Aksoy (2010) to estimate the infiltration of groundwater in the sewer systems. By using a nonlinear storage model on the influent measurement of one year, the study fractionates the influent and distinguishes the groundwater base flow from the average sewage flow, assuming the latter is constant. Supporting the analogy with the river, the same study also shows a strong correlation between the flow in the sewer and the water level of the nearby river.

Weiss et al. (2002) describe two easy methods of estimating I/I. One is the moving minimum method: the "*sum of sanitary sewage plus I/I at any day is set equal to the minimum daily inflow during the past 21 days*" (Weiss et al. 2002: 14). Although it lacks a physical background, according to the sensitivity analysis done in the study it provides a good estimate.

The other method described is the triangle method. It is based on two assumptions: the sanitary sewage flow is constant and the infiltration and inflow is highest after wet weather periods. Data of the daily mean inflow at the treatment plant are ranked in ascending order for one year. On the resulting graph, shown in figure 1.2, the infiltration and inflow is distinguished from the surface runoff and the constant sanitary flow. This is done with by attributing the highest flows to the days with storm water runoff; the higher flow on the dry days then comes from infiltration and inflow (Weiss et al. 2002).

These methods are also used in the DWAAS methodology made by STOWA, in which it is referred to as the Weiss Brombach method (Kerk and Wieringen 2005). DWAAS — translated as dry weather flow analysis system — works with the existing data and consists of the following steps: (1) comparing the daily influent with the water use in the area, (2) determining the theoretical dry weather flow, (3) moving minimum method on a 7 day period, (4) determining the amount of extraneous water using the Weiss-Brombach method, (5) assessing the results, (6,7) looking at weekend and seasonal effects. Here step three and four are the moving minimum and the triangle method described by Weiss et al. The last two steps look at the influence of discharges of large companies and try to see groundwater infiltration variations.

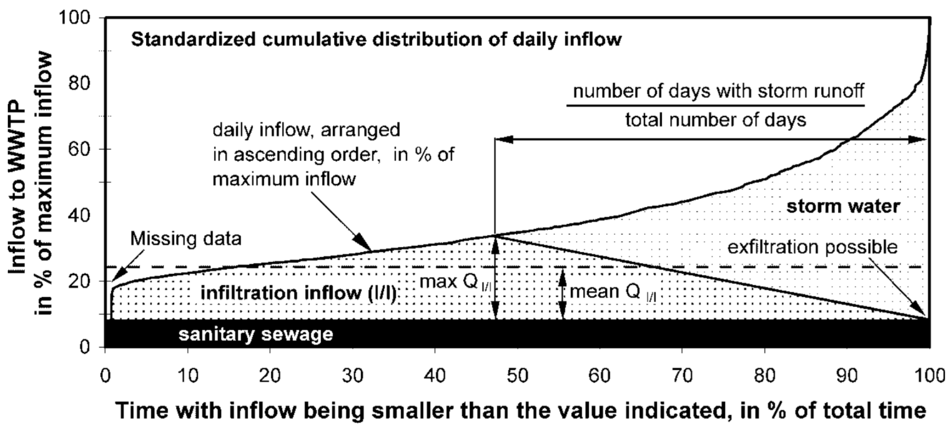


Figure 1.2: Triangle method (Weiss et al. 2002)

The chemical methods look at the water quality and make use of the difference in properties of the clean parasite water and the water the sewer is supposed to transport. The extraneous water alters the overall quality of the water that arrives at the treatment plant, which allows for its quantification.

To indicate the amount of infiltrated groundwater during dry weather, Kracht and Gujer (2005) use COD as a tracer, because the COD concentration in the groundwater can be assumed as negligible. A continuous COD measurement allows for a time

series analysis of the flow of both infiltration and wastewater flow (Kracht and Gujer 2005). Bares et al. (2012) elaborated on this to measure the extraneous water at a wastewater treatment plant in Prague. After using similar measuring techniques — COD and flow measurements and a mixing model — they extended the limited data using Monte Carlo analysis with positive results, thereby overcoming the limit to dry weather periods (Bares et al. 2012).
It is possible to use the same method but with different water quality parameters. An example is the use of total suspended solids (TSS) measurements, which are cheaper to monitor than COD but give less accurate results (Bares et al. 2009). Aumond et al. (2008) measured conductivity and turbidity of a mixed combined and sanitary sewer system. This showed that mostly conductivity in relation with flow rate measurements are able to estimate infiltration and inflow on a daily scale.

Another inherent indicator of the source of the water are the different stable isotopes of H_2O. When the drinking water — becomes sewage — comes from a different source as the local groundwater — a proxy for the infiltrating water— these isotopes can be distinct enough to quantify the two different sources of water. This therefore requires a thorough investigation in the homogeneity in isotopes of both the ground and drinking water in the area of interest. Using the same mixing model this can be used to measure the groundwater infiltration in the sewer pipes quite accurately (Kracht et al. 2007). HouHou et al. (2010) elaborate on this by adding the measurement of the isotopes of dissolved sulphate. The sulphate isotope helped to find the origin of the different water inputs, though due to instability it was unable to quantify the different contributions (HouHou et al. 2010).
In his master thesis, Schilperoort (2004) studies the infiltration and inflow in the Netherlands using this method. Because at some locations the groundwater resembled the drinking water too much, the method could not be used for each site studied. Using the isotopes did show interactions between the groundwater and surface water at some locations, which can be useful information on its own. When the amount of I/I could be quantified, it was around 39% of the dry weather flow. Schilperoort rightly states that this amount can not be directly compared to the values found with for example the DWAAS method, as the extraneous water found with the isotopes is the quantity during the time of measurement, while the other methods use the annual average.

Little research has been done on the accuracy of the different methods. One such study done by De Bénédittis and Bertrand-Krajewski (2005) compares different methods that estimate the infiltration in different sub-catchments. In addition to time series of the total influent, the studied methods determine the wastewater flow based on different data: annual drinking water consumption, number of inhabitants and reference values of discharge per capita, a residual night flow, and the continuous measurements of pollutant concentration. Most methods were comparable in indicating the sub-catchment with the most infiltration; the methods varied greatly in the indicated amount of infiltration. It was also found that methods based on night flow tend to consistently overestimate the infiltration when compared to other methods, but also gave the lowest uncertainty. Methods that combine flow data in combination with pollutant concentrations gave the most accurate results (De Bénédittis and Bertrand-Krajewski 2005).

Using stable isotopes is unfortunately not as accurate either. Prigiobbe and Giulianelli (2009) used a Monte Carlo random sampling to quantify the error propagation in using the isotope $_{18}O$ in hydrograph separation. This concluded that the uncertainty on the infiltration ration, which is the ratio between infiltrated groundwater and the waste water flow, is 20%. However the study also emphasises that this uncertainty can be very site specific and has be to evaluated on an individual case basis. Kracht et al. (2008) showed that using the stable isotopes of water gave similarly accurate results as the time series analysis of pollutographs.

How much is there?

It turns out that the percentage of extraneous water in the sewer system can be quite significant. In most cases it is expressed in terms of a percentage of the annual dry weather flow; this then varies from values around 20% (De bénéditis and Bertrand Krajewski 2005) up to 60% (Kerk and Wieringen 2005). A more direct and better quantification is in relation to the total influent reaching the treatment plant as done by Weiss et al. (2002). In this comparison of 34 treatment plant influents in Germany the average percentage of extraneous water is about 35%. Even in the case of a separate system misconnections and infiltration can contribute to a high amount of extraneous water; the study of Bugajsk et al. (2017) show that in the studied treatment plant this amount varied between 26 and 48% fo the total sewage inflow.

In the Netherlands the percentage of the total influent that is extraneous water is similar, with 26% of all the treated water being extraneous water. This is a little more than all the rainwater that is treated, which accounts for 24% of the total flow (Infographic extraneous water, Appendix 1).

Effects of extraneous water

All this extra — and mostly clean — water is transported towards and treated at a treatment plant. Since the capacity of the treatment plants and pumping stations are well capable of handling this extra water during dry weather, there has been little incentive to tackle this problem. But what are the consequences?

One way to look at it, is that the whole process of infiltration and inflow is a pollution problem (Weiss et al. 2002). When this relatively clean water enters the sewer system it mixes with the sewage and this mix is then treated at the treatment plant. The resulting outflow of the treatment plant has to fulfil certain standards, but the quality of this water is mostly lower than the quality of the original source of the parasite water. In this way the infiltration and inflow thus reduces the overall water quality.

There are a couple of studies that look more in detail to the effects of extraneous water on the affected wastewater treatment plant. A study in the research project 'Sealing of sewer pipes' by Rödel et al. (2017) looks at the effects of two different treatment plants, by comparing measurement data on moments of high and low infiltration and inflow. The overall trend found here is that a decrease in extraneous water leads to a decrease in energy consumption in the plant, and decreases the emitted pollution load.

STOWA — the Dutch Foundation of Applied Water Research — also has multiple studies that look at extraneous water. A general study on the reduction of the hydraulic load of treatment plants, sums up the reasons why a reduced influent flow, and thus a reduced amount of extraneous water, can be beneficial: it improves the recovery of nutrients and energy from wastewater, in reduces the needed investment costs for new treatment technologies, and similar to the findings of Rödel it reduces the energy costs and improves the effluent quality (Korving et al. 2015).

A preceding study of STOWA studies the effects of a reduced amount of rainwater or extraneous water on the treatment plant. This study looks more in detail to the working of the treatment plant by modelling the performance of a low loaded treatment plant (100.000 i.e.), coupled to an average Dutch sewer network. By coupling influents with different reductions in rainwater or extraneous water to a treatment plant model the effects on the performance are evaluated. This shows that the reduction in extraneous water has a limited or no effect on the effluent concentrations for nitrogen, but due to the reduced flow the effluent load is reduced (Geraarts and Langeveld 2008).

A similar recent study by Dirckx et al. (2019) comes to the same conclusion. They elaborated the same method as the STOWA study of 2008 by modelling four different case studies with both sewer and treatment plant models. Here the effects on the effluent quality are similar: a limited improvement on the effluent concentrations, but a reduced pollution load due to the reduced flow. Dirckx shows that the effects on the water quality are mostly due to a reduction of combined sewer overflows (CSO) in winter.

Present research

The goal of this thesis is to see the effects of removing extraneous water from the influent on the performance of the connected treatment plant. This research therefore continues on the mentioned research of STOWA — the effects of extraneous water in a sewer system on the performance of the connected wastewater treatment plant — by applying the

figure 2.1: Catchment areas of Dokhaven (above) and Willem Annapolder (below). The location of the treatment plants is marked by the blue squares. Willem Annapolder is connected to long pressure mains which are drawn in this figure. The catchment area of Dokhaven only has short pressure mains that connect the rest of the catchment area to the treatment plant.

same principles on two specific case studies: the domestic wastewater treatment plants of Dokhaven in Rotterdam (Zuid-Holland) and Willem Annapolder in Kapelle (Zeeland). It is also similar to the work of Dirckx (2019), albeit without the study on the occurrence of combined sewer overflows.

Besides studying the effects on two specific cases, this thesis adds one important aspect that has been missing in previous studies: the effect of different types of sewer systems in combination with the performance of the treatment plant. Willem Annapolder treats wastewater from surrounding municipalities which are connected by long pressure mains. The effect of removing extraneous water on the concentrations in the pressure mains has not been modelled before in combination with the performance in the treatment plant.

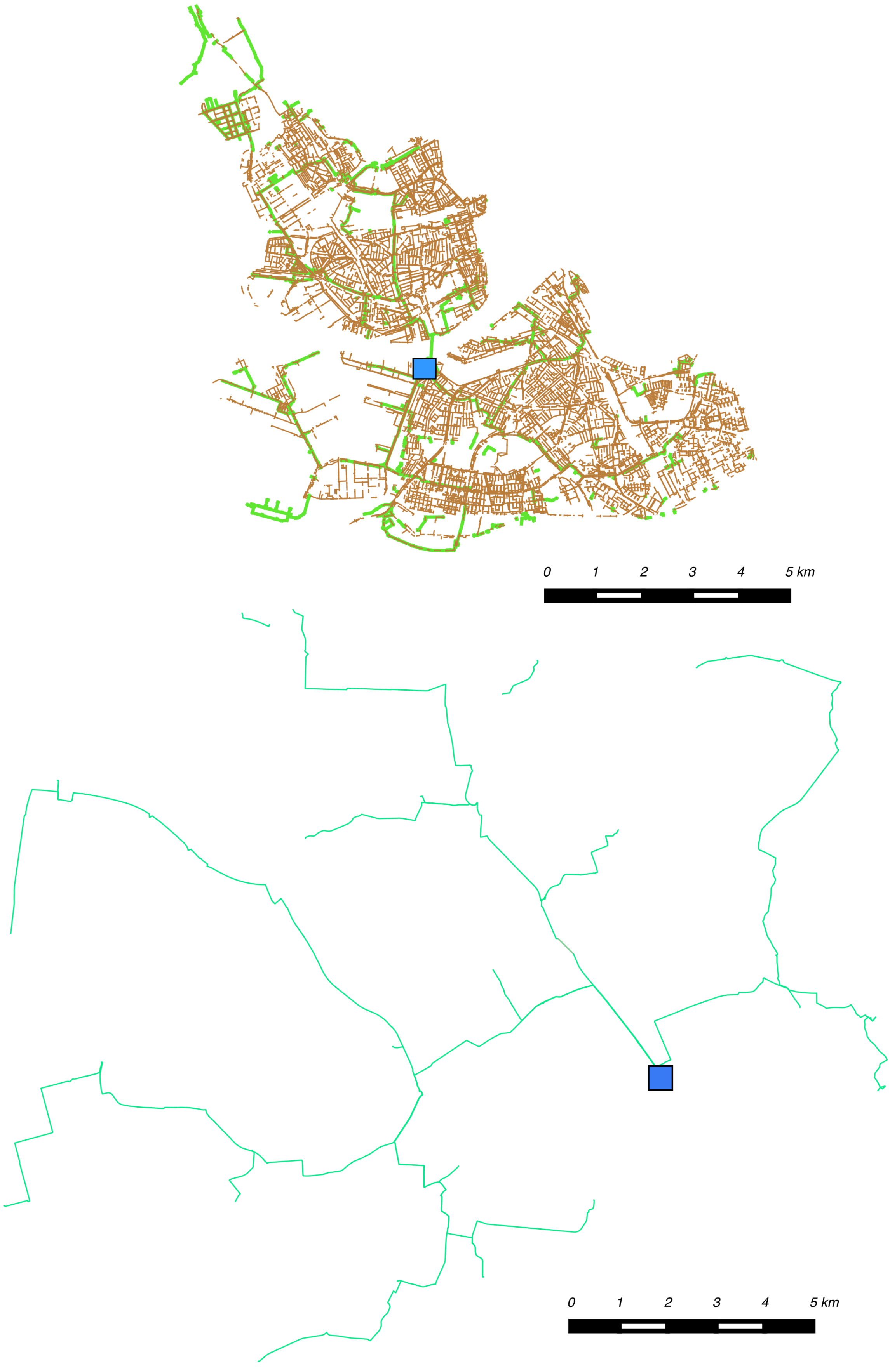


Figure 2.1: Sewer network of Dokahven (top) and the pressure mains connected to Willem Annapolder (bottom). In dokhaven the gravity sewers are represented in the brown lines. These are connected to the pressure mains that transport the water to the treatment plant represented by the blue square.

2
Case studies

This thesis studies two case studies: the treatment plant of Willem Annapolder and the treatment plant of Dokhaven. Both treatment plants are known to have a high percentage of extraneous water in their influent. The operating waterboard of Willem Annapolder — Waterschap Scheldestroom— has reducing extraneous water as one of the four trajectories in its current program SAZ+ (Samenwerking (afval)waterketen Zeeland — Cooperation in the sewage chain Zeeland). For this study the percentage of extraneous water was estimated with the Weiss Brombach method for the influent of Willem Annapolder. This is 16% of the total influent flow for the year 2015; values in previous years varied between 9 and 22%.
In Dokhaven the extraneous water in 2010 is estimated to be 34000 m^3/d, which results in a dry weather flow which is 68% higher than the theoretical dry weather flow (Vosse 2013). Over the course of a year this is approximately 30% of the total influent flow to the treatment plant.

Willem Annapolder and Dokhaven are different types of treatment plants: the first is a low loaded plant with a configuration for biological phosphorus removal with pre-denitrfication; the latter is a high loaded plant with an A-B system.
Not only the type of treatment plant is different, but they are also connected to a different kind of sewer system as can be seen in figure 2.1.1. Dokhaven treats the wastewater from the city centre of Rotterdam in which it is located. Willem Annapolder treats the wastewater of municipalities in its surrounding which are connected by long pressure mains. Also the hydraulic loading is different, with Willem Annapolder receiving an average of 25400 m^3/d and Dokhaven 114100 m^3/d. This makes the residence time in the sewer much longer in the case of Willem Annapolder.

Dokhaven treatment plant

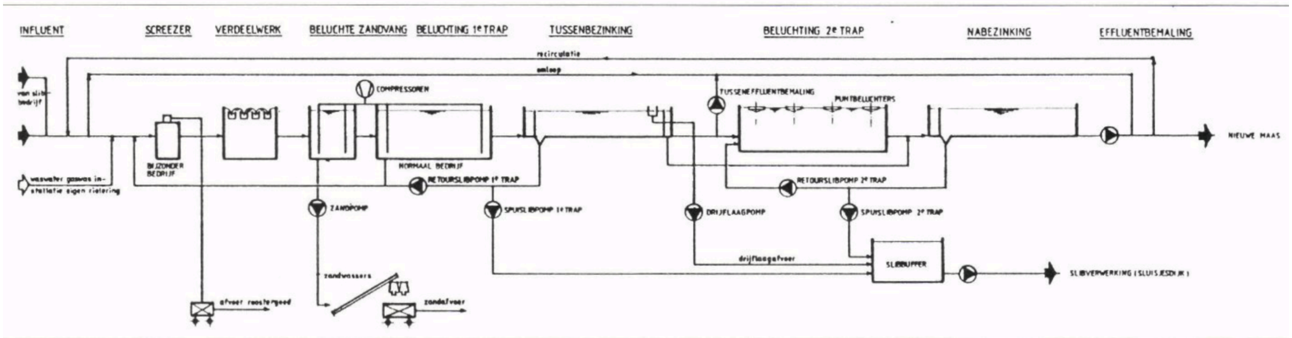


figure 2.2: plan DOKHAVEN (van der Vlies 1987)

Together with the plant Kralingseveer, Dokhaven treats the wastewater from the densely populated city centre of Rotterdam, with Dokhaven connected to the oldest part of the city. The wastewater is transported through a mostly combined sewer system. Eventually five pressure mains connect the different catchments in the city to the treatment plant.
The sludge from the treatment at Dokhaven is treated at a nearby facility at Sluisjesdijk. Next to the wastewater from the city, Dokhaven also receives the pre-treated rejection water from this process.

For the location of the plant the engineers followed the sewer lines from the city which are all directed at the river. The plant is therefore located at the location of an old harbour at the southern shore of the Maas. Because this is in a residential area the plant is build completely under ground. This makes it very difficult to change anything in the set up of the plant, which is now operating close to its maximum capacity.

The treatment plant setup follows the A-B process configuration which are laid out over parallel water lines. This system consists of two activated sludge tanks with accompanied settlers. The influent first passes a screener after which it is divided over the eight parallel A stages. Here the water first passes an aerated grit chamber which removes the largest solids. The grit chamber is aerated to save space and functions as a first aeration step. The flow then goes through the first of the two consecutive activated sludge systems: the high loaded A stage. In this stage the mixed liquor is first aerated by bubble aeration. Here most of the COD is removed by aeration and adsorption. After the aeration the mixed liquor is led to the accompanied settler. The sludge from this settler is diverted back to either the screens or directly back to the A-stage aeration tank. A constant flow of sludge is discharged as surplus sludge.
The B-stage is divided into four parallel streams, which means the effluent water of two A stage settlers are combined into the influent for one of the aeration tanks in the the low loaded second phase (B). The B stage aeration is done by four consecutive point aerators. After the aeration the mixed liquor again flows to the accompanied settlers. The sludge from the settler is partially wasted as surplus sludge, but most is recycled back to the B stage aeration tank. The water from these settlers is partially recycled back to the screens to be retreated again. The rest is discharged to the surface water as effluent (Meijer 1988).

Willem Annapolder treatment plant

The treatment plant of Willem Annapolder is designed for biological phosphorus and nitrogen removal, which is done in the anaerobic anoxic oxidation tank (AAO) configuration.
The sewage from the four connected areas comes together at the start of the treatment plant where it goes through the screens to take out the largest debris. After the screens the wastewater from the site joins the flow and the mix goes to the primary clarifier.
The effluent of the primary clarifier is combined the returning sludge flow from the secondary settlers and this resulting stream enters the biological reactor. This reactor has an anaerobic, anoxic, aerobic (A^2/O) configuration and is build as a concentric plug flow reactor with the first anaerobic stage in its centre (figure 2.3, upper right circle). The anaerobic tank is split into 5 segments in order to achieve the correct hydraulic retention time and sludge loading. These conditions are created to favour the growth of the polyphosphate accumulating organisms (PAO). From the anaerobic reactor the flow enters the fixed anoxic reactor, where it is combined with the recycled —nitrate rich— mixed liquor from the aerobic tank. The last stage

of the biological reactor is the combined aerobic/anoxic reactor. This reactor is aerated by three plate aerators. The size of the aerated volume in the combined aerobic anoxic reactor is configured in such a way that the total anoxic sludge volume is about 65% to have an adequate denitrification.

figure 2.3: Aerial photograph of the treatment plant Willem Annapolder

After this the mixed liquor is partially recycled back to the anoxic tank and partially divided over four secondary clarifiers. Two of the settlers are relatively new, where number three has the same surface area as the two old settlers. Settler number four is larger than the other three, as can be seen in figure 2.3 as the settler next to the A^2/O reactor, and it is from this settler that part of the sludge is extracted as surplus sludge. The rest of the sludge combined with the sludge from the other three settlers is diverted back to the anaerobic tank to maintain a high biomass level. The water flowing from the settlers is discharged onto the adjacent surface water of the Wester-Schelde.
The excess sludge from primary settler and secondary settler number four are dewatered and treated on site in an anaerobic digester. This digester is also fed with sludge coming from the nearby wastewater treatment plant of Waarde. The reject water from this sludge treatment process and the other on site wastewater streams are pumped to the primary clarifier.

Data used

Similar data sets are available for both the case studies, which are the standard measurements done at the wastewater treatment plants. The data used cover one calendar year: the year 2015 for Willem Annapolder and 2014 for Dokhaven. These are logged flows and concentration measurements inside the treatment plant and 24 hour average measurements of the influent and effluent concentrations taken every six or seven days. For the logged data hourly values are used. This data frequency allows to see the daily flow patterns and the dynamics during and after a rain event in the influent flow.

3 Method

This study aims to see the effects of the removal of extraneous water in the sewer system on the connected wastewater treatment plant. The primary focus is herein is the effect on the nitrogen removal. This is done by connecting two different models. The first is an influent generator that simulates the changes in concentration due to rain events; the resulting concentrations are the input for the second model which describes the processes in the treatment plant. The models of the treatment plants are made in BioWin. Both the influent and the BioWin models are made for the two case studies described above.

The following paragraphs explain the used models. First the construction of the influent model is described. This is followed by the set up of the models of the treatment plants. Finally a small sensitivity analysis is presented to see the effects of error propagation between the two different model types.

3.1 Influent

The performance of the treatment plants are studied with two different influents: one that represents the current situation and one in which the extraneous water is reduced. These influents are constructed in the following steps.
For the present situation the hourly measured flow data were used. The related influent concentrations are created with an empirical influent. In the case of Willem Annapolder this model was extended with a simple model that describes the transport through the pressure mains. The resulting concentrations were then calibrated with the measured influent concentrations at the respective treatment plants.
These concentrations had to be further characterised in order to have a complete input for the BioWin model of the treatment plants. The ammonia concentrations resulting from the influent model for example were translated into N-kjeldahl concentrations required by BioWin.
Finally the influent concentrations are adjusted to a flow in which the extraneous water is reduced. By comparing the results of the BioWin model between the two different types of influent — normal situation and with reduced extraneous water — something can be said of the effects of the latter.

3.1.1 Influent generator

The time series of the influent concentrations are generated by the empirical model that is developed by Petra van Daal for her PhD research on real time control of treatment plants (Langeveld et al. 2017)(van Daal-Rombouts 2017). This model simulates the behaviour of the concentrations in the sewer system based on the influent flow measurements and is calibrated with the available measured concentrations. In this thesis the model was used to calculate the concentrations of ammonia, the chemical oxygen demand (COD), total phosphorus, and total suspended solids (TSS).

The basis of the influent model are the dry weather flow (DWF) pattern and the corresponding concentrations. When the influent flow increases outside the expected boundary for dry weather, this indicates a rain event. The corresponding influent concentrations then vary according to the increase and decrease of the flow. The model therefore distinguishes different types of rain events: small, medium and large events. Each type is coupled to certain processes that dilute or increase the concentration of the pollutants. This way an hourly variation of the concentrations was generated based on the hourly varying flow measurements.

In the following paragraphs the different steps of the model are explained. First the dry weather flow and concentrations are determined. This is followed by the categorisation of the different types of rain events. The event determines the processes that change the concentrations of the influent flow, which are explained in the third part. In the case of Willem Annapolder, the impact of the pressure mains are incorporated in the influent model. This is explained in the last paragraph.

Dry weather flow pattern and concentrations

To determine the dry weather flow pattern in the influent flows at the treatment plants, the measured flow data is divided into influent on dry and wet days. The dry days are determined by the following formula: *'if during a 2-days time- span in total less than 0.5 mm of precipitation has been recorded the last day can be considered a DWF day'* (Schilperoort et al. 2012). This rule is applied on the available precipitation data for the two areas. The used data comes from the KNMI (Royal Netherlands Meteorological Institute) which has a rain gauge in Bergschenhoek for the Rotterdam area and a rain gauge in Kapelle for the area connected to Willem Annapolder. This rainfall data consists of the cumulative rainfall over a 24 hours period between 8 am of the preceding day till 8am of the day linked to the measurement *("RD = daily precipitation amount in 0.1 mm over the period 08.00 preceding day - 08.00 UTC present day")*. The measured rainfall thus refers to the amount of the previous day, which has to be taken into account in the selection of the dry days.
In the vicinity of Willem Annapolder there is a second rain gauge in Wilhelminadorp. This rain gauge shows the same pattern, albeit with a slight difference in peaks. Since the gauges have the same dry days only the gauge in Kapelle is used for the determination of the dry days.

In figure 3.1.2 all the flows measured at Willem Annapolder on dry days are plotted in one week. The mean dry weather flow is determined by averaging all the flows that happen on the same hour of the day. The same is done for the upper 95 percentile, which is used by the influent model as a threshold. Especially in the case of Willem Annapolder there is a different flow characteristic in the weekends. The mean and upper 95 percentile are therefore calculated separately for weekdays and weekends.

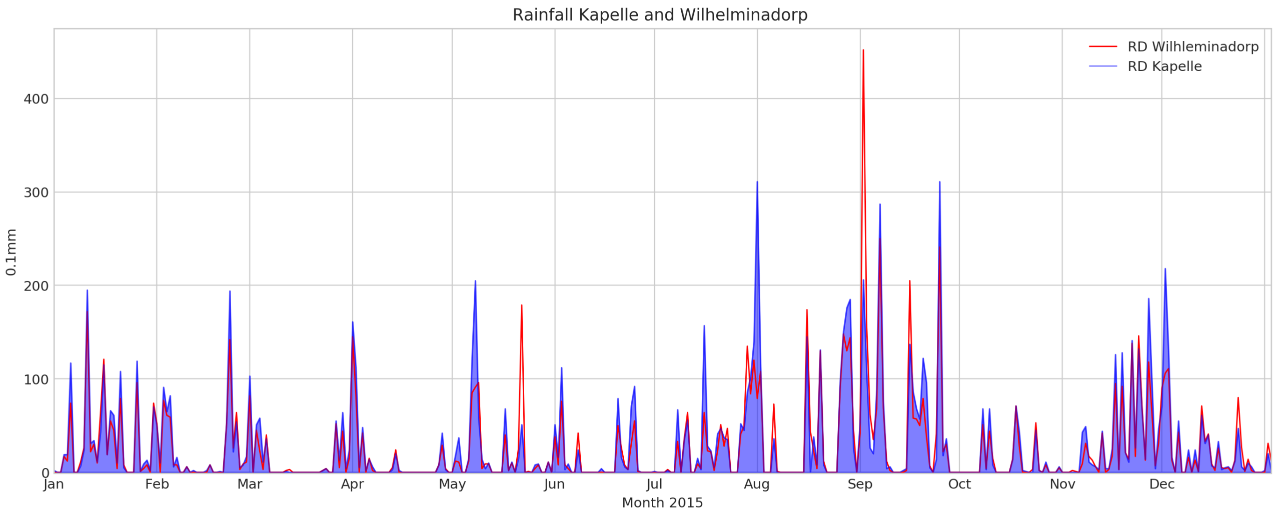


Figure 3.1.1 Rainfall gauge in Kapelle and Wilhelminadorp, Zeeland

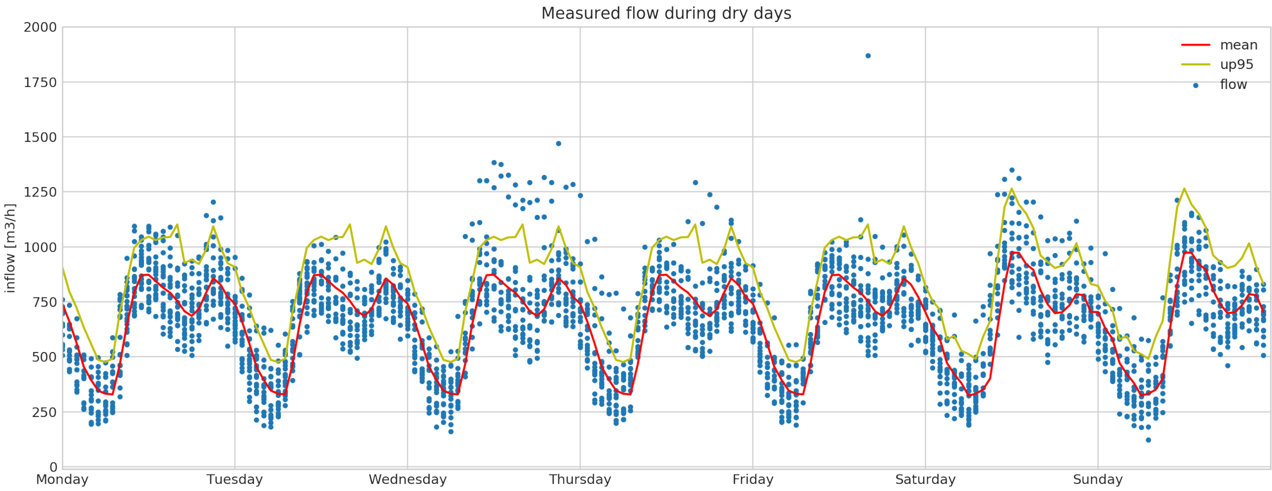


Figure 3.1.2 Dry weather flow (DWF) pattern Willem Annapolder

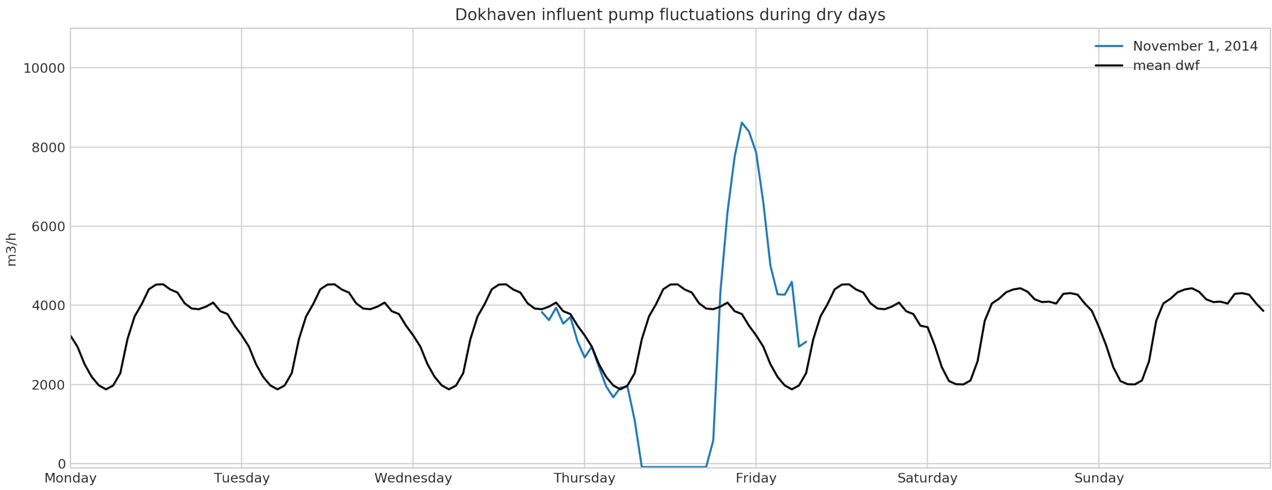


Figure 3.1.3 Pump fluctuation during dry weather

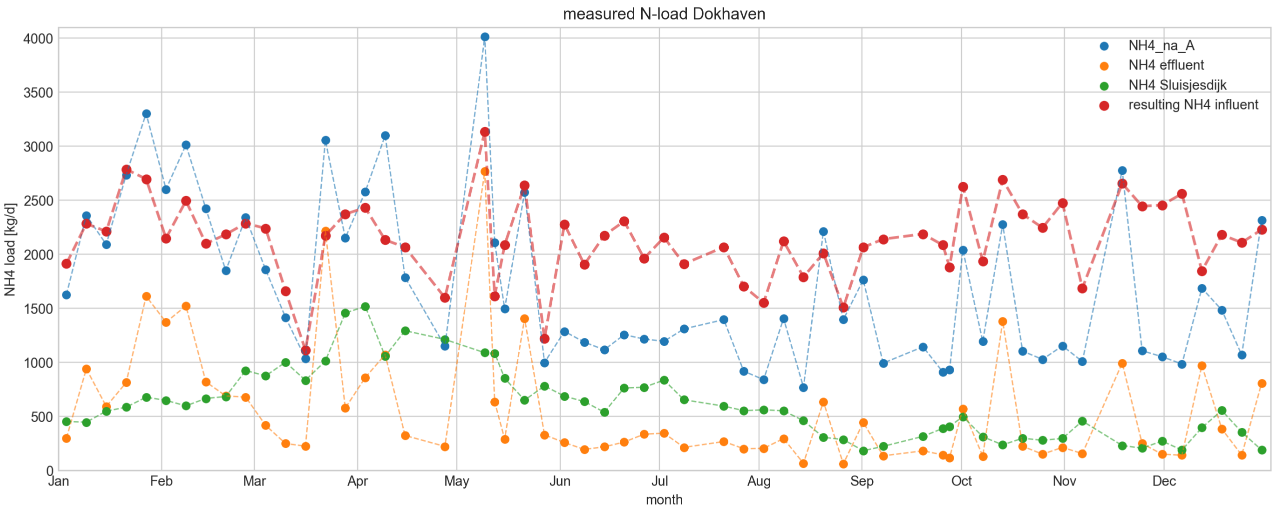


Figure 3.1.4 Measured and calculated ammonia load [kg/d = mg/l * m3/d /1000]

Some high peak flows were also measured during dry days. These are mostly the consequence of the pumps being turned off for maintenance and catching up after, as shown in figure 3.1.4, or measurement errors. These moments were filtered out for the determination of the mean dry weather flow and upper 95 percentile, since the increase of flow is not a consequence of of rainfall.

