Interactions within wastewater systems

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Proefschrift

ter verkrijging van de graad van doctor
aan de Technische Universiteit Delft,
op gezag van de Rector Magnificus prof.dr.ir. J.T. Fokkema,
voorzitter van het College voor Promoties,
in het openbaar te verdedigen op vrijdag 10 september 2004 om 10.30 uur
door Jeroen Gerardus LANGEVELD
civiel ingenieur
geboren te Delft.

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Dit proefschrift is tot stand gekomen met ondersteuning van:
Arcadis i.s.m. HKV Lijn in water en Vertis
Grontmij
Witteveen+Bos

Printed: OPTIMA Grafische Communicatie – Rotterdam

ISBN 90-77595-72-4

Keywords: *interactions, sewer system, wastewater treatment, wastewater system*

Printed in the Netherlands

to M.A.M. Langeveld - van der Valk
to Letty and Silke

Voorwoord

Allereerst wil ik mijn beide promotoren, François Clemens en Jaap van der Graaf bedanken. De afgelopen jaren waren geweldig en ik vond samenwerken met jullie erg inspirerend. De manier waarop wij in dit onderzoek van eerste idee tot proefschrift als team hebben gefunctioneerd is onvergetelijk. Doorgaans geldt dat twee kapiteins op een schip er minstens één te veel is. Mij gaf dit juist de ruimte en het vertrouwen om het gehele onderzoeksgebied flink te exploreren omdat er altijd wel iemand was die de juiste koers in het oog hield.
François, de leus 'live life to the max' lijkt speciaal voor jou gemaakt. Ik vind het bewonderenswaardig hoe je werk en privé combineert. Ik hoop dat ik ook de komende jaren deelgenoot mag zijn van je bestorming van de internationale wetenschappelijke ladder, zoals ik dat ook mocht zijn van je eerste schreden als hoogleraar.
Jaap, bedankt dat je in 1999 mij en het onderzoek 'Interacties' zag zitten. Het opstarten van dit onderzoek is slechts een van de vele voorbeelden van je geweldige gevoel voor timing. Daarnaast bewonder ik de manier waarop je altijd de juiste snaar weet te raken in discussies en vergaderingen. Ik ben erg blij dat ik de afgelopen jaren deze kunst heb mogen afkijken.
De vliegende start van het onderzoek Interacties lag niet alleen aan de chemie tussen François, Jaap en mijzelf. Jan Wiggers, mijn afstudeerhoogleraar, heeft me in de twee jaar dat hij mij onder zijn hoede had alle ruimte gegeven om een brede, inhoudelijke basis op te bouwen. Jan, bedankt voor de kans en het vertrouwen te mogen beginnen met een promotieonderzoek. Helaas heb je in 1999 wegens privé-omstandigheden vroegtijdig de TU verlaten. Ik ben erg gelukkig met het feit dat jij alsnog plaats neemt in mijn promotiecommissie.
Ab Dirkzwager, als steunpilaar van het eerste uur wil ik je bedanken voor je raad en advies.
Jan Kop, op jou lijkt de tijd geen vat te krijgen. Je betrokkenheid bij 'jouw' vakgroep is ongelooflijk en ik ben je zeer erkentelijk voor de steun achter de schermen in de roerige 'hoogleraarloze' periode van begin 1999. Je blijkt wat mij en mijn goede vriend Hans Korving betreft het gelijk aan je kant te hebben gekregen.
Hans Korving, ik ga toch meer schrijven dan 'idem'. Ik heb grote bewondering voor het feit dat je je promotieonderzoek hebt volgehouden en voltooid, ook al was het pad erg hobbelig. Onze levendige discussies hebben sterk bijgedragen aan de kwaliteit van dit proefschrift en daarnaast heb ik altijd met veel plezier met je samengewerkt, hetgeen in de toekomst in welke vorm dan ook hopelijk een vervolg krijgt.
Naast Hans Korving hebben vooral Arjen van Nieuwenhuijzen en Jasper Verberk bijgedragen aan de teamspirit binnen de sectie. Ik hoop dat de nieuwe generatie onderzoekers met eenzelfde gedrevenheid en genoegen mag werken.
Arjen, jouw invloed op mijn werk is groter dan je wellicht zult denken. Op jouw aanraden heb ik de titel van het proefschrift aangepast! Ik hoop dat wij elkaar de komende jaren nog veelvuldig zullen zien, niet alleen in Delft bij Interacties II, maar ook in Amersfoort en Nijmegen.
Jelte van der Heide en Jaap de Koning wil ik in een adem noemen. Jullie hebben mij beiden een niet te onderschatten luisterend oor geboden.
Daarnaast gaat mijn dank uit naar alle overige (oud-)collega's van de sectie gezondheidstechniek, al was het maar om het lachen om mijn grappen tijdens de koffiepauzes. In het bijzonder wil ik nog Rob Veldkamp, Tonny Schuit, Cees Boeter, Martijn Klootwijk en Cornelis de Haan bedanken voor hun hulp tijdens mijn soms wel wat wilde experimenten.
Verder wil ik alle mensen die de experimenten in Loenen, Beekbergen, Ulvenhout en Katwoude mogelijk hebben gemaakt, in het bijzonder Paul van Berkum, Marie-Claire ten Veldhuis, Hielke van der Spoel en de enthousiaste medewerkers van Witteveen+Bos, Norbert Mulder, Guy Henckens, Lennard Stigter en Harry van Mameren, bedanken.
Na het meten volgde vaak het modelleren. Guus Stelling, bedankt voor je hulp bij het doorgronden van de numerieke schema's. Mark van Loosdrecht, bedankt voor je opmerkingen bij het gebruik van ASM modellen.

Wat het werken aan een universiteit vooral leuk maakt is het feit dat er zo veel jonge, enthousiaste en getalenteerde mensen rondlopen. Het begeleiden van afstudeerders in hun afstudeerproject heb ik ervaren als het 'werk' dat de meeste voldoening geeft. Jamie Reuvers, Niels Willemsen, Christiaan Schaum, Marjolijne Herbergs, Jeroen Stok en last but not least Rémy Schilperoort: bedankt dat ik jullie mocht begeleiden!
Het onderzoek was niet mogelijk geweest zonder de steun van de werkgroep Interacties. Ik wil dan ook de partners binnen het onderzoek en in het bijzonder de afgevaardigde werkgroepsleden Jan Zuidervliet en Hans Aalderink (Arcadis), Hans Geerse en Hans Hartong (HKVlijn in water), Johan van Dijk (Vertis), Saskia Holthuijsen en Saskia van 't Veen (Grontmij) en tenslotte Peter de Jong en Marcel Boomgaard (Witteveen+Bos) bedanken voor hun bijdrage aan het onderzoek. De brede samenstelling van de werkgroep en de specifieke kwaliteiten van elk van de werkgroepsleden heeft in hoge mate bijgedragen aan het succes van het onderzoek. Door de hoge frequentie van samenkomen en de afsluitende gezamenlijke etentjes (goed idee, Jaap!) heb ik met velen van jullie een bijzondere band kunnen opbouwen.
In the international context, I am indebted to Richard Ashley and Jean-Luc Bertrand-Krajewski for their useful comments on my manuscript, the German HSG for the exchange of ideas and information and all others who contributed by exchanging knowledge and information.
Binnen het onderzoek is een groot aantal onderwerpen verkend, wat een aantal zeer interessante ideeën voor verder onderzoek heeft opgeleverd. Hoewel ik deze ideeën graag zelf verder had uitgewerkt ben ik erg blij dat ik het stokje mag overdragen aan twee opvolgers, waar ik zeer veel vertrouwen in heb. Cathelijne Flamink en Rémy Schilperoort, het afgelopen jaar heb ik met veel genoegen met jullie samengewerkt en ik hoop dat wij elkaar de komende jaren, ook buiten het onderzoek om, veel zullen zien.
Verder mag ik mij gelukkig prijzen met twee goede vrienden als paranimfen, die beiden een belangrijk aandeel in het tot stand komen van dit proefschrift hebben gehad. Marcel Boomgaard, het was geweldig om met je samen te werken aan onder andere de in hoofdstuk 5 beschreven optimalisatieroutines. Met name onze congressen in Orlando en Portland, met alles erop en eraan, waren onvergetelijk.
Wouter Sint Nicolaas, ik ben je veel dank verschuldigd voor het doorploeteren van de eerste versie van elk hoofdstuk. Na het 'Woutfilter' kon ik ze met veel vertrouwen indienen bij François en Jaap. Samen met jouw lieve Reen ben je erg belangrijk voor me.
Mijn vrienden, (schoon-)ouders en naaste familie wil ik bedanken voor hun begrip voor het feit dat ik het afgelopen jaar vrijwel al mijn energie in mijn proefschrift heb gestoken. Ik hoop dat ik jullie de komende tijd weer de aandacht kan geven waar jullie recht op hebben. Dit laatste geldt zeer zeker ook voor mijn lieve Letty en Silke. Silke, dank je wel voor het plezier dat je me geeft en dat je me zowel thuis als op de TU, waar je voor een pasgeborene wel erg veel tijd hebt doorgebracht, hebt laten doorwerken. Letty, ik weet dat ik erg veel van je heb gevraagd en wil je hier als laatste bedanken voor al je steun en voor de ruimte die je me hebt gegeven.

Jeroen Langeveld
Nijmegen, september 2004

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Chapter 1 Introduction and scope

1.1 Urban wastewater systems

Urban wastewater systems consist principally of two subsystems: sewers and wastewater treatment plants, both discharging to the receiving waters, as illustrated by figure 1.1. Sewer systems have to collect and transport wastewater and storm runoff out of the urban areas, while the wastewater treatment plants have to reduce the pollution load on surface waters. Since the main functions of the two subsystems differ strongly, sewer systems and wastewater treatment plants have often been regarded as separate entities both in practice and in science. Nevertheless, the history of their development is strongly linked, as is exemplified by the brief historic overview in the next section.

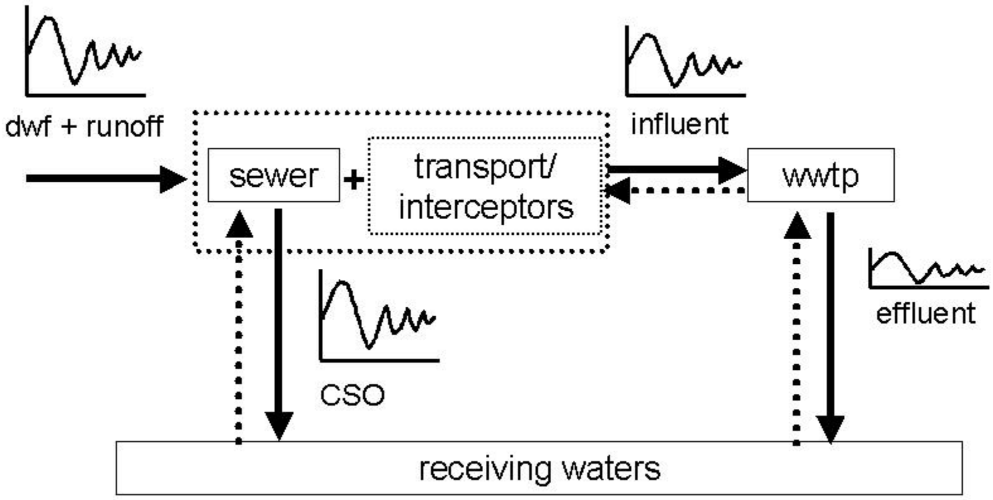


Figure 1.1 Components of the urban wastewater system and their interactions. These interactions may be physical (a flow of water and its components) or non-physical (a flow of data or information) in both directions. (dwf = dry weather flow, CSO = combined sewer overflow, wwtp = wastewater treatment plant).

1.1.1 History of urban wastewater systems in the Netherlands

Although sewer systems have been known for centuries (e.g. Babylonian sewers existed as early as 4500 BC [Akker, van den (1952)], the well known Cloaca Maxima in ancient Rome has been built 200 BC), it was not until the second half of the 19th century that the first development of today's 'modern' wastewater systems started. This development was mainly due to [Zon, van (1986)]:

- increasing degree of urbanisation due to industrialisation;
- activities of medical doctors, such as John Snow [Snow (1855)], which led to growing knowledge on relations between public health and local living conditions and sanitation;
- *tremendous* (odour) nuisance within cities [Zon, van (1986)]. Odours were considered to be an indicator of unhygienic conditions and should therefore be diminished.

A great number of solutions to the perceived problems have been developed by 'inventors', engineers and hygienists [Zon, van (1986)]. Protecting public health and, at that time from an economical point of view even more important, reuse of the nutrients have been the key issues in the development of various systems, of which only the Liernur system [Krepp (1867)] (vacuum system), cesspools, collection system with barrels and a gravity sewer system have been applied at a larger scale in the Netherlands [Akker, van den (1952); Zon, van (1986)].

Due to the rapid development of the drinking water supply, see figure 1.2, the amount of wastewater to be transported out of the urban areas increased strongly. As a result, the gravity sewer system proved the only system to be able to collect most of the diluted (and therefore worthless with regard to reuse of nutrients) sewage and to transport this out of the urban area. The development of the gravity sewer system is shown in figure 1.2.

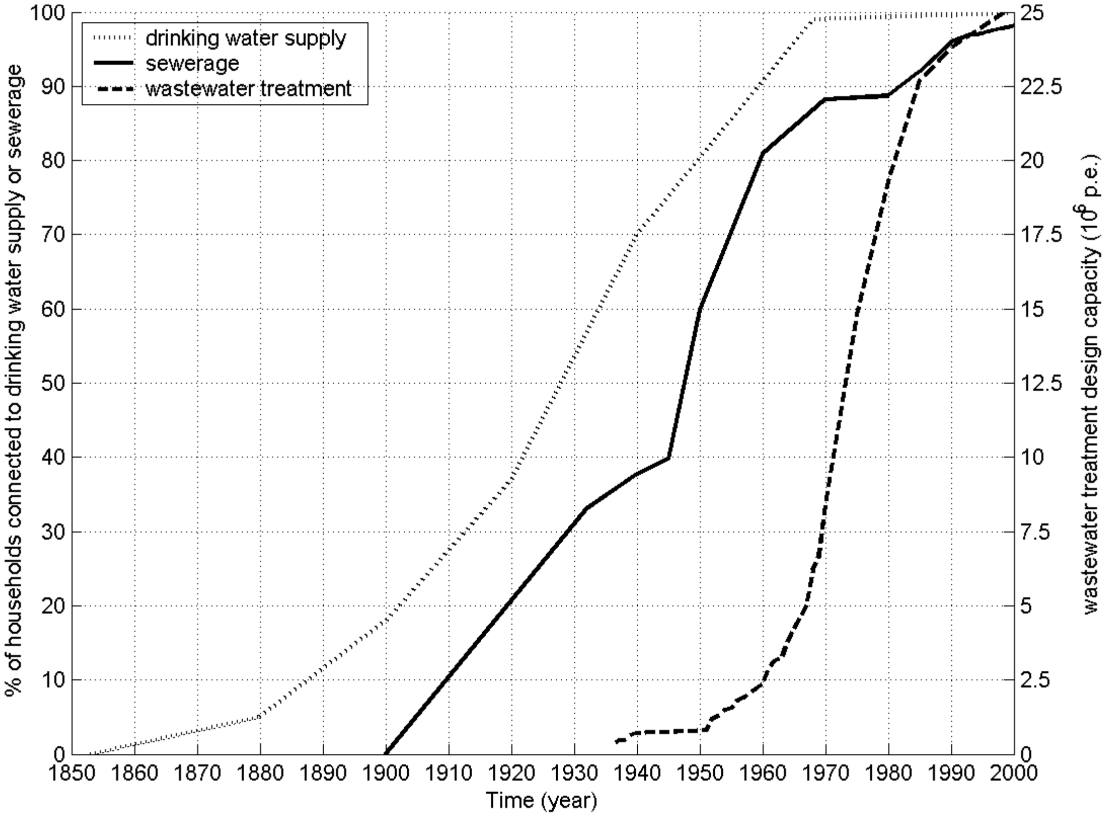


Figure 1.2 Development of the sanitary infrastructure in the Netherlands [CBS (2003); Stichting Rioned (2002); Dirkzwager and Kiestra (1995)]. All three systems show a development typical for large-scale technological systems [Bijker *et al.* (1987)]: an initial phase of invention, testing and innovations, where technology is the most important factor, a phase of transfer and growth, where the system as a whole gets momentum and develops rapidly until reaching full expansion and the final phase of consolidation. During the latter phases the importance of technology diminishes rapidly and organisational, legal and financial aspects take over. A perfect example is the development of the wastewater treatment capacity, which was enhanced by the 1970 Pollution of Surface Waters Act (WVO Wet Verontreiniging Oppervlaktewater).

As early as the end of the 19th century, especially around larger urban areas, such as London and Paris, it became evident that the free discharge of sewage in the rivers caused adverse effects on the river water quality, leading to anaerobic conditions, undesirable odours and blockage of rivers by sludge embankments (Thames) [Akker, van den (1952)].
In London and Paris, this problem was dealt with by the application of land treatment, while in the Netherlands flushing through of urban surface waters (dilution) or building of outfalls towards the sea or larger surface waters were seen as solutions of the water pollution. However, these solutions showed only temporary relief, due to an increasing population, urbanisation and industrialisation.
This led to the development of the first wastewater treatment plants, which aimed at a reduction of the settleable biodegradable matters in the wastewater. The first treatment plants consisted of mechanical treatment or land treatment. At the end of the 19th century, the first biological treatment plants (trickling filters) became available.

Nevertheless, it was only until the late 1960s and 1970s that large-scale construction of wwtps took place in the Netherlands, as illustrated in figure 1.2 [Groeneveld (1994)]. In this period, the increased discharge of untreated wastewater (and the enhanced persistency of its pollutants) by far exceeded the self-cleansing capacity of the receiving waters, with severe oxygen depletion in several rivers and lakes as a result. Consequently, the Dutch government declared the 1970 Pollution of Surface Waters Act (WVO Wet Verontreiniging Oppervlaktewater). In the beginning of the 1970s, wwtps aimed at reducing biodegradable matter (biochemical oxygen demand, BOD) in wastewater. Later on, from 1978 onwards the focus shifts towards nutrients, especially the nitrification of ammonium-nitrogen, which led to effluent standards for Kjeldahl-nitrogen [Dirkzwager (1997)]. Since 1985 the focus has also been on the control of eutrophication and therefore the effluent standards for phosphate and nitrogen have been strengthened to today's strict levels, as shown in table 1.1.

Table 1.1 Effluent standards for wwtps [Dirkzwager (1997); Dirkzwager and Kiestra (1995); Nieuwenhuijzen, van (2002)]

Period	Type of standard	Today's status of standard, in mg/l 10-day average (flow proportional daily composite samples), except for N-total (yearly average).
until 1970	no standards	
1970 – now	BOD suspended solids	20 mg O_2/l 30 mg SS/l (SS = suspended solids)
1978 – now	Kjeldahl nitrogen	20 mg N/l
1990 – now	P_{total}	2 mg P/l, new + existing plants < 100,000 p.e. (population equivalent) 1 mg P/l, new + existing plants > 100,000 p.e.
1992 - now	N_{total}	15 mg N/l new plants (existing plants from 2005)<20,000 p.e. 10 mg N/l new plants (existing plants from 2005)>20,000 p.e.

The focus on water pollution control also had an influence on the design of sewer systems. Until 1951 combined sewer overflow (CSO) structures were designed to start discharging once the wastewater was diluted 5 to 10 times [Akker, van den (1952)]. Since each wwtp was designed to deal with 2 to 4 times dry weather flow (dwf), it was often impossible to reach the desired dilution. In 1951, [Ribius (1951)] developed the straightforward method of overflow frequency. Sewer systems were only allowed a certain overflow frequency (usually 10 times per year), which introduced the possibility to find a trade-off between in-sewer storage capacity and hydraulic capacity of wwtps.

In the early 1970s, the CSO discharges were considered to be too polluting, which led to the large-scale introduction of the separate sewer systems. However, it was soon realised that separate sewer systems have a major drawback: the inevitable faulty connections. It is known that about 5% of the connections to a separate sewer systems may be wrong [Clemens (2001a)]. Measuring campaigns in the Netherlands [NWRW (1989)] showed that the annual pollution load to the receiving waters from combined sewers and from separate sewers has the same order of magnitude.

In order to cope with this drawback, 'improved' separate sewers systems have been introduced since the middle of the 1980s. These systems consist like ordinary separate sewer systems of a sanitary sewer system and a storm sewer system. The system is 'improved' as a part of the storm water (and possibly the wastewater entering the storm sewer through faulty connections) is discharged to the wwtp. The pumping, or interceptor, capacity, is typically about half of the capacity of a combined sewer system, which reduces the peaks arriving at a wwtp significantly.

In 1985 it was agreed upon in an international context within the North Sea Action Plan and the Rhine Action Programme that the nutrient discharges from all sources within the Rhine catchment and to the North Sea should be reduced by 50% relative to the 1985 pollution loads. In the Netherlands, therefore, it was agreed that this could be reached with a 'reference system' or a system with an equivalent annual pollution load. This reference system is defined by a national committee [CUWVO (1992)] as:

> *Any combined sewer system should emit a pollution load less than or equal to a fictitious system having 7 mm (70 m^3/ha) in-sewer storage, 2 mm storage in a settling facility and a pumping capacity equal to the dwf production plus 0.7 mm/h (7 m^3/h/ha)*

In addition to the measures necessary to equal the 'reference system' performance, a number of water boards have their own regulations in order to protect vulnerable receiving waters.

Table 1.2 gives an overview of the various environmental standards as applied to (parts of) the wastewater system in practice in the Netherlands during the last 60 years.

Table 1.2 Environmental standards for sewer systems.

Period	Type of standard	Remarks
until 1951	dilution during wet weather flow (wwf)	depending on the sensitivity of the receiving waters a dilution factor of 3 to 10 should be reached before the CSO may start working [Akker, van den (1952)]
1951 – 1992	overflow frequency	acceptable overflow frequency (calculated by simple reservoir model) varies between 3 and 10 CSO events/annum, depending on the water board and type of receiving waters [Ribius (1951)].
1992 – 2001	overflow volume	Each sewer system should perform at a certain minimal level, equivalent to the performance of a 'reference' sewer system with an internal storage capacity of 7 mm, a stormwater settling tank of 2 mm and a (interceptor) pumping capacity of 0.7 mm/h + dwf. This performance level is measured by the annual overflow volume, to be calculated by a (simplified) hydrodynamic or reservoir model [CUWVO (1992)].
2001 – now	overflow loads	In 2001 the reference system performance has been defined more clearly and equals an annual discharged chemical oxygen demand (COD) load of 50 kg COD per hectare of impervious area. The COD concentration of the spilled CSO volume to be taken into account is fixed at 250 mg COD/l [CIW (2001)]. Basically, this approach can be addressed as 'volume based', given the fixed concentrations.
1992 – now	receiving water quality assessment	the 'waterkwaliteitsspoor' or water quality assessment for receiving waters may be categorised as an immision[a] based approach. The approach assesses the impact of CSOs on the receiving waters. When necessary regarding the quality of receiving waters, additional measures compared with the 'basic performance level' may be necessary [CUWVO (1992)].

[a] emission standards deal with the discharged pollution, whereas immission standards require a certain state in the receiving waters. In the latter case, the total pollution load that can be discharged depends on the required receiving water quality [Lijklema (1995)].

This brief description illustrates the strong relation in the development of the sewer system and of the wastewater treatment, in which, however, both systems were continuously regarded as separate entities. This is also reflected in the separate responsible agencies. Besides, their development shows the continuously changing requirements to be met by the wastewater infrastructure.

Moreover, a comparison of table 1.1 and 1.2 illustrates the large difference in the assessment of wastewater treatment plants and sewer systems. The performance of sewer systems is assessed by *model calculations*, while the performance of a wwtp is assessed by *measurements*. Modelling, either static or dynamic, of the performance of a wwtp is usually only applied during the (re-) design of a wwtp or for developing control strategies.

1.1.2 Today's wastewater infrastructure

Today's Dutch (waste)water infrastructure is among the most developed in the world. Over 99.9 % of the population has access to public water supply and 98 % of the houses are connected to a sewer system discharging to a wwtp [(RIONED (2001)]. The wastewater discharged to the wwtps is treated effectively, as illustrated by table 1.3.
The water boards are responsible for the wastewater treatment and the municipalities for the sewer systems.

Table 1.3 Characteristic data for today's wastewater infrastructure [CBS (2003), RIONED (2002)].

sewer (reference year 2002)			
- total length	86.452 km		
- mechanical sewers	16 %		
- gravity sewers, of which:	84 %		
- combined	64 %		
- (improved) separated	36 %		
wwtp (reference year 2000)			
- total number	391 #		
- design capacity	25.2 10^6 person equivalent (p.e.)		
- actual load	23.2 10^6 p.e.		
- flow	5.7 10^6 m^3/d		
	influent composition (averages)	**effluent quality** (averages)	**treatment efficiency** (flow proportional weighted average)
COD	470 mg O_2/l	51 mg O_2/l	90 %
BOD_5	180 mg O_2/l	7 mg O_2/l	97 %
N_{total}	44 mg N/l	11 mg N/l	66 %
P_{total}	7 mg P/l	2 mg P/l	79 %

Within the European Union a lot of progress has been made concerning the wastewater infrastructure during the last two decades. Although the Dutch level of households connected to a sewer system and subsequent wastewater treatment is still extraordinary high, most EU countries are close to the point where a further increase in connectivity will no longer be economically feasible. Only with respect to connectivity to wastewater treatment some progress still has to be made.

Table 1.4 Today's European wastewater infrastructure [Rioned (2002)].

country	% of households connected to sewer system		% of households connected to sewer system and wwtp	
	1980	**1998**	**1980**	**1998**
Austria	-	82	38	81
Belgium	-	82	23	38
Denmark	89[a]	89	79[a]	89
France	-	81[b]	-	79[b]
Germany	-	92[b]	-	92[b]
Luxembourg	-	88[b]	81	88[b]
Netherlands	86	98	72	98
Norway	80	80	34	73
Portugal	35	82	2	-
Switzerland	-	96	73	96
UK	95	94[c]	82	84[c]

[a] 1985
[b] 1995
[c] 1997

Sewer systems
Today's sewer systems are the result of over a century of new ideas, design philosophies and technological improvements. Since the building of sewer systems took place gradually, each wastewater system may consist of a combination of four distinguishable types of sewer systems, see figure 1.3:
- combined sewer system
- improved combined sewer system: combined systems with additional storage capacity
- separated sewer system
- improved separated sewer system: separated system where a part of the stormwater is discharged to a wwtp

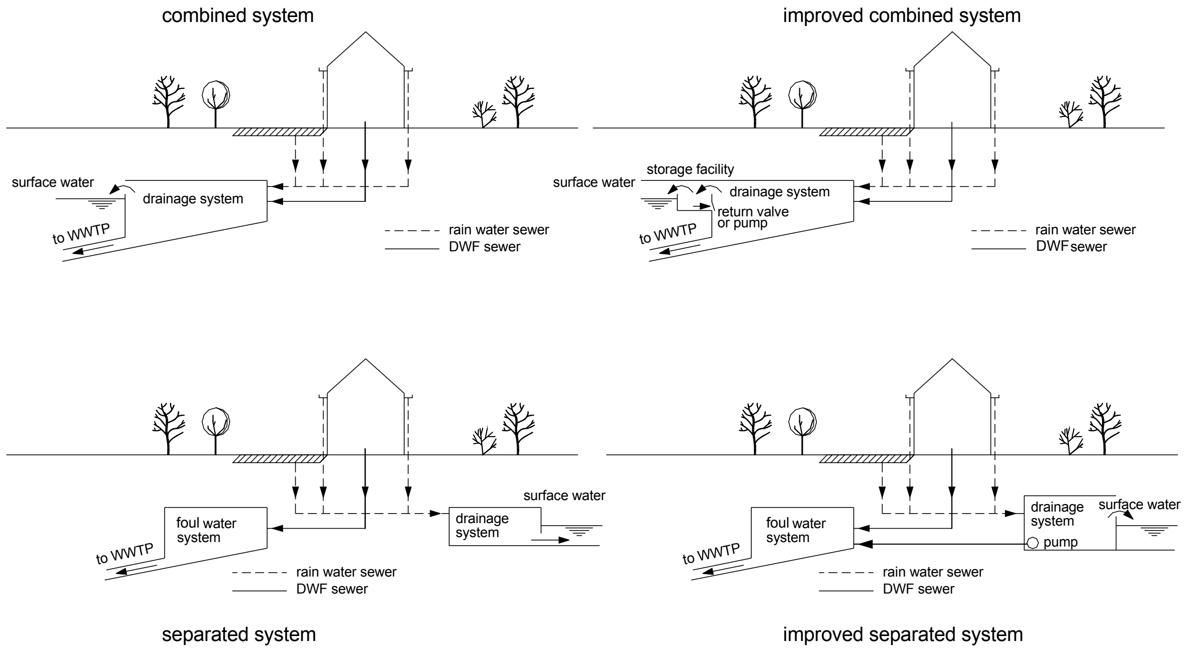


Figure 1.3 Types of gravity sewer systems (reproduced with permission) [Clemens (2001a)].

Within wastewater systems, the contributing sewer systems can either be linked to each other or directly to a wwtp, both under gravity or through pressure mains. Since the combined sewer system is still the predominant type, (64 % of all sewer systems are of the combined type, [RIONED (2002)] the behaviour of wastewater systems under transient conditions is often dominated by the combined sewer systems. In this respect, the layout of wastewater systems, including (pressurised) interceptor sewers, is of major importance.

Wastewater treatment plants
Today's wastewater treatment plants are, just like sewer systems, the outcome of continuous efforts to comply most economically with the changing requirements by applying best available technology. As a result, a great number of different types of wwtps exists, see figure 1.4, although activated sludge treatment is the predominant type with 95% of the installed capacity [CBS (2003)]. In order to be able to deal with the stringent effluent standards for nutrients, (ultra) low loaded systems are increasingly applied in the Netherlands [Nieuwenhuizen, van (2002)].

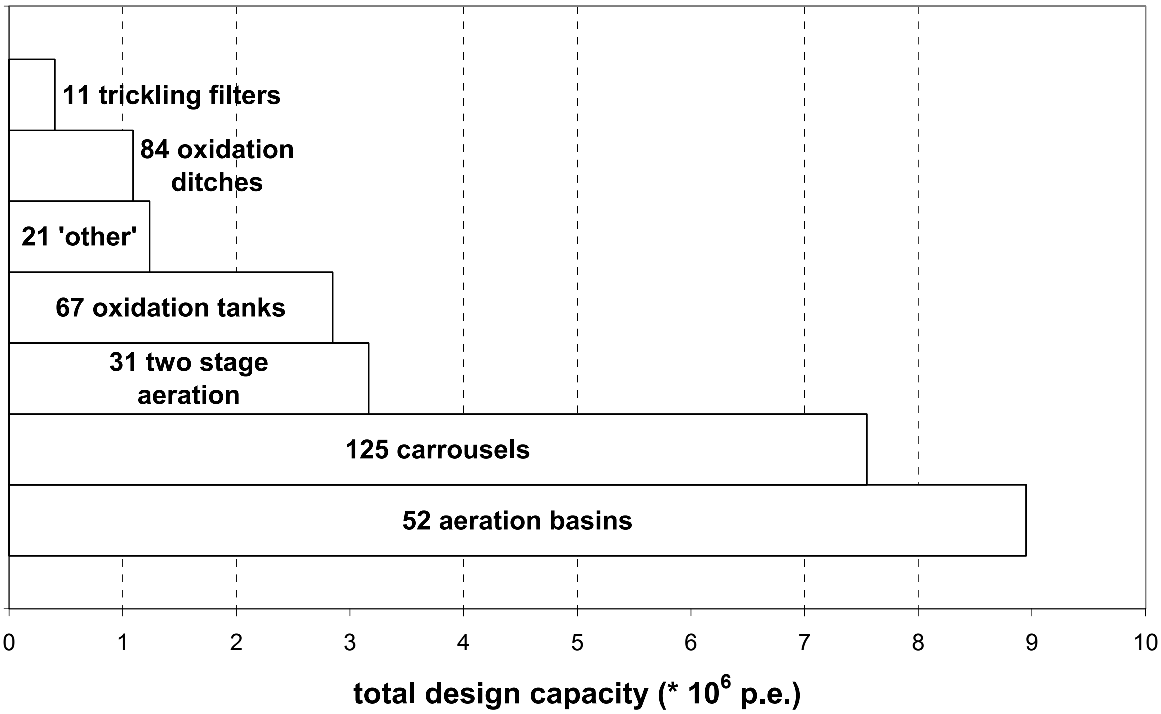


Figure 1.4 Number and design capacity of the various types of biological processes applied in Dutch wastewater treatment systems in 2000 [CBS (2003)].

Receiving waters
In general, the quality of receiving waters within the Netherlands has been considerably improved since the declaration of the 1970 Pollution of Surface Waters Act. Approximately 5.4 billion Euro has since been invested in the improvement of sewer systems and the construction and improvement of wastewater treatment plants [Nieuwenhuijzen, van (2002)].
The most important point sources of pollutants have since been eliminated and the most obvious problems have been solved. Oxygen depletion nowadays only occurs in small receiving waters after significant CSO events. However, the eutrophication of the receiving waters still remains an important problem. Moreover, new problems, like the impact of endocrine disrupters on the aquatic ecosystem [Flamink (2003)], continue to emerge.
As a result of the European Water Framework Directive (WFD) [WFD, (2000)] within the near future (before 2015) the emission standards for sewer systems and wwtps locally will be tightened even more to assure a good ecological receiving water quality. Since 75% of the Dutch wwtps discharged into relatively small regional receiving waters, where the wwtp effluent has a relatively large contributing effect to the receiving water quality, the EU WFD will locally result in a tightening of the emission standards.

1.2 Wastewater system optimisation

The wastewater infrastructure within the Netherlands is nearly complete, in the sense that almost all the sewage is collected and treated to a certain extent. Table 1.4 shows that, although to a lesser extent, the same holds true for many European countries.
Wastewater systems are generally designed and constructed to comply with the regulations in force. Table 1.1 and 1.2 illustrate that the regulations do have a tendency to change in an unpredictable manner. Therefore, designing sewer systems able to comply with all imaginable future regulations is virtually impossible. As a result, it is very likely wastewater systems need to be improved as soon as the regulations set by the responsible authorities change, thereby starting a new series of improvements and optimisations. Optimisation is in this case defined as complying with the standards at minimal costs. Until recently, sewer systems and wastewater treatment plants were improved or optimised separately. However,

since the beginning of the 1990s (INTERURBA I [Lijklema *et al.*, (1993)]; German Gesammtemissions Gruppe [Durchschlag *et al.* (1992); Otterpohl *et al.* (1994a)]; Sewage into 2000 [Kruize (1993)]), it has been increasingly acknowledged that the combined emission from the sewer systems and the wwtps determine the total loading from the urban water system on the surface waters. As a result, nowadays, it is widely accepted that an integrated assessment of the emissions from sewer systems and wastewater treatment plants is necessary when attempting to reduce the total impact of the urban water system on the receiving waters (INTERURBA II [Matos (2002)], Urban Pollution Management (UPM) procedure [FWR (1998)]).
However, assessments of wastewater system performance can be done in many ways and at many levels of detail, ranging from an estimation of annual loads to constant monitoring of the performance and dynamics of the wwtp and the sewer system.

1.2.1 Today's approach in the Netherlands

A large and still increasing number of wastewater system optimisation studies have been performed in the Netherlands. The success of these studies, however, was sometimes rather limited due to all sorts of procedural, political and organisational problems. In order to facilitate wastewater system optimisation studies, the RIONED foundation issued a guideline [Stichting RIONED (2003a)], see appendix XI, structuring the process of optimisation studies. This guideline, however, does not address two main limitations of today's wastewater system optimisation studies.
Firstly, today's wastewater system optimisation studies can be categorised as volume-based. This is due to the fact that, nowadays, the performance of the sewer system is assessed by its annual overflow volume, see table 1.2. In addition, in most wastewater system optimisation studies wwtp performance is assessed by the effluent quality during dwf and the hydraulic capacity of the secondary clarifiers. As a result, qualitative aspects and the dynamic interactions between sewer systems and wastewater treatment are completely neglected.
Secondly, the optimisation procedure itself is in most cases based on expert judgement combined with 'trial and error'. At best, a sensitivity analysis is performed on a calibrated model in order to judge which CSO or catchment deserves most attention. More often, however, uncalibrated models are applied to check which of a limited number of selected alternatives is the 'optimal solution' [Boomgaard *et al.* (2001a)]. The use of advanced optimisation algorithms, such as Genetic Algorithms, is still limited, although their potential for optimisation of urban water systems is well-known [e.g. Rauch and Harremoës (1999)].

1.2.2 International approaches

Internationally, today's paradigm is to assess the urban water system as an integrated system. Although the paradigm is unambiguous, its translation into practical approaches is not. In literature almost as many approaches as research groups involved are found, indicating no approach has as yet been proven to be superior.

Only FWR (1998) defines the interactions within wastewater systems:

…'Interaction' between the sewer system and the sewage treatment works (STW) means either:
- *there is a river quality problem downstream of the STW due to a complex interaction of CSO, storm tank and STW effluents impacting upon the river quality; or,*
- *the STW effluent quality deteriorates significantly during wet weather and causes a river problem, regardless of the CSO discharges. [FWR (1998)]*

Although many researchers developed integrated models, as illustrated by table 1.5, only a few discussed systematically the necessary level of detail of process descriptions within these integrated models:

- Fronteau *et al.* (1997) compared the state variables, processes and parameters for sewer and wastewater treatment plant models;
- Rauch *et al.*, (1998) describe the result of a COST working group meeting where general requirements for integrated wastewater models based on receiving water objectives have been developed;
- FWR (1998) The UPM manual gives suggestions for the type of model to be selected for each component of the urban water system within a UPM study;
- Leinweber (2002) discusses the requirements for sewer and wwtp models to be included in an integrated assessment of wastewater systems.

Table 1.5 Examples of packages for integrated modelling.

Package	Sewer	STW	Receiving waters	
SIMBA	SIMBAsewer	SIMBA		[Ellingsson *et al.*(1999); Alex *et al.*(1999)]
	KOSMO	SIMBA		[Leinweber *et al.* (1999)]
ICS	MOUSE	STOAT	MIKE	[Clifforde *et al.* (1999); Hernebring *et al.* (1999)]
Synopsis	KOSIM	ASM1	DUFLOW	[Schütze *et al.* (2002)]
	Hydroworks	SIMBA		[Juillard *et al.* (2001)]
WEST				[Meirlaen *et al.* (2001)]

Practical applications, however, are still limited. This is mainly due to the laborious calibration of the integrated models, necessitating an enormous amount of data and the fact that the individual sub-models have been designed to suit other purposes. [Rauch *et al.*, 2002]. Besides, knowledge on the interactions within wastewater systems is still limited, thereby hampering the further development of appropriate models.