Contrary to the original model of van Daal-Rombouts, the dry weather concentrations are kept constant in this model. These were calculated with as the mean of the measured concentration during dry days. In the case of Dokhaven there is no direct measurement of ammonia at the influent. Here the ammonia concentrations are however measured at the end of the A stage of the treatment and in the effluent. Together with the measurements of the flow and concentrations from Sluisjesdijk these measurements were used to estimate the ammonia in the influent. For a more detailed explanation of the set up of the wastewater treatment plant of Dokhaven one is referred to chapter 3.2.2.
It can be assumed that no ammonia is converted in the A stage due to its very short sludge retention time. The ammonia concentration at the end of the A stage can then be regarded as the same as the concentration which enters the A stage, which is a combination of the influent, the effluent recirculation and return flow from the sludge treatment at Sluisjesdijk.

$$C_{in} = \frac{Q_A C_A - Q_{SD} C_{SD} - Q_{rec} C_{effl}}{Q_{in}}$$

From Sluisjesdijk 40 N Kjeldahl measurements are available. These are converted to ammonia concentration by a constant NH4/Nkj factor. From the yearly report it is found that the average NH4-N concentration that leaves Sluisjesdijk is 147mg/L (Besten Noteboom 2014). With an average N kjeldahl concentration measured of 266mg/L this factor is 0.55. The resulting average ammonia concentration in the influent is then calculated from these measured values. This data is then combined with the N kjeldahl measurements of the influent to attain the correct fractioning of the nitrogen in the influent, and to attain the average dry weather ammonia concentration for the influent model.

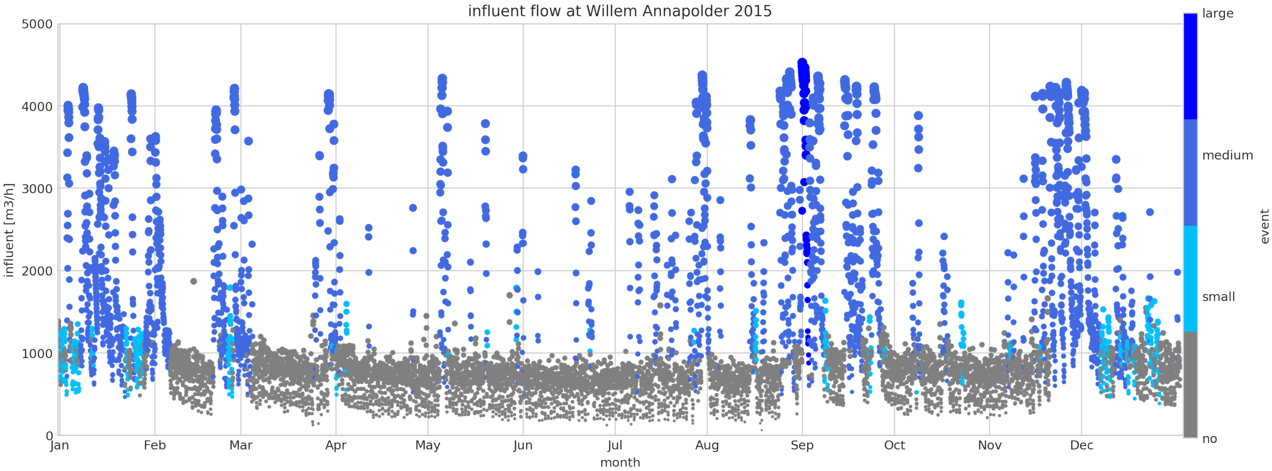


Figure 3.1.5. Total influent flow at Willem Annapolder, categorised by event: no event, small, medium or large rain event

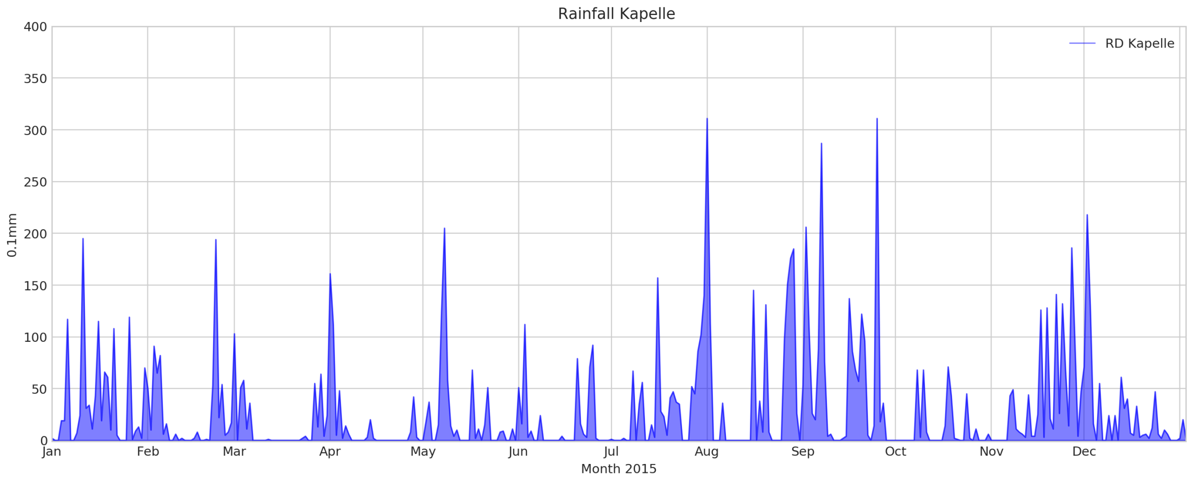


Figure 3.1.6. Rainfall at Kapelle for corresponding events.

Rain events

During dry weather the concentrations stay at a constant dry weather concentration. When the influent flow significantly increases, it means rain has fallen and the concentration should therefore change in accordance. The influent model distinguishes different types of rain events to determine what should happen to the concentrations.

The threshold that signals a potential rain event is the upper 95 percentile of the dry weather flow determined before. Whenever the measured influent flow surpasses this threshold, the moment is registered as the start of a possible rain event. This rain event ends when the flow again drops below the same threshold. In the model all these events are part of a collection called events1.

Not all the moments where the influent flow surpasses the threshold are real or significant rain events. When the volume that enters the plant during a rain event does not surpass the expected volume during 24 hours of dry weather flow, this event is not taken into account. It is also chosen not to include events with a duration less than 2 hours. In the model all the events that pass this filter are part of the collection events2.

All the remaining rain events are then divided into three different kinds: small, medium and large events. The kind of event determines what happens to the concentrations in the influent. In order to distinguish which event is taking place the stormwater flow — the difference between the influent and the 95 percentile —, volume and duration of the event are used based on the criteria shown in table 2.1.1.

For the threshold of the large events van Daal-Rombouts uses the measured water level in the influent reception tank (Langeveld et al. 2017). Because no such data is available for Dokhaven and Willem-Annapolder the three events with the largest influent flow per time are selected. However, since this selection is arbitrary it is chosen to make no distinction in behaviour of the concentrations between medium and large events.

A medium event occurs when the maximum stormwater flow during the event exceeds a certain threshold. This stormwater threshold is chosen to be the mean of the 95 percentile of the dry weather flow. The resulting threshold values correspond to the threshold used by van Daal, which is 4000 m3/h for the city Eindhoven (Langeveld et al. 2017).

The small events are the remaining rain events that surpassed the filter but are not large enough to be a medium or large event.

Similarly to the original model of van Daal-Rombouts, two consecutive events are merged when the time between them is smaller than one or three hours for medium and small events respectively. If a smaller event directly follows a larger event, it is also merged with the preceding event (Langeveld et al. 2017).

Table 2.1.1: Overview of event criteria. The duration of an event is the time between the start and end of the event. Qsw, the stormwater flow, is the influent flow above the 95 percentile. This is also used to calculate the volume of an event, which is the integration of the stormwater flow over the time of the event.

	Willem Annapolder	**Dokhaven**
Large event (event3)	Volume / Duration > 2000 m3/h	Volume / Duration > 8000 m3/h
Medium event (event9)	$Q_{sw\ max}$ > 860 m3/h	$Q_{sw\ max}$ > 4500 m3/h
Small event (event8)	Duration > 1h & volume> volume 24h dry weather flow	

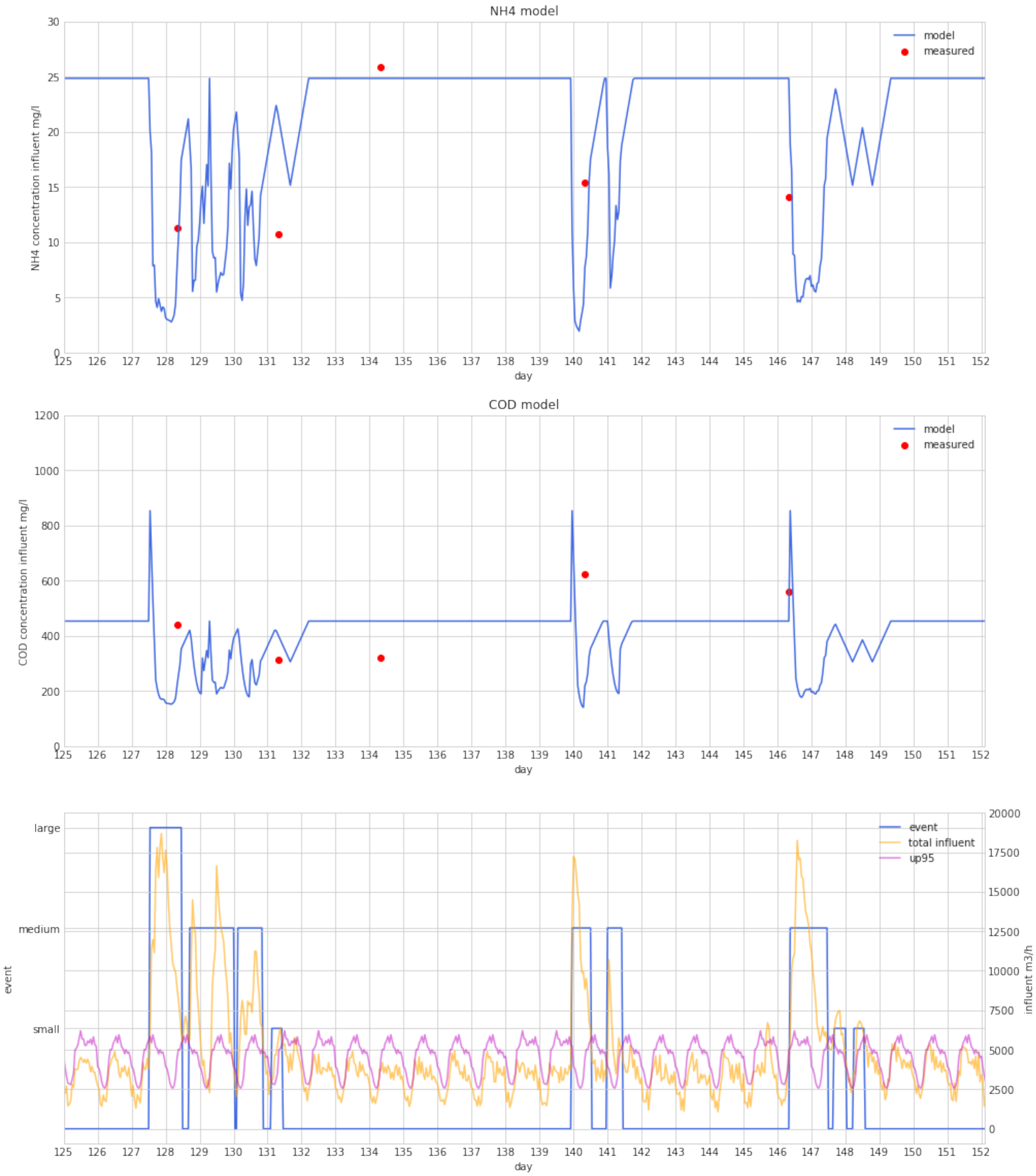


Figure 3.1.7-3.1.9. The resulting change in ammonia and COD concentrations for the corresponding events. The bottom plot shows the events (blue) that correspond with the influent flow (yellow) surpassing the upper 95 percentile (magenta).

Dilution processes

Different processes are modelled that change the incoming concentrations depending on the type of event that occurs. During a small event, the concentration of the modelled substances are diluted by a 'dilution triangle'. This triangle linearly dilutes and recovers the concentration during a certain amount of time. The rate of dilution is the same as the recovery rate (a6), just as the timespan of dilution is equal to the timespan of the recovery (t6) (Langeveld et al. 2017).

During medium or large events three processes are modelled that influence the concentration. The main process is the dilution, which is based on the ratio between the 95 percentile of the dry weather flow and the actual influent flow at that moment (Q_{DWF}/Q_{actual}). This ratio is multiplied by a dilution factor. This factor takes into account the possible increase or decrease (a1>1) in pollution load due to in-sewer-stocks. (Langeveld et al. 2017).

$$C_{WWF}(t) = C_{DWF}(t)(a_1 \frac{Q_{DWF}(t)}{Q_{actual}(t)} - a_1 + 1)$$

The second process simulates the observed delay in the dilution, during the onset of the storm event. This is modelled by a parabola that decreases the concentration to the minimum during the onset. This minima is represented as the dilution depth, which is the minimal of the ratio Q_{DWF}/Q_{DWF} during the onset. The parameter a_2 is the duration of the onset stage.

$$C_{WWF}(t) = C_{DWF}(t)\left(\frac{dilution\_depth}{a_2^{\ 2}}(t - t_{endonset})^2 - dilution\_depth + 1\right)$$

At the end of the event the restoration process starts, in which the concentration linearly increased again towards the dry weather concentration, with the restoration factor a_3.

$$C_{WWF}(t+1) = C_{WWF}(t)(1 + a_3)dt$$

For the concentration that are related to the particulate matters, like COD and total suspended solids, an additional process is added. This accounts for the increase in concentration at the beginning of the event due to the flushing of in sewer stocks. The process is simulated by the parameters of the maximum concentration peak during the first flush (a_4) and the duration of the recovery from the first flush (a_5). Since there needs to be time to build up the sewer stocks, this process only takes place when two medium or large events follow each other after more than 12 hours.

All the parameters in the equations — a_1 till t_6 are determined by the calibration of the influent model with the measured concentration data.

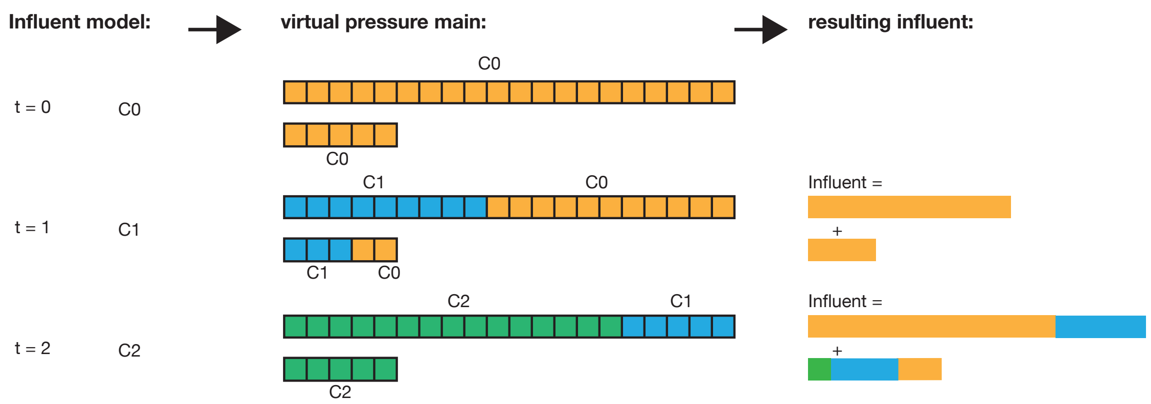


Figure 3.1.10. The addition of the virtual pressure main after the general influent model. By modelling the transport through the pressure main the shift in dynamics between flow and concentrations is taken into account.

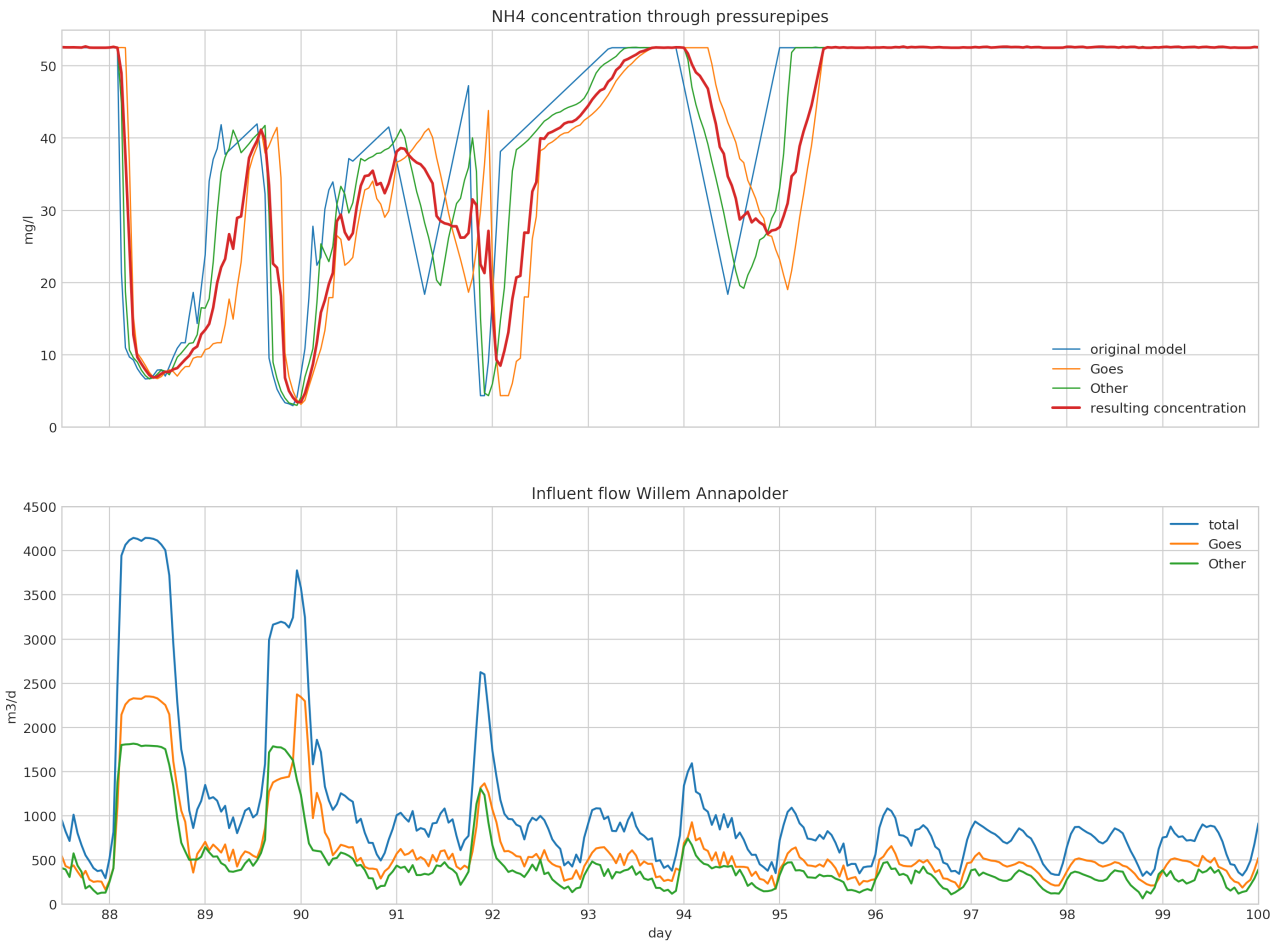


Figure 3.1.11. Influent ammonia concentrations and flows for the different pressure mains

Pressure mains

Contrary to Dokhaven, that lies in the middle of the city who's wastewater it treats, Willem Annapolder receives its sewage from multiple municipalities in the region. The sewer system that transports the wastewater consists of multiple long pressure mains, which has an influence on the dynamics of the pollution load.

When sewage is transported by a pressurised system, the pipes function as plug flow. During a rain event the water that is already inside the system has to be pumped to the plant before new stormwater can enter the system. This means the flow of wastewater is increased before the effects of the dilution are noticeable at the influent of the treatment plant. Because the sewage present in the pressure mains before the rain event has a dry weather concentration, this also increases the load on the connected treatment plant at the onset of the event.

The influent model described above is designed to describe the behaviour of the concentrations in the influent from a gravitational sewer. For Willem Annapolder this model is adjusted to take into account the effect of the pressure mains. This is done by adding virtual pipes that simulate the transportation through the pressure mains and thereby the delay in the dilution processes.

The influent arrives at Willem Annapolder through four different mains that are connected to four different areas. For each of these the hourly flow time series are available. This data shows that most of the wastewater comes from the area of Goes, which accounts to approximately 58% of the total influent. Since this area accounts for most of the volume, the other three areas are seen as one in the model for simplification.

The area of Goes is connected to the plant by a pressure main of about 12 km and a volume of 5529 m3. In the other system that represents the three other areas, the largest pressure main is taken as the dominating cause of delay. This is the main from Nisse drieweg, with a volume of 1357 m^3 , which is about one quarter of the volume of the other pressure main (the following largest volumes are 850 and 336 m^3). The pipes are simulated in the model as two volumes that are divided into a sequence of small pipe segments. Each of the segments of the pipe holds a part of the influent with corresponding concentrations.

At every time step, the concentrations are calculated with the influent model using the same processes and events as described above. Since no concentration measurements are available inside the catchment area, no distinction is made between the areas in the calculation of the concentrations with the influent model. Every timestep (one hour) this gives each new volume of influent its corresponding concentrations. This volume is then divided over the two virtual pipes according to the flow data available of the four areas. The same volume, but with the concentrations present in the pipes from the previous timestep, leaves the pressure pipes at the other end and becomes the influent of the treatment plant. The corresponding concentrations of the volume that leaves the pipe is calculated as the weighted average of the concentrations of the segments present at the end of the pipe. The concentration at every segment of the pipe is then again determined— the newly calculated concentration at the start of the pipe with the rest of the pipe filled with the volumes and concentrations of the previous time steps.

In some cases, especially for the smaller pressure main representing the three other areas, the incoming volume can be larger than the volume of the pipe. The influent then becomes a mix between the concentrations present in the whole pipe and part of the newly calculated concentration for the volume of the influent that exceeds the volume of the pipe. At the end the pipe is completely filled with the newly calculated concentration of that timestep.

At the end of the two pressure pipes, the influent is again mixed. The resulting concentration is the volume weighted average of the two concentrations that exit the pipes. This then results in the correct influent at the treatment plant that incorporates the shift in the dynamic of the concentrations relative to the flow.

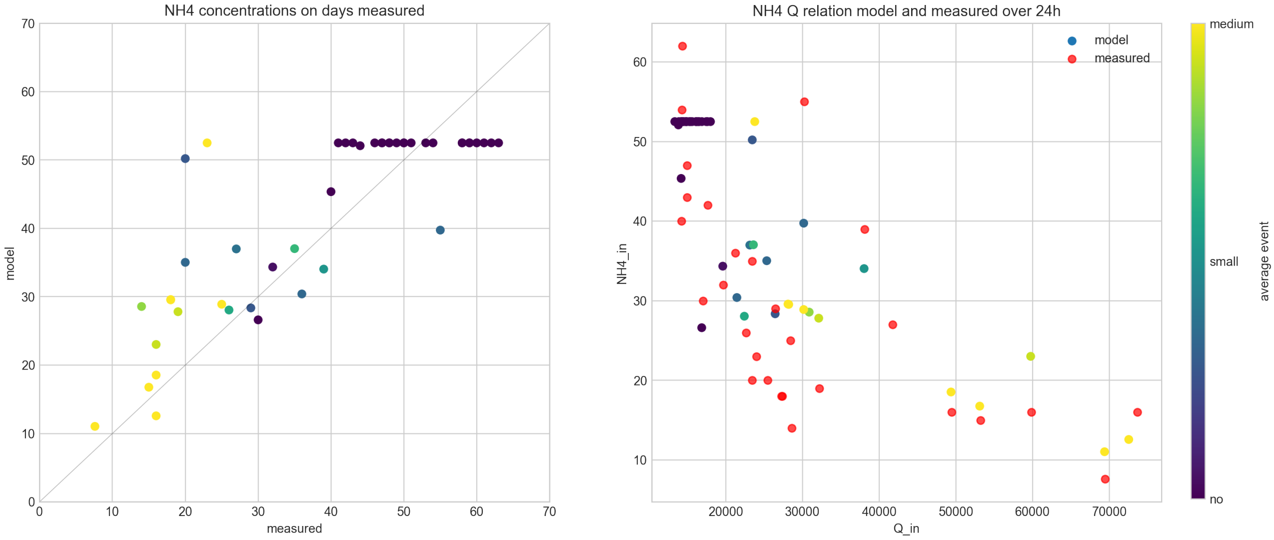


Figure 3.1.12. (a) Modelled ammonia concentrations plotted against the measured concentrations for Willem Annapolder. (b) Ammonia concentrations plotted against the influent flow. An increase in flow corresponds to a decrease in concentrations up to a certain minimum.

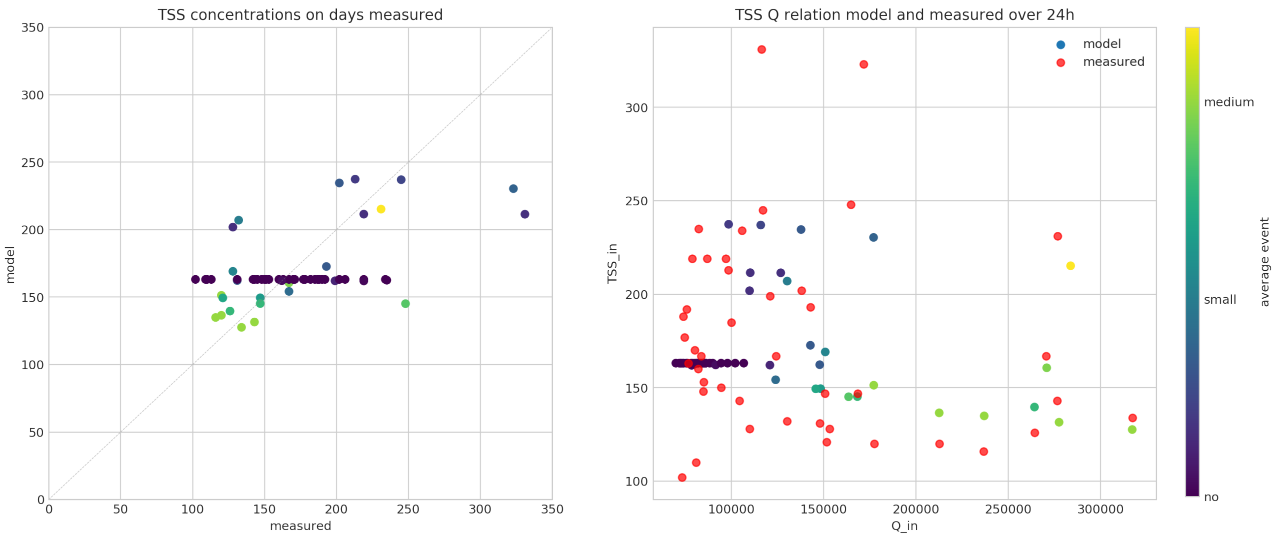


Figure 3.1.13. Same principle as figure 3.1.12 but for total suspended solids. Due to flushing of the sewer stocks the suspended solids show a different behaviour.

Calibration and accuracy of the influent model

The parameters that influence the processes described above are determined by comparing the model results with the measured values in the influent. The measured values are 24 hour averages measured from 8am to 8am the next day. In order to compare the model results with this data, the modelled concentrations are also averaged over the same time period. The parameters of the influent model are then altered to reduce the overall difference between the measured and modelled values. This calibration process is done by trial and error.

The model results are visually checked with the measured data (see figure 3.1.12). By plotting the measured concentrations (x-axis) over the modelled concentrations (y-axis) for the same moments, it can be seen wether the model over- or understates the resulting concentrations (see figure 3.1.12a). The other figures of the resulting concentrations can be seen in appendix 3.

Besides the visual inspection the root mean square errors (RMSE) are calculated and minimised. Since a fixed error is introduced by using a constant dry weather concentration, only the error on days with a rain event is evaluated. After this error is minimised, the total error is calculated. The resulting errors vary between 10 an 20% for ammonia, phosphate and COD, which is comparable to the errors in the results of van Daal-Rombouts (2017). The resulting concentrations and errors are shown in appendix 4.

Only the model for the suspended solids is less accurate. The behaviour of the solids in the sewer is hard to predict. While the other substances show the clear decrease in concentration with increased flow, this is not seen in the measured data for the suspended solids (figure 3.13).

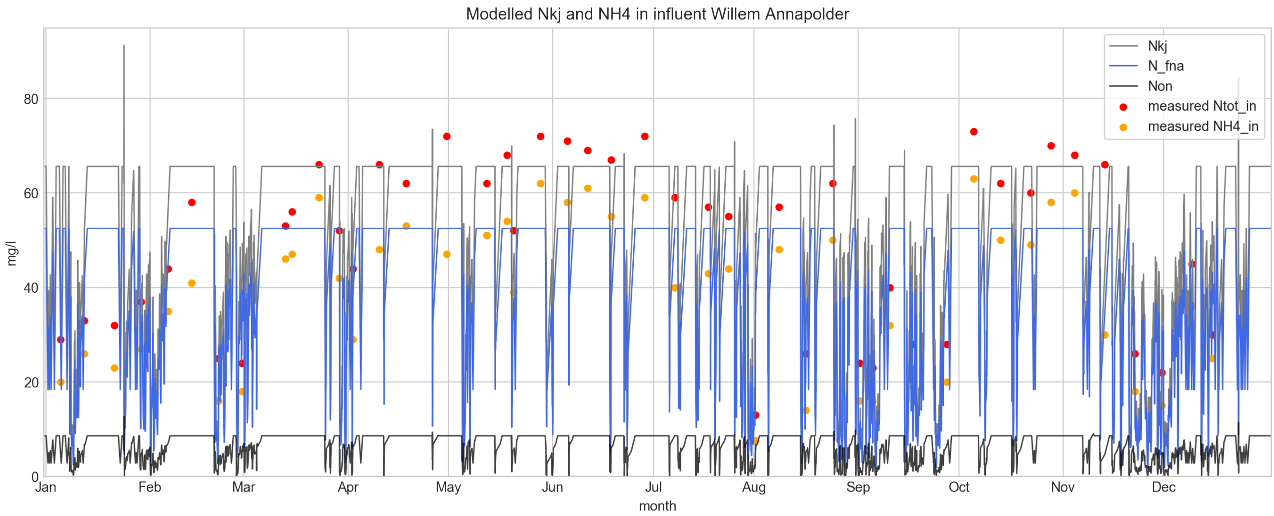


Figure 3.1.14. Modelled and measured ammonia-N and Nkjeldahl N concentrations. The Nkjeldahl is created as the sum of the different components with the organic nitrogen (N_{ON}) used to close the mass balance with the used fractions.