1.3 Scope of the research

Today's Dutch approach of wastewater system optimisation does not seem to be fully in line with the widely advocated and accepted integrated approach on urban (waste)water systems. As knowledge on the dynamic behaviour of both sewer systems and wastewater treatment plants is constantly increasing and modelling tools are widely available, this may seem rather surprising. However, research has mostly been focused on either sewer systems or wastewater treatment plants, rather than taking the interactions between sewer systems and wastewater treatment plants into account. Therefore, knowledge on the interactions between sewer systems and wastewater treatment plants seems to be too little available to be applied in daily practice and wastewater system optimisation studies currently applied in the Netherlands.

The objective of this thesis is to identify the possibilities to extend today's Dutch volume based approach for wastewater system optimisation to a water quality based approach by taking into account the dynamic interactions within wastewater systems.

1.4 Outline

Chapter 2 discusses the state of the art of the broad field of sewerage and wastewater treatment. Knowledge deficits related to extending current Dutch volume based approach for wastewater system optimisation to a water quality based approach are identified. The sensitivity of wastewater treatment plant performance to influent fluctuations and the possibility to predict and quantify these influent fluctuations with current sewer process models are the main research topics identified.

Chapter 3 describes the methodology and results of the analysis of the sensitivity of wastewater treatment plant performance to influent fluctuations. In this respect, the sensitivity of wwtp performance is the guiding principle for the exploration and testing of sewer quality modelling approaches as highlighted in chapter 4.

In chapter 5, two optimisation algorithms, Genetic Algorithms and Simulated Annealing, are shown to be capable of solving the typically non-linear, multi-objective optimisation problems as encountered within wastewater system optimisation studies. The material presented in this chapter is the product of a co-operation with Marcel Boomgaard.

Chapter 6 discusses the relevance of the interactions within wastewater systems on wastewater system performance. In addition, the result of an assessment of wastewater system performance is shown to depend on wastewater system characteristics, environmental conditions and selected wastewater system performance indicators.

Finally, chapter 7 gives the overall conclusions and recommendations.

Chapter 2 State of the art

2.1 Introduction

Wastewater systems comprise sewerage and wastewater treatment. Historically, both subsystems have been regarded as separate entities. Nowadays total wastewater system performance is a key issue. The loading of wastewater systems fluctuates in terms of both quantity and quality. The capacity of wastewater systems to deal with these variations in the loading, however, is finite. Consequently, the performance of the wastewater system, assessed by the pollution load discharged via the wwtp effluent and the CSOs, is subject to these fluctuations in the wastewater system loading. The capacity to deal with fluctuations in the loading of wastewater systems is determined by the design and operation of sewer systems and wwtps, involving many processes taking place in each subsystem.
This chapter discusses the fluctuations in the wastewater system loading and identifies dominant processes taking place in sewer systems and wwtps affecting the dynamic response of the wastewater system to its loading. The state of the art with respect to these dominant processes is the starting point for the research described in the chapters 3 to 6.

2.2 Urban wastewater systems and receiving water quality

Urban wastewater systems can have a significant impact on receiving water quality. The wwtps continuously discharge their effluent, whereas sewer systems, although usually much more intermittently, discharge through CSOs, SSOs (sanitary sewer overflow) or storm water outfalls. The importance of the combined emissions from sewer systems and wwtps has been widely acknowledged since the beginning of the 1990s [Lijklema *et al.* (1993)].

However, the emission from the wastewater system is only one of the factors determining the quality of the receiving waters. Other sources, such as natural and agricultural sources, may contribute significantly to the pollutants loading of the receiving waters. Above all, the characteristics of the receiving waters determine the effect of the total loading, as illustrated by table 2.1 [House *et al.* (1993)].

Table 2.1 Qualitative assessment of receiving water impacts of urban discharges [after House, *et al.* (1993)]

Receiving water	Water quality				Public health	Aesthetics	
	dissolved oxygen	**nutrients**	**sediments**	**toxics**	**microbials**	**clarity**	**sanitary debris**
streams					+		
- steep	-	-	-	+	++	-	++
- gradual	+	-	+	+		-	++
rivers							
- small	++	-	+	+	++	-	++
- large	+	-	+	+	++	+	++
estuaries							
- small	+	+	+	+	++	+	++
- large	-	-	+	-	++	+	++
lakes							
- shallow	+	++	+	+	++	+	++
- deep	+	+	+	+	++	+	++

- least likely
+ likely
++ most likely

[House *et al.* (1993)] distinguish three basic categories of concern for receiving water quality:
- water quality changes
- public health risks
- aesthetic deterioration

2.2.1 Water quality changes

The proceedings of INTERURBA I highlight the abundance of processes occurring in receiving waters subject to discharges from the urban wastewater systems [House *et al.* (1993)]:
- physical processes: transport, mixing, dilution, flocculation, erosion, sedimentation, thermal effects and re-aeration
- biochemical processes: decay of organic matter, adsorption and desorption of metals and organic micropollutants
- microbiological processes: growth and die-off

The impact and importance of each individual process depends on the relevant time and space scales, as illustrated by figure 2.1.

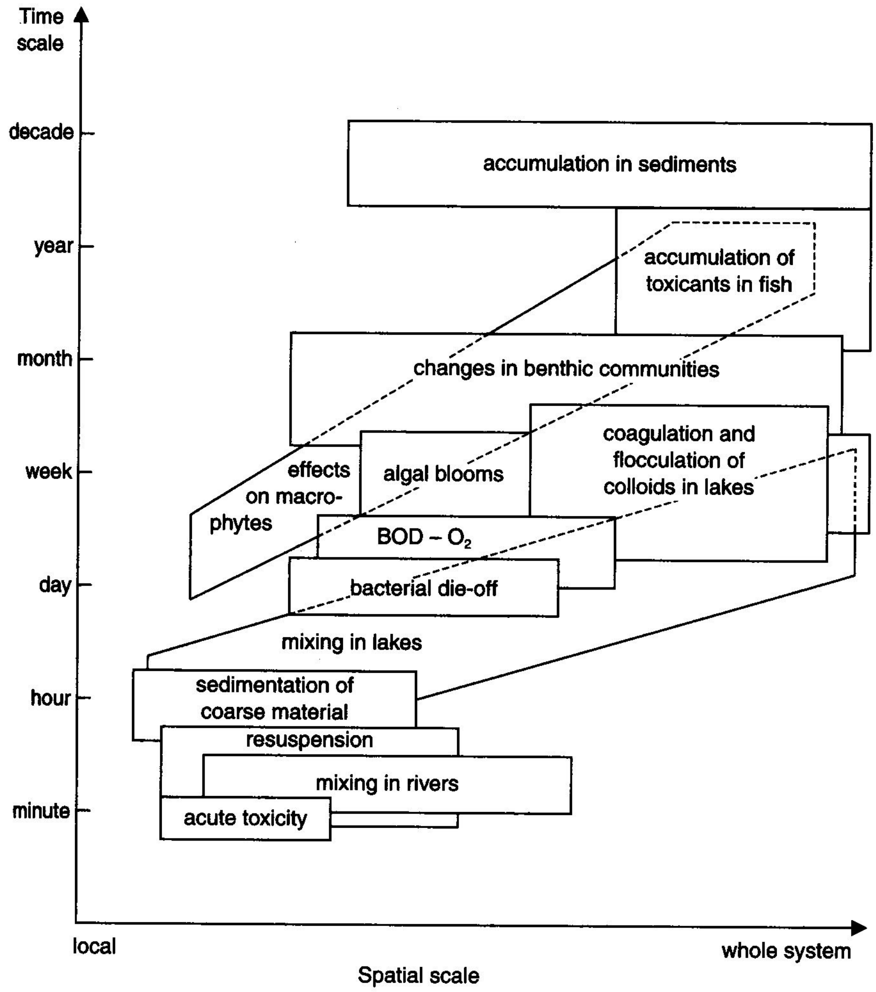


Figure 2.1 Time and space scales for receiving water impacts (after [House *et al.* (1993), Aalderink and Lijklema(1985)].

With respect to the impacts on receiving waters, normally a distinction is made into short term (acute, hours), medium term (delayed, days) and long term (accumulating, weeks to years).

Oxygen depletion is one of the first well studied water quality problems [Streeter-Phelps (1925)]. During storm events, wastewater systems may significantly affect the receiving water dissolved oxygen (DO) concentration by:

- increased discharge and subsequent deoxygenation of BOD (or more general COD) and nitrification of Kjeldahl-nitrogen (short term effect)
- discharge of wastewater with a low DO concentration (short term effect)
- discharge of sediment and subsequent increased oxygen uptake by sediments (medium term effect)

Toxicity is another important water quality parameter. Related to the discharge from urban wastewater systems, acute toxicity seems to be mostly due to increased levels of ammonia (also depending on temperature and pH), whereas long term toxicity seems to be related to parameters such as accumulating metals and hydrocarbons. Recently, the ecotoxicologic impact of substances like endocrine disruptors has been acknowledged [Jobling *et al.* (1998); Flamink (2003)].

Eutrophication is typically a long-term water quality problem. Therefore, the annual loading rather than the event based peak concentration of total nitrogen and total phosphorous to the receiving waters is of main interest.

2.2.2 Public health risks

Public health risks related to sewage and wastewater are mostly associated with exposure to bacteria and viruses. This may involve either direct exposure to affected receiving waters, e.g. during the recreational use of the receiving waters (swimming), or indirect exposure through the food chain. An example of the latter is watering of cattle near CSO structures. In the Netherlands, an inventory showed 2% of the CSOs to be potentially hazardous with respect to public health and 4% with respect to cattle watering [CIW (2001)].
In general, high levels of bacteria and viruses may be found in CSO volumes as well as in wwtp effluent. E.g. Faecal coliforms levels in both combined wastewater [Ashley and Dabrowski (1995)] and wwtp effluent normally amount to 10^5-10^6 coliforms/100 ml [Metcalf & Eddy (2003)], whereas in a CSO Cryptosporidium and Giardia levels of 10^1 and 10^2/100 ml have been measured [Gibson *et al.* (1998)]. The low infectious doses of Cryptosporidium and Giardia (resp. 1-10 and less than 20 organisms) indicate the potential health hazards of CSOs [Metcalf & Eddy (2003)].
The 1976 EU guideline for bathing water requires a maximum level of 2000 faecal coliforms/100 ml. However, recent epidemiological research indicates this level is still too high as only bathing water with faecal coliform levels of less then 220/100 ml does not increase the risk of gastro-enteritis [Medema (2002)]. In the new EU bathing water directive a value of 500 /100 ml is likely to be adopted. A reduction of approximately 10^4 seems to be necessary to be able to safely recreate in affected receiving waters.

2.2.3 Aesthetic deterioration

Aesthetic pollution seems to be mainly associated with gross solids. These gross solids are defined as solids with a specific gravity between 0.9 and 1.2, which can be captured by a 6 mm mesh screen, and which are large enough to be perceived as individual solids. [Davies *et al.* (2002), Digman *et al.* (2002)]
Especially in the United Kingdom (UK) this aspect receives a lot of attention, although only a minor relation exists with the ecological water quality [Gujer and Krejci (1989)].

2.2.4 Water quality problems and parameters associated

The selection of appropriate parameters is rather important when assessing the performance of wastewater systems. In engineering practice, only parameters required by the standards are considered to be important. In the Netherlands, nowadays, this would result in a focus on (annual) effluent concentrations and CSO volumes only, see chapter 1. Ideally, each parameter of the wastewater, discharged either through CSOs or the wwtp effluent, causing

concern for the receiving water quality should be taken into account as soon as its contribution to a receiving water quality problem could be considered to be significant.
Each water quality problem is typically related to a number of substances. Table 2.2 gives an overview of receiving water problems and the related parameters.

Table 2.2 Receiving water problem and related parameters

Perceived receiving water (quality) problem	Associated parameter	Contributing flow	
		CSO volume	wwtp effluent
oxygen depletion	BOD/COD and N_{Kj}	X	X
eutrophication	N_{total}, P_{total}	X	X
public health	Faecal Coli, etc.	X	X
acute toxicity	NH_3, in conjunction with T and pH	X	X
erosion	Q	X	-
aesthetic pollution	gross solids	X	-
ecotoxicity	endocrine disruptors	X	X

Within the European Water Framework Directive (EU WFD) [WFD (2000)] quality standards for receiving waters will be established aiming at ensuring a good ecological water quality. The list of substances will contain 22 prioritary substances [Hellings and Dalen, van (2002)], thereby largely increasing the list of important parameters as summarised in table 2.2. Nevertheless, the ‘old parameters’ will remain relevant as the new list of 22 parameters comes on top of the old parameters.

Moreover, the EU WFD [WFD (2000)] formally changes the way of thinking from emission to ‘immission’, as often advocated [Rauch *et al.* (1998, 2002), Meirlaen *et al.* (2001), FWR (1998)]. In the ‘immission based’ approach the quality of the receiving waters sets the standard for discharges from the wastewater system. The Fundamental Intermittent standards [FWR (1998)] are a perfect example of this approach. The standards are expressed in terms of concentration and duration thresholds for a range of return periods for the DO and un-ionised ammonia concentration, depending on the desired ecosystem.

2.3 Dynamics of flows within wastewater systems

The inflow of wastewater systems, consisting of dwf and storm runoff, shows considerable fluctuations in terms of both flow and composition. Furthermore, these fluctuations may change significantly while passing through the wastewater system, as indicated in figure 2.2. Depending on the characteristics of the sewer system, the fluctuations of the inflow to the wastewater system can be reduced, unchanged or even amplified, resulting in typical fluctuations in wwtp influent. Since the hydraulic capacity of a wwtp is limited and a wwtp effectively reduces the fluctuations in composition of the wwtp influent, the wwtp effluent shows comparatively less fluctuations. In addition, the limited annual number of CSO events show considerable fluctuations in terms of flow and composition.

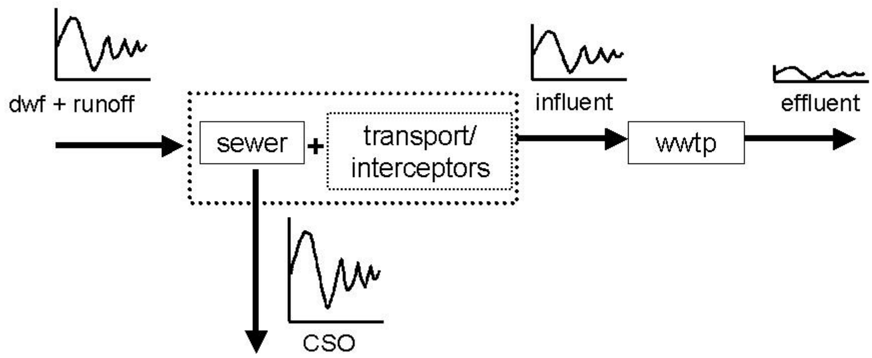


Figure 2.2 Fluctuations of flows and substances through the wastewater system.

2.3.1 Fluctuations in wastewater system inflow

The inflow to a wastewater system consists of:

- dwf, which can de subdivided into:
  - domestic wastewater
  - non-domestic wastewater, i.e. commercial and industrial wastewater
  - infiltration/inflow
- storm runoff

The domestic wastewater production in Western Europe on average amounts to 150 litre per person and day [EEA (2004)]. Table 2.3 gives an impression of the concentrations of the main pollutants in fresh wastewater. The wastewater production shows diurnal and weekly patterns in both flow and composition. The usage of appliances, such as toilet, washing machine or shower is quite individual and shows a great variability. Nevertheless, the overall diurnal or weekly pattern, even at household level, is usually quite noticeable [Almeida *et al.* (1999), Friedler *et al.* (1996), Butler *et al.* (1995)]. The composition of the wastewater depends on the installed appliances (e.g. the COD load may rise as soon as kitchen grinders are applied) and public behaviour. The importance of public behaviour has recently been recognised by [Ashley *et al.* (2002a)], indicating the influence of socio-economic characteristics and ethnic origin on especially gross solids discharged into the sewer system.

Table 2.3 At source wastewater concentrations in mg/l

	Range of literature values as reported by [Almeida (1999)]	**Average (peak) measuring results 2 households [Almeida *et al.* (1999)]**	**Measuring results at 8 locations [Graaf, van der *et al.* (1988)] range, mean and STD (standard deviation)**
COD	500 – 1110	1094 (1432)	668 – 1983, 966 ± 250
BOD	200 – 542	-	246 – 707, 397 ± 110
NH_3-N	19 – 92	23.3 (56.3)	
NO_3-N	0 – 6	3.6 (4.7)	
$N_{Kjeldahl}$-N	50 – 74	-	77 – 222, 111 ± 32
P_{total}			19 – 46, 27 ± 8[a]
TSS	146-697	548 (840)	134 – 519, 270 ± 134

[a] Measuring data date from period before phosphates have been banned from cleaning agents
TSS total suspended solids

A discussion on the characteristics of commercial and industrial wastewater is considered beyond the scope of this research as in most cases domestic wastewater is the dominant factor. In cases where industrial discharges are large compared to domestic discharges special attention may be needed.

Apart from wastewater, extraneous flows, often described as infiltration and inflow, contribute to the total dwf. Infiltration is groundwater entering the collection system, whereas inflow is the sum of a variety of flows, such as drainage water, cooling water, and pumped water from construction sites, in situ soil cleaning facilities or wrong connections.
The infiltration and inflow into sewers can amount to over 50% of the annual dwf volume. German research, analysing 4 years of influent data at 34 treatment plants, revealed that on average annually 35% of the wwtp influent originates from infiltration and inflow, whereas only 30% originates from ‘real’ sewage. Another 35% of the annual inflow is due to storm water runoff [Weiß *et al.* (2002)]. Research in the Netherlands [STOWA (1996), (2003), Schilperoort (2004)] confirms these results. In general, groundwater can be considered to be clean compared to domestic wastewater with respect to important parameters such as COD and Kjeldahl-nitrogen. As a result, the inflow of groundwater results in a dilution of the concentrations of dissolved substances in the dwf.

Storm runoff is determined by the rainfall and the time variant characteristics of the contributing surfaces. Before the rain enters the sewer system, several processes are likely to have changed the runoff pattern [Ven, van de (1989)]:
- wetting losses;
- interception;
- infiltration;
- storage in surface depressions;
- evaporation;
- flow towards the sewer system.

Moreover, during runoff towards the sewer system significant quality changes of the rainwater take place as illustrated by table 2.4.

Table 2.4 Concentration ranges of rain and runoff.

Parameter	rain	roof runoff	road runoff	total runoff
P_{total} (mg P/l)	0.01[a], 0.1[g]	0.3[h]	1.5[h]	
BOD (mg O_2/l)		3 – 42[d]	14- 32[d]	
COD (mg O_2/l)	17-19[g]	12 – 132[b], 5 – 198[d], 22[h]	10 - 235[b], 25 – 171[d], 49[h]	
SS (mg/l)	-	2 – 7[b], 7 – 211[d]	8 – 230[b], 10 – 206[d]	
$N_{Kjeldahl}$ (mg N/l)		0.8 – 8.6[b]	0.2 – 5.2[b]	
NH_4 (mg N/l)	1[a], 1.2[g]	0.3 - 1. 4[c], 4[h]	0.2[h]	1-5[e], 5[f]

[a] yearly average for the Netherlands [Stolk (2001)]
[b] range measured in grab samples on a number of locations in the Netherlands [Oldenkamp and Campen, van (1990)]
[c] range measured in Bayreuth, Germany [Förster (1996)]
[d] event mean concentrations (EMC) [Gromaire-Mertz *et al.* (1998)]
[e] [Ashley and Crabtree (1992)]
[f] [Bertrand-Krajewski *et al.* (1995)]
[g] [Goettle and Krauth (1980)]
[h] Karlruhe/Waldstadt, cited in [Xanthopoulos and Hahn (1994)]

2.3.2 Fluctuations in sewer system outflow: wwtp influent and CSO

The wwtp and the CSO are the main outlets of a combined sewer system. The sewage lost by exfiltration or through surcharged manholes is considered to be beyond the scope of this thesis.

Wwtp influent

Wwtp influent fluctuates at a range of time scales. Table 2.5 gives an overview of the yearly average concentration in Dutch influent from 1980 till 2000. The data show the yearly average concentrations to fluctuate over the years due to changes in the annual flows. The annual loads of all pollutants are quite constant, with the well-known exception of phosphate due to the ban on phosphates in detergents after 1985. The constant level of the influent loads indicate that the long term behaviour of the inhabitants is fairly constant, as both the total number of inhabitants and the connectivity to the wastewater infrastructure sewerage have only changed marginally during the last decade.

Table 2.5 Yearly average concentrations and total loads (between brackets and in 10^6 kg/a) of Dutch influent [CBS (2003)].

parameter	1985	1990	1995	1996	1997	1998	1999	2000
COD (mg O_2/l)	573 (800)	595 (933)	510 (921)	603 (921)	570 (916)	456 (930)	480 (915)	470 (921)
BOD (mg O_2/l)	220 (304)	222 (349)	185 (331)	229 (347)	224 (360)	173 (348)	185 (346)	180 (354)
N_{total} (mg N/l)	53 (70.1)	52 (81.3)	47 (84.0)	55 (82.9)	53 (84.7)	42 (85.6)	45 (85.2)	44 (84.8)
P_{total} (mg P/l)	15 (18.7)	9 (14.4)	8 (13.8)	9 (13.5)	9 (13.6)	7 (13.7)	7 (13.3)	7 (13.3)
Flow (10^9 m^3/a)	1428	1673	1851	1651	1697	2146	2014	2097
Precipitation (mm/a) [KNMI (2003)]	801	765	782	632	686	1109	863	897

In literature often seasonal fluctuations are mentioned [Nielsen and Nielsen (2002)]. A well-known seasonal variation is the difference in dwf during winter and summer periods, as exemplified by figure 2.3. Mostly, these variations can be explained by variations in inflow and infiltration [Brombach *et al.* (2002)].

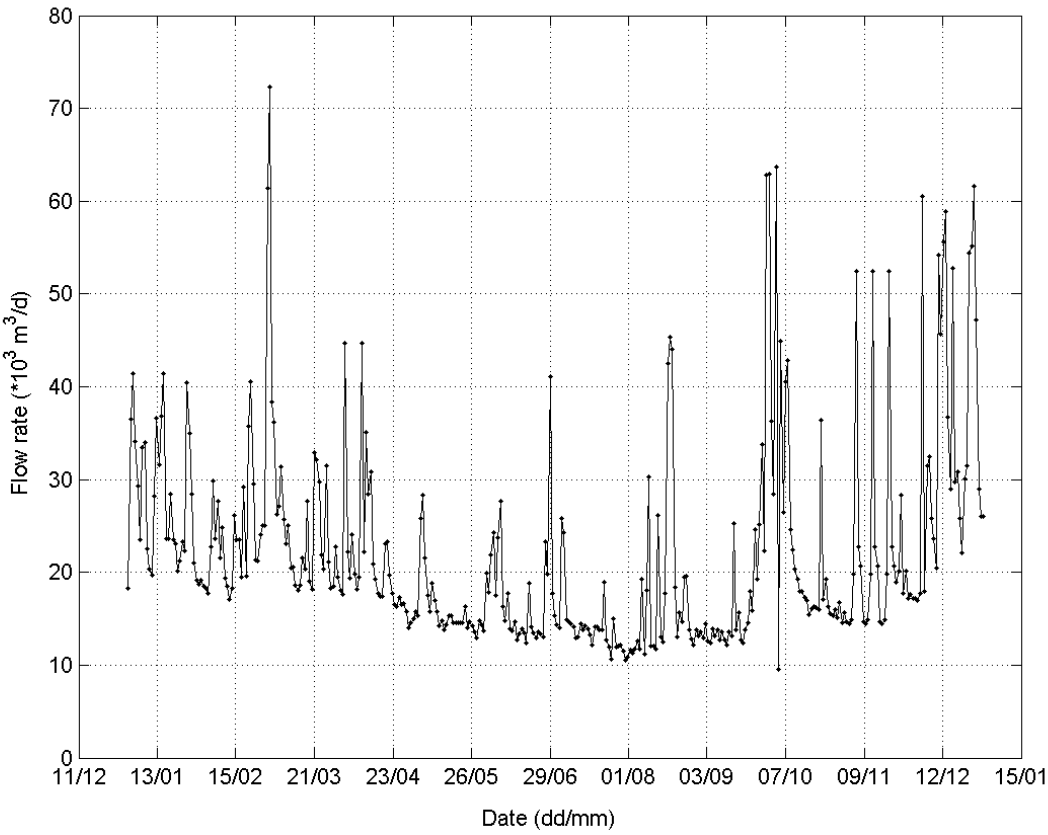


Figure 2.3 A year of daily influent flow rate for wwtp Zaandam [data from: Herbergs (2001)].

Another seasonal variation is the difference in the biodegradability of the wastewater. The ratio between COD and BOD is known to change over the seasons. Moreover, a measuring campaign on 12 wwtps revealed that also the fractionation in terms of biodegradability changes over the seasons [STOWA (1994)]. Apart from changes in biodegradability, which are most likely due to temperature effects, some typical season dependent variations may occur:

- increased salt concentrations due to road de-icing;
- high level of nutrients due to falling blossom or leaves.

Influent usually shows characteristic diurnal patterns during dwf in both composition and flow. Normally, the morning peaks in ammonia and flow do coincide [Krebs *et al.* (1999), Urbaniak (1998)] as a consequence of the diurnal inflow pattern [Friedler *et al.* (1996), Butler *et al.* (1995)]. However, especially settleable pollutants, such as parts of the COD and SS, show significantly different diurnal variations, see figure 2.4. Their diurnal concentration profile not only depends on fluid transport, but rather on in-sewer processes such as sedimentation, re-erosion and transformation.

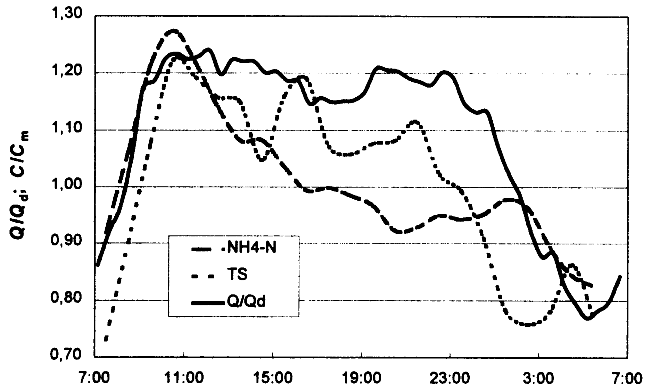


Figure 2.4 Diurnal variations of flow rate, ammonia and TSS concentration normalised with daily averages. Measured at inlet wwtp Dresden, 500.000 p.e. [reproduced with permission from: Krebs *et al.* (1999)].Q = flow, Q_d = mean dwf, C = concentration, C_m = mean concentration

The most significant fluctuations in the wwtp influent, however, occur during wwf. In terms of flow these variations are limited to the installed hydraulic capacity of the wwtp. This capacity is determined by the design philosophy, which varies between countries, as exemplified by table 2.6. In terms of wastewater quality a broad range can be observed, ranging from rather low concentrations due to dilution to increased concentrations due to the release of pollutants available within the sewer system (i.e. sewer sediment and biofilm). Table 2.7 gives an illustration of measured (by 24-hour flow proportional composite samples) fluctuations in influent concentrations during both dwf and wwf.

Table 2.6 Wwtp treatment capacity.

country	biological capacity	hydraulic capacity	remarks
Belgium [Carrette *et al.* (2000)]	$3*Q_{14}$	$6*Q_{14}$	
Germany [ATV (1991)]	$2\ Q_s + Q_f$	$2\ Q_s + Q_f$	Q_s = 85 percentile of daily peak dwf, Q_f = ‘fremdwasser’, annual mean of inflow/infiltration
the Netherlands	$4.5*Q_{10}$	$4.5*Q_{10}$	Q_{10} = hourly flow of 1/10 of total daily dwf. With 60 m^2 of impervious area per person and 120 litre per person and day the Q_{dwf} = 0.2 mm/h. The total installed pumping and treatment capacity equals 0.7 mm/h + Q_{dwf} In situations with significant I/I the Q_{dwf} usually will be somewhat larger than 0.2 mm/h, thereby reducing the dwf-multiple treated at the wwtp.
UK [FWR (1998)]	$3*Q_{dwf}$	$3\text{-}\ 6 * Q_{dwf}$	Q_{dwf} = 24 hour average dry weather flow [Butler and Davies (2000)]

Table 2.7 Fluctuations in influent composition during wwf in mg/l. Please note that the given values are daily averages.

Parameter	wwtp Utrecht 1995-1996	wwtp Katwoude 2000-2002	wwtp Haarlem 1999
COD (mg O_2/l)	132 – 718	114 – 1110	89 – 569
BOD (mg O_2/l)	56 – 285	-	34 –195
$N_{Kjeldahl}$(mg N/l)	8 – 61	14 – 87	8.2 – 40
P_{total}(mg P/l)	2.9 – 8.8	-	1.1 – 8.8

The daily averages illustrate the importance of influent fluctuations. However, as the data given in table 2.7 is the result of a 24 hour composite sample, the influent pollutograph is likely to show much more fluctuation. Figure 2.5 gives an overview of fluctuations measured at the inlet of the wwtp of the city of Dresden (500.000 p.e.). The dissolved fractions (NH_4, PO_4) show significantly different fluctuations compared to the suspended solids TSS.

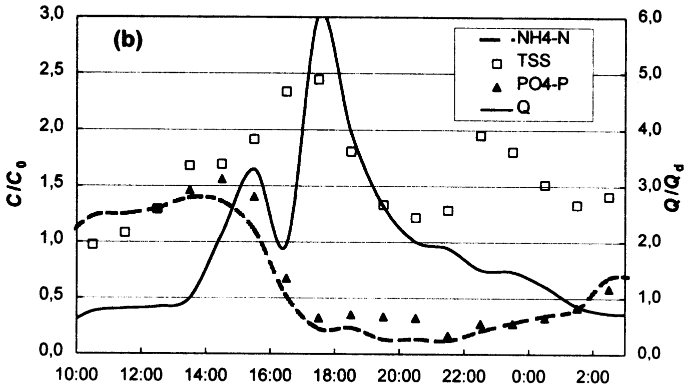


Figure 2.5 Fluctuations during wwf of flow rate, ammonia, TSS and PO_4 concentration normalised with daily averages. [Reproduced with permission from: Krebs *et al.* (1999)].

[Kühn and Gebhard (1998); Bruns (1998) and Krebs *et al.* (1999)] define a number of distinct phases in the influent pollutograph during storm events:

1) increase of flow rate and subsequently an increase of the load arriving at the wwtp due to the 'push' of wastewater with dwf concentration levels. This phase is the more distinct the more wastewater is stored downstream in either large interceptor sewers or rising mains.
2) increased concentration of suspended solids as eroded sewer sediments starts to arrive at the wwtp. These sediments are usually transported with a velocity lower than the fluid velocity [e.g. Bertrand-Krajewski (1993 *et al.*)]
3) arrival of diluted wastewater at the wwtp
4) return to dwf equilibrium. Equilibrium for dissolved compounds will be reached as soon as all remaining storm runoff has been transported (pumped) towards the wwtp. Reaching equilibrium for suspended solids may last longer since it takes time before all depressions within the sewer system are filled again with sediment.

These phases give only a general description of the dynamic changes in the influent during a storm event, as ancillary structures, such as retention tanks, may significantly influence the influent profile during wwf.

CSO

The CSO spill volume varies significantly between storm events. The composition of this volume has been the topic of many studies, although in literature only a limited number of well-described measuring campaigns can be found. The results of these campaigns are given in table 2.8 and show for each location a huge variability in terms of event mean concentrations.

Table 2.8 Range CSO event mean concentrations (mean between brackets).

	Dorp-Oost, Vlist[a]	**Kerkrade**[b]	**UPM data**[c]	**Loenen**[b]	**'Le Marais', Paris**[d]
BOD (mg O_2/l)	9 – 105 (35)	15.0- 232 (74.6)	125	8.9 – 141 (39.9)	67 – 296 (181)
COD (mg O_2/l)	35-600 (160)	60.6 – 725 (243)	390	52.2 – 877 (271)	123 – 736 (428)
$N_{Kjeldahl}$(mg N/l)	5 – 22 (11)	3.8 – 31.7 (13.4)	-	3.3 - 26.3 (10.4)	-
P_{total}(mg P/l)	1 –5.6 (2)	0.9 – 7.5 (3.0)	-	0.9 - 7.2 (2.9)	-
TSS(mg/l)	10 – 660 (105)	56.3 – 1081 (320)	420	20.9 – 1201 (303)	105 – 559 (307)

[a] [WRW *et al.* (1999)]
[b] [Bakker *et al.* (1989)]
[c] flat/ average catchments [Threlfall *et al.* (1991) cited in FWR (1998)]
[d] [Gromaire-Mertz *et al.* (1998)]

Moreover, also during CSO events the concentration of the overflowing wastewater changes considerably. As each storm event is different, in literature many examples can be found of attempts to characterise the course of the event. The most applied method is to check the occurrence of a first flush. Unfortunately, many definitions of first flushes exist:

- based on concentrations:
  - initial concentration peak relative to concentrations during event [Thornton and Saul (1987)];
  - concentration peak relative to baseline concentration [EPA (1993)], quoted by [Bertrand-Krajewski *et al.* (1998)];
- based on mass- volume curves:
  - a first flush is present as soon as the maximum gap between mass – volume curve and bisector as plotted in figure 2.6 is larger than 20% [Geiger (1994)];
  - maximum divergence between cumulative percentage of mass and cumulative percentage of flow plotted against the cumulative percentage of time [Gupta and Saul (1996)];
  - a first flush is present as soon as at least 80% of the total mass is conveyed by 30% of the flow [Saget *et al.* (1996); Bertrand-Krajewski *et al.* (1998)].

An analysis of available data from the NWRW (project [NWRW (1989)] shows Dutch sewer systems not to be sensitive to first flushes according to the Saget definition, see figure 2.6. This result is in line with the results of Bertrand-Krajewski *et al.* (1998)], based on data from the QASTOR database [Saget and Chebbo (1996)] containing 197 rainfall events, where only 1% of the events showed a clear first flush. [Fenz and Nowak (1998)] state, based on data from Germany, Austria and Denmark, that for most combined sewer systems 55 to 70 % of the pollutant mass is to be expected within the first 50% of the volume.
One of the problems, however, in assessing the results of the first flush analysis in literature is the lack of information on measuring set up and data handling. E.g. the Dutch data underlying figure 2.6 have been measured at the CSO. Therefore, the first part of each storm event is not taken into account, since the first millimetres of runoff will not overflow as they are stored within the sewer system.

Nonetheless, the analysis of the first flush itself gives interesting information on the dynamics within sewer systems during storm events. Especially the differences between the first flush behaviour for the various parameters of interest (BOD, COD, TSS, NH_4, PO_4, see figure 2.5), the large variations between catchments and between storm events for the same catchment indicate that a number of different processes occurring within the sewer system affect the pollutograph.

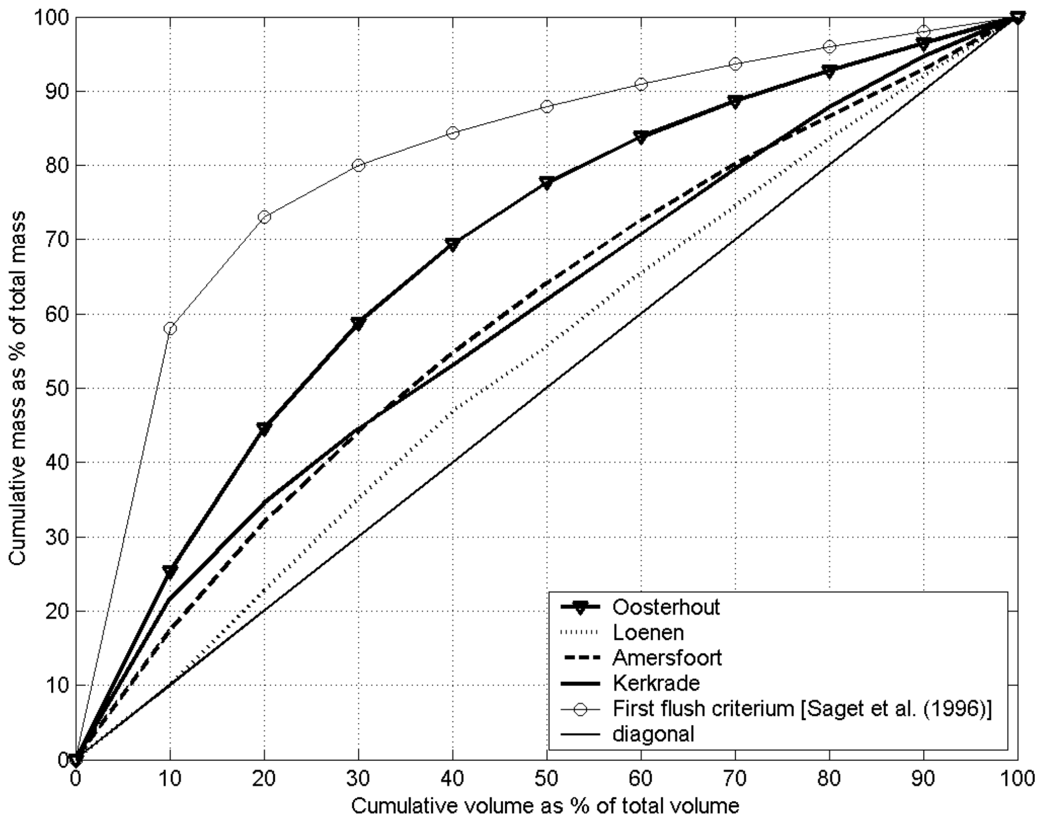


Figure 2.6 Mass – volume curves for COD. Average (volume-weighed) curves for selected (based on reliability and data density in terms of number of samples per event) for 4 catchments from the NWRW project [NWRW (1989)].