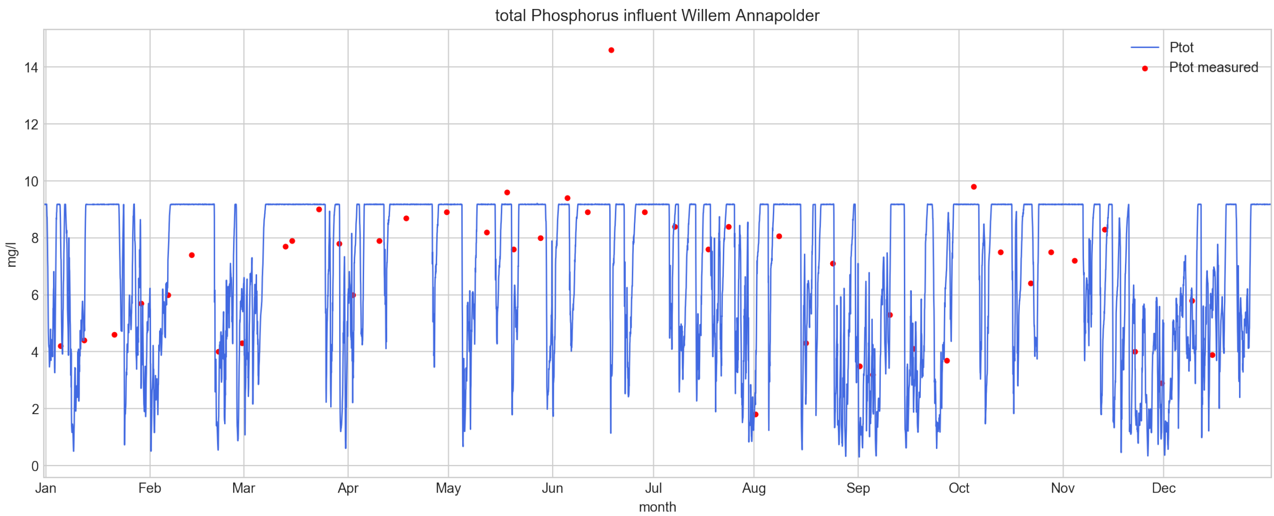


Figure 3.1.15. Resulting total phosphorus concentration and the measured concentrations in 2015 for Willem Annapolder.

3.1.2 Influent Characterisation

In order to use the calculated influent concentrations in the BioWin models of the treatment plant, small adjustments are needed. The influent input in BioWin consists of total COD, total Kjeldahl nitrogen (TNkj), total Phosphorus, Nitrate N (all in mg/L), pH, Alkalinity (mmol/L), insoluble suspended solids (ISS), Calcium, Magnesium and dissolved oxygen (mg/L). These concentrations are combined with the corresponding influent flow, which needs to be in m3/day.
Next to the varying concentrations, it is possible to change the different fractions of the COD, Nitrogen and Phosphorus that are present in the influent. These fractions are constants and are used to convert the modelled ammonia into the total Kjeldahl N.

Nitrogen

In Biowin the influent total Kjeldahl N includes the following substances (BioWin Manual):

$$TNkj = NH_4 + N_{US} + organismsN + X_{IN} + X_{ON} + S_{ON}$$

This follows the characterisation of Nitrogen as described by Melcer (2004). A large part of it is in the inorganic soluble ammonia NH4. This is usually about 75% of the incoming nitrogen. In the case of Willem Annapolder measurements of both ammonia as N Kjeldahl are present of the influent. The flow weighted average ratio NH4/Nkj is taken from these measurements, resulting in a factor of .76 (WAP 15 conc vracht). For Dokhaven the calculated ammonia concentration and the measured N Kjeldahl are used, which result in a ratio NH4/Nkj of 0.53.
A small fraction comes from the soluble inorganic nitrogen N_{US}— about 2%. Most other components are related to the COD concentration. X_{IN} is the unbiodegradable particulate nitrogen, which is given by the N-fraction of the unbiodegradable particulate COD. Then there is the nitrogen present in the organisms. This is determined by using the default stoichiometric value of 0.07 mgN/mg COD, which is the same for all the different organisms in the model. The only significant organisms present in the influent are the ordinary heterotrophic organisms, which are 2% of the COD at default in BioWin, but which can be 5 to 15% of the total influent (HENZE et al. 1995). Lastly there are the remaining organic nitrogen parts X_{ON} and S_{ON} for the particulate and soluble fraction respectively. In BioWin, as in other ASM2 model fractioning, these are calculated as the difference between the total Nkj and all the other parts.

The total Nkjeldahl in the influent can thus be calculated by the summation of the different constituents. The basis is formed by the calculated ammonia concentrations. Added to this is the nitrogen linked to the incoming COD concentrations, which is calculated in the influent model. During dry weather the COD fraction is approximately 21% of the total influent. During the start of a rain event however, the COD increases, while the ammonia concentration decreases, which causes the the nitrogen from the COD to become 80% of the total influent Nkj at some moments. In order to keep the fractions of ammonia and COD related nitrogen constant over the time, the remaining organic nitrogen is calculated in such a way that the mass balance is correct over the whole year, and that the constant fractions of the total Nkjeldahl correspond with the calculated amounts.

Phosphate

The characterisation of the phosphate is done similarly to the ammonia. In BioWin the phosphate in the influent is a fixed fraction of the total phosphorus: $F_{PO4} = PO_4/P_{tot}$. For Dokhaven this fraction is taken as the percentage of soluble phosphorus in diluted wastewater as reported by STOWA (Mels et al. 2001); in the case of Willem Annapolder the fraction is calculated from the measured influent concentrations.
The total phosphorus, which is the concentration needed as input for BioWin, is the sum of the soluble phosphate and the phosphorus in the different particulates: the incoming biomass, the phosphorus in the unbiodegradable particulates and the phosphorus in the biodegradable organic particulates (X_{OP}):
These particulate concentrations are all related to the incoming COD and are calculated with their related fractions. In

$$Ptot = PO_4 + organismsP + X_{IP} + X_{OP}$$

BioWin, the X_{OP} fraction is used to close the mass balance; in the influent model this is used to make the sum of the different phosphorus components match the measured values while having constant fractions.

Since no phosphate measurements are done at the influent for Dokhaven, the influent model is made with the measured total phosphorus concentration. This is changed to phosphate with the used fraction, after which the particulate phosphorus is added. In the case of Willem Annapolder the phosphate concentrations in the influent model are directly based on the measured phosphate concentrations. The total phosphorus needed for the BioWin influent is then made by adding the particulate fractions to the phosphate concentrations from the influent model.

COD

The COD concentration for the BioWin influent is the same as the concentrations calculated above. This is the total COD, but just as the total Nitrogen Kjeldahl it consists of different fractions. The COD characterisation can have a large influence on the performance of certain processes during the treatment. A higher biodegradable fraction of COD increases the pre-denitrification and phosphorusus removal at the beginning of the water train, while the amount of particulate matter influences the sludge production (Roeleveld and Loosdrecht 2002).

In BioWin the COD is partitioned the same way as in the ASM2d model, though with different names. The different fractions are F_{bs} (readily biodegradable), F_{US}, F_{UP}, and F_Z which represents the COD present in the biomass. Just as in the ASM2d model the readily biodegradable fraction is divided into acetates (S_A) and readily fermentable biodegradable substrate (S_F), which is registered by the fraction of acetate in the total readily biodegradable substrate F_{ac}. The fraction slowly biodegradable (X_S in ASM) is not a parameter in BioWin but follows as the resultant from one minus the other fractions. BioWin also makes a distinction within the slowly biodegradable between colloidal and particulate matter by using the fraction F_{XSP} — representing the latter.

Some of the fractions can be calculated from the measured values. Roeleveld and Loosdrecht (2002) evaluated the characterisation procedure from STOWA (the Dutch Foundation of Applied Water Research). They show how to calculate the characterisation for the model influent from the measurements done. The unbiodegradable soluble COD can be determined from the COD in the effluent. Since it is inert and soluble it is not altered by any of the steps in the treatment plant. Here they make a distinction between a high and low loaded treatment plant.

$$S_I = 0{,}9 \cdot COD_{eff,sol} \text{ (low loaded wwtps) (WAP)}$$

$$S_I = 0{,}9 \cdot COD_{eff,sol} - 1.5 \cdot BOD_{5,eff} \text{ (high loaded) (DOK)}$$

With these formula the S_I is calculated for Willem Annapolder (low loaded) and Dokhaven (high loaded). For both plants it was taken into account that the S_I present in the effluent is coming from both the influent as well as the return flow from the sludge treatment. This fraction varies over time, but since the model only accepts a constant, the flow weighted average is taken. This results in a F_{US} of 0.08 for Willem Annapolder (0.12 if the digester reject water is not taken into account), and 0.061 for Dokhaven (.064 without the flow from Sluisjesdijk). For the other fractions it is necessary to have more detailed measurements of the influent, which are not available. For the first run before calibration the other fractions are set on the average values found by Roeleveld and Loosdrecht (2002). These are then adjusted where needed in the calibration step. The starting values and the resulting calibrated values can be found in appendix 5.

Other

For the ISS concentrations, the TSS was modelled similarly to the COD concentration. From literature the value of ISS is then approximately 15% of the TSS, from the often used VSS/TSS ratio of 0.85 (Ekama and Wentzel 2004). The alkalinity and calcium and magnesium concentrations are assumed to be constant. The alkalinity of the influent is taken to be 80 eqv/m3. This value is not measured but chosen in a way that it will not cause problems for the modelled treatment processes (Henze et al 1997: 50). The nitrate and dissolved oxygen concentration are assumed to be constant and zero.

3.1.3 Sludge treatment effluent

Both the Dokhaven and Willem Annapolder treatment plant have an additional stream of pollutants coming from sludge treatment. The sludge from Dokhaven is treated at a nearby facility at Sluisjesdijk. The reject water from this process is transported back to Dokhaven. Even though this water is already partially treated at Sluisjesdijk it remains an important source of ammonia for the plant.
At Willem Annapolder the sludge is treated on site with an anaerobic digester for methane production. From time to time it also receives sludge from the treatment plant of Waarde, increasing the total pollution load. The water effluent of this sludge treatment goes to the plant sewer, which enters the water treatment at the primary settling tank.

Hourly flow data is available for both streams, but only limited data is available on the concentrations of the pollutants or the factors that contribute to the variation in concentration. For Dokhaven there are 58 measurements of the concentrations. The hourly concentrations for the model are linearly interpolated between these measured concentrations. For the concentrations of the terrain sewer at Willem Annapolder only 28 concentration measurements are available and these show a wider variability. Therefore the hourly concentration is taken as the average measured concentration.

Figure … shows the resulting influent loads for COD and ammonia for Dokhaven and Willem Annapolder respectively. A complete overview of all the loads from the sludge treatment is shown in appendix 3. Even though the flows of the sludge treatment effluents are much smaller, the concentrations are high enough to become to a significant extra load on the treatment plants. The mean load from the terrain sewer at Willem Annapolder compared to the influent load is 19, 25 and 74% for NH4, COD and Ptot respectively. At Dokhaven the mean load from Sluisjesdijk compared to the influent for the same substances are 30, 17 and 32% respectively. It is therefore important to take them into account when modelling the treatment plant.

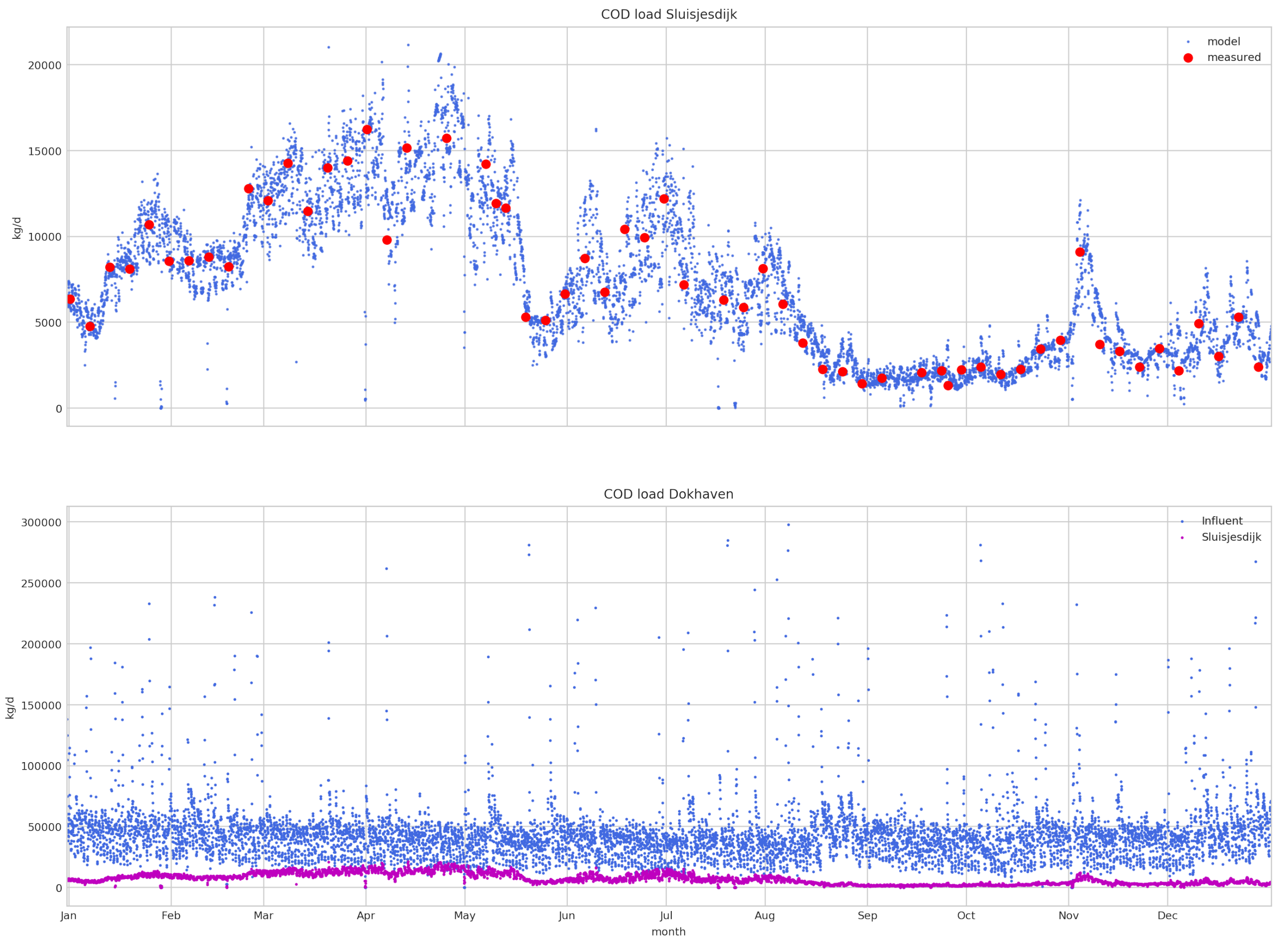


Figure 3.1.17. COD load coming from the sludge treatment at Sluisjesdijk. Above plot shows the load, the bottom plot shows the load from Sluisjesdijk in comparison to the influent load at Dokhaven.

The resulting flows of pollutants are also characterised. In the case of Willem Annapolder measurements are available of both Ptot, PO4, Nkj and NH4 (WAP15 conc vracht). The fraction NH4/Nkj and PO4/Ptot are calculated from these measurements, which are 0.6 and 0.8 respectively for the weighted average. The COD fractioning is chosen to be the same as the influent.

In the case of Dokhaven only measurements of Nkj and Ptot are available. The amount of ammonia in this stream is taken from the annual performance report (jaarraportage 2014). Here the average amount of ammonia, nitrite and nitrate-N that leave the Annamox reactor are measured, which are 147, 7 and 101 mg/L respectively. Using this average value the fraction NH4/Nkj for the Sluisjesdijk return flow becomes 0.55.
The different COD fractions in the model are taken from the calibrated fractions in the Dokhaven model of Royal Haskoning DHV (van Opijnen 2017). This is characterised by a large fraction of inert solubles (47.7%).

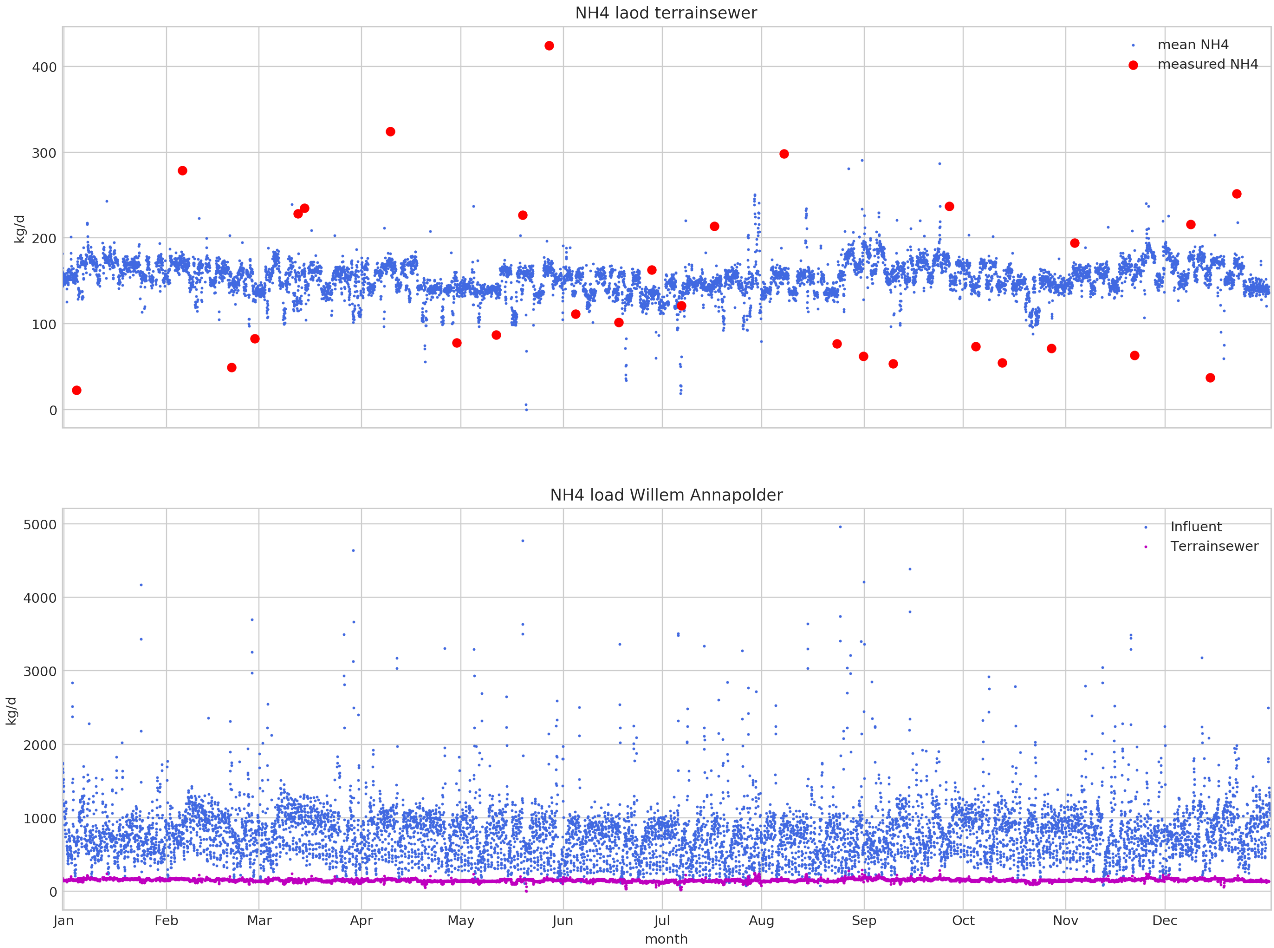


Figure 3.1.18. (a) NH4 load from the reject water of the sludge treatment at Willem Annapolder. This load is modelled with a constant ammonia concentration (blue scatter). (b) The plot below shows the load from the reject water together with the load from the influent.

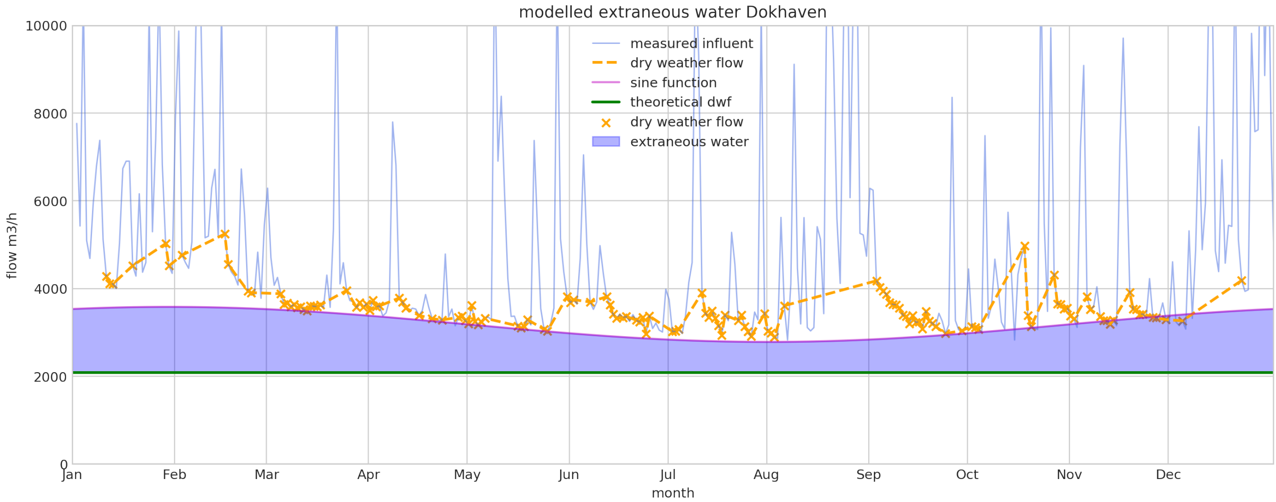


Figure 3.1.19. Modelled extraneous water (blue surface) in the influent at Dokhaven. The variation follows the minimum dry weather flow variation.

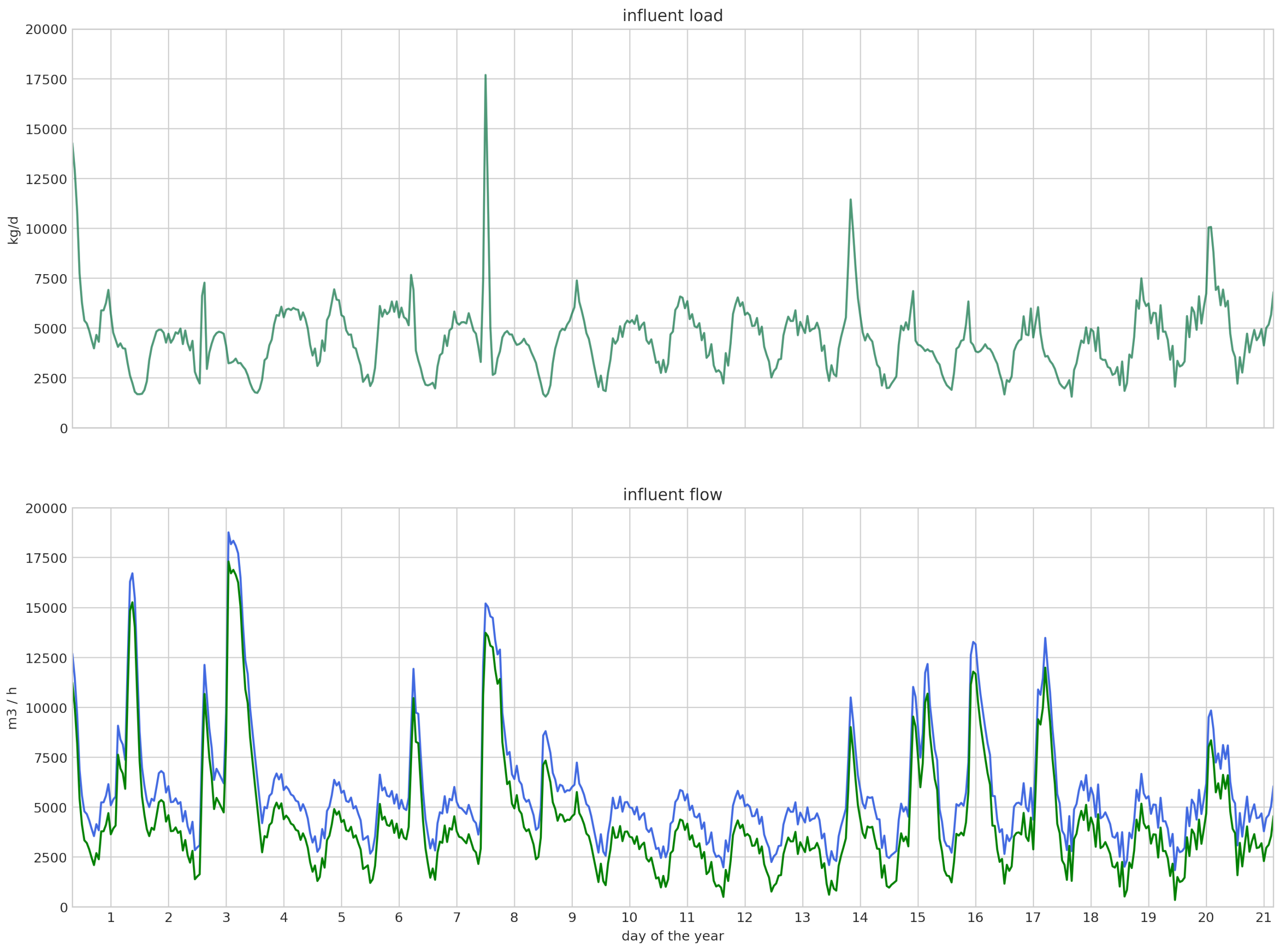


Figure 3.1.20. Influent load and influent flow for the calibrated influent model (blue) and the influent model with reduced extraneous water (green) at Dokhaven. The load is kept the same (so the blue and green lines overlap), but the flow is reduced.

3.1.3 Extracting extraneous water

The goal of the models is the find the effect of a reduction of extraneous water in the influent, on the performance of the wastewater treatment plants. The first influents generated above are used to calibrate the BioWin models for the current situation. When these are calibrated, these models can be run with a new influent in which the extraneous water is reduced. This way the effect of a reduction in extraneous water can be studied. The way the new influent is created is similar to the method used by Dirckx et al. (2019)

The amount of extraneous water in the influent of the two treatment plants is found in earlier studies.
From the study done for Rotterdam in 2010, the flow of extraneous water to Dokhaven is about 34000 m3/d, where the measured dry weather flow is 68% higher than the theoretical flow (Vosse 2013).
For Willem Annapolder the amount of extraneous water is estimated with the DWAAS method for the different connected catchment areas and varies between 15 and 30% of the total flow for the reliable measurements. From the data for the different areas, a weighted average amount of extraneous water is made for the system of Goes and for the combination of the other areas. This results in an average of 25% of the total influent for Goes and 20% of the total influent of the other three systems to be extraneous water. When the measured values of extraneous water were unreliable they were omitted from this average, which is the case for Wilhelminadorp, Biezelinge, s'Gravenpolder, Baarland and Kraaijertsedijk (Langeveld 2016). This data allows to calculate the new influents without extraneous water for both the Goes area and combined other area.

When assuming a constant theoretical dry weather flow, the influent flow data shows that the amount of extraneous water is not constant over the year. The variation follows the change in the minimum measured dry weather flow. A similar seasonal variation in extraneous water is also shown by de Ville et al. (2009) and used by Dirckx et al (2019) to create a similar new influent. This variation is simulated by linking the amount of extraneous water to a sine function, which follows the moments of minimum dry weather flow. The total amount of extraneous water is then calculated as the surface between the sine function and a lower flow limit. In the case of Willem Annapolder this lower flow limit is chosen in such a way that the total surface matches the estimated yearly extraneous water. In Dokhaven the theoretical dry weather flow is used as the lower limit. This makes the total amount of extraneous water removed about 77% of the estimated total amount.

Since the infiltration and inflow are seen as relatively clean, its reduction will cause a related increase in the concentration of pollutants. But since no new pollutant are added, the pollution load does not change. The new concentrations are calculated by multiplying the previously calculated concentrations by a concentration factor. This factor is the ratio of the old influent flow over the new flow without extraneous water. In order to prevent the concentration factor to become extremely high at moments of low flow, the minimum of the new flow is limited at the measured minimum flow in normal conditions. Even though the load would still be the same at that moment, the possible high concentration values during low flow could give problems in the models of the treatment plant.

In the case of Willem Annapolder the pressure mains are again taken into account, by simulating the transport through the pressure mains after the new influent concentrations are calculated. This allows to see the effect of a reduction in extraneous water on the concentrations after the transport. Because the dry weather concentration is now higher, the concentration of the water in the pressure pipe will be higher at the onset of a storm event. This results in an increased load when the flow through the pipe is increased to make room for the rainwater. This can be seen in figure… which shows the resulting ammonia load at the influent with reduced extraneous water.
A similar result is found by Rutsch et al, who studied the propagation of the pollutions in sewer pipes (2005). This study also concluded that a decrease in infiltration causes an increase in the pollution loads at the start of the pulse.

After the new concentrations are calculated, the calibrated fractions are used to provide the correct Nkj and Ptot concentrations in the new BioWin influent.

3.2 Wastewater treatment plants

For the treatment plant simulation two models — one for Dokhaven and one for Willem Annapolder — are made in BioWin. BioWin is a wastewater treatment process simulator of EnviroSim and allows for an easy representation of the treatment plant. The basis of BioWin is the BioWin biological model, which is a combination of an activated sludge and anaerobic digestion model (Liwarska-Bizukojc et al. 2011).

As far as the available data allows, the models are made and calibrated following the STOWA protocol (Hulsbeek et al. 2002). The STOWA protocol is made by the Dutch Foundation of Applied Water Research (STOWA) with the idea to create general guidelines for making and calibrating activated sludge models. These guidelines are based on the experience of many different professionals using SIMBA, which is a simulation package for wastewater treatment plants that runs in Simulink and has been used for many years. The protocol is based on the use of the ASM 1 model, which means that the focus is mostly on COD and nitrogen removal.
The BioWin biological model is more elaborate than the ASM 1 model as it takes into account more processes. Since the goal of this model study is on the water line, with the focus on ammonia removal, the same protocol can be used to a large extent.

The purpose of both models is to see the difference in performance between the current situation and the situation when the extraneous water is reduced. Since this is linked to the performance of the control structures and the variation of the influent during storm events a detailed model is required. This allows to see the difference in behaviour during dry weather flow, the onset of heavy rainevents and long rainy periods.
It is chosen not to include the sludge processes in the model as the reduction of extraneous water will have limited effect on this. All the processes that happen to the water in the treatment are represented in the models.

The STOWA protocol recommends the different parameters which should be adjusted for each step of the calibration. These parameters for the ASM1 model are also mentioned as the most sensitive in BioWin (Liwarska-Bizukojc and Biernacki 2010). However, before any kinetic or stoichiometric model parameters are changed, the characterisation of the COD in the influent and certain control parameters are used to change the model outcome. Since no information on the actual influent characterisation is available, this is an unknown parameter with a large influence. The same holds for the operational data of the plants. Even though most of the internal flows are measured and the control structures are clear, these are not all certain and very sensitive (Meijer et al 201). An important aspect herein is the aeration in the model (Cierkens et al. 2012).

Finally it has to be noted that this is an iterative process. For clarity only the final decisions in calibration are explained instead of all the iterations that led to the correct model.

3.2.1 Willem Annapolder

3.2.1.1 BioWin model

Data
The model of the treatment plant only looks at the processes that treat the water: the primary settler, the anaerobic, anoxic and aerobic reactor and the following secondary settlers.
The model is made with the data from the routine measurements. This includes the hourly measurement of the temperature in the aeration, hourly measurements of the control parameters for the aeration — ammonia and dissolved oxygen (DO) concentration— and hourly flow data of the sludge flows: the excess primary sludge, the sludge underflow from the secondary settlers, and the surplus sludge flow taken from the underflow of secondary settler number four. Lastly there are lab measurements of ammonia, COD, phosphorus and phosphate measurements in the influent and effluent.
The model is run with the measured flow data for the influent flows, the primary sludge flow and the sludge underflows from the secondary settlers.

Where possible the data is checked on errors. These were especially important in the sludge underflow from the secondary settlers. At some moments the sludge flow from settler number three becomes zero, which causes an outflow of sludge from the settler in the model and a drop in the solids concentration in the biological reactors. When these reactions are not seen in the measured data of the solids content, the zero flow is marked as an error in the measurement and corrected with a linear interpolation between the value before and after the error.

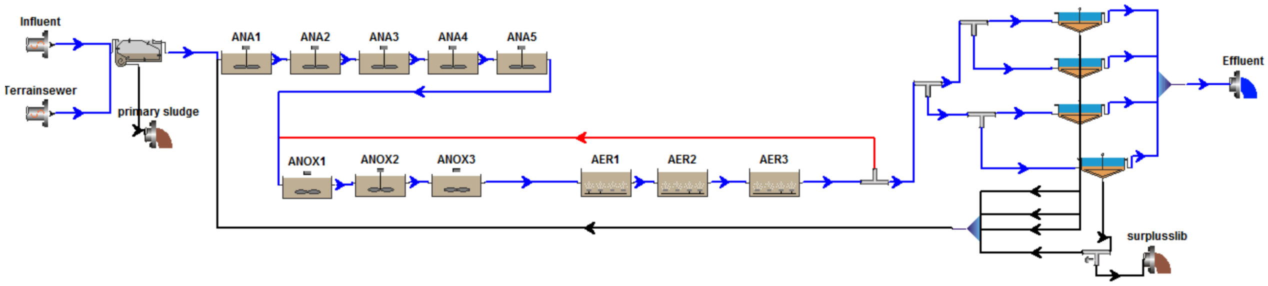


Figure 3.2.1: BioWin model layout for Willem Annapolder

Model setup

The dimensions of the tanks follow the dimensions described by the waterboard (Waterschap Zeeuwse eilanden 2006), though the thanks from the AAO reactor are split into different sequential tanks. An overview of all the dimensions is shown in appendix 5.

Influent

As input the model uses two COD influent modules. One module takes the calculated influent flow and concentrations that are calculated in the influent model. In this module the accompanied fractions are also defined.
The second influent comes from the terrain sewer, which also contains the reject water from the on site sludge treatment process. This influent is also described above.

Primary settler

The primary settler is modelled as an ideal primary clarifier with a constant removal performance and constant sludge waste of 50 m3/h, which is the same as the measured sludge flow. The performance of the settlers has a large influence on the sludge content in the biological tanks. Since only limited measurements are done on the TSS concentrations before and after the settlers, the removal performance is modelled as constant. For the primary clarifier a removal percentage is modelled as a constant 64.9%, which is the 90 percentile of the measured removal of suspended solids.