2.3.3 Fluctuations in wwtp effluent

The composition of wwtp effluent is known to fluctuate significantly less than the composition of wwtp influent [Urbaniak (1998)]. Table 2.9 gives an overview of the statistics of one year of daily effluent data of wwtp Wervershoof (low loaded wwtp, capacity 219,000 p.e.). The mean effluent concentration complies with today's standards as given in table 1.1, although occasionally high concentrations can be noted. Figure 2.7 illustrates the dynamic fluctuations in wwtp effluent composition for ammonium.

Table 2.9 24 hour average fluctuations in effluent of wwtp Wervershoof, 2000 [Stok (2003)]

	N	**min – max**	**mean ± STD**
flow (m^3/d)	365	16,901 – 145,337	46,133 ± 21,311
COD (mg O_2/l)	60	23.6 – 53.5	39.3 ± 6.8
BOD (mg O_2/l)	6	3.0 – 18.1	3.7 ± 1.3
NH_4 (mg N/l)	257	0.1 – 19.1	2.6 ± 3.1
NO_3 (mg N/l)	257	0.0 – 13.8	3.5 ± 2.2
P_{total} (mg P/l)	-	-	-
TSS (mg/l)	254	-	± 0.4

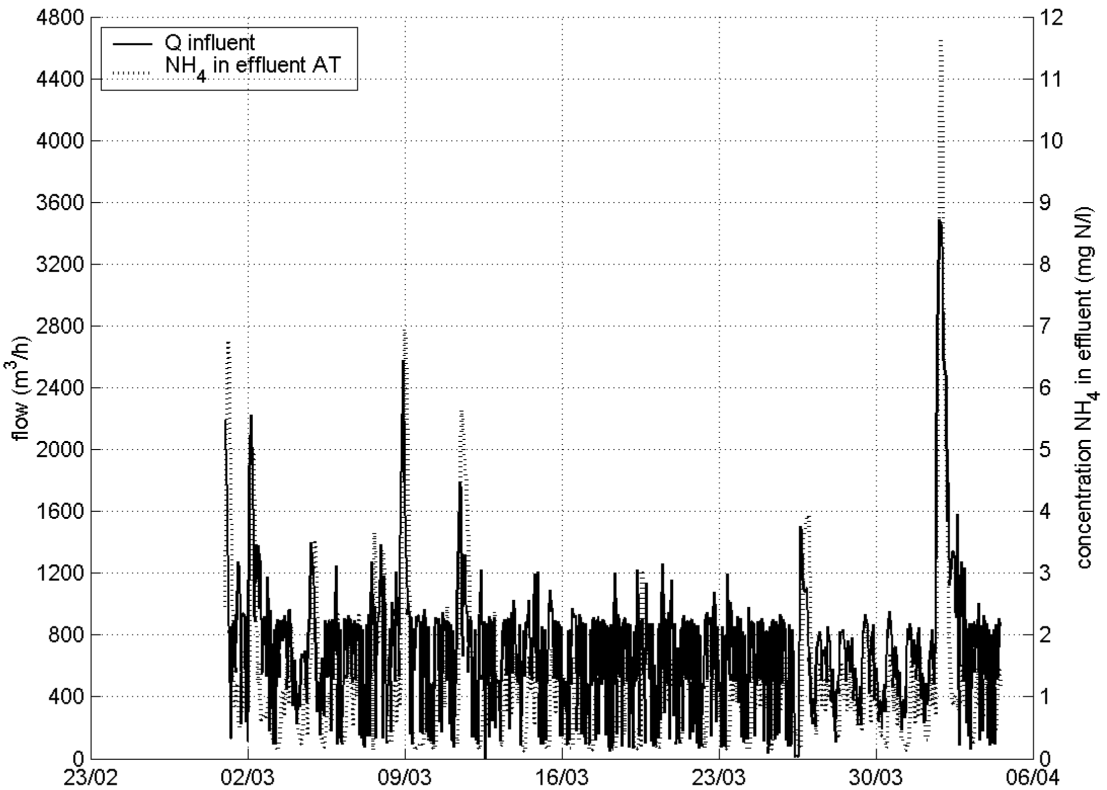


Figure 2.7 Effluent ammonium concentration vs. influent flow, measured at wwtp Hoek van Holland, 28 February to 4 April 2003. Fluctuations in effluent quality for ammonium seem to be correlated to high influent flows as each time the influent flow rises above 1200 m^3/h the ammonium concentration in the effluent rises. Data from [Veldt, van der (2003)].

2.3.4 Characterisation and origin of pollutants

With respect to the interactions within wastewater systems it is necessary to have knowledge of the importance of the numerous processes taking place within the sewer system and the wastewater treatment plant. Ideally, only those processes that have a significant influence on the interactions within wastewater systems should be taken into account when assessing these interactions in e.g. a wastewater system optimisation study.
In order to be able to 'pinpoint' these processes, this section elaborates the characteristics and origins of the most important pollutants as given in table 2.2.

Fractionation of wastewater

Fractionation of wastewater comprises establishing a distribution of pollutants over particle size ranges. Fractionation is sometimes performed at wwtp influent to be able to select the most appropriate (pre-)treatment technique [Nieuwenhuijzen, van (2002)].

Table 2.10 gives the average fractionation of influent measured at 5 Dutch wwtps. With respect to organic compounds these results are consistent with values generally found in literature [e.g. Levine *et al.* (1985)]. For the distribution of phosphorous and nitrogen consistency could not be checked because of a lack of data in literature [Nieuwenhuijzen, van (2002)].

Table 2.10 Average fractionated influent composition as % of total load (44 samples of 5 wwtps, standard deviation in brackets) [Nieuwenhuijzen, van (2002)].

fraction parameter	dissolved (< 0.1 µm)	supra dissolved (0.1–0.45µm)	colloidal (0.45-1.2µm)	supra colloidal (1.2 – 5 µm)	suspended (5 – 63 µm)	settleable (> 63 µm)
TSS	-	-	-	-	52 (±18) %	48 (±18) %
Turbidity	-	-	7 (±4) %	10 (±7) %	62 (±17) %	21 (±12) %
BOD_5	48 (±12) %	-	-	14 (±6) %	30 (±8) %	8 (±4) %
COD	36 (±10) %	3 (±4) %	2 (±2) %	11 (± 6) %	27 (±11) %	21 (± 9) %
N_{total}	83 (±25) %	1 (±1) %	3 (±3) %	4 (±2) %	5 (±3) %	4 (±4) %
P_{total}	53 (±18) %	3 (±3) %	3 (±1) %	5 (±2) %	30 (±12) %	6 (±3) %

During storm events also sewer sediments may be eroded. Within sewer sediment, most of the pollution load is typically associated with the smallest particles, as illustrated by table 2.11.

Table 2.11 Percentage of total particulate pollutant load associated with the different particle size fractions during wwf [Bertrand-Krajewski *et al.* (1993)].

fraction parameter	>1 µm < 50 µm	50–250 µm	> 250 µm
COD	68 %	4 %	28 %
BOD	52 %	20 %	28 %
$N_{Kjeldahl}$	16 %	58 %	26 %

Biodegradability

Apart from the fractionation in terms of particle size fractions the pollutants within the wastewater can be characterised by their biodegradability. The biodegradability of the wastewater changes significantly during transport through the sewer system [Nielsen *et al.* (1992)] and even more during treatment at the wwtp. As a result, the traditional parameters BOD and COD have only a limited value with respect to predicting their pollutant potential within receiving waters [Servais *et al.* (1999)].

In order to be able to determine the biodegradability (or treatability) of the wastewater the total COD can be partitioned into fractions. Normally, the following fractions are distinguished [Henze (1987)]:

- inert matter, consisting of:
  - soluble inert matter
  - suspended inert matter
- biodegradable substrate, consisting of:
  - readily biodegradable substrate
  - slowly biodegradable substrate, consisting of
    - rapidly hydrolysable substrate
    - slowly hydrolysable substrate

These COD fractions may be further partitioned according to specific needs in modelling of transformations of COD [Hvitved-Jacobsen *et al.* (2002)].

Besides, the oxygen uptake rate (OUR) may be used as an indicator for the biological activity, where a high OUR indicates a high biological activity.

Origins of wet weather pollution

The main sources of wet weather pollution in combined sewer systems are:

- runoff
- dry weather sewage
- deposits in sewers
- slime or biofilm on sewer walls

Given the nature and variability of these four sources, their predominance in terms of contribution to wet weather pollution shows a broad range, as illustrated in table 2.12. This is easily understood, as e.g. the relative contribution of dry weather sewage and runoff depends on the storm intensity and magnitude.

Table 2.12 Event based contribution of sewage, runoff and in-sewer stocks to storm water pollutant loads.

source	parameter	sewage	runoff	in-sewer stocks sediments	in-sewer stocks biofilm
Paris – 'Le Marais' [Gromaire *et al.* (2001)] (10 – 90 percentile and (median) of 30 events)	TSS	4 - 43 (21)%	9 – 25 (15) %	40 – 81 (64) %	
	COD	9 – 62 (34)%	10 –29 (15)%	26 – 72 (51) %	
	BOD	11 – 63 (39)%	5 – 12 (7) %	32 – 80 (54) %	
Zürich, [Krejci *et al.* (1987)] (average over 4 events)	TSS	6 %	35%	39 %	20%
	COD	20%	22%	35%	23 %

Unfortunately, in literature not much attention has been paid on the contribution of each source to the nitrogen loads. Given the typical ratios of N/COD of the various sources, the annual contribution of each source can be estimated from the available literature data. The estimated values show the sewage to be the main source for nitrogen during storm events.

Table 2.13 Estimated annual contribution of sewage, runoff and in-sewer stocks to storm water nitrogen loads.

parameter	Sewage	runoff	in-sewer stocks sediments	in-sewer stocks biofilm
COD[a]	59 %	9 %	20 %	11%
$N_{Kjeldahl}/COD_{total}$	0.11[b]	0.04[c]	< 0.01[d]	0.07[e]
estimated $N_{Kjeldahl}$	83 %	5 %	3 %	10 %

[a] annual values [Chebbo *et al.* (2003)]. Division between sediment and biofilm based on [Krejci *et al.* (1987)]
[b] [Graaf, van der *et al.* (1988)]
[c] [Oldenkamp and Campen, van (1990)]
[d] [Arthur and Ashley (1998)]
[e] [Huisman (2001)]

Especially with regards to SS, COD and BOD the in-sewer stocks can be a major source of pollution during storm events. In order to be able to deal with the great diversity in sewer solids [Crabtree (1989)] proposed a sewer solids classification. This classification has three main categories:

- coarse granular mineral deposits in the base of the sewer (type A)
- mobile, fine grained deposits with a high organic content at the water-sediment interface (type C)
- biofilms located on the wall at the area in contact with the sewage (type D)

Generally, type A sediment is predominant in terms of mass, while type C and D are predominant in terms of pollutant potential due to the high concentration of associated pollutants. Nowadays, a lot of attention is being paid to the water-sediment interface, see section 2.4.3.

Each type of sewer solid can be associated with at least one type of transport, thereby complicating attempts to describe the transport of sewer solids.

2.4 Sewer systems

Sewer systems can be considered to be reactors where physical, chemical and biological processes interact between the aqueous, solid and atmospheric phases. Moreover, these processes take place at a broad range of time and space scales. Each process within the sewer system affects to a certain extent the pollutographs at the outlets of the sewer system: the CSO and the wwtp. The extent in which the in-sewer processes affect the pollutographs depends on:

- physical properties of the sewer system; such as dimensions, material and slope of pipes, lay-out of manholes, and pumping stations and special structures. Moreover, the condition of the assets (e.g. growth of roots through open joints, pump failure [Korving (2004)]) themselves may be very important.
- hydrodynamic conditions; determined by the combination of the total inflow of wastewater, groundwater and storm runoff and the physical properties of the sewer system.
- environmental conditions, such as temperature.

In literature, normally a distinction is made into 4 groups of important processes [Ashley *et al.* (1999)]:

- hydrodynamics
- advection-dispersion
- sediment transport
- water quality processes

2.4.1 Hydrodynamics

The hydrodynamics within sewer systems can be described by the well-known 1-dimensional De Saint-Venant equations [De Saint-Venant (1871)]:

Momentum balance:

$$\underbrace{\frac{\partial Q}{\partial t}}_{I}+\underbrace{\frac{\partial}{\partial x}\left[\beta\frac{Q^2}{A}\right]}_{II}+\underbrace{gA\frac{\partial h}{\partial x}}_{III}+\underbrace{c_f\frac{Q|Q|}{R_h A}}_{IV}=0 \qquad \text{(eq. 2.1)}$$

Mass balance:

$$\frac{\partial Q}{\partial x}+\frac{\partial A(h)}{\partial t}=0 \qquad \text{(eq. 2.2)}$$

where:

Q	discharge	(m^3/s)
A	cross-sectional area	(m^2)
B	width of the free water surface	(m)
g	gravitational acceleration	($\approx 9.813\ m/s^2$)
R_h	hydraulic radius	(m)
c_f	resistance constant	(-)
h	water level	(m)
x	location along x-axis	(m)
t	time	(s)
ß	Boussinesq's number	(-)

under the assumption of:
- hydrostatic pressure
- homogeneous and incompressible fluid
- 1-dimensional flow

The De Saint-Venant equations have no known analytic solution and therefore have to be solved numerically. As this is computationally demanding a number of typical simplifications of the equations is often applied, as given in table 2.14. For clarity, the four terms of the momentum balance have been numbered:

I acceleration term
II convective term
III gravitational term
IV friction term

The third term can also be rewritten as:

$$gA\frac{\partial h}{\partial x} = gA\left[\frac{\partial a}{\partial x} + \frac{\partial z_b}{\partial x}\right] = gA\left[\underbrace{\frac{\partial a}{\partial x}}_{IIIa} - \underbrace{i_b}_{IIIb}\right] \qquad \text{(eq. 2.3)}$$

where:

a	water depth	(m)
z_b	bottom level	(m)
i_b	bottom slope	(-)

Term IIIa is the pressure term and term IIIb the gravity term.

Table 2.14 Approximations of the De Saint-Venant equations (after: [Havlik (1996) and Clemens (2001a)].

	dynamic wave	**diffusive wave**	**kinematic wave**
terms of momentum equation taken into account	I + II + III + IV	III + IV	IIIb + IV
account for downstream backwater effects and flow reversal	yes	yes	no
attenuation of flood waves	yes	yes	no
account for flow acceleration	yes	no	no

Table 2.14 can be used to select the most appropriate approximation of the De Saint-Venant equations. However, the hydrodynamic conditions in the sewer system affect almost every in-sewer process. As such, good knowledge of the hydrodynamics is a prerequisite for studying the behaviour of sewer systems.

[Clemens (2001a)] states that current hydrodynamic models are capable of properly describing the hydrodynamics within the sewer system, with an accuracy in the calculated water levels of a few centimetres. This statement has been confirmed by further research, as described in chapter 4.

2.4.2 Advection-dispersion

The advection-dispersion process describes the movement of the pollutants within the aqueous phase through the sewer system. Typically, advection-dispersion involves the transport of dissolved and fine suspended substances within the wastewater, but also of bigger particles whilst eroded.
The advection part describes the carrying along of pollutants with the flow at the average flow velocity, whereas the dispersion describes to what extent the actual transport of

pollutants deviates from the average flow velocity. Most available sewer models, e.g. MOUSE, Hydroworks and SOBEK, also provide modules for advection-dispersion [Bouteligier *et al.* (2002a)], based on the general equation for the 1-dimensional description of the advection-dispersion process in sewer systems:

$$\frac{\partial c}{\partial t} + u\frac{\partial c}{\partial x} - K\frac{\partial^2 c}{\partial x^2} = 0 \qquad \text{(eq. 2.4)}$$

where:

c	concentration	(kg/m^3)
u	flow velocity	(m/s)
t	time	(s)
x	space step	(m)
K	dispersion coefficient	(m^2/s)

The simplification to a 1-dimensional description of the transport of pollutants within the aqueous phase has implications for the quality of the simulation results.
During storm runoff manholes may surcharge resulting in a complex 3-dimensional flow field. Tracer experiments have been carried out under both lab [Guymer and O'Brien (1995); Guymer and O'Brien (2000), Guymer at al. (1998)] and field [Boxall *et al.* (2003)] conditions. These experiments showed that the exchange of pollutants between the flow and the wastewater within the surcharged manhole depends on both the flow and the level of surcharge. The concentration profiles measured in the experiments under surcharged conditions are often significantly skewed (i.e. they have a long tail). As the advection dispersion equation can only predict spatial concentration profiles of Gaussian shape, [Guymer *et al.* (1996)] suggested the use of an aggregated dead zone model, which is more able to describe the measured concentration profiles.
[Pedersen and Mark (1990)] circumvented this problem by introducing the submerged jet theory, which limits the mixing of the flow with the above standing liquid to a certain maximum.

Moreover, [Huisman *et al.* (2000)] showed MOUSE (using the Manning approach) to have a numerical dispersion larger than the actual physical dispersion. As a result, calibration of the dispersion coefficient, even under dwf conditions, proved to be impossible. This result seems to contradict [Garsdal *et al.* (1995)], claiming to be able to determine the dispersion coefficient. However, this discrepancy can be easily explained as [Garsdal *et al.* (1995)] based their conclusions on an assessment of travel times rather than on a concentration profile.

Despite the limitations of the 1-dimensional description of the advection-dispersion process, [Bouteligier *et al.* (2001)] have successfully applied Infoworks to describe the ammonia concentrations during wwf, see figure 2.8. Infoworks uses a simplified version of the advection dispersion equation, taking only advection into account. The physical dispersion is implicitly accounted for by the numerical dispersion and the representation of the manholes as being completely mixed. [Bouteligier *et al.* (2001)] emphasise that Infoworks is capable of properly describing the ammonia concentrations during wwf only as long as the hydrodynamic conditions are well-described and a reliable dwf profile is available.

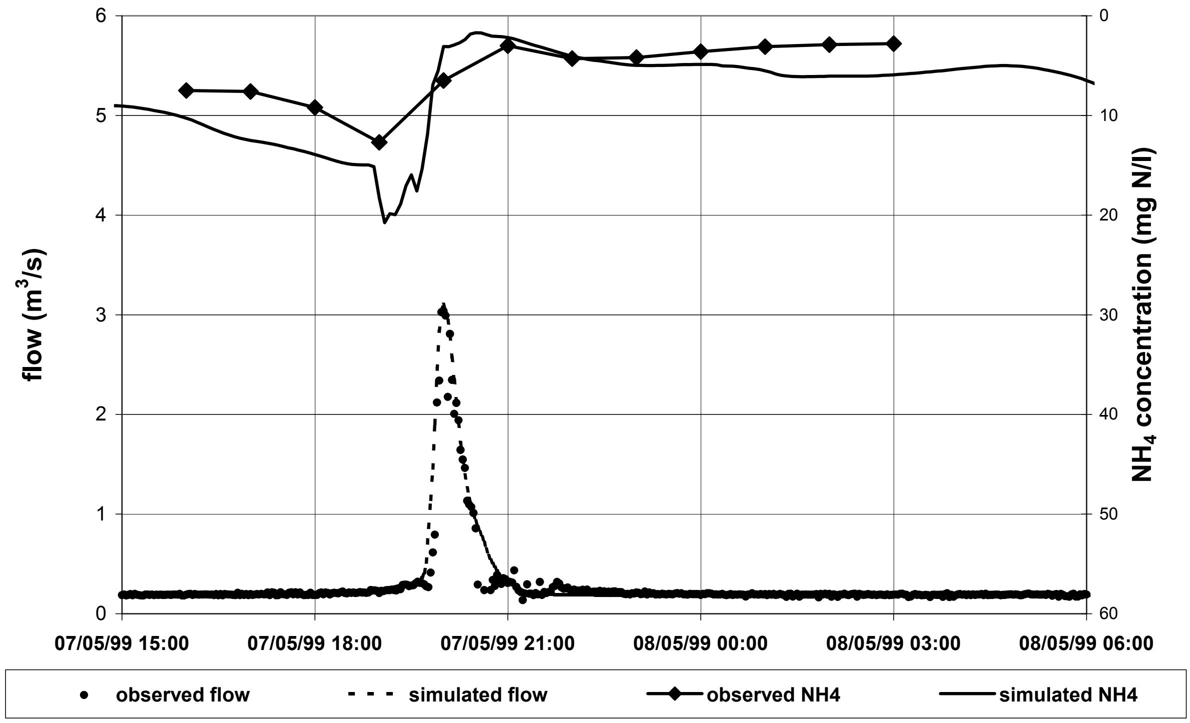


Figure 2.8 Modelling of ammonium during wwf [reproduced with permission Bouteligier *et al.* (2001)].

2.4.3 Sediment transport

Describing sediment transport necessitates knowledge on the deposition, erosion and subsequent transport of sewer sediment and its associated pollutants. The difficulty of field measurements, combined with the extreme temporal and spatial variability, has hampered the development of knowledge on sediment transport [Ashley *et al.* (1999)]. Nonetheless, some knowledge is available, typically related to the distinctive sediment fractions:

- suspended solids
- gross solids
- near bed solids
- granular bed load
- bed deposits
- biofilms.

Each type of sewer solid can be associated with at least one type of transport, thereby complicating attempts to describe the transport of sewer solids.

Suspended solids typically comprise up to 80 – 90% of the total mass of solids transported in the sewer [Ashley *et al.* (1994)]. According to [Ashley and Verbanck (1996)], full suspension will be developed as soon as $\eta \leq 3$, with η given by equation 2.5.

$$\eta = \frac{w_s}{\kappa u_*} \leq 3 \qquad \text{(eq. 2.5)}$$

where

η	sedimentation parameter	(-)
w_s	particle settling velocity, for heterogeneous suspension	(mm/s)
u_*	fluid bed shear velocity	(mm/s)
κ	von Karman's constant, ≈ 0.4	(-)

The suspended solids are transported at a rate in the order of the average flow of the fluid and are subject to dispersion due to spatial sediment concentration variations and velocity gradients. The dispersion of fine suspended solids can be described with the well-known

descriptions usually used for dissolved compounds, as illustrated by [Guymer *et al.* (1998)], for suspended solids with a d_{50} of 47 µm. The boundary between fine and normal suspended solids, however, is unclear. Normally, an upper limit of the particle size of 100 µm for suspended solids is adhered to. Solids larger than 100 µm are supposed to be settleable [Levine *et al.* (1985)].
The suspended solids concentration within the sewer is reported to show a distinct profile, with a sharp increase of the solids concentration near the sediment bed. Especially the concentration profile near the sediment bed is hard to describe. The [Rouse (1937] parabolic profile has been applied with limited success [Ashley and Verbanck (1996)], whereas Verbanck's [Verbanck (2000)] results for a two layer approach (flow divided in an upper 75%, described by the Rouse equation and a lower 25% described by the Coleman equation [Coleman (1982)]) still need to be confirmed for other sewer systems.

Gross solids are of interest with respect to aesthetic pollution [Jefferies (1992)]. Their modes of transport ranges from advection (as long as the water level and velocity are sufficient) to (a mix of) the sliding dam and velocity decrement mechanism of solid movement [Littlewood and Butler (2003)]. [Digman *et al.* (2002)] describe a model with a sophisticated module for calculating the loading of gross solids, which could be used to generate an input for the sewer solids tracker [Schütze *et al.* (2000)].

Near bed solids, i.e. the solids at the water-sediment interface as discussed above, receive a lot of attention nowadays because of the associated pollutant loads. At the moment no less than three types of 'near bed' material have been defined:

- quiescent highly organic material [Oms *et al.* (2002)] deposited in backfall areas in bulk deposits. This material has been observed in Parisian sewers and is also referred to as 'la crème' [Oms *et al.* (2003)].
- near bed solids moving slowly near the bed, as defined by [Arthur *et al.* (1996), Arthur and Ashley (1998)]
- 'fluid sediments' moving in an 'inner suspension' region just above the bed [Verbanck (2000)]

Each type of near bed solid will be eroded at small (< 1 N/m^2) shear stresses, therewith representing an immediate available stock of sediments at the onset of a storm event.

Granular bed load and bed deposits are mainly of interest with respect to blockage of sewers. Especially when bed forms, such as dunes or ripples have been formed, the hydraulic roughness increases dramatically [Kleijwegt (1992)]. Recently, the development of bed forms of real sewer sediment has been measured in controlled experiments in an annular flume [Tait *et al.* (2003), Schellart (2002)].
Apart from the grit removal at the wwtp the granular sediment has no interference with wwtp performance. Moreover, the granular bed load normally is transported by saltation and therefore is not likely to contribute to the discharged CSO loads.

Biofilm consists of active biomass attached to sewer walls, often referred to as 'slimes'. As they are largely organic and are easily eroded, they may contribute significantly to the pollution load during storm runoff, see table 2.12. The biofilms will be discussed in more detail in following section on transformations.

Contemporary sediment transport models do not yet include every sewer sediment fraction as discussed above. The fractions included nowadays are wash load, i.e. the transport of dissolved and fine suspended fractions, suspended load and bed load. Near bed solids are not explicitly accounted for, as as yet it remains unclear whether these should be modelled via bed-load type formulae or via 'near bed' suspension [IWA (2004)].

Not accounting for the near bed solids is not the only limitation of contemporary sewer sediment transport models. [Bouteligier *et al.* (2002a)], [Margetts (2000)] and [Heip *et al.* (1997)] have scrutinised the performance of the water quality models of Wallingford Software (Hydroworks/Infoworks) and DHI Software (Mouse/MouseTRAP). The main results of their work are:

- wash load can be modelled satisfactory with the implemented advection-dispersion equations [Bouteligier *et al.* (2002a)]
- suspended load is modelled by using an advection-dispersion equation for transport and an erosion-deposition criterion, for which in Infoworks the Ackers-White theory is used. [Bouteligier *et al.* (2002a)] state that, as the particles of the suspended load are mainly organic, Ackers-White is used outside of its application range and therefore results should be considered with reservation
- bed load can be modelled in Mouse using one of the four possible sediment transport formulae Engelund-Hansen, Ackers-White, Engelund-Fredsoe-Deigaard and Van Rijn. Infoworks offers only the Ackers-White criterion and models the transport of eroded sediment by advection, which validity can be questioned. Moreover, Infoworks is reported to have no feedback between the quality module and the hydraulic simulator and to limit the deposited sediment depth to be more than 10% of the pipe diameter [Margetts (2000)], although in the latest version this problem seems to have been solved [Bouteligier *et al.* (2002a)]. Mouse, on the other hand, also has limitations, such as the impossibility to simulate the transport of sediment through pumps, problems with negative gradient pipes and, due to the feedback between water quality and hydraulics, excessive computational times [Margetts (2000)].

The main conclusion from the analysis of two main sewer software products is that nowadays it is still not possible to accurately model sewer sediment transport due to a lack of knowledge on sediment transport and software induced errors. It is not to be expected that within short notice the sewer sediment models will come close to the level of accuracy potentially achieved by the hydrodynamic models.

Nonetheless, papers on the apparently successful application of these models are abundant [e.g. Zug *et al.* (2002), David (2002), Masse *et al.* (2001)], illustrating that any formula implemented in software will eventually be used by possibly ignorant users. Moreover, the quality of model results often seems to depend more on the user's qualification, experience and care than on the actual performance of the model [Russ (1999)].

2.4.4 Water quality processes

Wastewater undergoes significant transformations during its transport through the sewer system. 'Fresh' wastewater with an age of a few minutes may be fairly different in terms of composition from wastewater that had been under transportation for 20 hours or more [Nielsen *et al.* (1992), Kaijun *et al.* (1995)]. The transformations take place within the bulk water, within the biofilm or within the sediment under aerobic as well as anaerobic conditions, see figure 2.9.

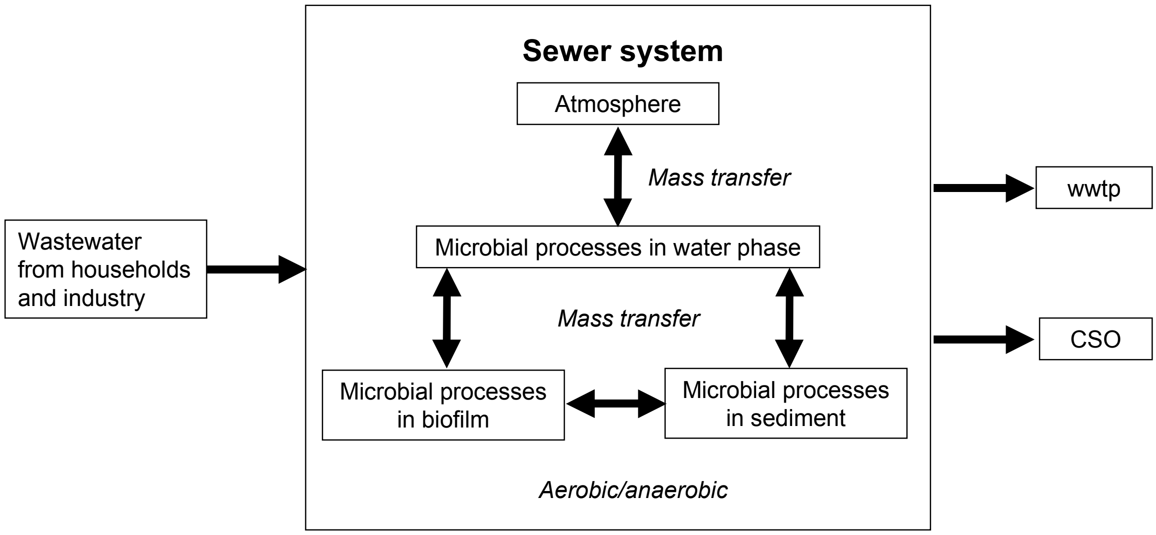


Figure 2.9 Processes within the sewer system (after: [Tanaka (1998)]).

Processes in the water phase related to wastewater transformations are hydrolysis, reaeration and microbial processes. The latter depends to a large extent on the sewage being aerobic or anaerobic. Under aerobic conditions the main processes are [Hvitved-Jacobsen *et al.* (1998a)]:

- growth of heterotrophic biomass
- removal of readily available biodegradable organic fractions
- hydrolysis of biodegradable substrate
- energy requirement for maintenance of heterotrophic biomass

Under anaerobic conditions the main processes involved are [Tanaka (1998)]:

- fermentation of readily biodegradable organic matter resulting in a production of volatile fatty acids (VFAs)
- hydrolysis of biodegradable substrate, although at a much lower rate (15% according to [Tanaka and Hvitved-Jacobsen (1998)]) than under aerobic conditions
- sulphate reduction

Processes in the biofilm may contribute significantly to the overall in-sewer transformations. With respect to aerobic conversions, the biofilm is reported to contribute between 30 to 50% to the overall aerobic conversions [Huisman (2001)]. The processes within the biofilm are controlled by the redox potential. As a result, biofilms typically are stratified as the top layer will be aerobic (provided that the sewage is aerobic) and the lower layers will be anaerobic as all oxygen and nitrate will be consumed. Typical processes related to the biofilm are [Huisman (2001)]:

- growth of fast growing heterotrophic bacteria
- nitrification and denitrification. Nitrifying bacteria play a minor role as heterotrophs will easily overgrow them. Denitrification occurs as long as the sewage contains nitrate, either from drinking water or infiltrating groundwater.
- sulphate reduction. This process becomes significant as soon as all oxygen and nitrate have been consumed.
- detachment of biofilm due to erosion or sloughing. The biofilm may contribute significantly to the active biomass within the bulk water.
- absorption of particles and pollutants.

Microbial processes in sewer sediment correspond to a certain extent with the processes in biofilms in terms of oxygen uptake and exchange of substrate and biomass. This correspondence has been used in models predicting in-sewer transformations, as the sewer sediment has been assumed to show the same biological activity as the biofilm, thereby reducing the complexity of the model [Almeida (1999)]. Nevertheless, as sewer sediment layers normally are significantly thicker than biofilms, the anaerobic activity in the deeper layers is likely to be more important.
On the other hand, sewer sediment is known to ‘age’ [Ristenpart (1995a)]. Older sediment normally has a higher density and lower BOD and COD contents [Ristenpart (1995b)]. This is of particular interest with respect to the pollutant potential of the sewer sediment, which may erode during storm events. Moreover, even the resistance to erosion seems to depend on the microbial processes within the bed, where consolidation and biofilm growth increases and the production of gas (e.g. methane) decreases the bed strength. [Vollertsen (1998)]

The microbial activity taking place within sewer systems has inspired many researchers to think of the sewer system in terms of a reactor rather than a transport medium only. The first incentive to do so were the problems related to the formation of hydrogen sulphide in rising mains. In the 1970’s control of the microbial activity in these mains by injecting oxygen or nitrate has been proposed [Boon and Lister (1975) and Boon *et al.* (1977)]. As a result, the potential of the sewer as a BOD removal reactor was recognised [Pomeroy and Parkhurst (1973)]. Later on, as the wwtp effluent standards could only be met with biological treatment of wastewater, the (pre-)treatment of wastewater within the sewer system became an interesting option. In this respect, the focus has been on two aspects:

- removal of as much COD as possible within the sewer system in order to either reduce the load to be treated by the wwtp or even to make further treatment unnecessary. In order to enhance the COD removal the supply of activated sludge [Koch and Zandi (1973), Green *et al.* (1985); Warith *et al.* (1998)], increased aeration [Koch and Zandi (1973); Mourato *et al.* (2003)] or the dosing of nitrate [Aesoy *et al.* (1998)] have been proposed.
- optimising the COD fractionation for subsequent treatment. Bio-P removal and denitrification at a wwtp requires easily biodegradable substrate. [Hvitved-Jacobsen *et al.* (2002)] tried to control the aeration in the 50 km Emscher interceptor sewer in order to produce as much VFAs as possible while at the same time preventing the production of hydrogen sulphide.

Recent research on in-sewer transformation processes has been dominated by the Activated Sludge Model (ASM) methodology [Henze *et al.* (2000)], which will be described in more detail in section 2.5.3. The ASM model family has been developed for wastewater treatment plants, but the process descriptions within the models have been proven to be applicable to sewer systems as well.
In Denmark research has resulted in the Wastewater Aerobic/anaerobic Transformations in Sewers (WATS) model [Hvitved-Jacobsen *et al.* (1998a)]. This model is based on ASM1 [Henze *et al.* (1987)] and includes aerobic and anaerobic transformations of COD, reaeration and sulphide formation. [Hvitved-Jacobsen *et al.* (2002)]. [Vollertsen (1998)] applied the same methodology to transformations of sewer sediment.
In the UK, [Almeida (1999)] developed a model in analogy of ASM1 and ASM2 [Henze *et al.* (1995)]. Her model, like the WATS model, needed the introduction of a more detailed fractionation of the COD than in the original ASM1 and ASM2 models.
In Switzerland, [Huisman (2001), Huisman and Gujer (2002)] has produced a model for in-sewer dry weather flow transformations based on the ASM3 type model [Gujer *et al.* (2000)]. The ASM3 model has the advantage of being able to take the storage of substrate within bacteria into account. As a result, there is no need to introduce new COD fractions to the original ASM3 model.

2.5 Wastewater treatment

Wastewater treatment plants have to comply with the standards, as described in table 1.1. Within Europe, these standards are of the not-to-exceed type, i.e. at all times the effluent requirements should be met. In the USA, the standards are defined at an acceptable level of compliance, i.e. a fixed number of times per year the standards may be exceeded [Metcalf & Eddy (2003)]. The latter approach takes into account the variability of wastewater treatment. This variability is due to:

- variability in influent wastewater flow rate and characteristics. This type of variability is the key-issue within this thesis and will be discussed thoroughly.
- inherent variability in wastewater treatment processes, mainly due to the presence of living micro-organisms.
- mechanical process reliability. An aspect hardly found in literature on wastewater system analysis, thereby illustrating the distance between scientists and practitioners. An example of the analysis of mechanical process reliability can be found in [Assezat (1989)].

Figure 2.10 gives an overview of a general process layout of a wwtp in the Netherlands. Each component of a wwtp will respond differently to changes in influent flow rate and composition. The main components and relevant processes will be discussed in this section.

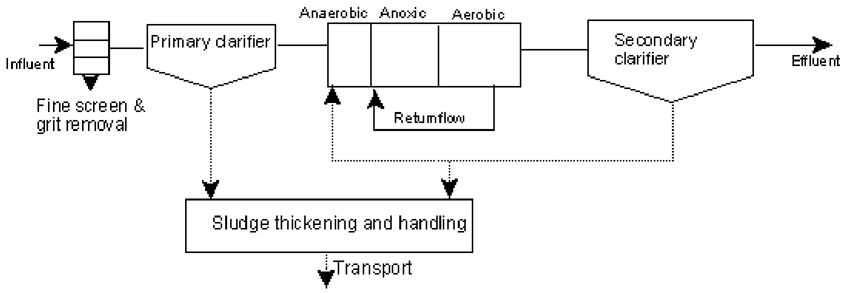


Figure 2.10 General process scheme of a wwtp (after: [Nieuwenhuijzen, van (2002)]).

2.5.1 Screens and grit removal

Screens at treatment plants aim at the removal of gross solids. During storm runoff the amount of gross solids to be removed by screens may be 5 to 10 times the amount normally encountered during dwf. Given the nature of the solids, screens are unsurprisingly the components of the wwtp showing the most failures per year [Wagner (1995)]. However, a failure of the screens will not necessarily lead to a deterioration of the quality of the wwtp effluent.

The same holds true for the grit removal, although an incomplete removal of grit may cause severe operational problems such as deposits in aeration tanks, damage to pumps or clogging of pipes. Especially intense summer showers after relatively long dry periods may introduce a sharp increase of up to 10 times of the total amount of grit to be normally removed [Londong (1990)].

The screens and grit removal are normally not taken into account in wwtp models.

2.5.2 Primary clarifier

The characteristics of the sewage are known to change during wwf. The mineral content increases, resulting in an increased settleability of the sewage and improved thickening of the primary sludge. As long as the sludge handling is able to deal with the increased amount of sludge no problems are to be expected for the subsequent activated sludge stage of the wwtp.

A critical problem with primary clarifiers during wet weather has been addressed by [Harremoës *et al.* (1993)]. The different retention behaviour of water quantities (the flow rate of primary clarifier effluent increases rapidly to wet weather flow rate) and concentrations

(concentrations will stay at dry weather levels for about the retention time of the primary clarifier) in the primary clarifier causes a peak load in the effluent of the primary clarifier. Even for low loaded treatment plants this may result in high concentrations (of especially ammonia) in the final effluent.
As a result, by-passing of primary effluent, as used to be applied often in Germany [Otterpohl and Dohmann (1996)], should only start as soon as the primary effluent is cleaner than the wastewater discharged through the CSO [Reuvers (2000)].