Anaerobic stage

The five compartments of the anaerobic tank are represented by five unaerated bioreactors. The dimensions of these tanks follow the dimensions of the five segments of the anaerobic reactor.

AO

Both the anoxic and combined anoxic/aerobic tank are plug flow reactors that go around the anaerobic tank in the center in two consecutive concentric rings. Each stage —one ring— is therefore modelled as three consecutive bioreactor tanks, with the anoxic stage being unaerated and the combined anoxic/aerobic stage aerated bioreactors. Dividing these stages into more compartments did not improve the model results.
In reality the mixed liquor is aerated by three plate aerators in the mixed aerobic anoxic reactor. In the model this is represented by one plate aerator in each aeration tank. In the original dimensioning of the plant the total aeration capacity was calculated as 6930 m3/h (handboek dimensionering, p13). In the model this provides each plate aerator with one third of the total capacity (2310 m3/h). In order to have an adequate total anoxic sludge volume, the plate aerators in these tanks only cover 60% of the aeration tank. All the other parameters for the aeration are kept at the default BioWin values for fine bubble aeration.

Recirculation

The effluent of the last aerobic reactor is divided by a splitter to return nitrate rich mixed liquor to the fixed anoxic tanks. This return flow is fixed at 3.5 times the flow that goes to the secondary settlers, with a maximum flow of 4115 m3/h (handboek dimensionering).

Secondary Clarifiers:
The four secondary clarifiers are modelled as four ideal secondary settlers with the same dimensions as in reality. In the model no biological reactions take place in the settlers. Since not the settlers are not equal in size, the mixed liquor from the AAO tanks is distributed over the tanks by splitters with a ratio that follows the different surface areas. This way the surface load of all the settlers is equal. This means the first two old settlers —number one and two— receive 43% of the flow, which is divided equally between them. The two new settlers —three and four— receive the other 57% of which settler number four, the largest, receives 63% (Handboek dimensionering). The four settlers are modelled with the same constant removal performance. This is calculated as 90 percentile of the difference in measured total solids content of the aeration tank and the effluent.

The temperature of the tanks is set to the measured temperature in the oxidation tank. All the other parameters are kept on the default values.

Control
The surplus sludge is diverted from the underflow of the fourth secondary settler. Contrary to the other flows in the model, this flow is not taken as measured but controlled by the BioWin controller. This is done with a PI controller that aims for a sludge residence time of 23 days, which is the average sludge retention time measured at Willem Annapolder in 2015 (bedrijfsrapportage 2015). The remaining sludge from the four secondary settlers is diverted back to the input of the first anaerobic tank.

Just as in reality the aeration in the model is controlled. The aeration is controlled with two parameters: the DO setpoint of the aerator and the measured ammonia concentration in the tank. The setpoint for DO is set at the recorded setpoints in 2015, which are1.3 mg/L for the period between the 22nd of January till the 22nd of May or 1.6 mg/L for the rest of the year. This is modelled the same in each aeration tank. In BioWin Controller the aeration is controlled with the ammonia concentration of the effluent of the last tank. This controller turns off the aeration in the three tanks when the ammonia concentration drops under 1mg/L. The aeration is turned on again when the concentration increases above 1.5 mg/L. These low and high boundaries for the aeration change on the 13th of March of the modelled year, when both are increased to 1.4 and 1.9 mg/L respectively. The lower boundary changes back to 1mg/L on the 25th of March, while the upper boundary stays at 1.9 for the rest of the year (handboek processbeschrijving p10).

This setup of the model forms the basis for further calibration.

3.2.1.2. Calibration

The calibration is done in the proposed order by the SIMBA protocol, which is sludge production, ammonia in the effluent and nitrate and phosphate in the effluent.

Sludge production

Since the surplus sludge flow is controlled by the SRT the sludge production is only further influenced by the COD fractioning of the influent and the return flow from Sluisjesdijk.

The annual performance report for 2015 shows an average surplus sludge production of 2346 kg DS/day. The final calibrated model has a lower mean sludge production of 1855 kg/d. Figure 3.2.2 shows the modelled and measured solids in the aeration tank. The control of the surplus sludge with a constant SRT results in an overall slightly lower sludge concentration in the biological tanks; the mean TSS concentration in the model is 3770 mg/L in comparison to 4457 mg/L measured in the lab.

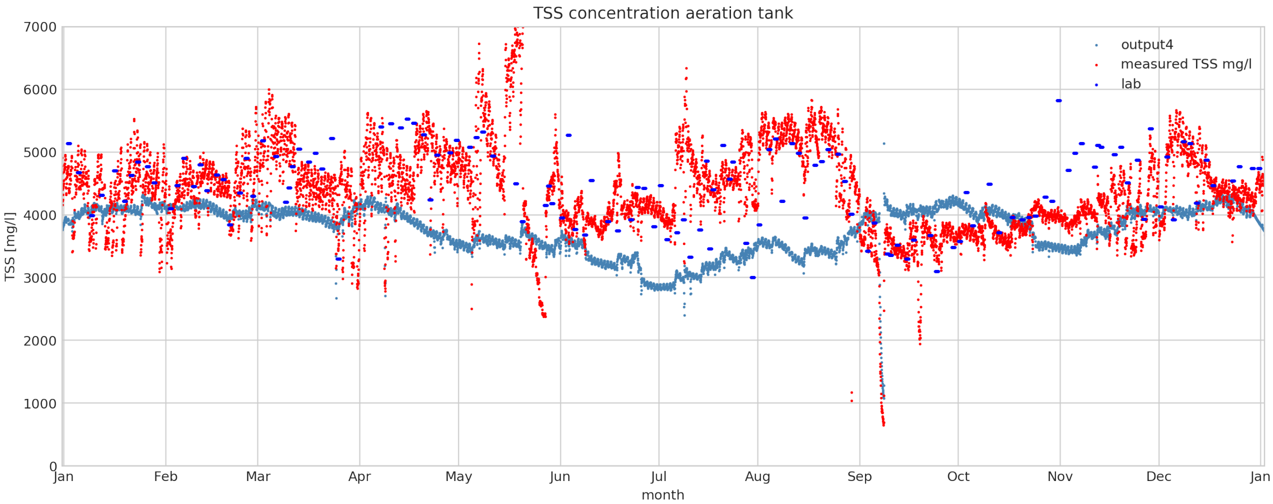


Figure 3.2.2. Biomass concentration in the bio-reactors at Willem Annapolder. The red line is the continuous measurement, the blue dots the lab measurements. In the model the excess sludge is controlled by a constant SRT, which results in light blue line

Ammonium concentration in the effluent

Using the default parameter values, the model shows an incomplete nitrification: The ammonia is converted to nitrite, but not all the nitrite is converted to nitrate. In most municipal wastewater treatment plants there is little to no nitrite present in the effluent, since the conversion from nitrite tot nitrate is the faster process of the two (Ekama and Wentzel 2008). Usually the amount of ammonia oxidising biomass is almost twice the amount of nitrite oxidising biomass (NOB) (Winkler et al 2012), which is the case for the default growth parameters in BioWin. But to increase the conversion from nitrite to nitrate the growth and decay rate of the nitrite oxidising biomass is increased to match that of the ammonia oxidising biomass. The DO half saturation for the NOB is lowered from 0.5 to 0.3 mg/L. This is done with the assumption there are no process inhibiting substances present in the influent.

The pre-calibrated model also showed a higher nitrification rate than shown in the measured effluent concentrations. Looking at the power use fo the aeration pumps, it can be seen that not all the three aerators are active at full capacity, but alternate with one aerator being at full capacity, one at half and one shut down. This is taken into account in the model by lowering the maximum aeration capacity of the aerators to 66% of the original capacity. Modelling of the alternating activity of the aerators in the three tanks did not improve the model results.

In addition to the change in aeration, the decay rate of the autotrophs are adjusted. These are the most uncertain kinetic parameters of the autotrophs and therefore the ones that are adjusted (Hulsbeek et al 2002). The decay rate of the NOB and AOB is lowered from the default value of 0.17 d^{-1} to 0.15 d^{-1}. This decrease can also compensate for the lack of AOB and NOB in the modelled influent.

Nitrate concentration in the effluent

The denitrification in the model is improved by changing the influent fractions and some of the kinetic parameters. In the first model the the denitrification was limited by a lack of biodegradable COD for the denitrifying bacteria. The influent COD fraction was therefore changed to include more biodegradable particulate matter (X_S). This is done by lowering the inorganic particulate fraction to the default value in BioWin (0.13), which is lower than the lowest fraction found by Roeleveld and Loosdrecht (2002). This resulted in more slowly biodegradable COD available in the anoxic reactors.

Another uncertainty that contributes to the lack of COD in the anoxic tank is the modelled performance of the primary clarifier. In the first model run this performance is set to match the removal of solids, measured before and after the clarifier. This removal however is higher than the measured removal of COD, which is therefore adjusted in the model to allow more COD pass through the primary clarifier.

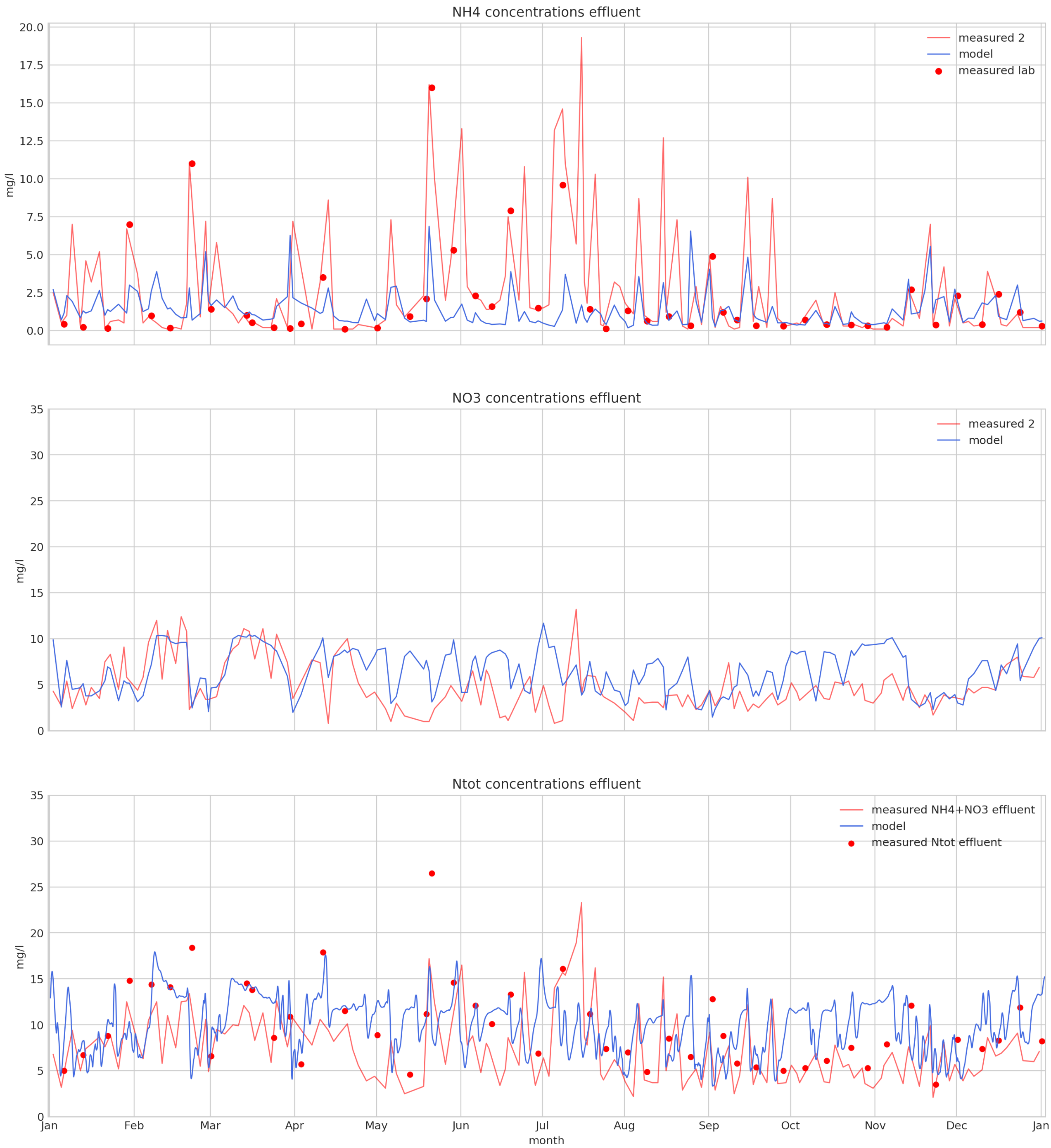


Figure 3.2.3. Effluent concentrations of the calibrated model and measured values for Willem Annapolder.

In addition to the available COD the following kinetic parameters are also adjusted: the anoxic hydrolysis factor, the anoxic growth factor for heterotrophs, the anoxic/anaerobic NOx half saturation, and the DO half saturation for the heterotrophic bacteria. These parameters are all mentioned in the STOWA protocol as the model parameters that influence the denitrification (Hulsbeek et al 2002).
The Anoxic hydrolysis factor decreases the hydrolysis of substrate under anoxic conditions. With the calibration this value is increased, which results in an increase in hydrolysis and therefore more available easily biodegradable substrate in the anoxic tank. The increase of the other three parameters also result in a better denitrification. The resulting values are shown in appendix 5.
With the new influent fractioning and the change in kinetic parameters, the effluent nitrate concentrations follow the measured concentrations. This also results in total nitrogen values that correspond with the measured concentrations.

3.2.2 Dokhaven

3.2.2.1 BioWin model:

The model of the Dokhaven treatment plant is based on two existing models. The first is a BioWin model from Cui, which was made for a master thesis study of the sludge treatment at Sluijsjesdijk. The second model is a SIMBA model made by (J van Opijnen from) Royal Haskoning DHV to study the effects of real time control on the effluent quality at Dokhaven. The model is fitted to the measured data from 2014, which is the same data used for the dynamic simulation of the SIMBA model.
The model of Cui is a steady state model. The water line of this model is used as basis and is adjusted with the control data from the model of Opijnen to create a dynamic BioWin model. Since the sludge treatment is not of interest in this study it is omitted from the model. The return flow from the sludge treatment at Sluisjesdijk is included by adding an extra influent source in the BioWIn model.

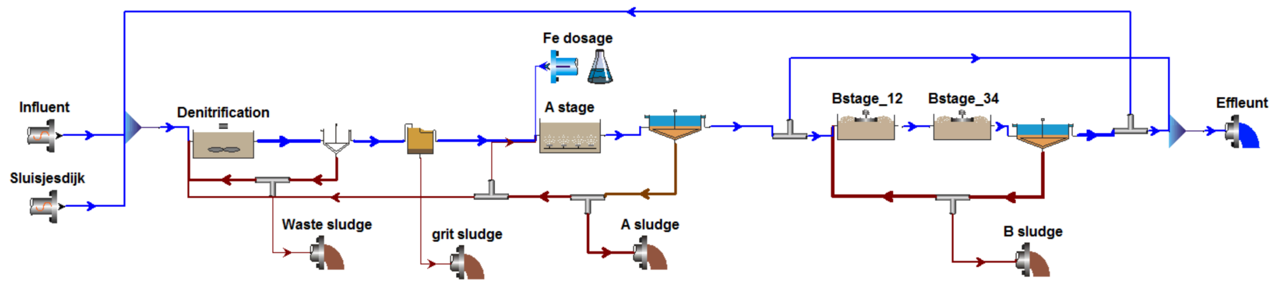


Figure 3.2.4. BioWin model layout Dokhaven treatment plant

Data:

Just as in the case of Willem Annapolder the data used for the model comes from the routine measurements. This includes hourly flow measurements and the lab measurements. The data is checked on errors when possible. Measurements errors were found in the underflow of the settling tanks in the A and B stage. When this flow has a negative value or becomes zero, but there is no measured increase in the amount of solids in the respective activated sludge tanks, these values are replaced by a linear interpolation of the two adjacent correct values. The same is done for the measured excess sludge from the A and B stage.

Model setup:

Virtual denitrification

The model can be divided into 4 elements, which are attached to the incoming influent and the resulting effluent flow.
The first element is a virtual segment, meaning a tank that is not present in the real wastewater treatment plant. This element consists of a small unaerated bioreactor followed by an ideal point settler which recycles the sludge back to this reactor. This element is originally installed by Cui to simulate the denitrification that happens at the screens (Cui 2012). In the model of Opijnen this element was present as a volume which represents the distribution tank and the screens.

Grit Chamber
The second segment is the grit chamber, which is unaerated since this option is not available in BioWin. Since there are no large elements in the influent, the sludge underflow from the grit chamber is kept very low (Cui 2012).

A-stage
After this the water goes through the AB system. The third segment therefore is the high loaded A part. This is modelled as one large aerated bioreactor, aerated with course bubble aeration. This is followed by an ideal settling tank. Unlike the BioWin model of Cui this settling tank is modelled without biological reactions taking place inside the reactor to reduce the run time of the model. The sludge from the settler is connected to the preceding bioreactor and the virtual denitrification tank in the beginning of the water line. The direction of the flow depends on the influent flow received by the treatment plant. If the influent is below 12310 m3/h the return sludge goes back to the denitrification tank and if the flow is higher it goes directly back to the A stage bioreactor. The underflow from the settler is kept constant. Part of this sludge flow is discharged as surplus sludge, which is modelled with the measured flow values.
The aeration tank is aerated by plate aerators, which aims to a dissolved oxygen setpoint of 0.4 mg/L. (Opijnen 20017)

Overflow
Before the water enters the B stage it passes through a bypass weir, which prevents the hydrological overloading the B-stage aeration tanks. Whenever the flow surpasses 14250 m^3/h the extra flow is directed to the effluent

B-stage
The last segment is the B stage and is modelled as two consecutive point aeration tanks followed by an ideal settling tank. It was chosen to use two tanks with point aerators instead of four separate tanks for simplification and because the control is the same for the first two aerators and the last two aerators. This did not influence the model performance.
The aeration capacity in these tanks is adjusted from the default values to match the power available in Dokhaven. Because the tank represents the aeration of the four parallel water trains, each tank is modelled as having eight point aerators. The first aeration tank operates at a constant maximum capacity, while the aeration in the second tank — point aerator number 3 and 4 in reality— is controlled.
The last aeration tank is connected to the accompanying B-stage settler, which is also modelled without biological reactions. This settler also has a constant sludge underflow, which is partially disposed as surplus sludge. The recycled sludge is pumped back to the first aerator tank of the B-stage. The surplus sludge flow is modelled with the measured flow values.

Recirculation
The cleaned water coming from the last settler is split into two flows. One is discharged as effluent and a part is recycled back to the beginning of the water train. It should be noted that in the period between september 2014 till may 2015 extra recirculation was taking place at Dokhaven. This was due to a fire that broke out in the chemical storage for the air treatment. This caused the air treatment of the plant to be unoperational until may 2015. Extra iron-chloride was dosed and the effluent circulation was increased in order to increase the oxidation and binding of smellcomponents [Besten Noteboom 2014].

At Dokhaven phosphate is removed by chemical precipitation. In the model a constant flow of iron chloride is added to the beginning to the A stage. 450 l/d with a 40% Fe Cl3 by weight.

The dimensions of the tanks can be found in appendix 4.

Control
In the model two elements are controlled. These are the aeration in the B stage and the recirculation of the effluent. The aeration in the B stage differs between the first and the second tank. The first aeration tank — aerator number 1 and 2 in reality— is constantly aerating at maximum capacity. The other tank is operated through BioWin Controller, where the DO setpoint is equal to the measured ammonia concentration at the end of the B stage, with a maximum of 2.25 and minimum of .75 mg/L.

The control of the effluent recirculation is done with the aim of having a constant high hydraulic load around 10000m3/h. This follows the description of the control by van Opijnen. In BioWin this is controlled by a proportional controller, which controls the recirculation flow proportionally to the difference between the aimed flow and the influent flow. The recirculation is also controlled by the measured ammonia and nitrate concentrations in the model. If the nitrate concentration is under 7mg/L there is no recirculation, since there is no need for further denitrification. If the ammonia concentration at the end of the B stage is higher than 4 mg/L the recirculation is limited to 2700 m3/h (Opijnen 2017).

The model described above, together with the calculated influent, forms the basis used for further calibration.

3.2.2.2 Calibration

Sludge

The sludge in the A and B stage are calibrated in order to have a correct solid content in the bioreactors. This is done by changing the surplus sludge flow, since this is the most sensitive parameter. Using the measured surplus sludge flow the sludge in the B stage aerators is around 8kg/l, while the average measured concentration is around 2700 g/l. Both the A and B stage surplus sludge flows and the influent COD fraction are adjusted to have a similar biomass content in the model as is measured.

The sludge content is also greatly influenced by the performance of the linked secondary settlers. When this decreases the solids concentration in the sludge return flow also decreases and thereby the solids content in the aeration tank. In order to match the variability in the measured concentrations, the performance of the settlers in the model varies with the difference between the measured solids concentration in the aeration tanks and the effluent flow of the settler. The resulting solids content is shown in figure 3.2.5.

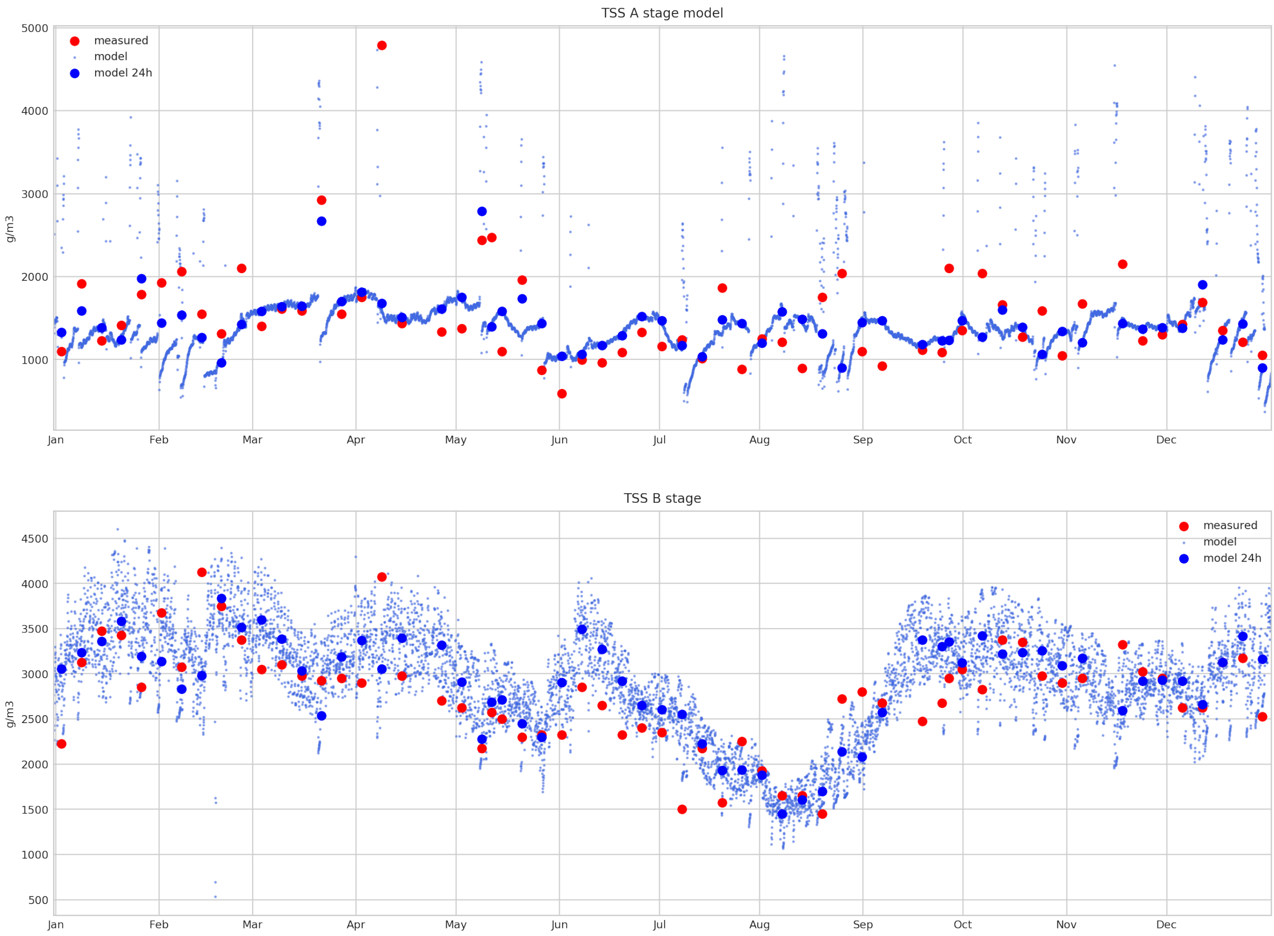


Figure 3.2.5 Solid content in the A and B stage aerators

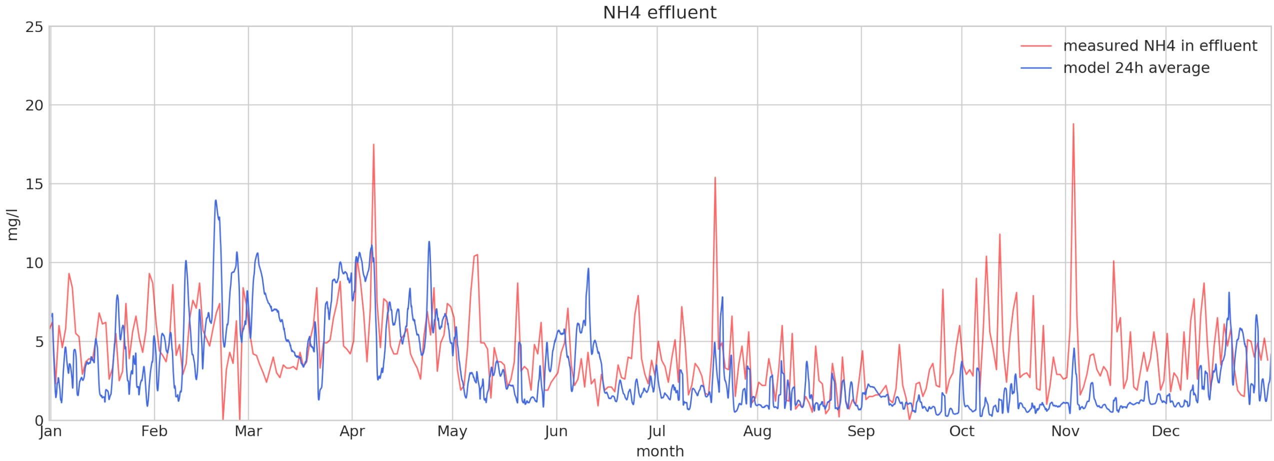


Figure 3.2.6. Effluent ammonia concentrations for the calibrated model for Dokhaven.

Ammonia in the effluent:

The first model run has a very low nitrification rate, which leads to ammonia concentrations in the effluent of 20mg/L. This is changed in two ways. The first is the most sensitive: the aeration in the model. Unfortunately no measurement data of the dissolved oxygen concentrations is available in either the A or B stage aeration tanks. The resulting oxygen concentrations in the model are therefore not compared to those in reality.

The A stage's main function is to reduce the COD by aeration. The COD in the A stage settling tank is measured and is twice as low as the modelled concentration. The aeration in the A stage is therefore increased by changing the DO setpoint from 0.4 to 0.8 mg/L, which is also the setpoint used in the original model by Cui. The increase in aeration in the A stage also makes up for the unaerated grit tank in the model, which is aerated in reality.

In the B stage the maximum aeration capacity is increased from 240 kW to 360 kW. For the autotrophs finally both the decay rate as the growth rate are changed. The decay rate is decreased to 0.13 d^{-1} and the growth rate is changed to 1.0 d^{-1} to further increase the nitrification.

With the default parameters there is also a very low conversion from nitrite to nitrate. To resolve this the growth and decay rate of the nitrite oxidising biomass are set at the same levels as the ammonia oxidising biomass. Additionally the DO half saturation for the NOB is decreased from the default 0.5 to 0.2 mg/L. This resulted in effluent nitrite concentrations shown in figure 3.2.7, with a peak concentration of more than 4 mg/L.

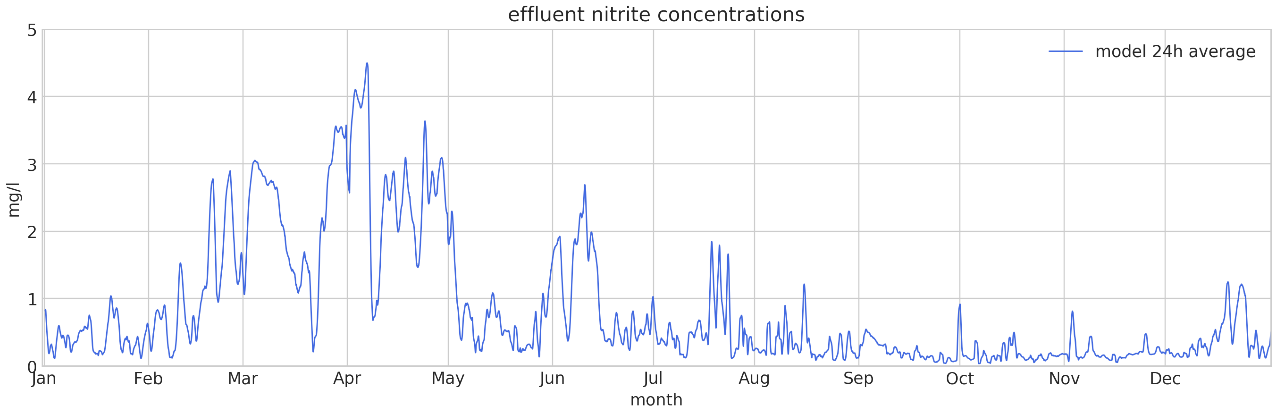


Figure 3.2.7. Effluent nitrite concentrations for the calibrated model for Dokhaven.

Nitrate and total Nitrogen in the effluent

The denitrification takes place in the virtual denitrification tank at the beginning of the water line. In the calibration the denitrification rate is decreased in comparison to the first model setup. This is done by decreasing the easily degradable COD fraction in the influent. This influences the available substrate for the denitrifying organisms and thereby the denitrification rate. The resulting effluent concentrations of NOx and total nitrogen are shown in figure 3.2.8. The measured NOx values are the sum of the nitrate and nitrite concentrations. The resulting total nitrogen concentrations in the effluent are shown in fur

Figure 3.2.8 Effluent NOx and total nitrogen concentrations for Dokhaven

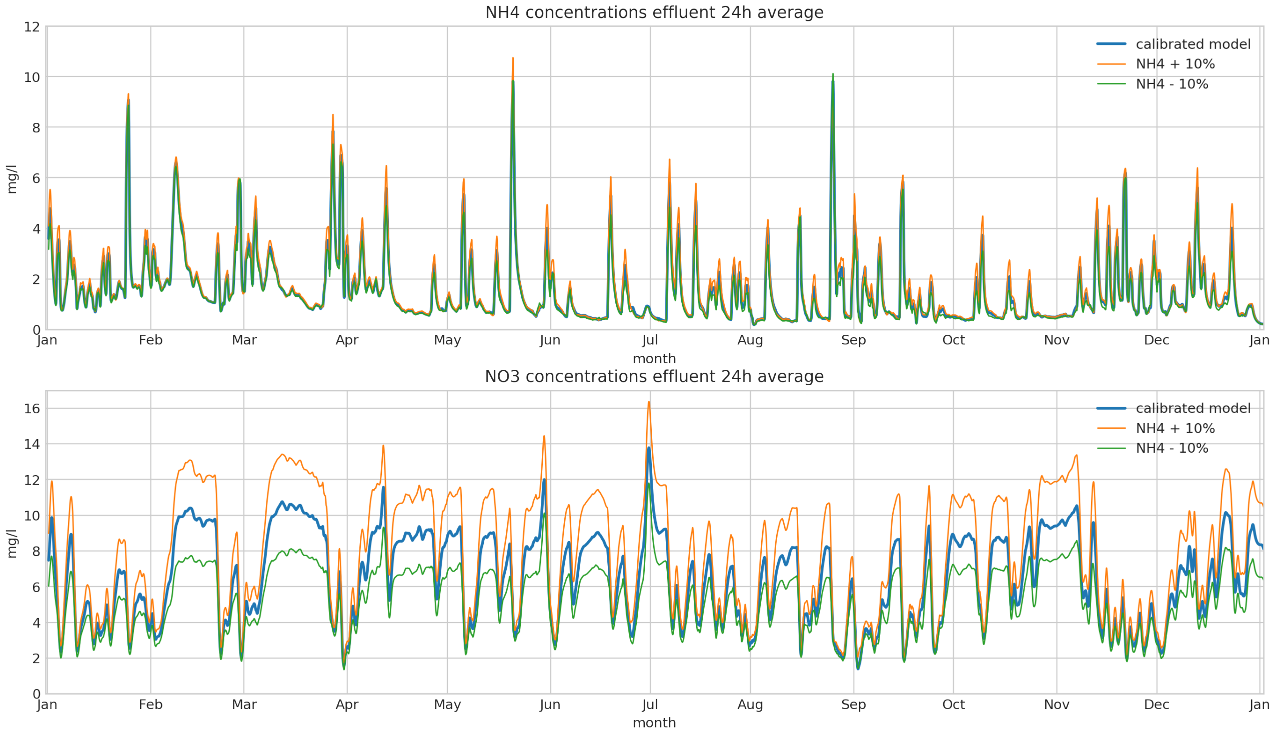


Figure 3.3.1: Variation in the effluent concentrations of ammonia and nitrate due to a 10% increase or decrease in the influent ammonia concentrations. Since the model has difficulties with the denitrification process this is also the most sensitive to the variations in the influent.

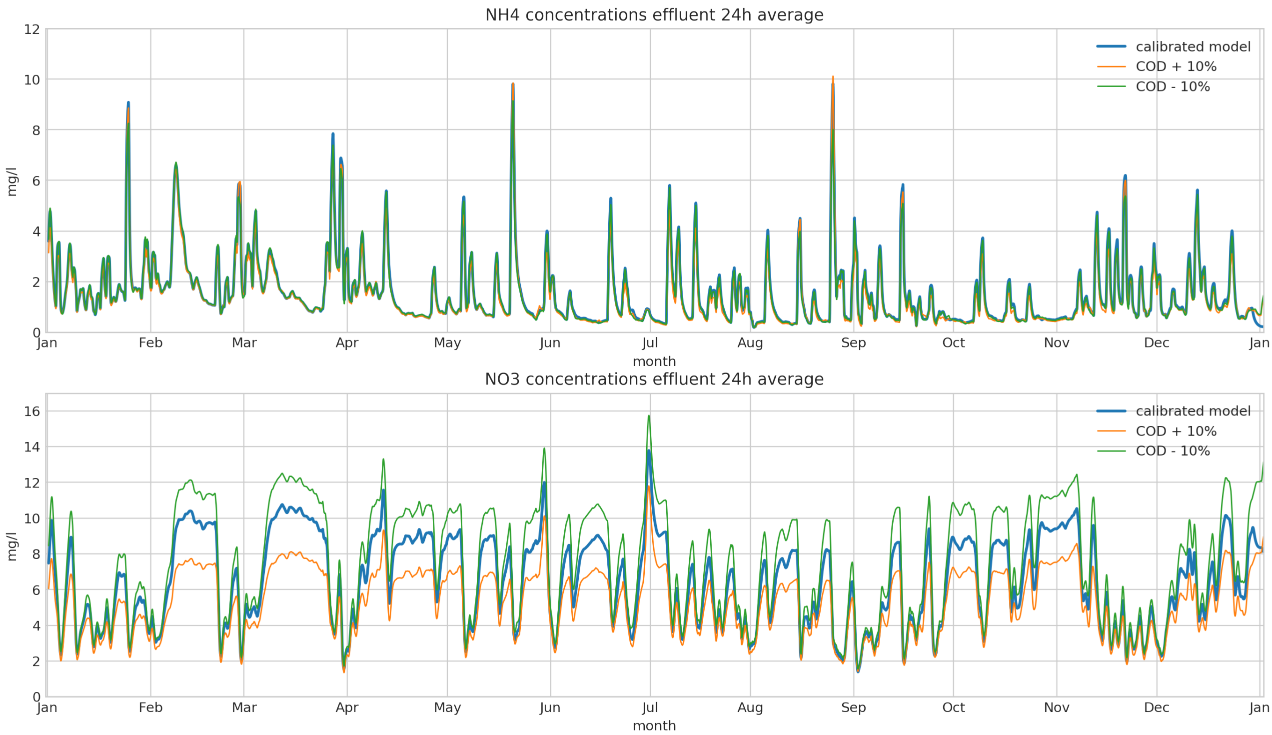


Figure 3.3.2: Variation in the effluent concentrations of ammonia and nitrate due to a 10% increase or decrease in the influent COD concentrations. The denitrification process increases when more biodegradable COD becomes available.

3.3 Sensitivity Analysis

The resulting effluent concentrations of the models are a result of how the calibrated influent passes through the calibrated model of the treatment plant. This means that when there is an error in the influent model, this results in a possible error in the effluent. A sensitivity analysis shows how an error in the influent propagates through the BioWin model.
This is done by running the calibrated BioWin models with an adjusted influent in which the ammonia or COD concentration is increased or lowered by 10%. This is done with the calibrated influent fractions.

3.3.1 Willem Annapolder

Figure 2.3.1 shows the resulting effluent concentrations for an increase and decrease of ammonia concentration in the influent. The ammonia concentrations in the effluent stay reasonably stable; the 10% increase and decrease result in an 8.5 % and 8.8% increase and decrease of the average effluent concentration. The difference is largest at moments of high flow as is expected. The resulting nitrate concentrations show larger variation. The change in influent concentrations result in a 22% increase and 18% decrease for an increase or decrease of the influent concentration. Since the calibrated model still has difficulty with the denitrification process, an error in the influent will be most visible here.
The denitrification is also shows the most variation when the COD concentration in the influent varies. Since the COD is characterised to have a large biodegradable fraction, an increase in COD in the influent gives more substrate available for the denitrification process and thus lower nitrate concentrations in the effluent. The 10% increase in the influent results of an 18% lower nitrate concentration in the effluent; a similar decrease causes a 16% increase.
The effect of the COD concentration on the ammonia in the effluent is an 8.5% increase and 8.8% decrease for an increase and decrease in the influent respectively.

Table 3.3.1: Overall changes in effluent concentrations and biomass content in the aeration tanks due to a 10% increase or decrease in the influent concentration of ammonia or COD.

Influent \ Effluent	NH4 [%]	NO3 [%]	TSS AT
NH4 + 10 %	8.5	22	
NH4 - 10 %	-8.8	-18	
COD + 10%	8.5	-18	1.0
COD - 10%	-8.8	16	-8.0

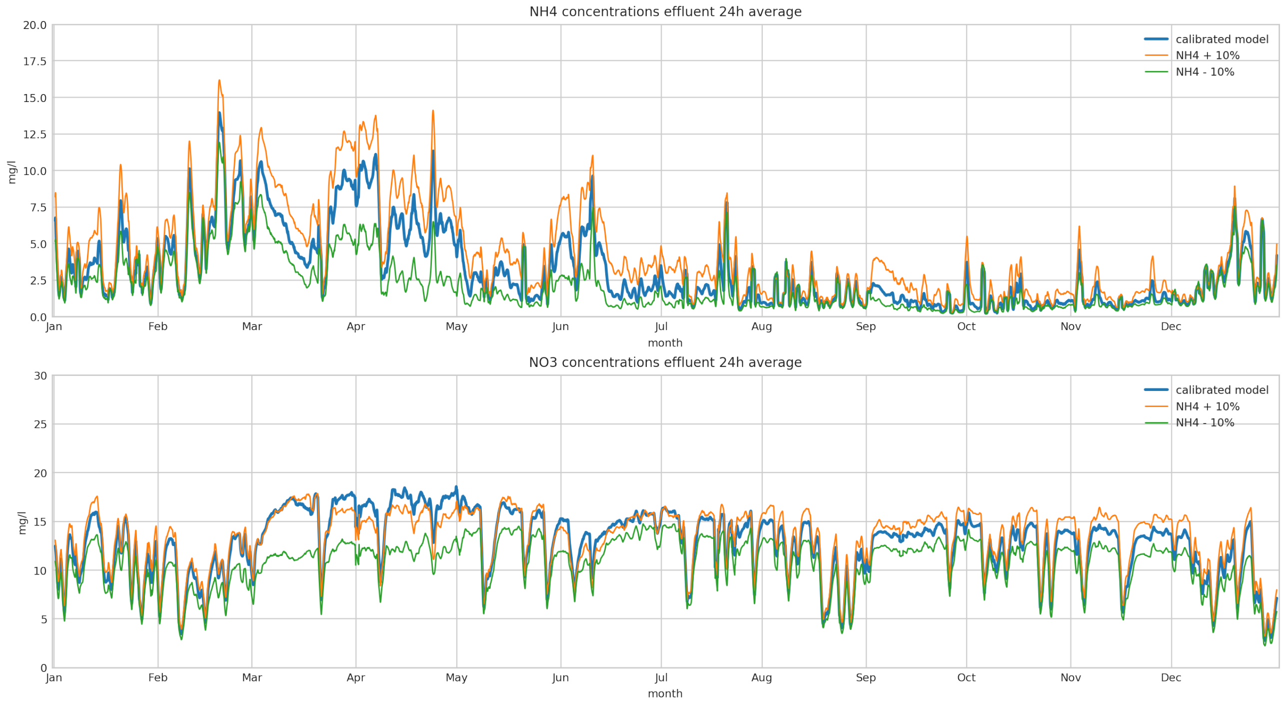


Figure 3.3.3: Variation in the effluent concentrations of ammonia and nitrate due to a 10% increase or decrease in the influent ammonia concentrations.

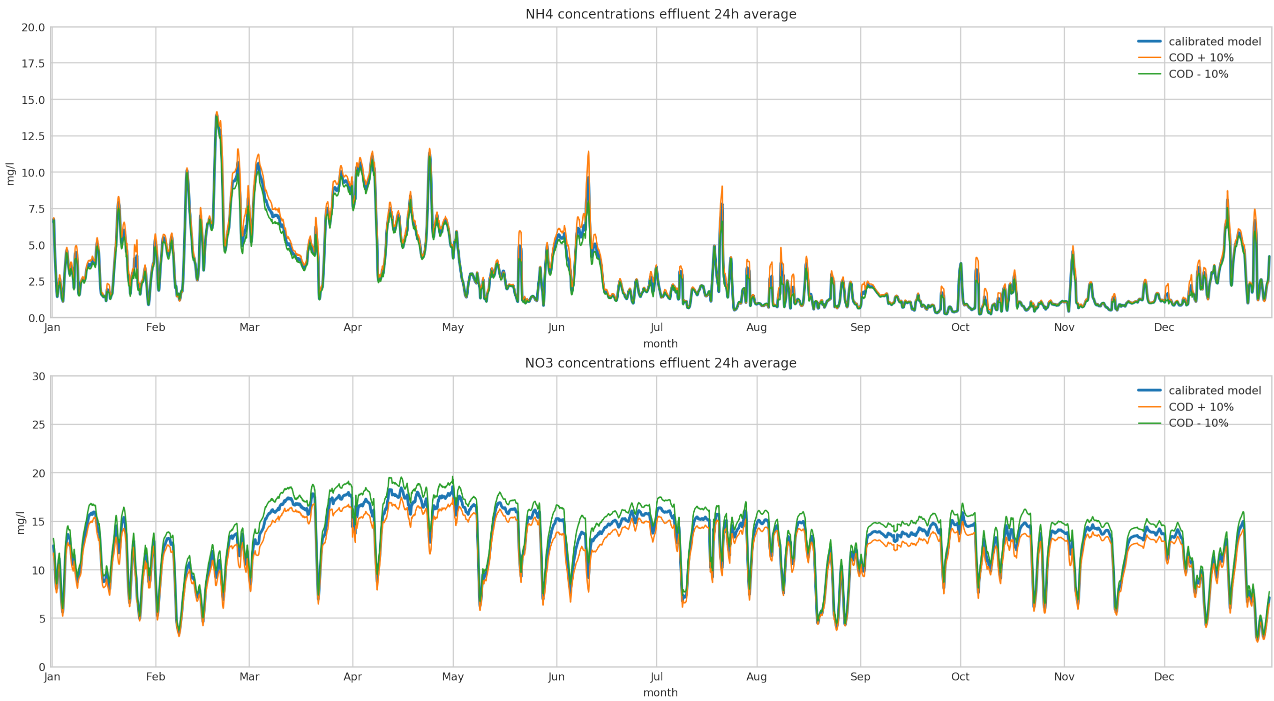


Figure 3.3.4: Variation in the effluent concentrations of ammonia and nitrate due to a 10% increase or decrease in the influent COD concentrations.

3.3.2 Dokhaven

The model of the Dokhaven treatment plant has trouble with the nitrification process during the first half year. This is also visible in figure 2.3.3: the effluent ammonia concentration is very sensitive to the variation in the influent concentration. A 10% variation causes a threefold variation in the effluent.

Consequently an increase in the ammonia influent has little influence on the denitrification process as less ammonia is converted to nitrate. The increase and decrease in the influent results in an increase of 4 % and a decrease of 17 % of the average effluent nitrate concentration.

The variation in the influent COD has only limited effect on the effluent concentrations. Just as in the case of Willem Annapolder an increase in COD in the influent decreases the average nitrate effluent concentration. As expected the COD influences the solid content in the aeration tanks, and thereby the sludge production to the same degree.

Table 3.3.2: Overall changes in effluent concentrations and biomass content in the aeration tanks due to a 10% increase or decrease in the influent concentration of ammonia or COD.

Influent \ Effluent	NH4 [%]	NO3 [%]	TSS A [%]	TSS B
NH4 + 10 %	34.0	4.0	-	
NH4 - 10 %	-30.7	-17.0		
COD + 10%	6.4	-6.0	8.0	9.0
COD - 10%	-4.7	7.0	-8.0	-8.0

4
Results

The BioWin models for Dokhaven and Willem Annapolder are calibrated for the measured effluent concentrations for the year 2014 and 2015 respectively. To see the the effects of an influent with reduced extraneous water, the calibrated models are run with new influents — with reduced flow and the associated increase in concentrations. As explained in chapter 3.1.3, the new influent concentrations are calculated in such a way that the load remains the same. The characterisation of the influent is also kept the same, as are the flows and concentrations from the sludge treatment. The results thus show the effects of a reduction in extraneous water on the modelled treatment plants. As the models are calibrated on their nitrogen removal, this is also the performance that is evaluated in the new situation.

4.1 Willem Annapolder

As can been seen in figure 4.1.1, the reduction of extraneous water at Willem Annapolder increases the effluent ammonia concentration. This effect is visible during dry weather but mostly in the effluent peaks at the onset of rainevents.
The latter can be explained by the effect of the pressure mains on the influent load: the dilution of the influent concentration comes later than the increase in influent flow. When the extraneous water is reduced, the initial plug has a higher dry weather concentration resulting in higher peak loads, as can be seen in figure 4.1.2.

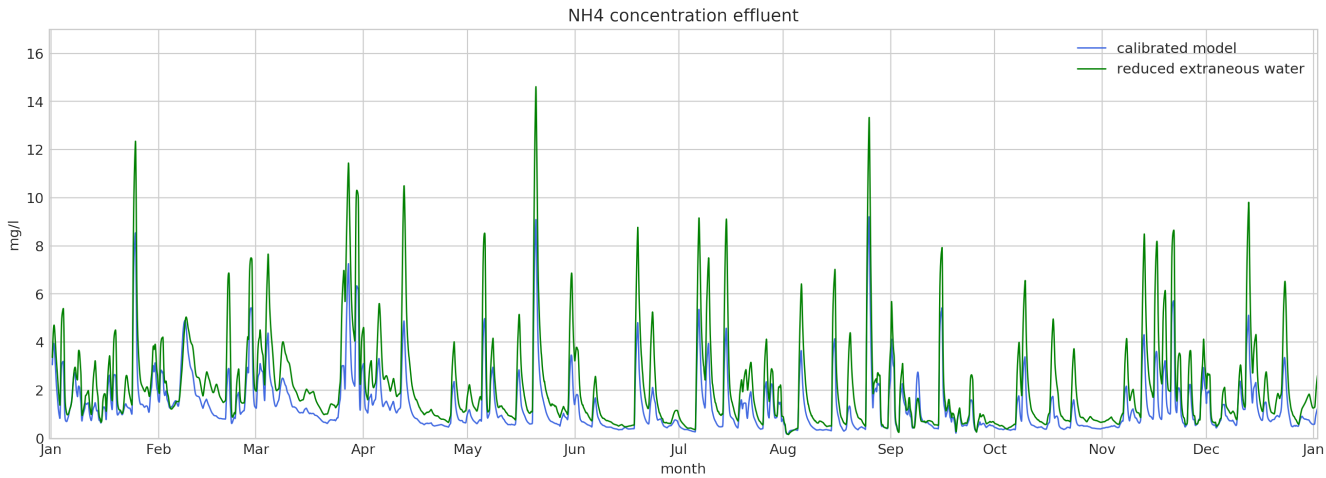


Figure 4.1.1: Effluent ammonia concentrations Willem Annapolder for the calibrated model and the model run without extraneous water. Values are moving averages over 24h.

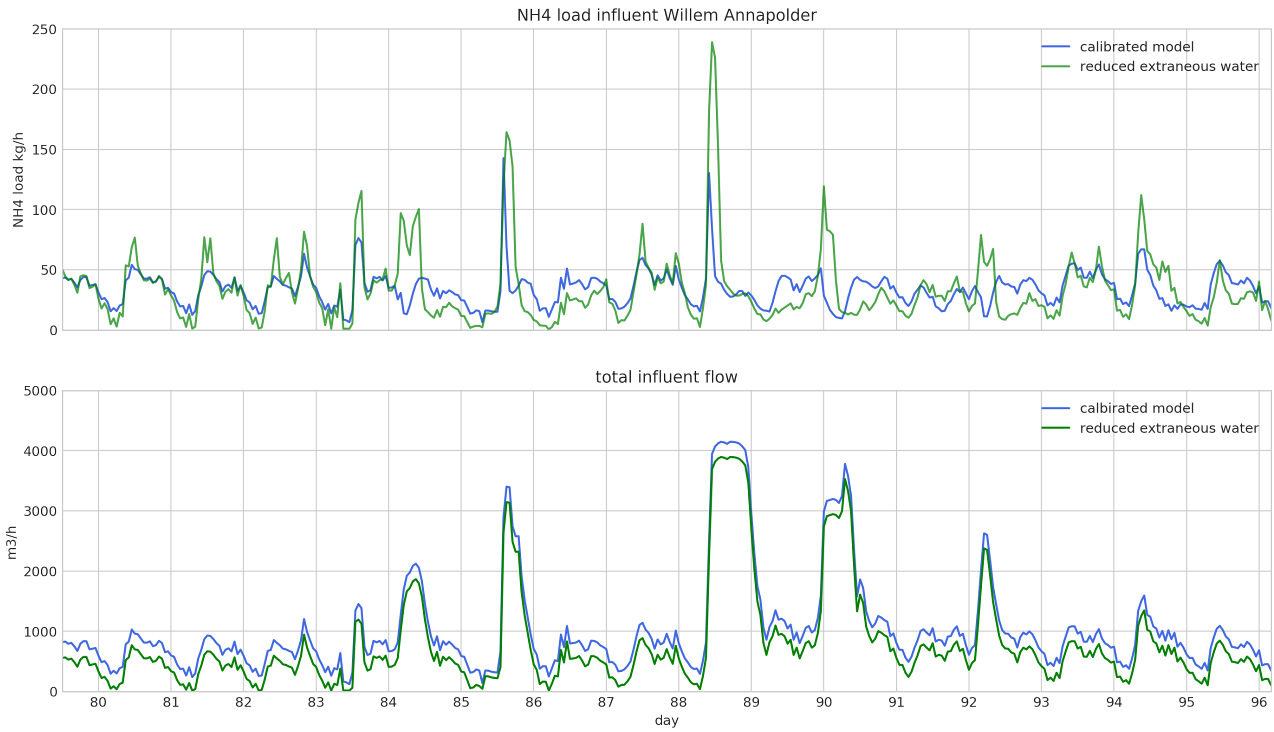


Figure 4.1.2: Influent ammonia load and flow for the end of March.

At Willem Annapolder the measurement of the effluent ammonia concentration show peaks in concentration at the start of rainevents. The reduction in extraneous water results in even higher peaks, as the treatment plant is unable to nitrify the high loads of ammonia coming in (figure 4.1.1 and 4.1.3).

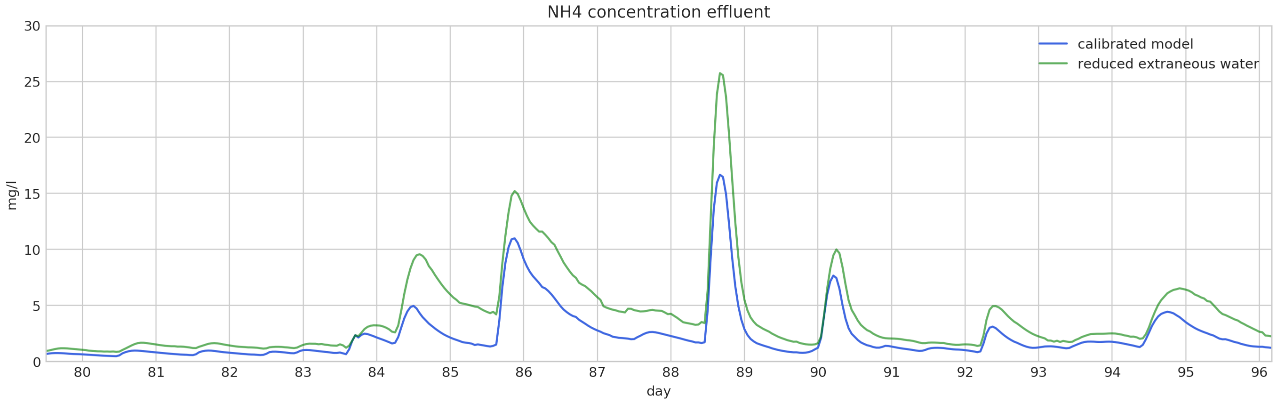
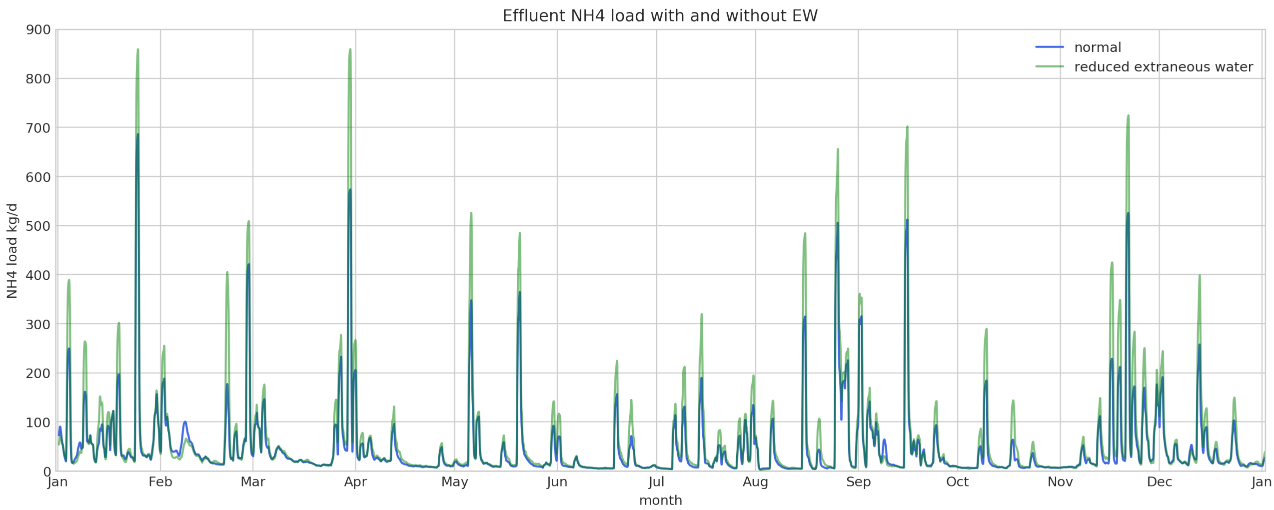


Figure 4.1.3: Effluent ammonia concentrations for the end of March.

Contrary to what is expected, the effluent ammonia concentration during dry weather before the peaks is also higher in the scenario with reduced extraneous water than in the calibrated model. Usually a longer residence time in the aeration tank increases the nitrification. The results show a small decrease in the nitrification rate (about 4,5 %) compared to the calibrated model. This is linked to less ammonia oxidising biomass (4% on average) in the model with reduced extraneous water.

Because the increase in effluent concentration of ammonia is higher than the proportional decrease in flow overall effluent load of ammonia on the receiving water also increases. This is mostly due to the high peaks during wet weather. During dry weather the increase in the effluent load is 8%; during rainevents this increase is 31%. Overall this results in an increase in the ammonia load of 29%.



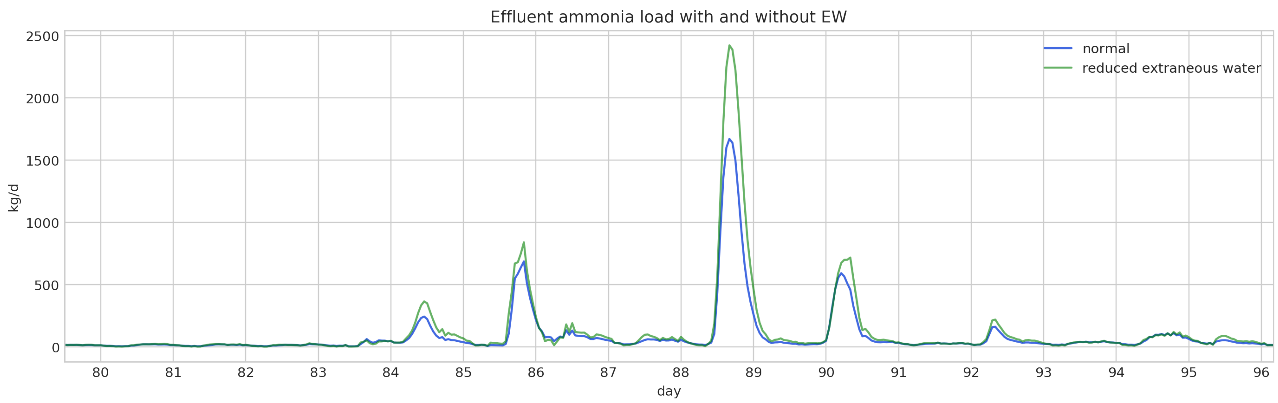


Figure 4.1.5: Effluent ammonia load for the end of March (same period as figure 4.1.2 and 4.1.3)

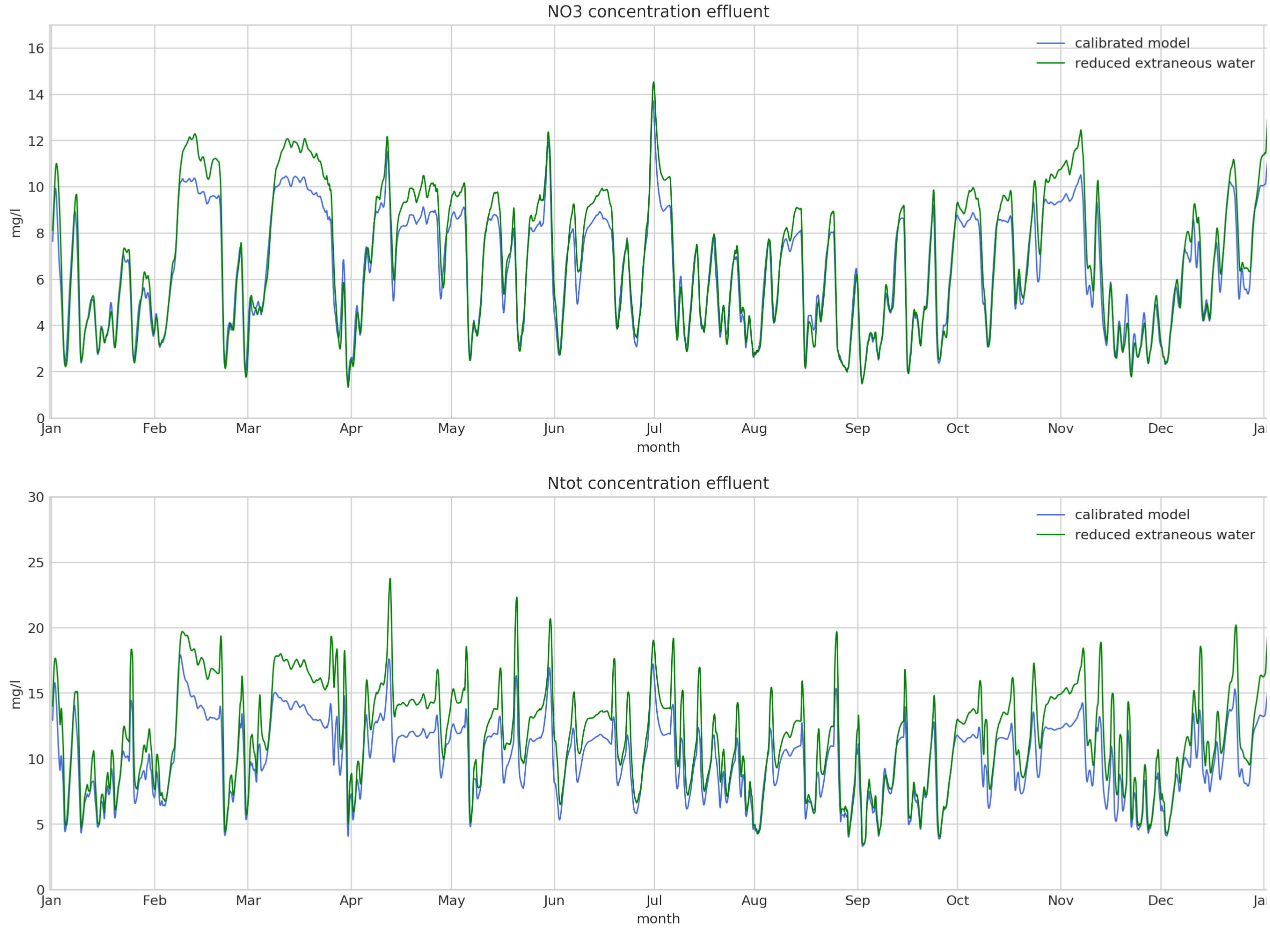


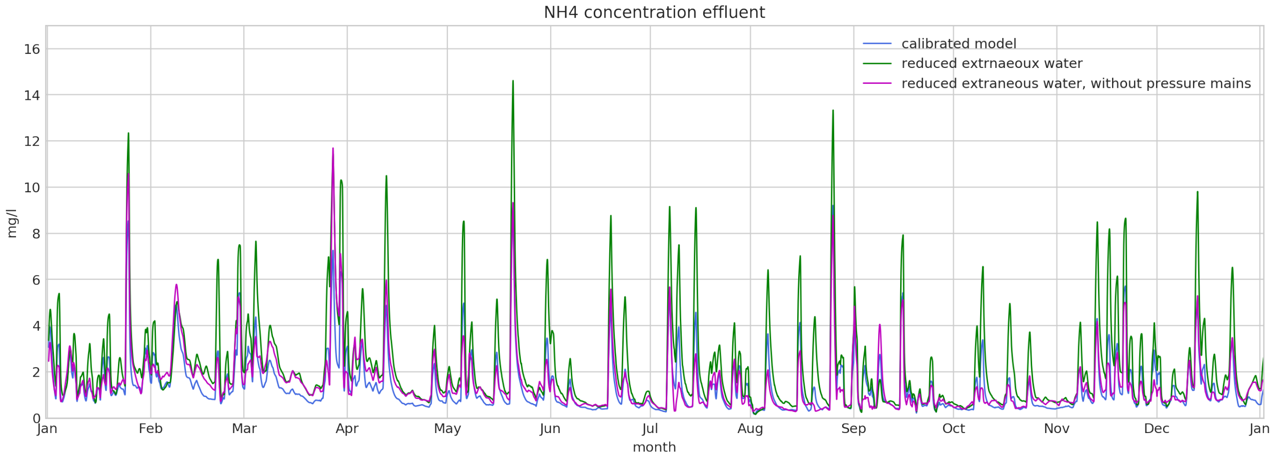
Figure 4.1.6: Effluent nitrate and total nitrogen concentrations at Willem Annapolder. The values are 24h moving averages

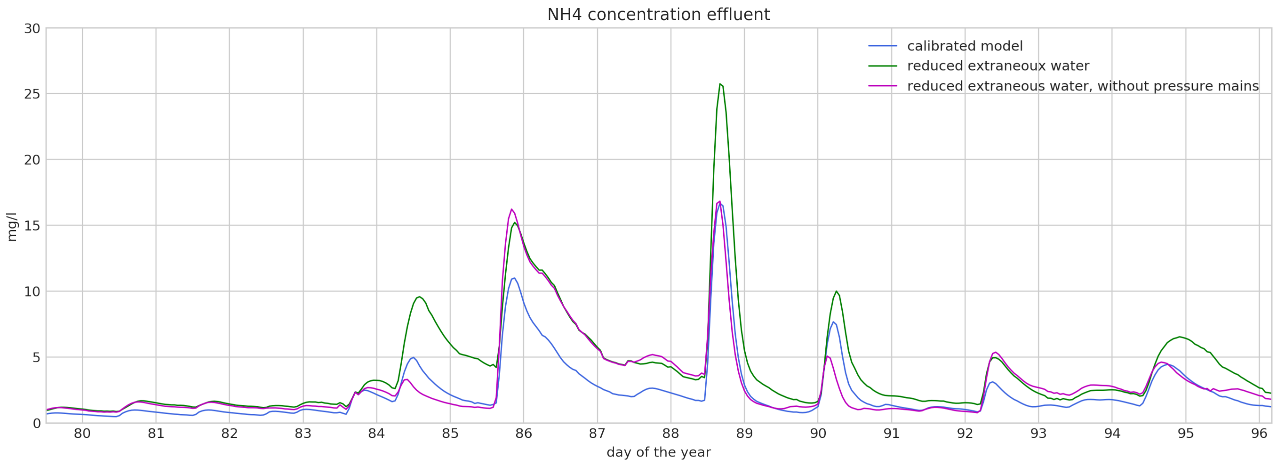
The resulting nitrate concentrations in the effluent are also slightly higher in the case with less extraneous water, with a 6% increase. The increase in ammonia and nitrate concentration result in an increase in the total nitrogen concentrations in the effluent which on average is 20% higher than in the calibrated model (figure 4.16).
However, since the nitrate concentration increased less than the decrease in flow, the overall effluent load does decrease about 9% on average. During the start of the rainevents the increase in ammonia concentrations in the effluent remains dominant, which still leads to higher peaks of nitrogen loads at the start of the rainevents. This causes the overall load during wet weather to only decrease by 3%; the load during dry weather decreases with 23% which results the 9% average decrease in total nitrogen effluent load (figure 4.1.7).