Mathematical models of primary clarifiers are scarce. The most applied model is a simple model approach as proposed by [Otterpohl *et al.* (1994b)]. This model roughly models the buffering behaviour of the primary clarifier and models the settling effect by simply taking a treatment efficiency for particulate compounds into account.
[Gernaey *et al.* (2001)] propose a model based on the Takács settling model [Takács *et al.* (1991)]. Apart from this more advanced settling model their model also includes ammonification. Although the model seems to properly describe the functioning of the primary clarifier, model calibration remains difficult due to a poor model parameter identifiability. Moreover, [Leinweber (2002)] states that the Otterpohl model is sufficient with respect to an analysis of the interactions within wastewater systems.

2.5.3 Activated sludge

The effect of wet weather fluctuations on the performance of the biological stage differs for each of the biological processes taking place.

The *removal of organic matter (BOD/COD)* is driven by both physical-chemical and biological mechanisms, primarily acting on particulate and dissolved organic matter respectively. Therefore, the fractionation of the influent during storm runoff is of importance. However, both the physical-chemical removal mechanisms and the uptake of soluble matter for biomass growth are fast processes when compared to the hydraulic retention time in activated sludge systems. As a result, in medium and low loaded wwtps, as dominantly applied in the Netherlands (figure 1.4), the soluble COD concentration in the effluent will be low and will not be affected by the loading of the wwtp, provided that enough aeration capacity is available.
The effluent load of COD, however, is likely to increase during storm events, as the COD concentration of the effluent will not decrease as much as the flow rate increases. As a result, the treatment efficiency diminishes during storm runoff, even though the wwtp removes more kilograms of COD than during dwf [Otterpohl and Dohmann (1996)].
Generally, the removal of organic matter during wwf poses no problems at a properly functioning wwtp. Therefore, fluctuations in the organic matter load of the influent are not important with respect to the organic matter content of the effluent.

Nitrification is performed by autotrophic bacteria. Their growth rate is an order of magnitude smaller than the fluctuations in ammonium loading occurring during wwf and as a result, the theoretical available nitrifying capacity is set by the autotrophic population at the onset of the storm event. This autotrophic population depends on the sludge loading, which has to be low enough to prevent the autotrophic bacteria from being overgrown by the heterotrophic bacteria. The actual available nitrifying capacity depends on the oxygen concentration, the temperature, the autotrophic bacteria concentration in the aeration tank and finally, the ammonia concentration. The first three aspects may all deteriorate significantly during a storm [Durchschlag *et al.* (1991), (1992)]:

- the oxygen concentration in the aeration tank decreases as soon as the respiration rate is higher than the aeration capacity
- the temperature of the storm runoff (especially in winter time) may be much lower than the dwf temperature

- the activated sludge concentration and subsequently the concentration of autotrophic bacteria in the aeration tanks may easily decrease with 30 % during a storm event due to storage of sludge in the secondary clarifiers.

Moreover, unlike the organic compounds, ammonia can not be stored in the biomass. As a result, as soon as the ammonia loading of the aerobic biological stage exceeds the nitrification capacity, both the load and concentration of ammonia in the effluent increase.
Peaks in the ammonia loading of the aerobic biological stage are often caused by a push-out of wastewater with a normal dwf concentration level from the sewer system [Krebs *et al.* (1999)] and the preceding treatment stages, comprising possibly a primary clarifier, a biological phosphorous removal (bio-P) and a denitrification tank [Otterpohl and Dohmann (1996)].
The nitrification process is very sensitive to fluctuations in the wwtp influent load during wwf. Consequently, many authors (e.g. [Bruns (1999), Müller and Krauth (1998), Bauwens *et al.* (1996)] have appointed the nitrification process as one of the critical wastewater treatment processes.

The *denitrification process* takes place under anoxic conditions and needs easily degradable COD as substrate. Both aspects are affected during wwf. Anoxic conditions may be violated due to an increased oxygen concentration in the wwtp influent, whereas the concentration of easily degradable COD decreases due to dilution and a reduction of hydrolysis of COD within the sewer system. As a result, the nitrate removal capacity decreases and the nitrate concentration of the activated sludge effluent increases. Apart from an increase of the nitrate concentration in the final effluent this imposes a potential danger for the functioning of the secondary clarifier due to the occurrence of denitrification in this clarifier.
Fortunately, unlike nitrification, the denitrification process can be supported by dosing acetate or another easily degradable carbon source. Moreover, the decrease in denitrification capacity due to the arrival of diluted wastewater happens later than the decrease in nitrification capacity, as the former process is not affected by the push-out phase [Bruns (1999)].

The *biological phosphorous removal* takes place under anaerobic conditions under the consumption of VFAs. Like the denitrification process, the bio-P removal may be hampered by the increased oxygen and the decreased VFA concentration in the influent. In addition, the consumption of glycogen during excessive aeration as normally occurring during storm events, is reported to hamper the bio-P removal [Brdanovic *et al.* (1998)]. As a result, the bio-P removal can come to a complete halt during wet weather [Meijer (2004)]. However, phosphorous can also be removed chemically by dosing precipitants. In the Netherlands, bio-P and chemical P removal are often combined. Moreover, with respect to the receiving waters the annual loads rather than peaks during wwf are important. Consequently, the reduction of the bio-P removal during wet weather is not considered to be a significant problem.

The development of the activated sludge models has been enhanced by the matrix notation introduced by the Activated Sludge Model no. 1 (ASM1) [Henze *et al.* (1987)], see appendix II. The systematic approach enabled researchers to easily communicate their models and research results. The main philosophy of the development of the ASM models has been to keep the process descriptions as simple as possible. Only the important processes are taken into account in the models. At the moment four versions of the ASM model family are available [Henze *et al.* (2000)]:

- ASM1, including COD removal, nitrification and denitrification
- ASM2, extending ASM1 with phosphorous removal
- ASM2D, essentially an improvement of ASM2 in terms of the inclusion of denitrifying phosphorous accumulating organisms (PAOs)

- ASM3. A model replacing ASM1. ASM3 also includes storage of organic compounds and separates the heterotrophs and the nitrifiers.

Despite the development of ASM3, ASM1 still remains the model mostly applied. Moreover, a simulation benchmark has been developed [Copp (2000)] in order to be able to judge whether the implementation of the ASM1 model into computer code has been performed properly. Copp [Copp (2000)] describes how eight (partly commercially available) models (e.g. WEST®, Simba®, GPS-X™) have been compared. The benchmark showed that the models almost all performed equally well in terms of simulation results.
The ASM models have, since their introduction, been widely applied by researchers and practitioners. Literature shows that the ASM models are capable of properly describing the performance of full scale wwtps [Veldhuizen, van *et al.* (1999a, 1999b), Meijer *et al.* (2001), Brdjanovic *et al.* (2000), Seggelke and Rosenwinkel (2002)].

2.5.4 Secondary clarifier

The secondary, or final, clarifier has to separate the activated sludge from the treated wastewater by settling. Within a secondary clarifier, a number of settling processes take place, ranging from free settling, flocculent settling, hindered settling to thickening. The performance of the secondary clarifier is a function of the loading and the sludge characteristics, e.g. the sludge volume index (SVI). During wet weather the volumetric load of activated sludge to the secondary clarifier increases. This higher loading causes storage of activated sludge within the clarifier, a rise of the level of the sludge blanket and possibly an increase in the suspended solids (and associated pollutants!) concentration in the final effluent.

The performance of the secondary clarifier is generally considered to be one of the key processes in the wwtp [Bruns (1999); Harremoës *et al.* (1993)] with the hydraulic loading as the most important driver. Given their importance, modelling of secondary clarifiers has attracted a lot of attention from researchers. However, the 'ideal overall' model is still lacking and available approaches range from simple 0-D completely mixed reactors [Freund *et al.* (1993)] to advanced 2-D models [Lakehal *et al.* (1999)]. With respect to the modelling of the effect of transient conditions, appropriate models have to be able to describe:
- concentration of suspended solids in effluent
- concentration of solids in return sludge
- storage of activated sludge within the secondary clarifier.

As the 0-D models are not capable of properly describing the storage of sludge within the clarifier and the 2-D models are computationally demanding, 1-D 'layer' models are dominantly applied [Leinweber (2002)].
The 1-D model of Otterpohl and Freund [Otterpohl and Freund (1992)] assumes a division into macroflocs and microflocs, with each having its own settling characteristics. The model allows for an easy parameterisation, as only the SVI, the estimated solids concentration in the effluent and the maximum amount of microflocs can be set.
The 1-D model of Takács [Takács *et al.* (1991)] is more complex, but allows for a better fit to specific problems, provided that enough data are available on settling velocities and suspended solids concentrations in the effluent.

2.5.5 Advanced treatment

Advanced treatment of wwtp effluent is considered to be the next step in wastewater treatment. The aim of advanced treatment could be as diverse as an improved effluent, improved overall sustainability or more specifically reuse of wastewater [Graaf, van der (2001)]. The performance of the advanced treatment steps depends on the performance of the wwtp rather than on the wwtp influent. Moreover, especially for reuse options, only a fixed proportion of the effluent will be subjected to advanced treatment.

With respect to the interactions within wastewater systems, advanced treatment is considered of minor interest.

2.6 Integrated wastewater system analysis

Integrated wastewater system analysis has received a lot of attention since the beginning of the 1990s (e.g. INTERURBA I [Lijklema *et al.* (1993)] and Sewage into 2000 [Kruize (1993)]. Nowadays, with respect to integrated wastewater system analysis two approaches have become dominant: the emission and the immission based approach. Within the emissions based approach wastewater system performance is assessed by the total emissions from sewer systems and wwtps. The immissions based approach focuses on the receiving water quality. However, irrespective of the selected approach, integrated wastewater system analysis necessitates knowledge of the dynamic behaviour of sewer systems, wwtps and their interactions. This section deals with the two types of interactions within wastewater systems as defined in line with the definition given in the UPM [FWR (1998)]:

- both the discharges from sewer system and wwtp are important with respect to either the total emissions from the wastewater system or the overall impact on receiving waters. The interactions in this respect come down to assessing which part of the wastewater system (sewer or wwtp) should preferably be improved in order to be able to comply with the requirements.
- physical interactions: the properties of and processes within the sewer system have a direct impact on the influent fluctuations in terms of flow and composition, whereas the actual hydraulic capacity of the wwtp affects in-sewer processes.

2.6.1 Integrated assessment

Although it is obvious that, as illustrated by table 2.2, both the sewer system through the CSOs and the wwtp via its effluent contribute to the main water quality problems, a number of difficulties exists with respect to the combined assessment of the two components of the wastewater system:

Point of discharge

In the Netherlands, but also elsewhere, many CSOs are located at the outskirts of the urban areas and discharge into local receiving waters. The wwtps are normally located outside of the urban areas and discharge into other, and typically larger, streams. As a result, the *total* emissions from the wastewater system have no relation whatsoever with the *local* receiving water quality. [Geerse and Lobbrecht (2002)] proposed a weighting methodology based on the specific sensitivity of the receiving waters. It could be argued that an immission based approach would solve this problem, as within the immission based approach requirements for the quality of each receiving water could be set and evaluated. However, also in this case it has to be decided for which stream which requirements may be breached first, therewith also necessitating a kind of weighting procedure [Matos *et al.* (2003), Rijsberman *et al.* (2001), Lundin *et al.* (1999), Vleuten-Balkema, van der (2003)]. As this weighting has to be the result of a political decision process, it is considered to be beyond the scope of this thesis to further elaborate on this topic.

Selection of parameters

The total emissions from the wastewater system have been discussed by a number of authors, resulting in the following parameters considered to be important:

- BOD. An early example of the total emissions concept has been given by [Huiswaard (1976)]. He theoretically calculated the yearly BOD load discharged through CSOs and wwtp and tried to minimise the total emissions at minimal annual costs. BOD has also been selected as total emissions indicator by [Durchschlag (1990)]
- COD. The German ‘Gesammtemissionsgruppe’ introduced the annual COD load as a parameter for the assessment of the total emissions from the wastewater system in order to assess the effects of building additional storage [Durchschlag *et al.* (1991,

1992)]. Within the total emissions approach, the COD loads from the sewer system and the wwtp are simply added. The COD discharged with the effluent of a wwtp, has entirely different characteristics in terms of biodegradability than the COD discharged through CSOs [Servais *et al.* (1999)]. Consequently, the beneficial effects of reducing the total COD load from the wastewater system still remain unclear.

- Ammonium. [Holzer and Krebs (1998) and Müller and Krauth (1998)] assessed the performance of a wastewater system by the total load of ammonia discharged per storm event. Ammonium has a number of advantages compared to COD and BOD. As it is a dissolved compound, it circumvents the problems with sewer sediment transport modelling. Moreover, ammonium, as it is closely interrelated with unionised ammonia, which is toxic to fish, is an indicator for acute receiving water quality problems. [Jack (1999), Jack *et al.* (1999)] applied both ammonium and BOD per storm event as parameter, revealing that the effect of building additional storage on the emissions differs strongly per parameter.
- Kjeldahl nitrogen. [Guderian *et al.* (1998)] used the daily kjeldahl nitrogen and COD load to assess the optimum flow to be treated at the wwtp. Like [Jack (1999)], they found the optimum to depend on the parameter selected.
- Total nitrogen. [Harremoës and Rauch (1996)] applied an analysis of the total emissions of the annual emissions of total nitrogen in order to be able to assess the accumulative pollution.

It is remarkable that with respect to the total emissions not much attention is paid in literature to parameters like phosphate, micropollutants and pathogens.

Assessment period
The aforementioned parameters are each assessed at a certain time frame. The early studies on total emissions focussed on annual loads [Durchschlag *et al.* (1991)]. More recent approaches utilise daily or event based loads. Ideally, the assessment period would reflect both the dynamics of the wastewater system studied and the time scales of the relevant processes within the receiving waters, as described in section 2.2.
[Jack (1999)] defines a Total Emissions Analysis Period (TEAP), which lasts from the onset of the storm event until full recovery of the wwtp. Even for moderate storms the TEAP may accordingly amount to 7 days, therewith likely overlapping subsequent storm events. As a result, application of the TEAP may result in having to calculate complete rain series.
[Schütze *et al.* (2002)] addressed the problem of selecting rain data for integrated modelling options. Ideally, continuous simulations should be performed in order to gain maximum insight in the statistics of wastewater system performance and possibly receiving water quality aspects. E.g. the Fundamental Intermittent Standards as described in the UPM manual [FWR (1998)] prescribe the evaluation of receiving water quality in terms of concentration/duration thresholds not to be breached with a certain frequency.
Today's available integrated models, however, are still too computationally demanding to be effectively applied for continuous simulation [Rauch *et al.* (2002)]. Some authors have solved this perceived problem with the development of simplified models (e.g. [Meirlaen *et al.* (2002), Willems and Berlamont (2002)]. Instead, [Schütze *et al.* (2002)] propose to reduce the calculation times by eliminating periods from the rain series unlikely contributing to the studied impacts. However, like [Jack (1999)], [Schütze *et al.* (2002)] concluded that due to the potentially long recovery periods of wwtps and receiving waters, the total length of the periods that can be eliminated is rather limited.

2.6.2 Integrated modelling of wastewater systems
Integrated models usually rely on linking individual submodels for the sewer system, wwtp and receiving waters respectively, therewith implicitly taking the physical interactions between the subsystems into account. All but one (WEST [Meirlaen *et al.* (2001)], a platform for fast, simplified integrated modelling) of the examples of integrated model packages given in table 1.5 have been developed by combining or linking available components. Therefore, it

can be questioned whether the interactions between the sewer system and the wwtp have been properly accounted for, as the individual model components have not been designed with the purpose of addressing the interactions in the wastewater system.

The individual components of integrated models can, basically, be linked sequentially or simultaneously. Sequential modelling suffices as long as no information from downstream (given the normal direction of wastewater flows) model components is necessary to give reliable results. Examples of sequential modelling are given by [Bauwens *et al.* (1996), Fronteau *et al.* (1996) and Seggelke and Rosenwinkel (2002)]. As soon as downstream information is essential, simultaneous modelling has to be opted for. Generally, simultaneous simulation will be needed:

- when applying integrated RTC (real time control), e.g. the wwtp hydraulic capacity is adjusted based on the level of the sludge blanket in the secondary clarifier;
- as a result of physical interactions between upstream and downstream components of the wastewater system, e.g. the water level of the receiving waters affecting the water level in the sewer.

As such, simultaneous modelling has been the aim of many developers of integrated model packages. The main problems encountered during this development are:

- incompatibility of state variables, e.g. BOD has been applied within sewer models like MOUSETRAP [Garsdal *et al.* (1995)] and river water quality models [Rauch *et al.* (1998)], whereas the ASM models use COD [Henze *et al.* (1987)]
- large differences in dominant time and space scales, e.g. spilling periods during CSO events may have a characteristic time scale of 50-100 seconds [Clemens (2001a)], whereas the characteristic time scale of the growth rate of autotrophic bacteria amounts to 3 days (based on their growth rate at 10°C [Henze *et al.* (1987)])
- large differences in complexity of submodels
- practical software and interface problems [Schütze (1998), Pfister *et al.* (1998)]

Various authors tried to overcome the perceived problems of integrated modelling. The most significant problem, the incompatibility of state variables has seen a number of solutions:

- assume values for variables not taken into account in the upstream subsystem
- introduce conversion factors, preferably based on known relations [Schütze (1998)]
- adjust both sewer and river models for compatibility with the ASM models. ASM-based sewer models have been developed by [Almeida (1999), Hvitved-Jacobsen *et al.* (1998a)]. An ASM compatible river model has been developed by an IAWQ taskgroup [Shanahan *et al.* (2001), Reichert *et al.* (2001) and Vanrolleghem *et al.* (2001)]. [Huisman *et al.* (2003)] present an integral and unified model for the sewer and wwtp based on ASM3.

An EU COST working group discussed the requirements for integrated wastewater models and proposed a problem oriented model selection in order to overcome the difficulties due to large differences in complexity of submodels [Rauch *et al.* (1998)]. Table 2.15 gives an overview of the minimum requirements for problem oriented integrated models. However, [Rauch *et al.* (1998)] did not discuss the minimum quality of the results obtained with the integrated models complying with the requirements of table 2.15 in terms of reliability and uncertainty.

Table 2.15 Minimum requirements for integrated models with respect to the assessment of three receiving water problems (after [Rauch *et al.* (1998)]).

Water quality problem		sewer system	wwtp	river
toxic peak loads (NH_3)	processes	rainfall-runoff, hydrodynamics, advection/dispersion	transport, mixing, nitrification	mixing
	state variables	N_{tot} (= NH_4 in worst case)	NH_4, autotrophic bacteria	NH_4, pH (measured)
hygienic impact (Faecal Coliforms, FC)	processes	rainfall-runoff, hydrologic analogy, mixing	-	transport, mixing, decay
	state variables	FC	$FC_{effluent}$ = constant	FC
oxygen depletion	processes	rainfall-runoff, hydrologic analogy, mixing, sedimentation in storm settling tank	transport, mixing, conversion, sedimentation in secondary clarifier	transport, mixing, conversion, reaeration, sediment oxygen demand
	state variables	COD, BOD	COD-fractions	BOD-fractions, DO

[Leinweber (2002)] has studied the requirements for models to be applied within the integrated approach of wastewater systems. Unlike many authors, Leinweber systematically discussed the necessity of integrated modelling, as well as the necessary model complexity of the submodels, for a number of water quality problems. As such, [Leinweber (2002)] also adopted the problem oriented approach. [Leinweber (2002)] concludes that the level of complexity of the models applied and the need for integrated models depends on both the water quality problem studied and the wastewater system characteristics. However, no methodology is proven to be able to select the most appropriate modelling tools in order to be able to take the interactions within wastewater systems into account.