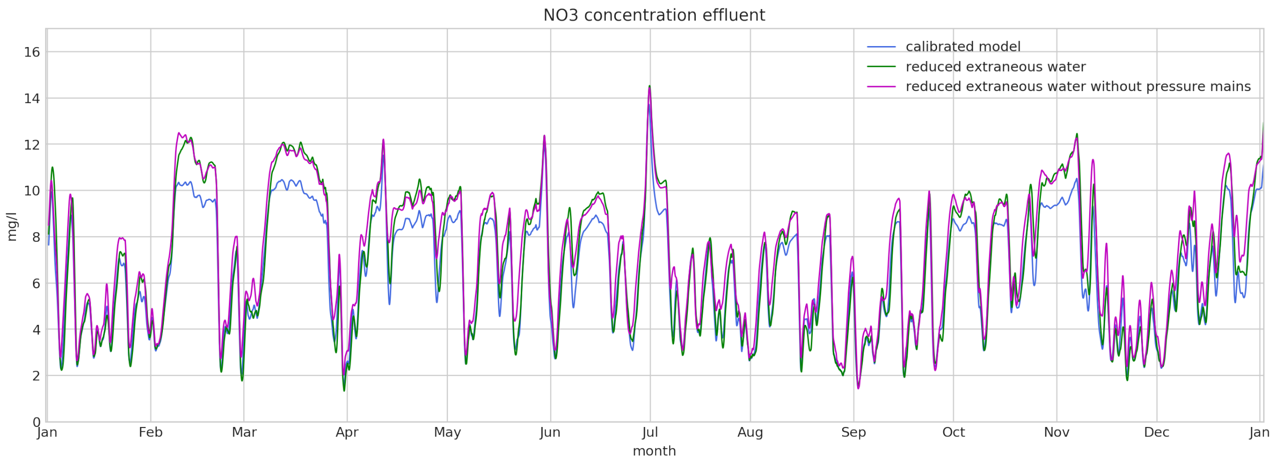
In the calibrated model, the average total nitrogen concentration in the effluent was 10 mg/l, which is close to the measured 9,8 mg/l for the modelled year. This is the same as the allows maximum average concentration in the effluent for large treatment plants — treating more than 20.000 i.e. — in the Netherlands. As the total nitrogen effluent concentration becomes larger with the reduction in extraneous water in the model, the resulting 12 mg/l total nitrogen surpasses this upper limit.

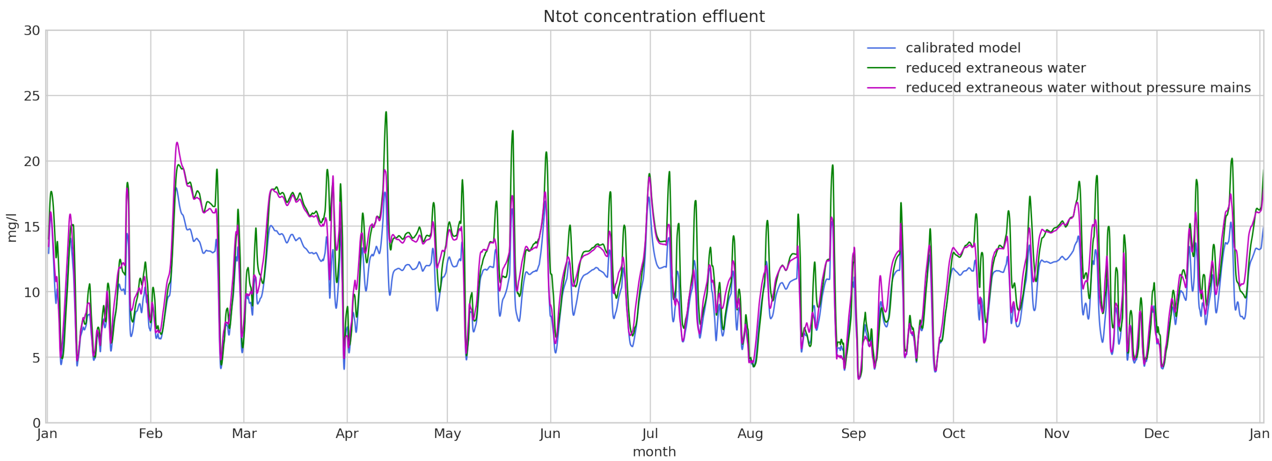
Figure 4.1.7: Effluent nitrate and total nitrogen load at Willem Annapolder. The values are 24h moving averages

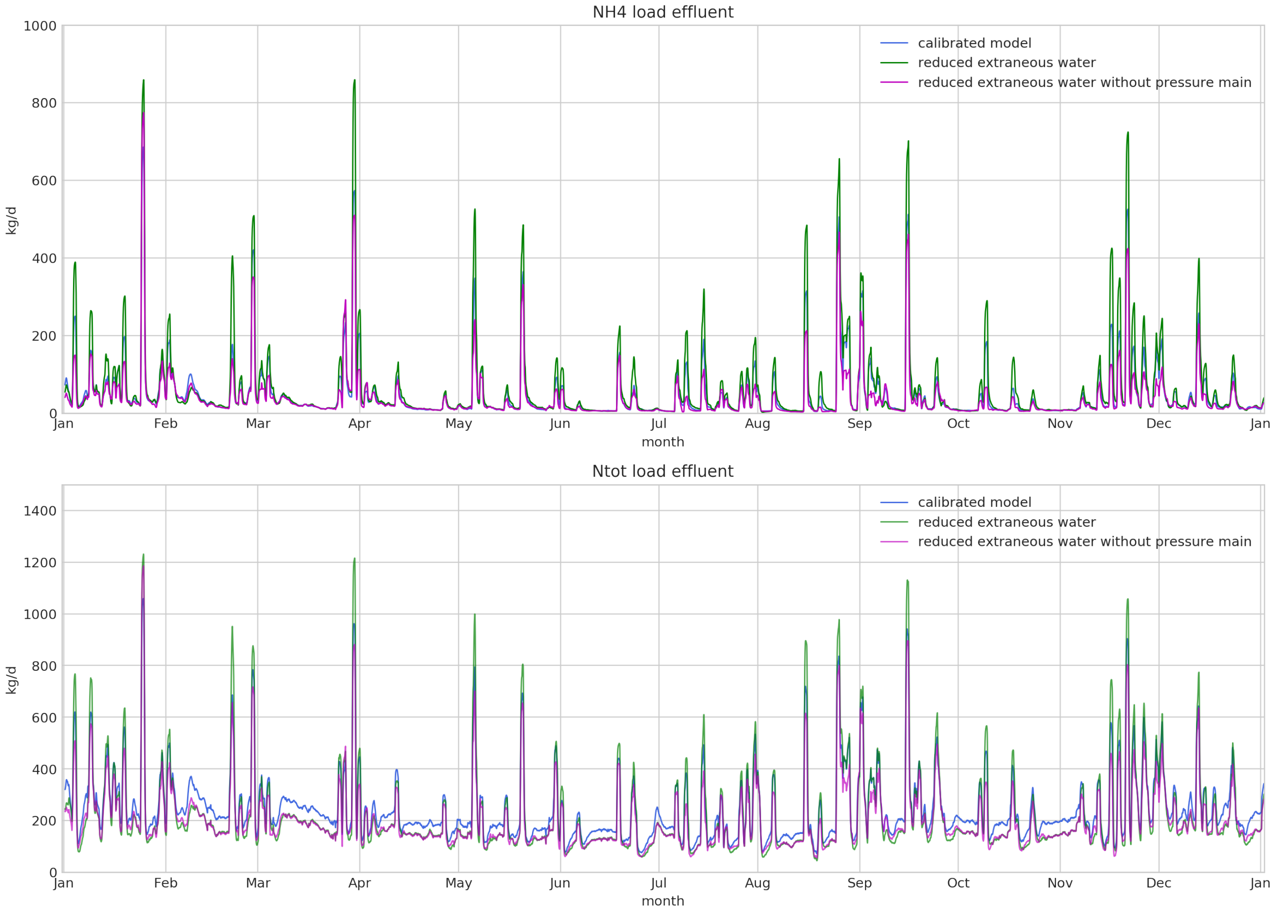
In order to distinguish the effect of the pressure mains and the reduction of extraneous water from each other, the BioWin model is also ran with another influent that only incorporates the latter. This is the time series of the concentrations made for the influent with reduced extraneous water before it goes through the virtual pressure mains as described in chapter 3.1.3.
Figure 4.1.8 and 4.1.9 show the resulting effluent concentrations and loads for the different nitrogen components. The results show that the effluent ammonia peaks are not present when the pressure mains are not modelled.
Just as the influent with the pressure mains, this new influent also causes an increase in the dry weather concentration of ammonia in the effluent. On average this is 33% higher than the concentrations from the calibrated situation; the influent with pressure mains and reduced extraneous water caused a 64% increase. The difference in concentrations are shown in table 4.1.1.
The differences are also clearly visible in the resulting effluent load. When the model is run without pressure mains, the reduction in extraneous water leads to an overall reduction in ammonia effluent load — 20% decrease instead of a 30% increase — and a stronger reduction in the total nitrogen effluent load — 16% instead of 8%.


NH4 concentration effluent
calibrated model
reduced extrnaeoux water
reduced extraneous water, without pressure mains
mg/l
month
Jan Feb Mar Apr May Jun Jul Aug Sep Oct Nov Dec Jan


NH4 concentration effluent
calibrated model
reduced extraneoux water
reduced extraneous water, without pressure mains
mg/l
day of the year


NO3 concentration effluent
calibrated model
reduced extraneous water
reduced extraneous water without pressure mains
mg/l
month
Jan Feb Mar Apr May Jun Jul Aug Sep Oct Nov Dec Jan


Ntot concentration effluent
calibrated model
reduced extraneous water
reduced extraneous water without pressure mains
mg/l
month
Jan Feb Mar Apr May Jun Jul Aug Sep Oct Nov Dec Jan



Figures 4.1.8 (left page) and 4.1.9: The top two plots of figure 4.1.8 show the 24h average effluent ammonia concentrations for the different models. The green peaks clearly show the effect of the pressure mains on the effluent concentration. The zoom for the period around the end of March show that the effluent ammonia concentration during dry weather is also higher than in the calibrated model. The nitrate concentrations are similar for both models with reduced extraneous water. 4.1.9 shows the effluent load for ammonia and total nitrogen. Because there are no high peaks of in the effluent, the effluent load for the model without pressure mains is lower.

Table 4.1.1 Effluent nitrogen concentrations and loads. The effluent ammonia concentration is highly influenced by the pressure mains when the extraneous water is reduced. The overall reduction in pollution load gained when the extraneous water is reduced is also less because of the pressure mains.

	Average effluent concentration [mg/l]			**yearly effluent load [1000 kg/y]**		
	NH4	**NO3**	**Not**	**NH4**	**NO3**	**Ntot**
measured	2.1	5.3	9.8			
calibrated model	1.4	6.5	10.0	429	1265	2162
reduced extraneous water	2.2	6.9	11.8	553	990	1972
reduced extraneous water without pressure mains	1.5	7.3	11.5	344	1050	1811

4.2 Dokhaven

In the new influent for Dokhaven the amount of parasitic water is reduced by 77% compared to the estimated total amount; this is an average reduction of the hydraulic load by 23%.
As can be seen in figure 4.2.1, the resulting effluent ammonia concentrations are not much effected. This is similar to the findings of Dirckx et al (2019), where the reduction of extraneous water only slightly improved the effluent concentrations. The increased ammonia concentrations in the influent and lower influent flow allow for an increase in the ammonia oxidising bacteria in the biomass (figure 4.2.2). The lower influent flow also allows for an increased recirculation of the effluent from the end of the B stage to the virtual denitrification tank (figure 4.2.3). This recirculation is controlled to aim for a constant hydraulic load on the A stage. On average the effluent recirculation in the model was increased by 12 percent. Both the influent recirculation and the increase in ammonia oxidising bacteria lead to ammonia concentrations which are more or less equal to the concentrations in the normal situation.
This is similar to the results of the case study of Utrecht in the STOWA report on the reduction of hydraulic load (Korving et al 2015). The studied treatment plant also has an A-B configuration with effluent recirculation. Though the study didn't model the effects on the treatment plant, they expected that the improved nitrification resulting from the decreased extraneous water comes from the increased recirculation.

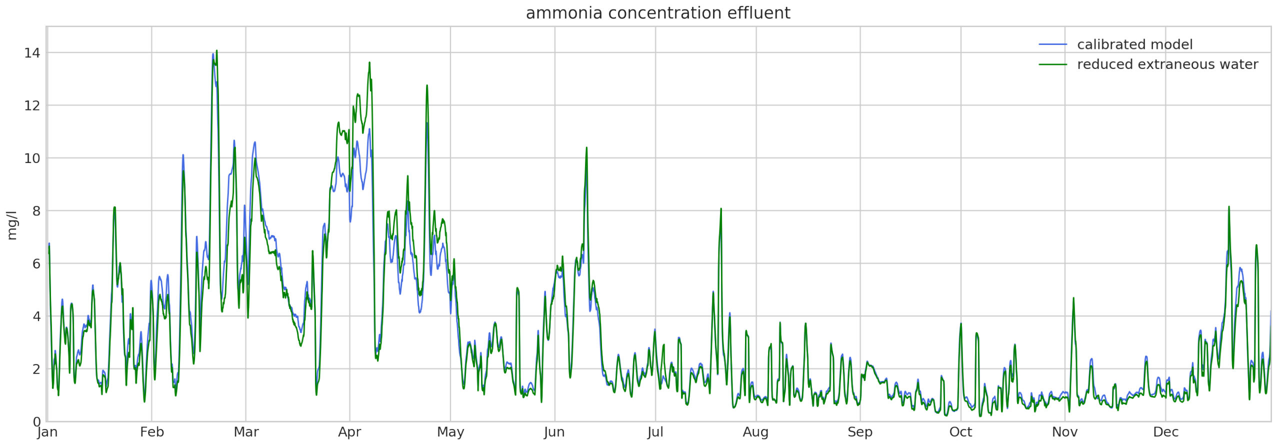


Figure 4.2.1: Ammonia concentrations in the effluent in the current situation and with reduced extraneous water

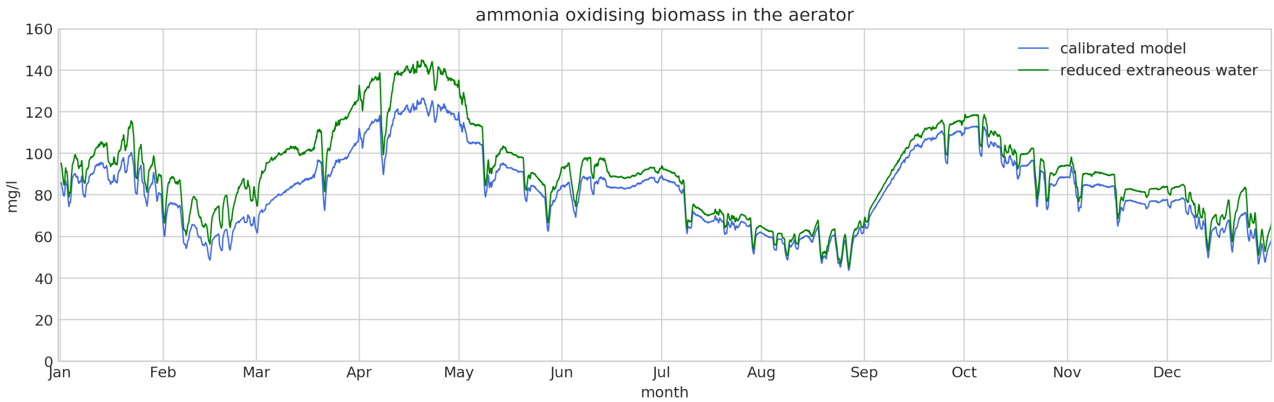


Figure 4.2.2: Ammonia oxidising biomass in the aeration tank.

As can be seen in figure 4.2.4, the NOx concentrations in the effluent become higher when the extraneous water is reduced; the denitrification rate did not increase along with the nitrification rate. The increase in recirculation of nitrate rich water from the end of the B stage back towards the start of the treatment process does allow for more nitrate to be converted. This can also be seen by the highest nitrate concentrations in the months March and April; here the recirculation is limited by the ammonia concentration, since the control allows for only one pump if the ammonia concentration is higher than 4mg/L.
The denitrification rate in the virtual tank could be limited by the available easily biodegradable COD, which is 16% less in the case with reduced extraneous water.

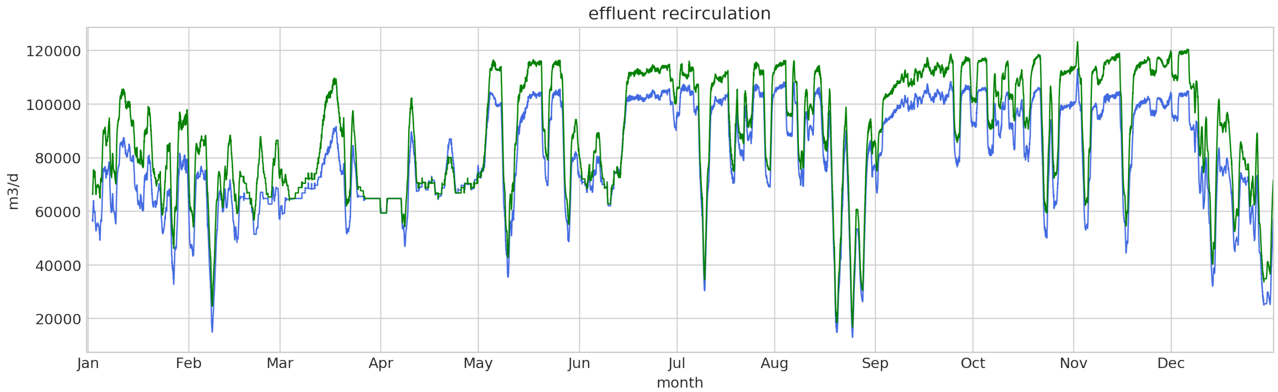


Figure 4.2.3: Effluent recirculation for the normal situation and when the extraneous water in reduced

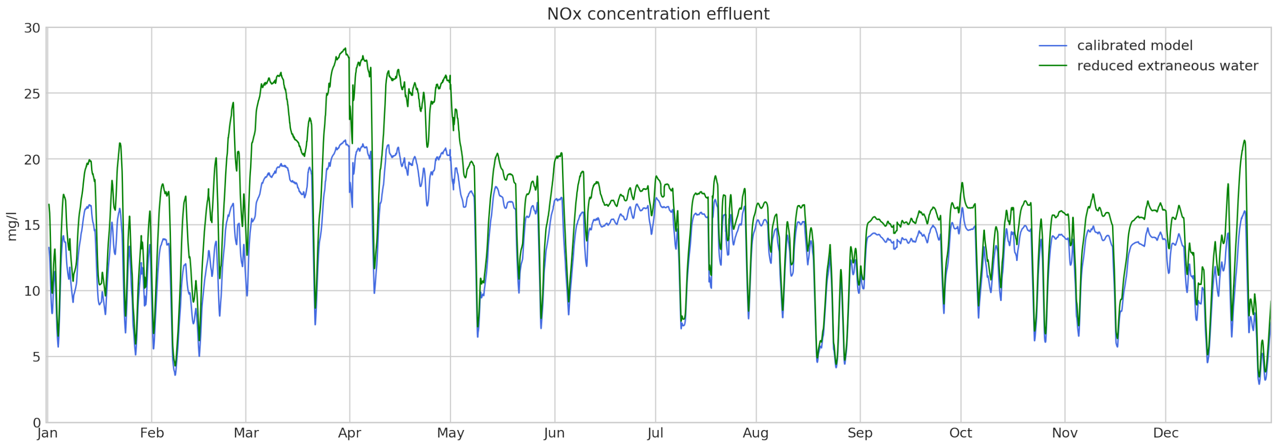


Figure 4.2.4: Nitrate concentrations in the effluent in the current situation and with reduced extraneous water

The increase in the nitrate concentrations result in an overall higher total nitrogen concentration in the effluent when the extraneous water is reduced (figure 4.2.5). On average the effluent concentration increased by 14%. Though the effluent concentration of the calibrated model were already higher than the measured values, due to the higher nitrate concentrations is becomes more difficult to reduce the total nitrogen in the effluent under the legal limit of 10mg/l.

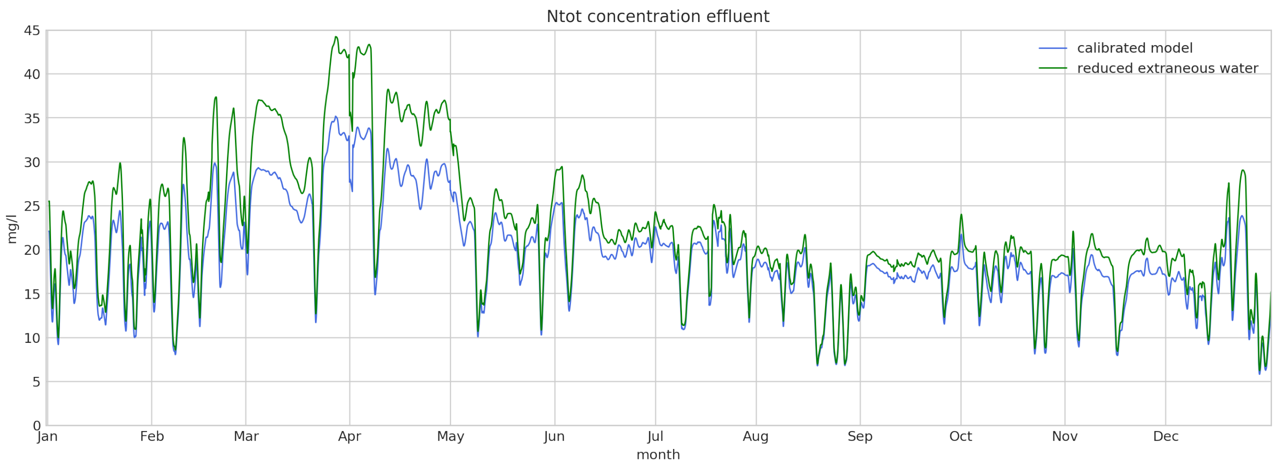


Figure 4.2.5: Total nitrogen in the effluent

The story becomes different when one looks at the pollution load on the receiving water. Figure 4.2.6 shows the resulting effluent load. Because the flow is reduced and the ammonia concentration in the effluent are similar, the effluent ammonia load is substantially decreased by 24%.
The nitrate concentrations in the effluent increased, but not as much as the flow decreased, resulting in a nitrate load on the effluent which is 13% lower. The total nitrogen load on the effluent decreases 16% on average. As these decreases in load come from the decrease in flow, the effects are mostly visible during dry weather, though the peaks from rainevents are also lower. The total nitrogen load is decreased by 13% during rain events; it is decreased 20% during dry weather.

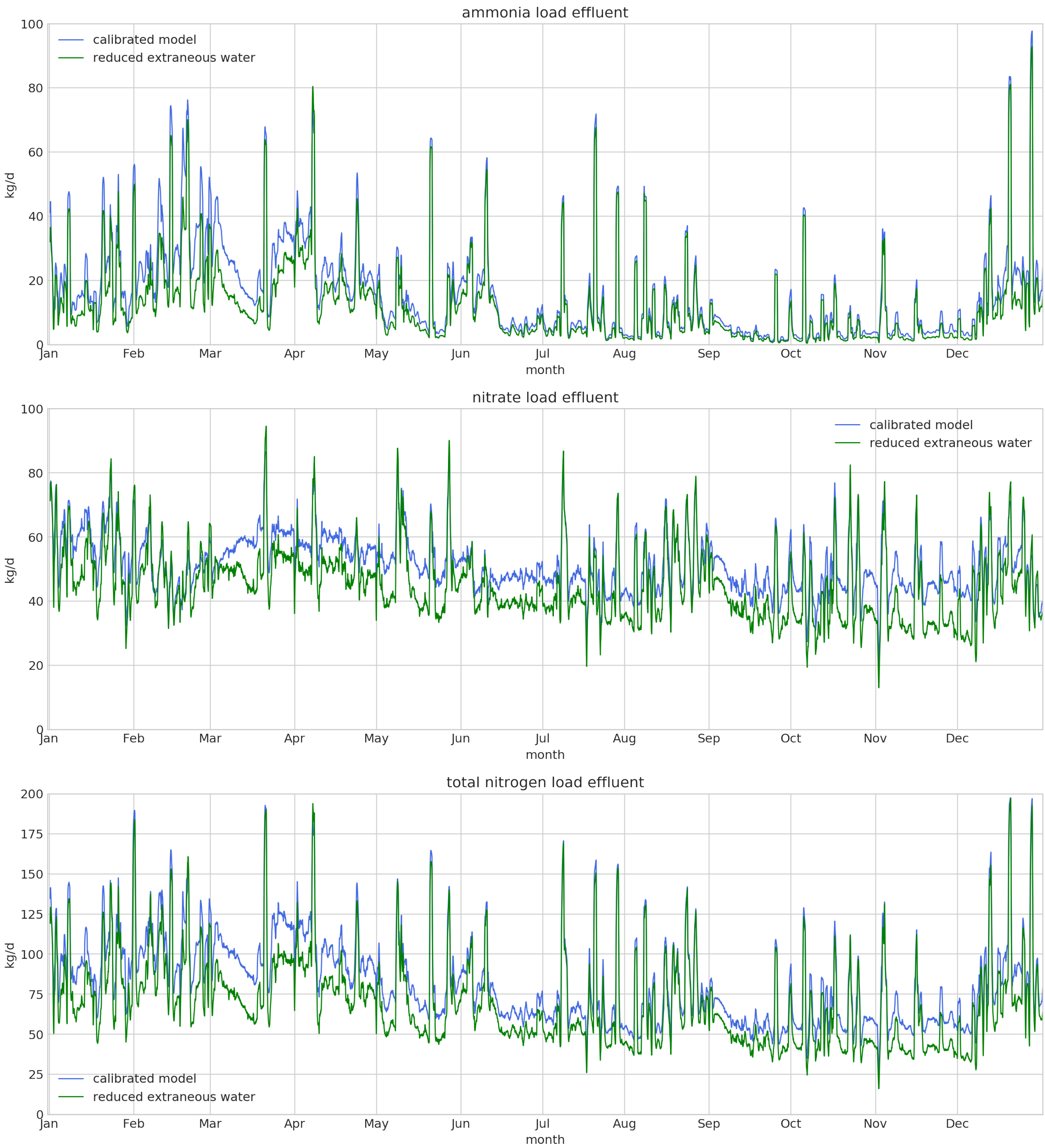


Figure 4.2.6: Effluent load Dokhaven models

In the model with reduced extraneous water the flows of the sludge streams — the underflow from the settler and the surplus sludge flows— are kept the same as in the calibrated model. This results in a small increase in the total amount of biomass present in the aeration tanks: 7 percent for the A stage and 4 percent for the B stage. This is linked to the increase in the ammonia oxidising biomass.

5

Discussion

COD

The influent model generates time series of the influent concentrations that simulate the dynamics during rain events. The resulting influent is used as input for the model of the connected treatment plant. This connection makes it possible to see the resulting dynamics in the workings treatment plant and the effects on the effluent. The two models used are relatively simple models that try to simulate complex processes. Obviously this has consequences on the accuracy on the results. Errors present in the first model will consequentially produce errors in the results of the BioWin model. The propagation of these errors was studied in the sensitivity analysis.

The results of the sensitivity analysis and the calculation of the error in the influent model show that the COD concentration in the influent is important to improve. The COD in the influent models has a RMSE of 20% of the dry weather concentration. The resulting effect of the error in the linked BioWin model depends on the characterisation of the influent. In the case of Willem Annapolder the biodegradable fraction is increased to increase the denitrification process in the anoxic reactor; an error in the influent COD thus influences the resulting nitrate concentrations. For the Dokhaven BioWin model the influent COD was characterised with a higher unbiodegradable fraction, which results in a model that is less sensitive the errors in the influent COD.
The influent COD concentrations can be modelled more accurately when more information is available that allows a better distinction between different rain events. In the case of Dokhaven, the COD concentrations during high flow are lower in the model than measured, in order to have better matching concentration during most medium events. Though Dokhaven is less sensitive to the difference in COD concentration in the influent, it does have an effect on the denitrification process which in the model eventually lead to high total nitrogen concentrations when the extraneous water is reduced.

Influent COD characterisation

The available biodegradable COD in the influent has an influence on many processes in the treatment plant. Because it is a sensitive unknown it is used in the calibration process of the BioWin models.
In the case of Willem Annapolder this resulted in an influent with a high fraction of slowly biodegradable COD for the denitrification proces in the anoxic tanks. During calibration it was found that this process was limited by the available easily biodegradable COD. Because all the fast degradable COD is consumed in the anaerobic tanks by the PAO's, the slowly degradable COD fraction is increased to have more COD available in the following anoxic tank.
In reality, it could be that this biodegradable COD does not come from the influent, but from the anaerobic digestion of the sludge. If the digestion has an incomplete hydrolysis, the reject water from this process — which is mixed with the influent — has a high acetate concentration.

In removing the extraneous water from the influent it is assumed here that the characterisation does not change. It is possible that the longer retention time could change the ratio of soluble over total biodegradable matter (Roeleveld and Loosdrecht 2002). This could thus further improve the denitrification process when extraneous water is removed.

Reject water

Both in the case of Dokhaven as Willem Annapolder the reject water from the sludge treatment are significant sources of pollutants. Unfortunately in the case of Willem Annapolder there are only limited measurements of its concentrations. More importantly, the sludge treatment at Willem Annapolder also treats the sludge from the nearby treatment plant Waarde, which is therefore a source of an unknown quantity of extra pollutants.

This could be the reasons for the measured high effluent concentrations during summer. In this period the aeration has trouble to achieve the desired levels of dissolved oxygen, resulting in high ammonia concentrations in the effluent. As the measured influent does not show the increase pollution load, the BioWin model also does not simulate these peaks in the ammonia concentration. It could be that the composition of the additionally treated sludge from Waarde changes due to linked industry or tourism, which then ends up as extra load on the Willem Annapolder treatment plant. More frequent measurements on the concentrations of the terrain-sewer would provide more insight into this matter.

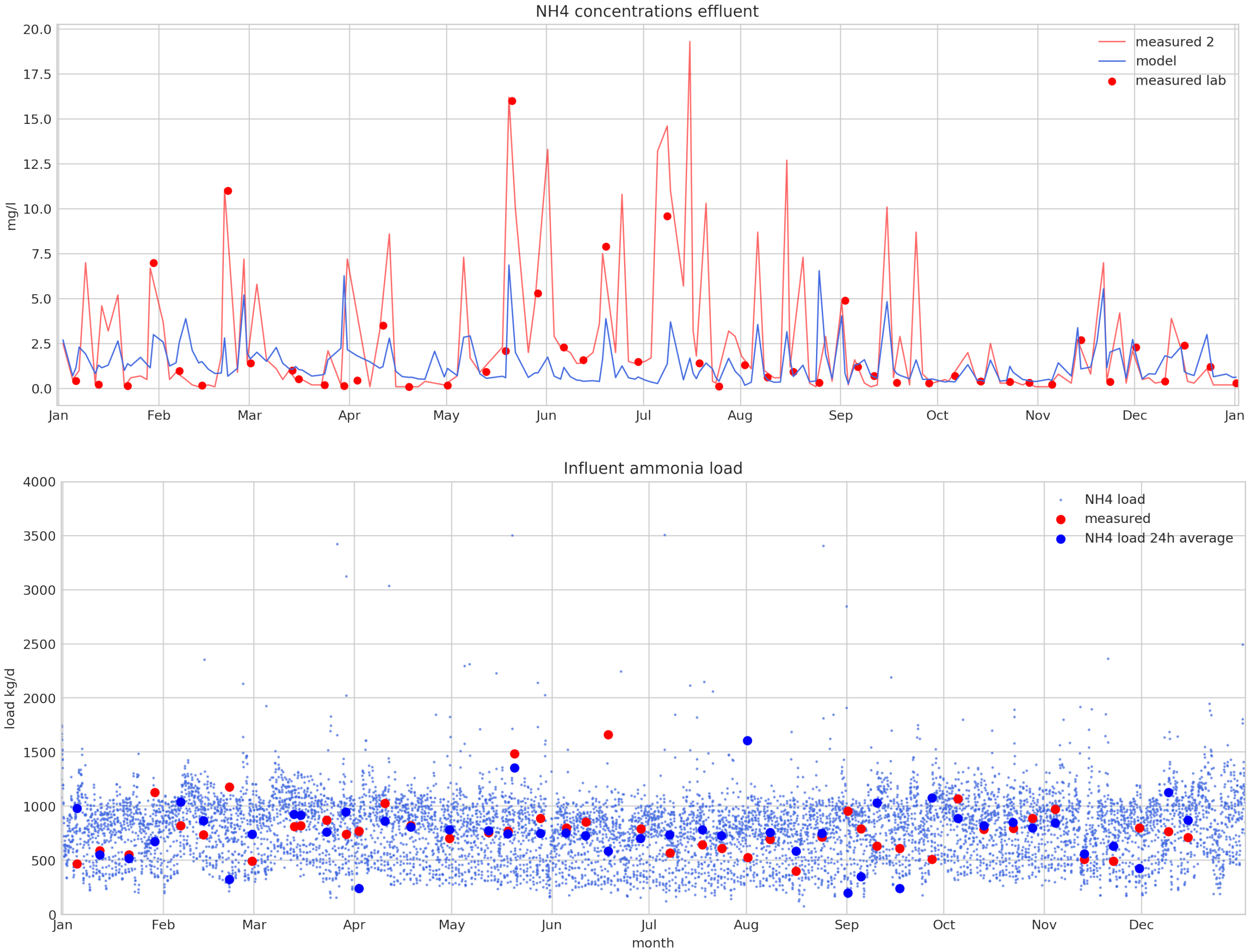


Figure 5.1: Not all the high peaks of ammonia in the effluent can be explained by the influent load, which is rather constant

Phosphate removal

The models of the treatment plants are calibrated on the biomass present in the bioreactors and the different effluent nitrogen concentrations, to simulate the nitrification and denitrification proces. Because of the limited scope of this thesis, the phosphor removal process was not studied.

In Dokhaven the phosphate is removed by chemical precipitation. In the aerator of the A stage a constant dose of iron salts is added to the activated sludge. These salts bind with the dissolved phosphate and are consequentially discharged through the waste sludge of the A stage. The coagulation process that binds the phosphate to the iron goes faster when the phosphate concentration increases. When the extraneous water is reduced one would thus expect a more efficient phosphate removal.

At Willem Annapolder the phosphate is removed biologically by phosphate accumulating bacteria (PAO's). In the anaerobic tanks these bacteria store energy by using the easily degradable COD in the influent while simultaneously releasing their stored phosphate. In the aeration tank the PAO's use this stored energy to grow and it is here that they take up the phosphate from the water. The phosphate is then discharged with the bacteria in the surplus sludge.
This process is very depended on the available easily degradable COD available in the anaerobic tank. It could be that, when the extraneous water is reduced, this fraction in the COD increases due to the longer residence time in the pressure mains. This could then increase phosphor removal process.

Concentration or pollution load?

The results for Dokhaven are comparable to the results found in earlier studies (Geraarts and Langeveld 2008) (Rödel et al 2017) (Dircks et al.2019): the effect on the effluent concentrations are limited, but the overall effluent load decreases. Wether or not it is useful to reduce the extraneous water in the influent depends on the criteria the treatment plant is judged on. At the moment this is done with the yearly average effluent concentrations that must be below a certain value depending on the substance. In this case an increase in concentration is unwanted. However, when the performance of the treatment plant is judged on the total amount of emitted pollutants to the environment — the pollution load— a reduction in the extraneous water is a useful measure. This also depends on the receiving water. Both Dokhaven and Willem Annapolder discharge their effluent on large rivers in which a high concentration of pollutant can easily dilute. If the treatment plant is connected to a small body of water, the effluent concentrations have a far greater effect on concentrations in the receiving water. In this case a higher load but with lower concentrations is preferred.

Pressure main

In this thesis two different treatment plants were modelled that are connected to two different types of sewer systems. Willem Annapolder receives its wastewater from long pressure mains, which have an effect on the dynamics of the incoming concentrations. In order to distinguish the effect of the pressure mains from the effect of the reduced extraneous water on the treatment plant, the BioWin model of Willem Annapolder is run with another influent. The results show that the high peaks of ammonia are caused by the pressure mains when extraneous water is reduced and is not an inherent effect of the BioWin model of Willem Annapolder.

Biomass concentration

What it also shows is that the increase in the dry weather effluent concentration of ammonia is not caused by the pressure mains, but by the plant being unable to nitrify the higher dry weather influent concentration. This is linked to the available bacteria in the bioreactors; the ammonia oxidising biomass at Willem Annapolder decreased in the model with reduced extraneous water. This is contrary to the model of Dokhaven, where the number of the same bacteria increased when the extraneous water was reduced.
The reduction in biomass at Willem Annapolder might be caused by an error in the control of the surplus sludge flow, which aims for a constant sludge retention time of 23 days. When the influent flow is reduced, it might also be possible to increase the sludge retention time, and thereby decreasing the dry weather effluent concentrations. By keeping the sludge retention time constant this effect was not taken into account in the results.
What is also not taken into account is the variation in growth parameters over the year. For both models the calibration was done over the whole year, which means the resulting parameters are fixed and chosen to be representative for both summer and winter. Though the temperature variation is taken into account, the dynamics of the biomass could be modelled more accurately if the model was split into a summer and winter period. This might also improve the representation of the effluent concentrations during summer at Willem Annapolder.

6 Conclusion

Extraneous water has an impact on many processes in the water cycle. This means that many factors have to be taken into account when one wants to reduce it. In the city it is closely related to the groundwater management; reducing the infiltration will increase the groundwater levels which can cause damage to buildings. On the other hand, when the wastewater treatment plant has to be extended to remove micro-pollutions the reduction in extraneous water leads to significantly lower investment costs.

This thesis studied the effects of a reduction of extraneous water in the influent on the performance of the wastewater treatment plant. The performance evaluated was the nitrogen removal capacity and the resulting effluent quality. Previous research has studied the general effects of less extraneous water at the influent; here the effects are studied on two specific case studies: the treatment plant Dokhaven in Rotterdam and Willem Annapolder in Kapelle.

Two models were made for each case study: an empirical model that simulates the concentrations of the influent and a BioWin model that simulates the biological processes of the linked treatment plant. Willem Annapolder treats the wastewater from multiple municipalities in its surrounding connected by long pressure mains. In this case the transport in the impact of the pressure main was also modelled. After the models of the treatment plant were calibrated for the year of the measurements, they were run again with a new influent in which the extraneous water was reduced.

For Dokhaven the results are similar to the previous studies: the reduction of extraneous water in the influent does not much change the effluent concentrations of ammonia and even increases the total nitrogen concentration in the effluent. However, due to the reduced influent and therefore effluent flow, the overall pollution load on the receiving water reduces significantly. On average, the total nitrogen concentration increased by 14%, while the effluent load decreased by 16%.

In the case of Willem Annapolder the results are different. This is mostly due to the effect of reducing extraneous water in the long pressure mains. The reduction of infiltration and inflow of clean water increases the dry weather concentration in the pipe. At the start of a storm event a plug volume with dry weather concentration is pushed to the treatment plant. A reduction in extraneous water thereby increases these peak pollution loads at the onset of each storm event. The measured effluent concentrations at Willem Annapolder show that the plant already has difficulties coping with these peaks in concentrations. The model shows that these influent peaks cause high peaks in the ammonia effluent concentration. On average the effluent concentration of ammonia for the situation with reduced extraneous water was 2.2 mg/l, which is a 63% increase from 1.4mg/l in the calibrated model. When the model was adjusted to commit the plug flow effect of the pressure mains, the resulting effluent concentration of ammonia was 1.5. This shows the importance of taking into account the characteristics of the sewer system when studying the effects of the reduction of extraneous water, which has not been done in previous studies.
Though the ammonia load on the effluent increases by 29%, the overall load of total nitrogen in the effluent load at Willem Annapolder still reduces by 9%. This brings forward the discussion how the performance of a treatment plant is evaluated the best: by effluent concentrations or overall load?

Similar to the previous studies done, this thesis shows that reducing extraneous water in the influent can be sightly beneficial to the effluent concentrations, but has an overall positive effect on the total nitrogen concentrations. However these effects are not general. When the wastewater has to be transported trough long pressure mains the effluent concentrations can also increase reducing the overall gain on the effluent load.

References

Besten, F.L.G., Noteboom, M. (2014). *Prestaties programma zuiveren 2014.* Retrieved from Waterschap Hollandse Delta, www.wshd.nl

Cierkens, K., Plano, S., Benedetti, L., Weijers, S., de Jonge, J., & Nopens, I. (2012). Impact of influent data frequency and model structure on the quality of WWTP model calibration and uncertainty. *Water Science and Technology*, *65*(2), 233-242.

Crouch, D. A (1995). *Case Study in Inflow and Infiltration Management*. In Seminar Publication National Conference on Sanitary Sewer Overflows (SS.Os) Washington D.C. April 1995
(p. 462).

De Bénédittis, J., & Bertrand-Krajewski, J. L. (2005). Infiltration in sewer systems: comparison of measurement methods. Water Science & Technology, 52(3), 219-227.

de Ville, N., Le, H. M., Schmidt, L., & Verbanck, M. A. (2017). Data-mining analysis of in-sewer infiltration patterns: seasonal characteristics of clear water seepage into Brussels main sewers. *Urban Water Journal*, *14*(10), 1090-1096.

Dimova, G., Ribarova, I., & de Carné, F. (2015). Coping with extraneous water in sewerage systems. In Understanding and Managing Urban Water in Transition (pp. 61-90). Springer Netherlands.

Dirckx, G., Fenu, A., Wambecq, T., Kroll, S., & Weemaes, M. (2019). Dilution of sewage: is it, after all, really worth the bother?. *Journal of Hydrology*.

Ekama, G. A., & Wentzel, M. C. (2004). Modelling inorganic material in activated sludge systems. *Water Sa*, *30*(2), 153-174.

Ekama, G. A., & Wentzel, M. C. (2008) Nitrogen removal. In Henze, M., van Loosdrecht, M. C., Ekama, G. A., & Brdjanovic, D. (Eds.). Biological wastewater treatment. (pp 87-138) IWA publishing, London

Ellis, J. B., Bertrand-Krajewski, J. L., Revitt, D. M., & Rieckermann, J. (2010). APUSS: Assessing the significance of infiltration and exfiltration on the performance of urban sewer systems. *Assessing Infiltration and Exfiltration on the Performance of Urban Sewer Systems (APUSS)*, 1.

Cui, F. (2012) Cold CANON: Annamox at low temperature, master thesis T.U.Delft, retrieved from: http://resolver.tudelft.nl/uuid:2bc15934-7e46-4d5e-98a6-d8f7cfdf8d9e

Geraarts, B., Langeveld, J. (2008). Het effect van afkoppelen van hemelwater op de RWZI. STOWA rapportage 2008-14

Handboek dimensionering: Ontwerpgrondslagen en dimensies rwzi Willem Annapolder, versie 1, Waterschap Zeeuwse Eilanden. October 2006

Henze, M., Harremoës, P., Jansen, J., Arvin, E., (1997). Wastewater treatment, Biological and chemical processes. Springer-Verlag Berlin Heidelberg

Karpf, C., & Krebs, P. (2011). Quantification of groundwater infiltration and surface water inflows in urban sewer networks based on a multiple model approach. *Water research*, *45*(10), 3129-3136.

Korving, H., Langeveld, J., van der Biezen, S., (2015). Reductie hydraulische belasting, Stichting RIONED / STOWA rapportage 2015-05

Langeveld, J. (2016). Aanpak rioolvreemd water. SAZ+ Samenwerking afvalwaterketen Zeeland. report by Partners4urbanwater, Nijmegen.

Langeveld, J., Van Daal, P., Schilperoort, R., Nopens, I., Flameling, T., & Weijers, S. (2017). Empirical sewer water quality model for generating influent data for WWTP modelling. *Water*, *9*(7), 491.

Liwarska-Bizukojc, E., & Biernacki, R. (2010). Identification of the most sensitive parameters in the activated sludge model implemented in BioWin software. *Bioresource technology*, *101*(19), 7278-7285.

Liwarska-Bizukojc, E., Olejnik, D., Biernacki, R., & Ledakowicz, S. (2011). Calibration of a complex activated sludge model for the full-scale wastewater treatment plant. *Bioprocess and biosystems engineering*, *34*(6), 659-670.

Meijer, H. A. (1988). Rotterdam-Dokhaven wastewater treatment plant; a large wastewater treatment plant in the midst of a developing residential quarter. *Water Science and Technology*, *20*(4-5), 267-274.

Meijer, S. C. F., Van Loosdrecht, M. C. M., & Heijnen, J. J. (2001). Metabolic modelling of full-scale biological nitrogen and phosphorus removing WWTP's. *Water Research*, *35*(11), 2711-2723.

Melcer, H. (2004). Methods for wastewater characterization in activated sludge modelling. IWA publishing.

Mels, A.R., van Nieuwenhuijzen, A.F., van der Graaf, J.H.J.M., Klapwijk, A., de Koning, J., Rulkens, W.H. (2001) Fysisch/chemische voorzuivering van afvalwater. STOWA rapportage 2001-20

Roeleveld, P. J., & Van Loosdrecht, M. C. M. (2002). Experience with guidelines for wastewater characterisation in The Netherlands. *Water Science and Technology*, *45*(6), 77-87.

Rödel, S., Günthert, F. W., & Brüggemann, T. (2017). Investigating the impacts of extraneous water on wastewater treatment plants. *Water Science and Technology*, *75*(4), 847-855.

Ruano, M. V., Ribes, J., De Pauw, D. J. W., & Sin, G. (2007). Parameter subset selection for the dynamic calibration of activated sludge models (ASMs): experience versus systems analysis. *Water Science and Technology*, *56*(8), 107-115.

Rutsch, M., Müller, I., & Krebs, P. (2005). Dynamics of rain-induced pollutographs of solubles in sewers. *Water science and technology*, *52*(5), 169-177.

Programma SAZ+ 2016-2020 Samenwerking (afval)Waterketen Zeeland, "Naar één Zeeuws Plan (Afval)waterketen, Samen op weg naar een toekomstbestendige waterhuishouding"

Schilperoort, R. P. S. (2004). Natural water isotopes for the quantification of infiltration and inflow in sewer systems. *Delft University of Technology*.

Schilperoort, R. P. S., Dirksen, J., Langeveld, J. G., & Clemens, F. H. L. R. (2012). Assessing characteristic time and space scales of in-sewer processes by analysis of one year of continuous in-sewer monitoring data. *Water Science and Technology*, *66*(8), 1614-1620.

Staufer, P., Scheidegger, A., & Rieckermann, J. (2012). Assessing the performance of sewer rehabilitation on the reduction of infiltration and inflow. *Water research*, *46*(16), 5185-5196.

van Daal-Rombouts, P. M. M. (2017). Performance evaluation of real time control in urban wastewater systems. PhD thesis. https://doi.org/10.4233/uuid:b1d5d733-b271-474f-ad32-b5fdde257161

van der Vlies, A. W. (1987). Besluitvorming en organisatie van het project awzi Dokhaven, *H2O*, 20(23), 564-578

van Opijnen, J. (2017). Dynamische modellering RWZI Dokhaven, *report from Royal HaskoningDHV*

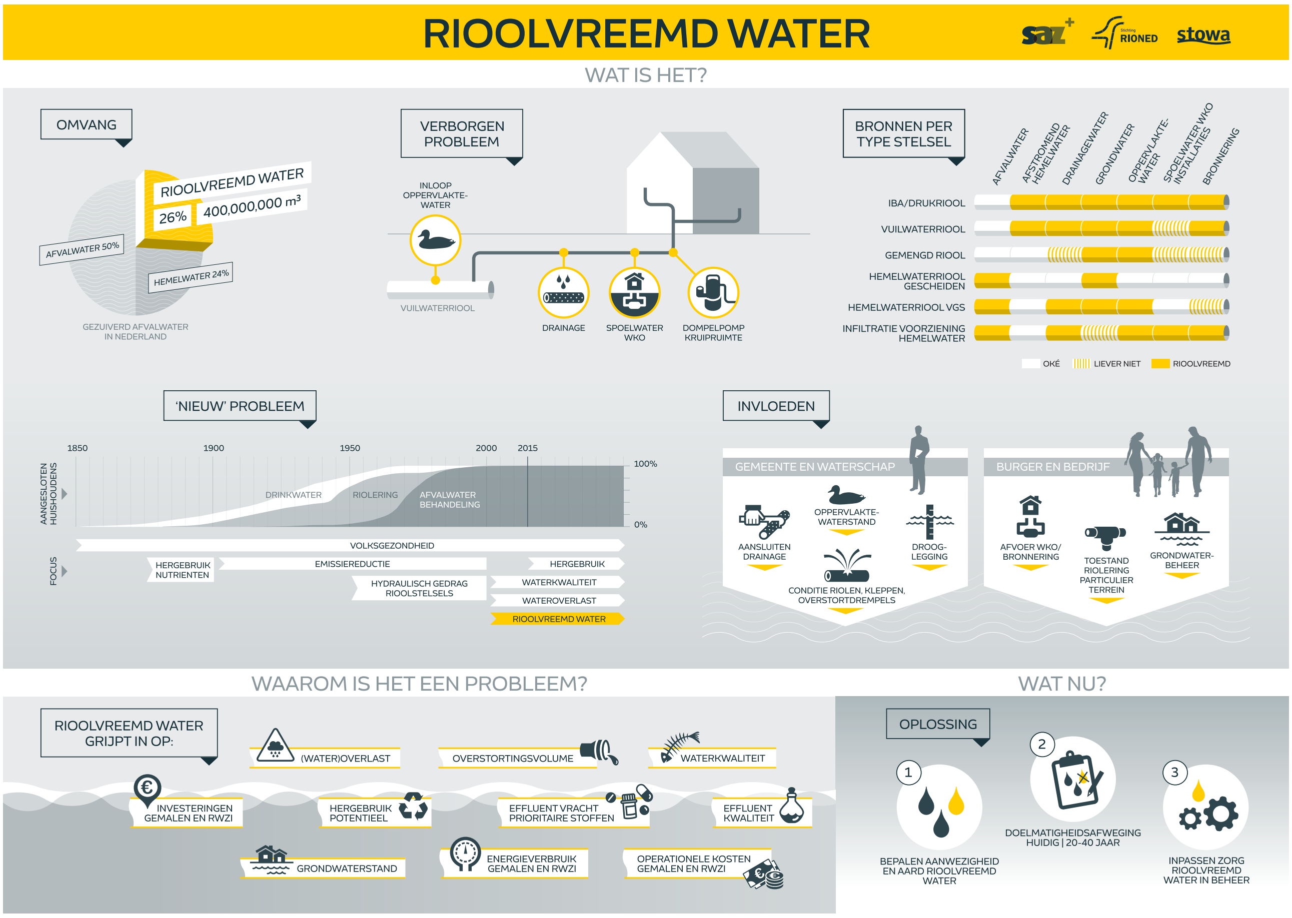
Vosse, M., (2013). Rioolvreemd water rioolwaterzuiveringsinrichting Dokhaven, *report from Royal HaskoningDHV*

Weiss, G., Brombach, H., & Haller, B. (2002). Infiltration and inflow in combined sewer systems: long-term analysis. *Water Science and technology*, *45*(7), 11-19.

Winkler, M. K., Bassin, J. P., Kleerebezem, R., Sorokin, D. Y., & van Loosdrecht, M. C. (2012). Unravelling the reasons for disproportion in the ratio of AOB and NOB in aerobic granular sludge. *Applied microbiology and biotechnology*, *94*(6), 1657-1666.

Wittenberg, H., & Aksoy, H. (2010). Groundwater intrusion into leaky sewer systems. *Water Science & Technology*, *62*(1).

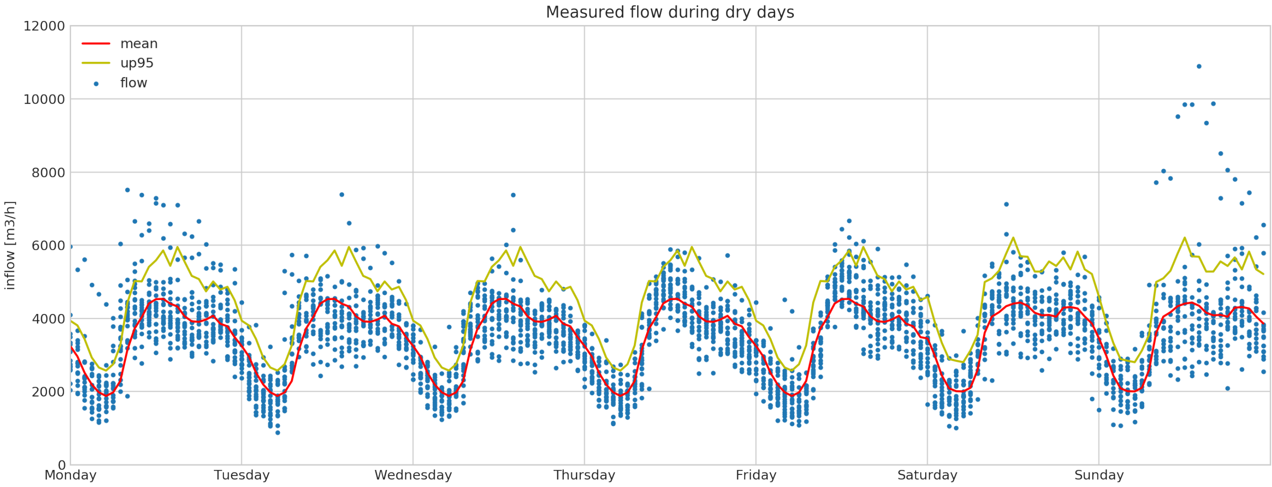
Appendix1.



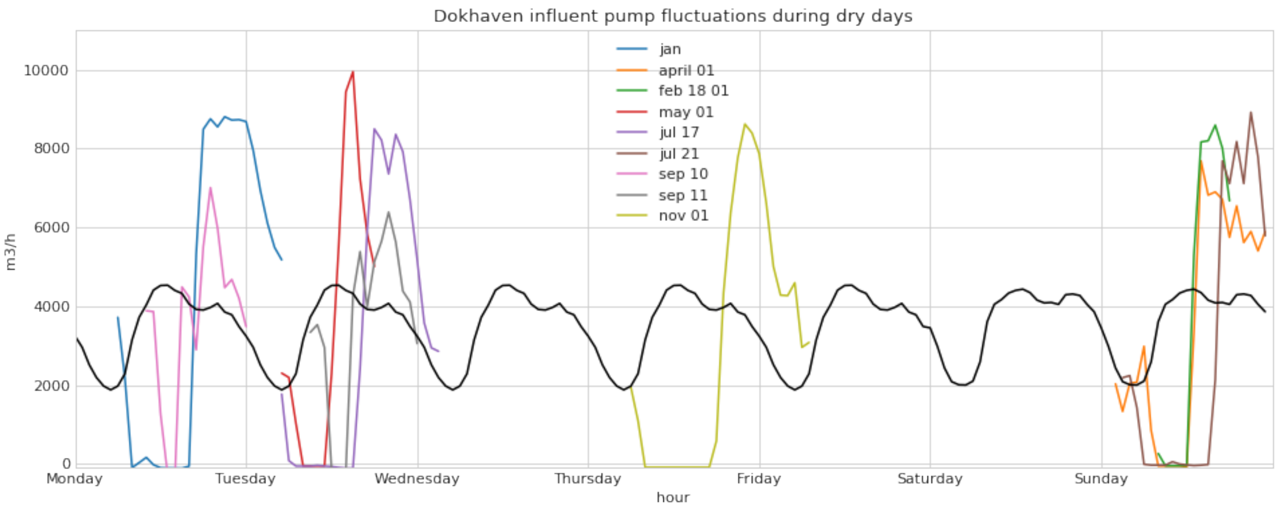
Infographic rioolvreemd water (extraneous water). retrieved from http://www.samenwerkenaanwater.nl/media/vh4260/rioolvreemdwater_infographic.pdf 22-03-2019

Appendix 2

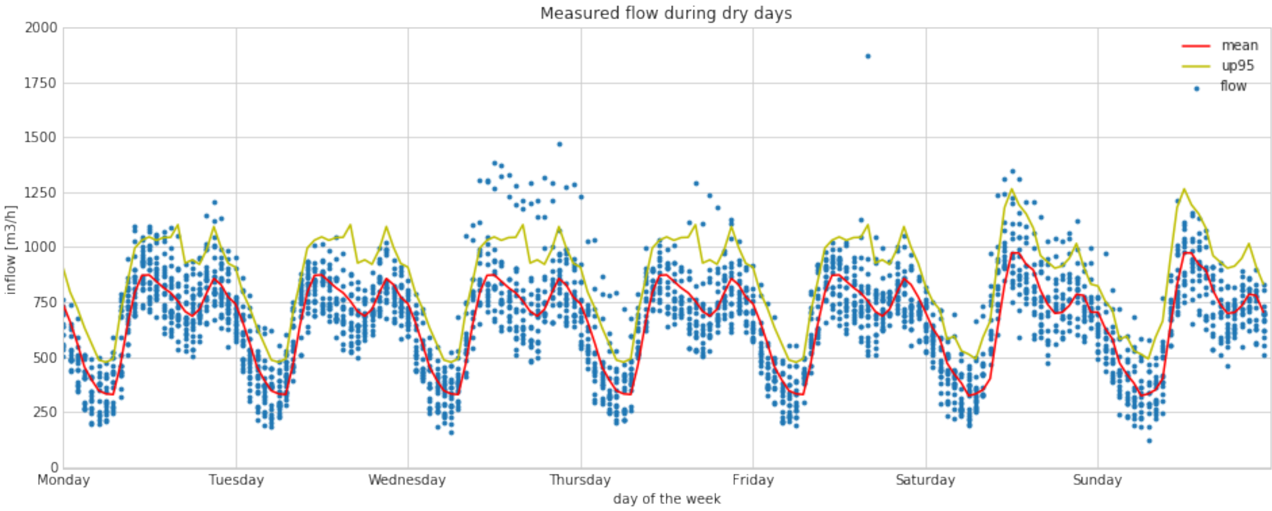
DRY WEATHER FLOW DOKHAVEN



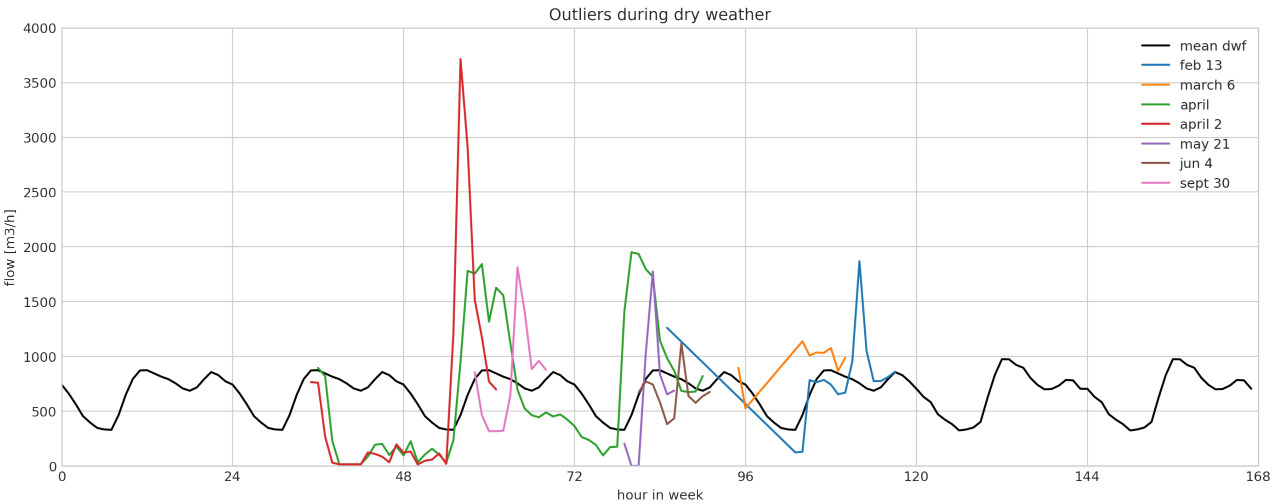
IRREGULAR PUMPING ACTIONS DURING DRY DAYS DOKHAVEN



DRY WEATHER FLOW WILLEM ANNAPOLDER

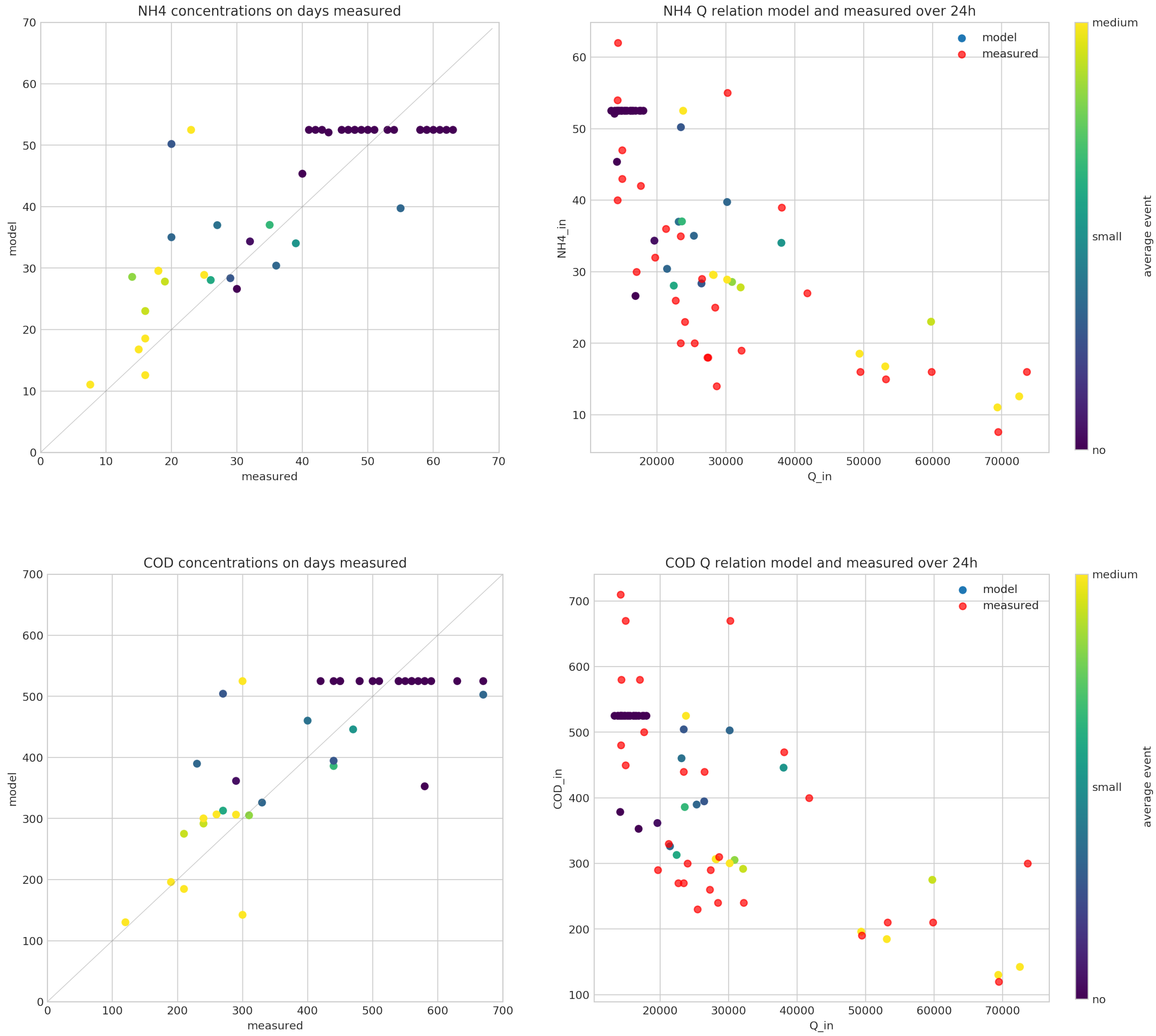


IRREGULAR PUMPING ACTIONS DURING DRY DAYS WILLEM ANNAPOLDER

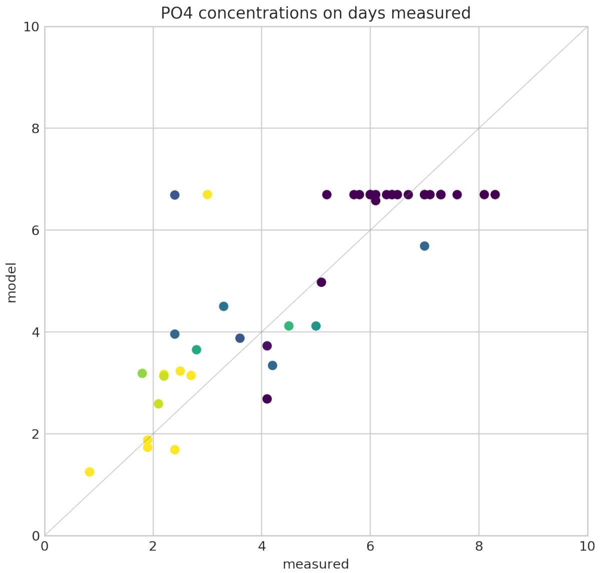


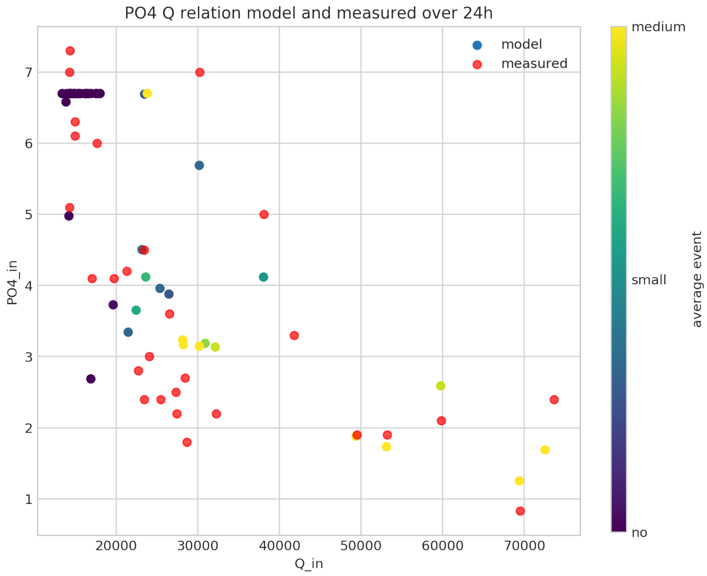
APPENDIX 3

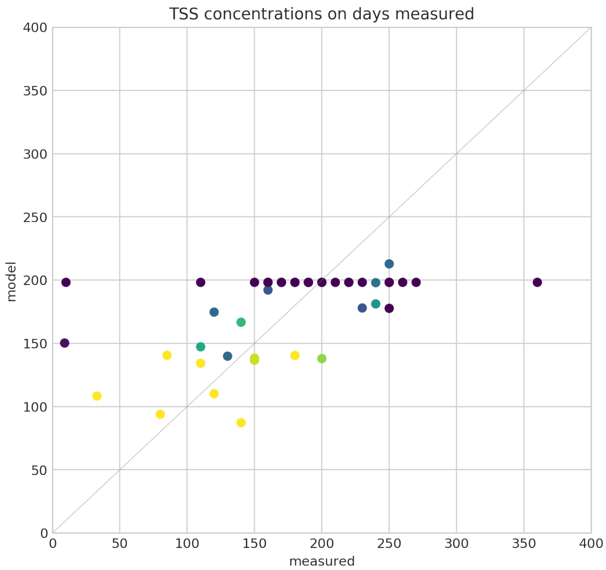
CALIBRATED INFLUENT MODEL CONCENTRATIONS WILLEM ANNAPOLDER



Calibrated parameters for the influent model. The first table shows the calibrated parameter values and overall error by van Daal-Rombouts. These values can be compared to the resulting parameter values and errors for Willem Annapolder and Dokhaven.

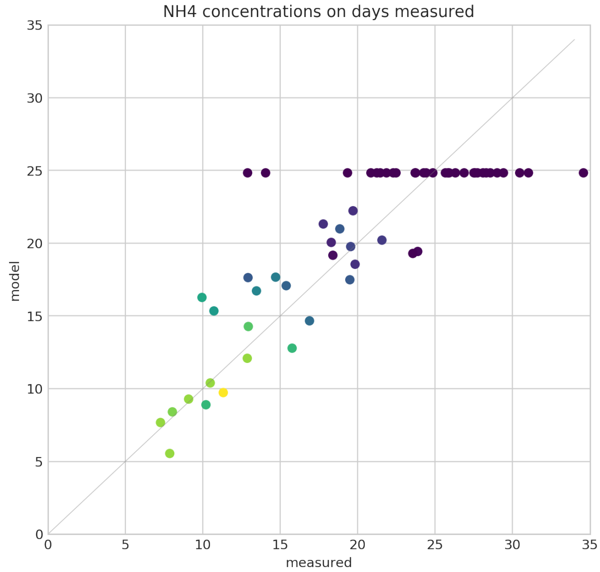

PO4 concentrations on days measured
model
measured

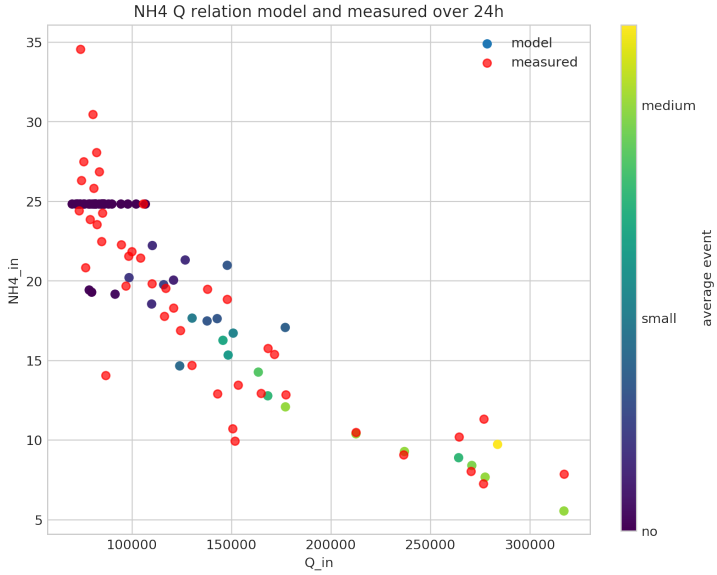

PO4 Q relation model and measured over 24h
model
measured
PO4_in
Q_in
medium
small
no
average event

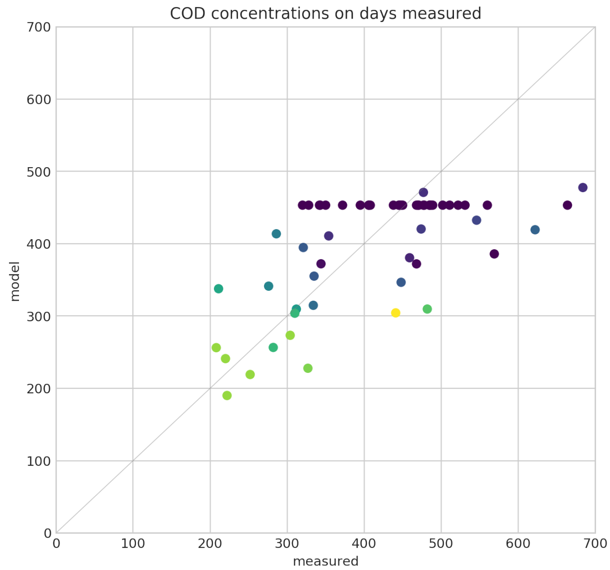

TSS concentrations on days measured
model
measured

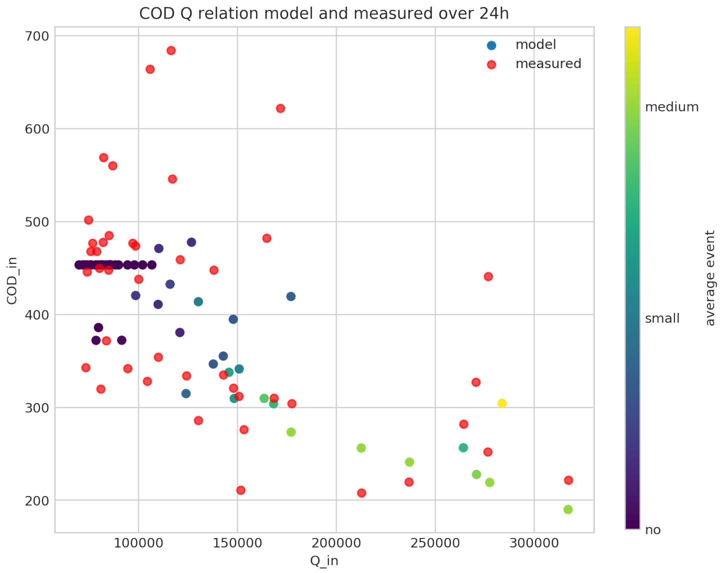

TSS Q relation model and measured over 24h
model
measured
TSS_in
Q_in
medium
small
no
average event

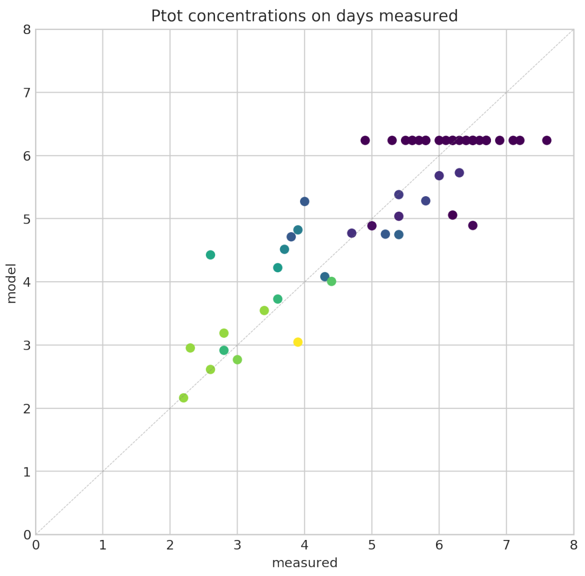
CALIBRATED INFLUENT MODEL DOKHAVEN:



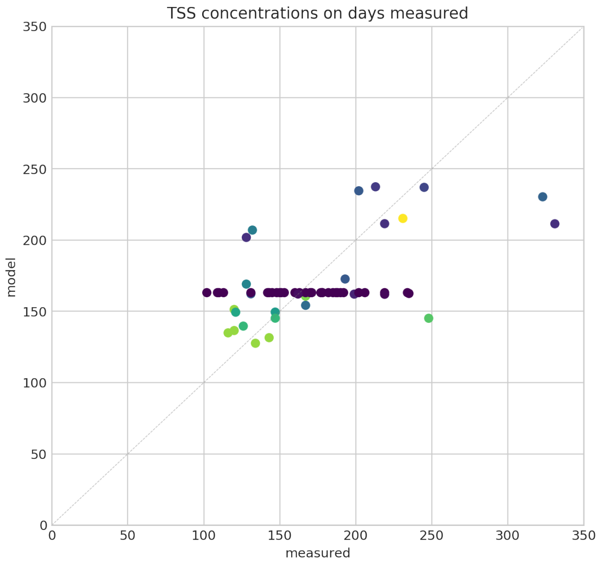


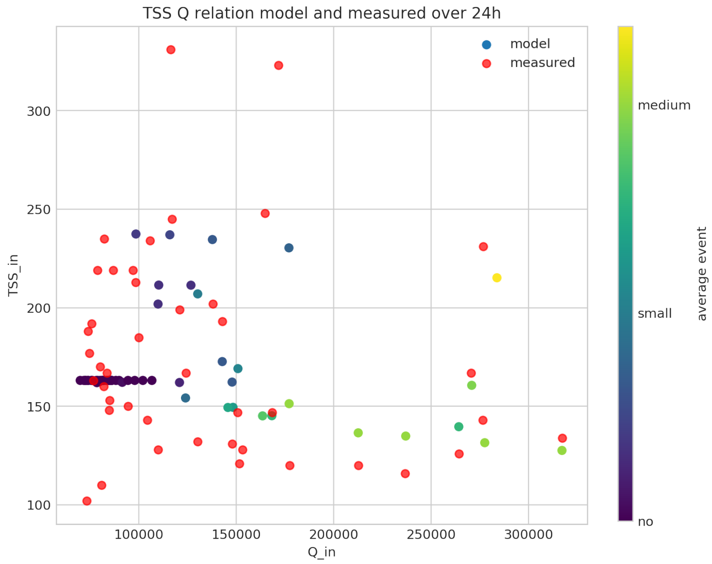





Ptot concentrations on days measured
model
measured


Ptot Q relation model and measured over 24h
model
measured
Ptot_in
Q_in
average event
medium
small
no


TSS concentrations on days measured
model
measured


TSS Q relation model and measured over 24h
model
measured
TSS_in
Q_in
average event
medium
small
no

Appendix 4

parameter values van Daal-Rombouts

model parameter	abbr	unit	NH4 model	COD model
dilution factor, L	a1,L	-	0.95	0.63
dilution delay time, L	a2,L	hour	2.05	5.7
dilution factor, M	a1,M	-	0.82	0.47
dilution delay time, M	a2, M	hour	1.92	9.82
recovery factor, M+L	a3	mg/(l*h)	0.9	0.504
peak first flush concentration	a4	mg/L	n.a.	48
recovery factor first flush	a5	mg/(l*h)	n.a.	68.4
recovery factor, S	a6	mg/(l*h)	2.12	0.06
RMSE / DWF_mean * 100%			**16%**	**18%**

parameter values Willem Annapolder

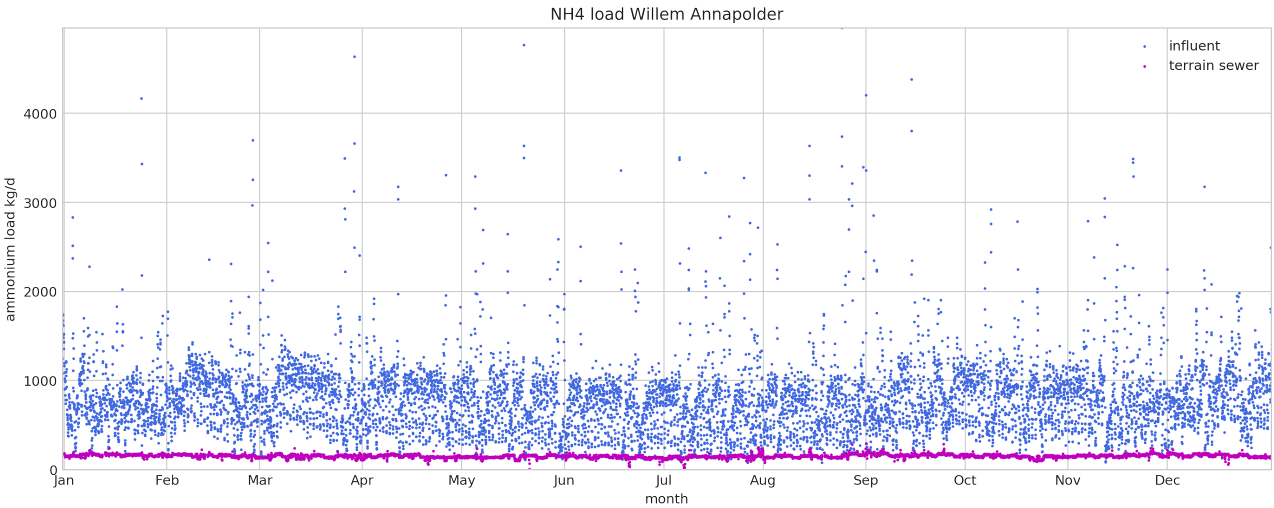
model parameter	abbr	unit	NH4	COD	PO4	TSS
dilution factor, L	a1,L	-	1.05	1	0.95	0.7
dilution delay time, L	a2,L	hour	2	8	3	11
dilution factor, M	a1,M	-	1.05	1	0.95	0.7
dilution delay time, M	a2, M	hour	2	8	3	11
recovery factor, M+L	a3	mg/(l*h)	0.01	0.01	0.025	0.05
recovery factor, S	a6	mg/(l*h)	0.05	0.045	0.04	0.035
length dilution time, S		hour	13	13	13	13
peak first flush concentration	a4	mg/(l*h)	n.a.	500	n.a.	250
recovery factor first flush	a5	mg/(l*h)	n.a.	75	n.a.	45
RMSE/DWF mean * 100%			**17.2%**	**20.0%**	**17.2%**	**26.2%**

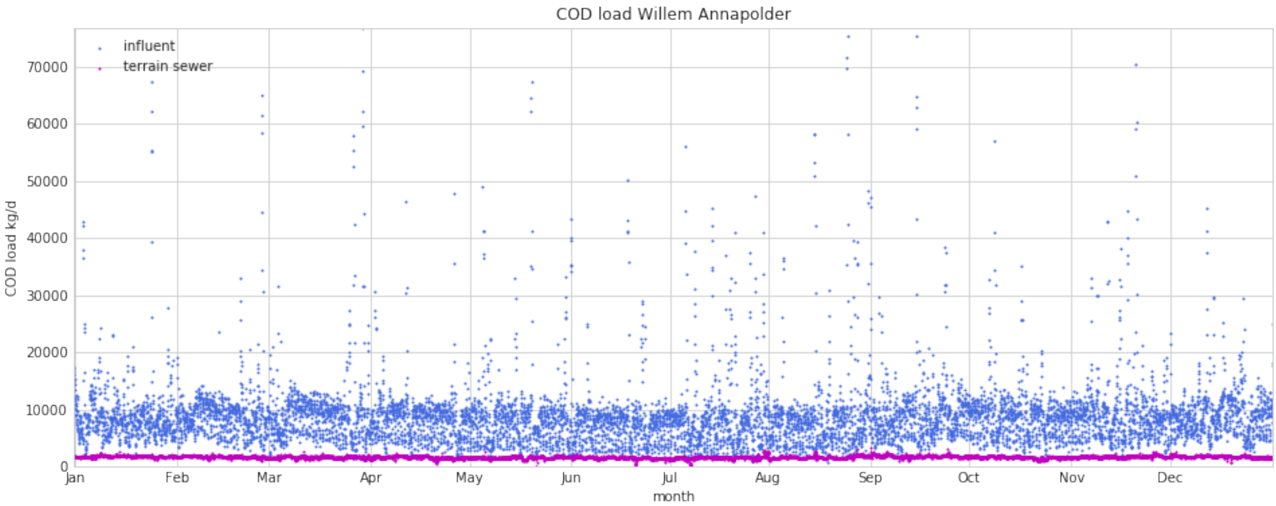
parameter values Dokhaven

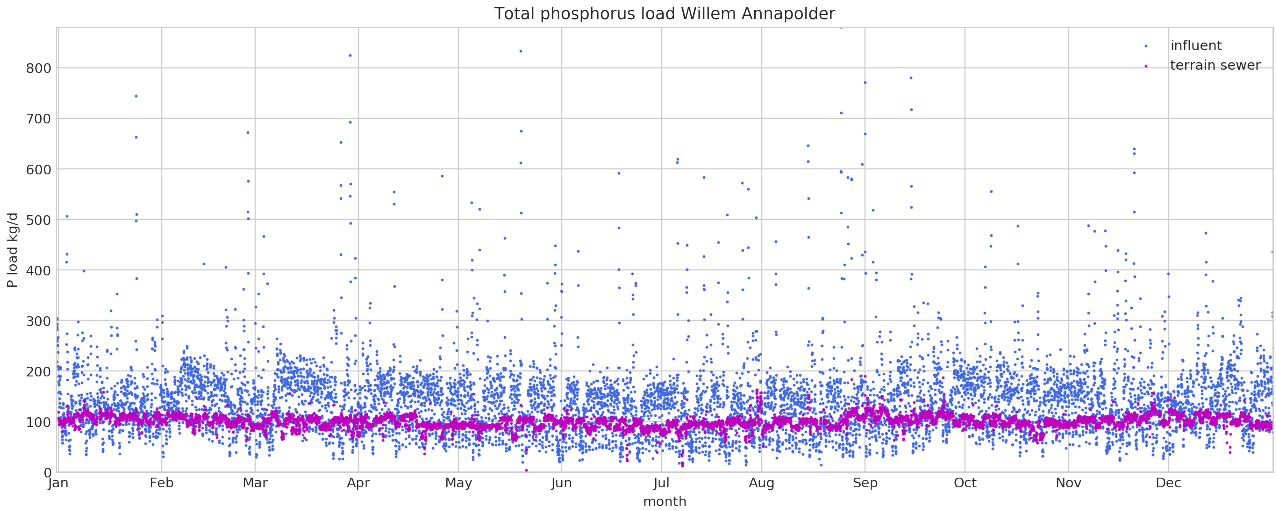
model parameter	abbr	unit	NH4	COD	PO4	TSS
dilution factor, L	a1,L	-	1.07	0.7	0.9	0.3
dilution delay time, L	a2,L	hour	1.9	5	2	5
dilution factor, M	a1,M	-	1.07	0.7	0.9	0.3
dilution delay time, M	a2, M	hour	1.9	5	2	5
recovery factor, M+L	a3	mg/(l*h)	0.03	0.5	0.025	0.08
recovery factor, S	a6	mg/(l*h)	0.03	0.025	0.03	0.001
length dilution time, S		hour	13	13	13	13
peak first flush concentration	a4	mg/L	n.a.	550	n.a.	370
recovery factor first flush	a5	mg/(l*h)	n.a.	70	n.a.	40
RMSE/DWF mean *100%			**15.2%**	**18.5%**	**10.5%**	**24.6%**

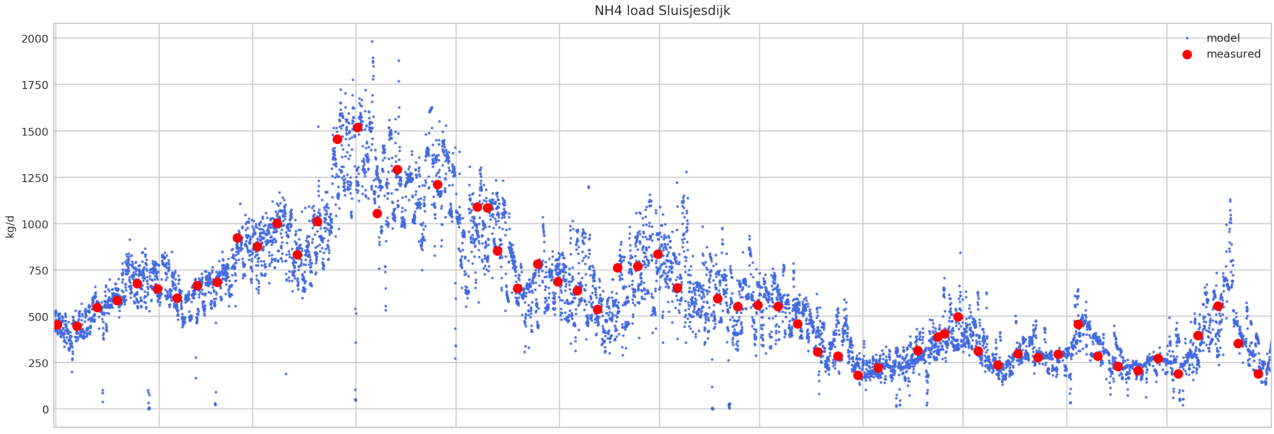
Appendix 3

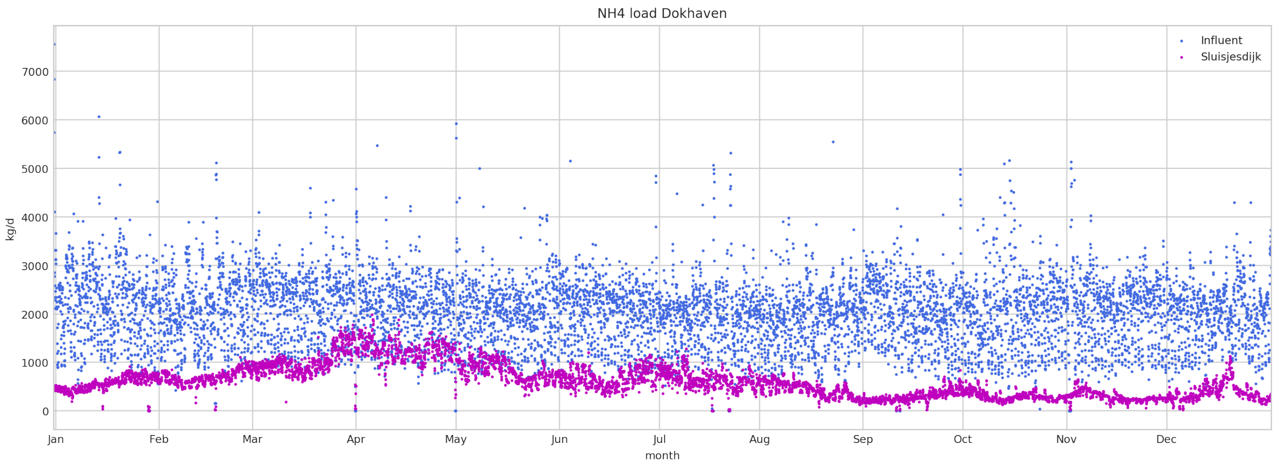
POLLUTANT LOAD INFLUENT AND SLUDGE TREATMENT:

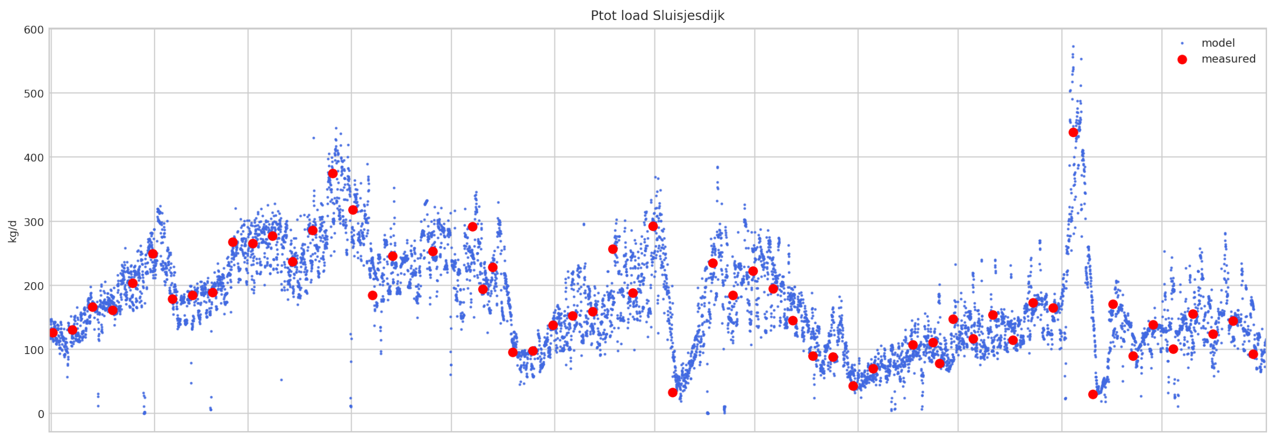


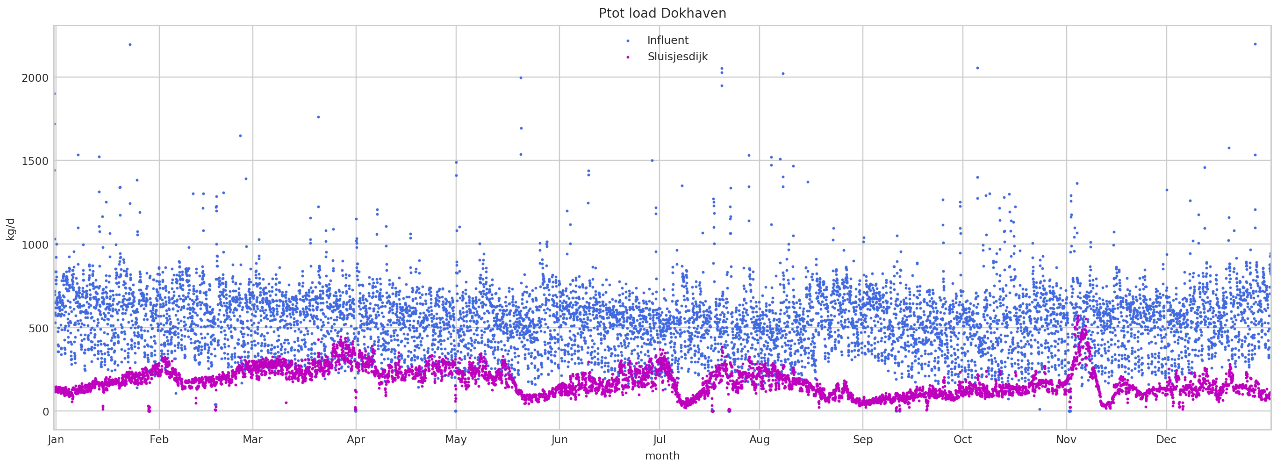





NH4 load Sluisjesdijk
model
measured
kg/d
2000
1750
1500
1250
1000
750
500
250
0


NH4 load Dokhaven
Influent
Sluisjesdijk
kg/d
7000
6000
5000
4000
3000
2000
1000
0
Jan Feb Mar Apr May Jun Jul Aug Sep Oct Nov Dec
month


Ptot load Sluisjesdijk
model
measured
kg/d
600
500
400
300
200
100
0


Ptot load Dokhaven
Influent
Sluisjesdijk
kg/d
2000
1500
1000
500
0
Jan Feb Mar Apr May Jun Jul Aug Sep Oct Nov Dec
month

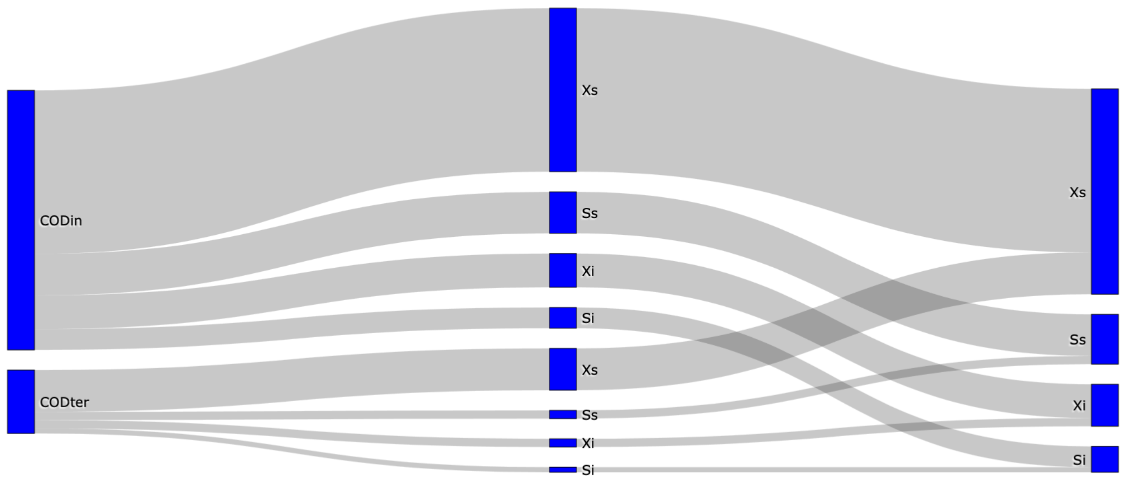
Appendix 4

RESULTING INFLUENT FRACTIONS:

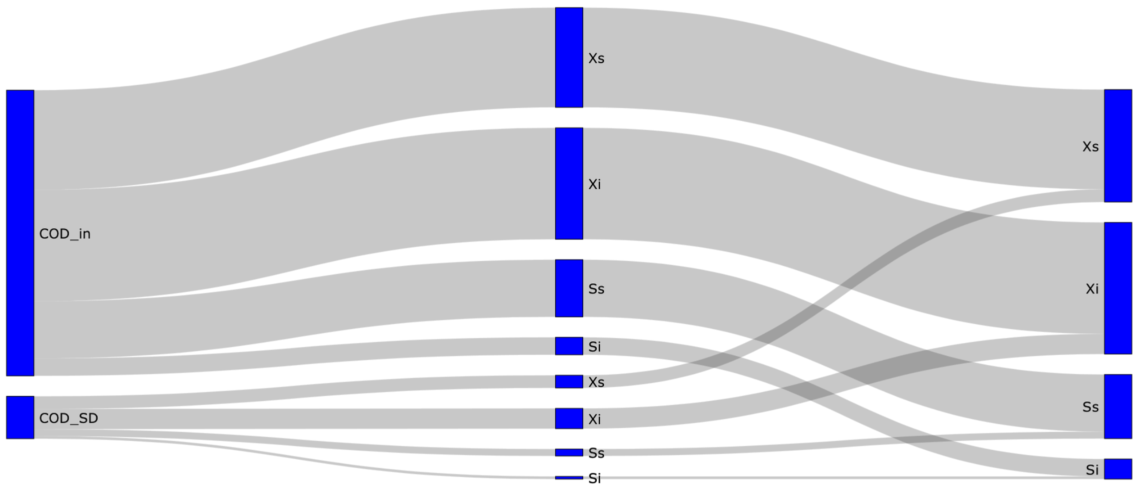
Fractioning of influent and sludge treatment effluent

BioWin Fractions	information	related ASM parameter	Default (BioWin)	WAP influent	WAP TS	DOK influent	DOK SD	Roeleveld Loos-drecht 2002
Fbs	readily biodegradable [g COD / g tot COD]	S_S	0.16	0.2	0.11	0.2	0.163	0.26
Fac	acetate [g COD / g readily biodegradable]	S_A	0.15	0.15	0.15	0.15	0.15	
Fxsp	non-colloidal slowly biodegradable [g COD / g slowly degradable COD]	X_S	0.75	0.5	0.75	0.75	0.75	
Fus	unbiodegradable soluble [g COD / g tot COD]	S_I	0.05	0.08	0.08	0.061	0.061	0.06
Fup	unbiodegradable particulate [g COD / g tot COD]	X_I	0.13	0.13		0.39	0.477	0.39
Fna	ammonia [g NH3-N / g TKN]	S_{NH4}	0.66	0.76		0.53	0.66	
Fnox	particulate organic nitrogen [g N / g organic N]		0.5	0.5		0.5	0.5	
Fnus	soluble unbiodegradable TKN [g N / g TKN]		0.02	0.02		0.02	0.02	
FupN	N:COD ratio for unbiodegradable particulate COD [g N / g COD]		0.035	0.035		0.007	0.035	
Fpo4	phosphate [g PO4-P / g TP]		0.5	0.71		0.57	0.61	
FupP	P:COD ratio for unbiodegradable particulate COD [g P / g COD]		0.011	0.011		0.01	0.035	
FZbh	biomass heterotrophic organisms	X_H	0.02	0.02		0.02	0.02	

COD fractioning Willem Annapolder



COD fractioning Dokhaven



Appendix 5

Dimensions Dokhaven model

Tank		value	unit
denetrification	volume	10000	m³
	depth	4	m
	width	25	m
	waste sludge	0.71	m³/d
grit tank	area	392	m2
	depth	4.33	m
	width	3.5	m
	capture of ISS	9.7	%
	underflow	0.714	m³/d
A stage aerator	area	1108.8	m2
	depth	4.32	m
	width	3.5	m
	DO setpoint	0.8	mg/L
	max airflow rate	13584	m³/hr
	density	10	%
A stage settling tank	area	6340.4	m2
	depth	2.6	
B stage point aerator 12	area	1479.7	m2
	depth	4	m
	width	27.2	m
	power	360	kW
B stage point aerators 34	area	1479.7	m2
	depth	4	m
	width	27.2	m
	DO setpoint	controlled	
B stage settling tank	area	11430	m2
	depth	2.5	m

Dimensions Willem Annapolder

Tank		value	unit
Primary settler	area	1660	m²
	depth	1.5	m
	diameter	46	m
	waste sludge		m3/d
anaerobic tank			
1	volume	80	m³
2-5	volume	655	m³
total	volume	2700	m³
	depth	6	m
	width	11.95	m
Anoxic	volume	3720	m³
	depth	6	m
	width	6.5	m
Aerobic/ Anoxic	volume	5580	m³
	depth	6	
	width	6.5	
Secondary settler 1	area	1135	m²
	depth	1.5	m
Secondary settler 2	area	1135	m²
	depth	1.5	m
Secondary settler 3	area	1135	m²
	depth	2.5	m
Secondary settler 4	area	1948	m²
	depth	2.5	m

RESULTING BIOWIN PARAMETERS:

Kinetic parameters

Parameter	Description	WAP	DOK	BioWin Default	ASM1 default	unit
$b_{A,NOB}$	decay rate NOB	0.15	0.13	0.17	0.15	1/d
$b_{A,AOB}$	decay rate AOB	0.15	0.13	0.17	0.15	1/d
$\mu_{A,AOB}$	maximum specific growth rate AOB		1.0	0.9		1/d
$\mu_{A,NOB}$	maximum specific growth rate NOB	0.9	1.0	0.7		1/d
r_{NO3}	reduction factor anoxic hydrolysis	0.6		0.28	0.4	-
r_g	reduction factor anoxic growth heterotrophic biomass	0.8		0.5	0.8	-
K_{OH}	DO half saturation coefficient heterotrophic biomass	0.1		0.05	0.1	mg O2/l
$K_{NOx, NOB}$	DO half saturation NOB	0.3	0.2	0.5		mg O2/l
K_{NO3}	anaerobic/ anoxic NOx half saturation coefficient	0.1		0.15	0.25	mg N/l