Both [Rauch *et al.* (1998)] and [Leinweber (2002)] illustrated that the problem oriented approach has a potential for selecting most appropriate models, i.e. the least complex model capable of answering the questions raised. However, none of the developers of the integrated physically based models given in table 1.5 have adopted this approach. As a result, the available integrated models are rather complex and require a lot of data to be able to properly calibrate the model [Vanrolleghem *et al.* (1999)]. Consequently, successful practical applications of integrated models are rather scarce [Rauch *et al.* (2002)], although applications for research, such as the model predictive control as studied by [Seggelke (2002)], exist.

2.7 Wastewater system optimisation

Wastewater system optimisation studies can be driven by many motives: non-compliance with the standards, cost effectiveness or emerging technologies. Moreover, as each wastewater system is unique, each optimisation study will have to be designed accordingly.
However, optimisation studies always follow the same basic principle of *evaluating* the performance of a number of *alternatives* against *criteria*.

- *criteria* can be expressed in terms of wastewater system performance indicators, e.g. translated into emission standards, such as the annual CSO volume, COD load or ammonium concentrations or costs, such as whole life costs [Cashman (2002)] or investment costs. In this respect, the criteria determine the scope of an optimisation study. Normally, only a few criteria, such as costs and pollutant loads are taken into account, although optimisation could also involve social, organisational and political aspects. It has to be stated that optimisation results are always to be assessed in relation with the criteria taken into account.
- *alternatives*, such as installing additional storage or pumping capacity, can be predefined or developed during the optimisation study
- *evaluating* necessitates tools capable of comparing the alternatives against the criteria.

Table 2.16 gives an overview of the features of a number of optimisation studies, illustrating the broad range of criteria, alternatives and evaluation tools. One striking feature of the highlighted optimisation studies is the fact that financial aspects have not been accounted for. Apparently, the focus has been on improved understanding of wastewater system performance or ‘proof of principle’ rather than practical application of the ‘optimal’ solution.

Table 2.16 Overview of optimisation studies.

Authors	criteria for assessment of performance	evaluated measures	evaluation tool
Durchschlag *et al.* (1991)	annual COD load	additional storage capacity	KOSIM and own wwtp model
Erbe *et al.* (2002b)	total COD and NH_4 load in reference period	activate in-sewer storage capacity, increase hydraulic loading of wwtp	PLASKI (surface runoff), SIMBA®sewer (sewer) and SIMBA® (wwtp)
Frehmann *et al.* (2002)	DO, NH_4 and COD concentration in receiving waters	RTC, reduce contributing impervious area, water saving measures	MOSI (surface runoff), SIMBA®sewer (sewer), SIMBA® (wwtp) and AQUASIM (river)
Guderian *et al.* (1998)	daily COD load, daily N_{Kj} load	hydraulic loading of wwtp	GEMINI (integrated model based on MWSIM (sewer) and ASM1 model
Jack and Ashley (2002)	total COD and NH_4 load in reference period	additional storage capacity	HydroworksQM (sewer) and GPS-X (wwtp)
Milina *et al.* (1999)	COD, P_{tot}, suspended solids and Faecal Coliforms	separation of storm runoff, increase pumping capacity, reduction of infiltration and inflow, RTC	PLASKI (surface runoff), SIMBA®sewer (sewer) and SIMBA® (wwtp)
Bruns (1999)	NH_4 concentration in wwtp effluent sludge blanket level in secondary clarifier of wwtp	RTC of hydraulic loading of wwtp (within practical limits)	own wwtp model

Moreover, in each study displayed in table 2.16 only a limited number of alternatives have been evaluated. However, wastewater systems often offer many opportunities for improvement. [Gill *et al.* (2001)] illustrate for a simple wastewater system, consisting of 4 catchments with 1 CSO, discharging to 1 centralised wwtp, that basically millions of combinations of extended storage and increased hydraulic capacity of the interceptors are possible. In engineering practice, for practical reasons only a number of all these options are normally evaluated.
However, as wastewater system optimisation studies all follow the same principle, automation of the laborious task of trying to find the optimal solution out of all possible options has been proposed recently [Gill *et al.* (2001)]. Genetic algorithms, or more general, heuristic methods, have been successfully applied to all sorts of combinatory problems. Within the field of urban drainage, they have been applied to model calibration [Veltri and Pecora (1999), Clemens (2001a)] and optimisation of RTC rules [Rauch and Harremoës (1999a)].
[Gill *et al.* (2001)] have applied a genetic algorithm (GA) for the optimisation of a wastewater system. SIMPOL, a simplified (spreadsheet) model described in the UPM manual [FWR (1998)] has been used as evaluation tool since it allows fast simulation. The criteria for performance of the wastewater system have been selected according to the UPM: BOD, ammonium and unionised ammonia concentrations in the river. In order to be able to compare the alternative solutions unambiguously, non-compliance with the standards has been capitalised by penalty costs. The value of the penalty costs, however, still lacks profound reasoning.

The literature on the optimisation of wastewater systems shows the potential of the application of heuristic methods, provided that the performance of the wastewater system can be properly evaluated. However, it would be worthwhile to search for alternatives for the somewhat subjective capitalising of environmental impacts.

2.8 Discussion and topics for further research

Section 2.3.3 showed that the performance of wwtps in terms of effluent quality varies in time. Furthermore, these fluctuations in effluent quality can be attributed to fluctuations in the influent flow and concentration [Leinweber (2002), Urbaniak (1998)]. As such, it was concluded that the performance of sewer systems indirectly affects wwtp effluent quality and that the interactions within wastewater systems are important with respect to wastewater system performance. The extent to which wwtp effluent quality is affected by influent fluctuations, however, is not quantified in literature. Consequently, it is not clear to what extent in-sewer processes, affecting fluctuations in wwtp influent flow and concentration, are important for wwtp performance.

The problem oriented approach, proposed by [Rauch *et al.* (1998)] for the development of integrated models, is a promising approach for setting requirements for sewer process models. The general idea behind this approach is that downstream wastewater system model components determines the necessary level of detail of upstream wastewater system model components, while keeping the models as uncomplicated as possible. This approach has recently been adopted by [Leinweber (2002)], who thoroughly discussed the requirements for integrated modelling of sewer systems and wwtps, although without quantifying them.

Within this thesis, the problem oriented approach was applied in order to be able to fulfil the research objective as given in chapter 1:

…identifying the possibilities to extend today's Dutch volume based approach for wastewater system optimisation to a water quality based approach by taking the dynamic interactions within wastewater systems into account…

The research, described in the following chapters, comprises:

- quantifying to what extent influent fluctuations, in terms of flow and wastewater quality parameters, have a significant effect on WWTP performance. This quantification is based on a fully calibrated and validated ASM1 model;
- deriving minimum requirements for sewer process models;
- analysing the extent in which current sewer process models are capable of meeting these requirements.

The result of this analysis is a description of the requirements for models to take the dynamic interactions into account when applying a water quality based wastewater system optimisation study.

In order to enhance the introduction of the water quality based wastewater system optimisation in the Netherlands, a procedure for wastewater system optimisation as well as optimisation techniques is developed, as the state of the art revealed that insufficient tools are available. In addition, the importance of wastewater system characteristics and wastewater system performance indicators in wastewater system optimisation studies is addressed.

Chapter 3 Wastewater treatment and influent fluctuations

3.1 Introduction

One of the conclusions of chapter 2 is that knowledge of the fluctuations in the influent in terms of flow and composition is essential to be able to predict wwtp performance in terms of effluent quality. The sensitivity of wwtp performance to fluctuations in the influent can be assessed by either measured data or simulations with a dynamic model. In literature, the latter is widely applied (e.g. [Leinweber (2002), Bruns (1999)]), whereas the former is often impossible due to a lack of data. Within this chapter both approaches were adopted in order to discuss the extent in which fluctuations in the influent are important to wwtp performance. Subsequently, the sensitivity of wwtp performance to fluctuations in the influent is used in a problem oriented approach to derive minimum requirements for sewer process models.

3.2 Material and methods

The performance of wwtps is affected by fluctuations in the influent, as described in chapter 2. In order to identify the extent in which influent fluctuations exert a significant influence on the effluent quality, the effect of influent fluctuations was analysed based on both measurement data and model simulations for a representative wwtp.
This section describes the selection of the parameters used in the analyses as well as the selection of the representative wwtp.

3.2.1 Selection of parameters and processes

The performance of a wwtp in terms of effluent quality is normally assessed by a limited number of parameters, listed in table 3.1. The sensitivity of these effluent parameters to fluctuations in influent flow and composition depends to a large extent on the dominant processes involved in the removal of these substances. Table 3.1 summarises the dominant physical-chemical and biological processes affecting the effluent quality.

Table 3.1 Dominant removal mechanisms at a wwtp (PC = primary clarifier, AS = activated sludge system, SC = secondary clarifier).

Parameter in effluent	Involved processes	primary process driver in influent	dominant time scale
COD/BOD	PC: settling	flow	PC: hydraulic retention time (hours)
	AS: adsorption and oxidation	COD/BOD load	AS: 1/growth rate (hours)
	SC: settling, thickening	Flow	SC: hydraulic retention time (hours)
NH_4	AS: nitrification	N_{Kj} – NH_4 load	1/growth rate (day)
NO_3	AS, SC denitrification	N_{Kj} – NH_4 load $COD_{biodegradable}$ load	1/growth rate (hours)
PO_4	PC: settling	flow	PC: hydraulic retention time (hours)
	AS: bio-P-removal	P_{tot}, VFA load	AS: 1/growth rate, uptake rate (day)
	chemical P-removal (precipitation)	P_{tot} load	reaction time (minutes)
	SC: settling, thickening	Flow	SC: hydraulic retention time (hours)
SS	SC: settling, thickening	Flow	hydraulic retention time (hours)

The selection of the parameters and associated processes to be taken into account within this chapter is based on the criterion of additional information on the sensitivity of the effluent quality to *qualitative* fluctuations of the wwtp influent.

COD removal at a wwtp is a rather robust process. For wide ranges of influent loads, the wwtp usually produces high quality effluent with respect to COD [Urbaniak (1998); Leinweber (2002)]. The same phenomenon was observed in data from Dutch wwtps, as illustrated for the low loaded wwtp ‘Katwoude’ in figure 3.1. Even influent loads of COD of up to 3 times the average dwf loading do not affect the effluent concentration of COD or BOD. In addition, the COD and BOD concentration in the effluent is not sensitive to fluctuations in the influent flow, as illustrated in figure 3.2. Consequently, the COD concentration in the effluent does not give information on the sensitivity of COD removal to fluctuations in the COD load and flow of the influent. Therefore, COD removal was not studied in detail.

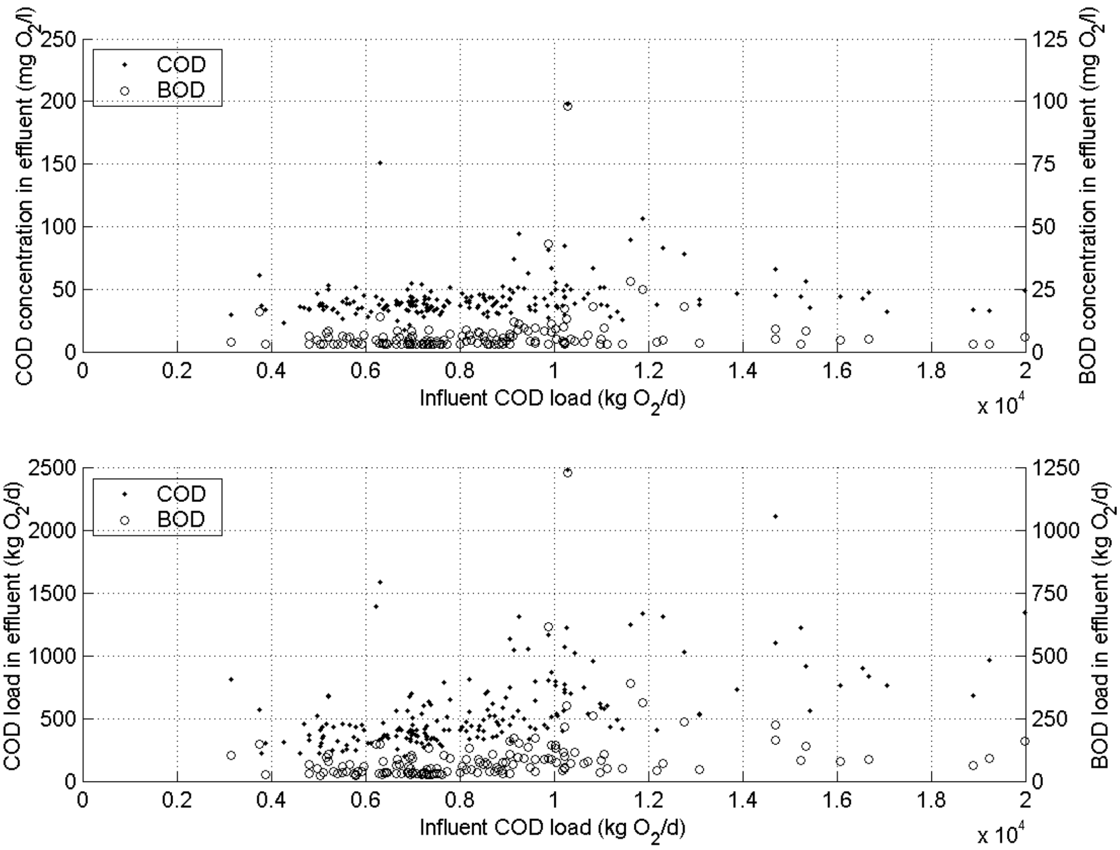


Figure 3.1 Influent COD load vs. effluent COD and BOD concentrations (upper graph) and COD and BOD load (lower graph). Wwtp Katwoude. Average dwf COD loading is 6815 kg O_2/d. COD and BOD concentrations are flow proportional 24-hour values. The effluent quality does not deteriorate with an increasing loading of the wwtp, which indicates that the wwtp is rather robust with respect to the COD influent loading. The high peaks in the effluent quality cannot be explained by the influent load.

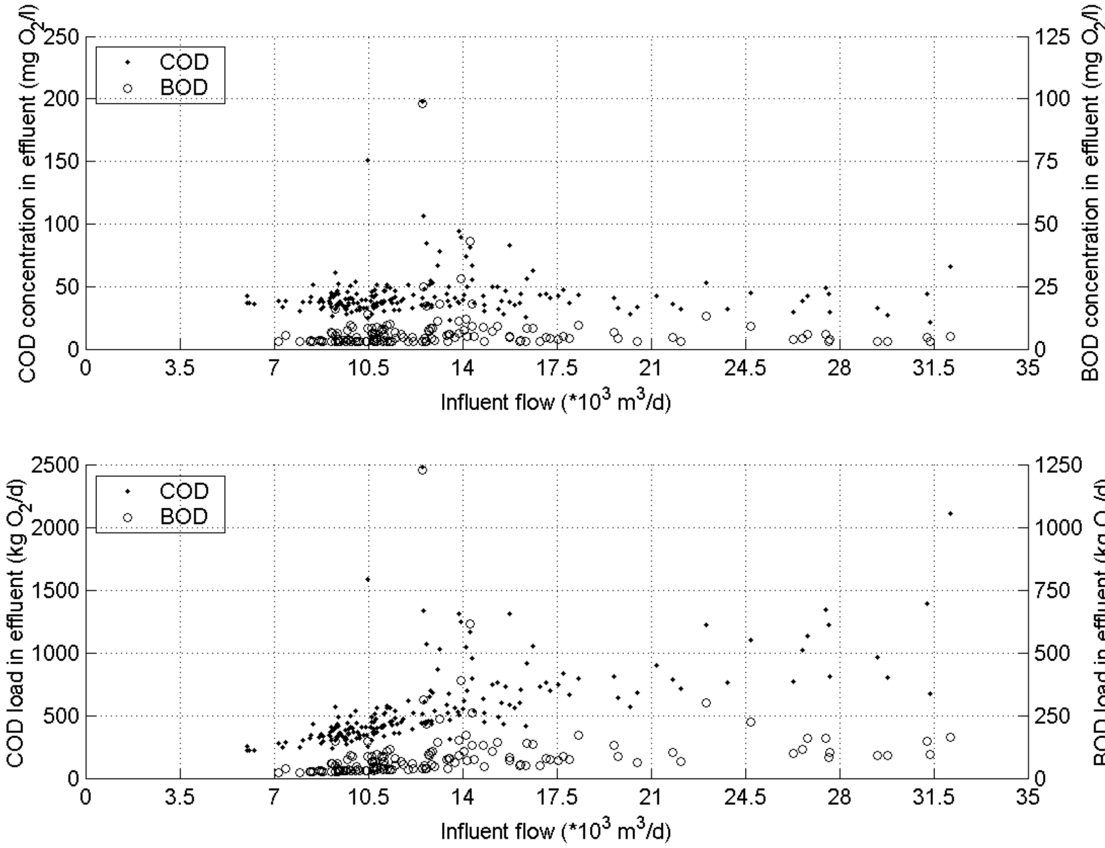


Figure 3.2 Influent flow vs. effluent concentrations BOD and COD (upper graph) and loads of BOD and COD in the effluent (lower graph), measured in 24 h flow proportional 24 hour samples. The effluent load of COD increases with the influent flow, although this effect is only due to the increased effluent flow, as the effluent concentrations do not increase with the flow.

Nitrogen removal normally involves nitrification and subsequently denitrification. Especially nitrification is reported to be very sensitive to influent fluctuations. [Durchschlag *et al.* (1991), Müller and Krauth (1998)]. [Bruns (1999)] identifies nitrification to be an indicator process for wastewater system performance, as this process reacts most sensitively and directly to influent fluctuations. Consequently, the sensitivity of nitrogen removal to influent fluctuations was analysed in more detail.

Phosphate removal can be realised by bio-P removal and/or physical-chemical P removal. Bio-P removal necessitates VFAs, anaerobic conditions and a sufficient reaction time [Meijer (2004)]. All factors may easily be violated during wwf, as the increased influent flow may have an elevated oxygen concentration and a, due to reduced hydrolysis in the sewer system, reduced VFA concentration. Contrary to the nitrogen removal, phosphate removal has with chemical precipitation an easily applicable alternative to biological removal. Chemical precipitation is often applied at wwtps in the Netherlands in addition to bio-P removal. As a result, phosphate removal is not considered to be relevant with respect to the impact of influent fluctuations on wwtp effluent quality. With respect to sludge production and operational costs (chemicals), however, the impact of influent fluctuations related to phosphate removal can be significant. A discussion of sludge production is considered beyond the scope of this thesis.

Secondary clarifiers are generally considered to be sensitive to hydraulic influent fluctuations [e.g. Bruns (1999), Müller and Krauth (1998) and Harremoës *et al.* (1993)]. Consequently, the secondary clarifier affects all pollutants associated with particles, such as COD, P_{tot} and N_{Kj}. As such, knowledge of the performance of the secondary clarifier is essential to be able to predict overall wwtp performance. However, given the importance of secondary clarifier to overall wwtp performance, secondary clarifiers are controlled in such a way that release of sludge, even as high influent flows, see figure 3.2, does not occur. Consequently, studying the performance of the secondary clarifiers on full scale wwtps will not result in improved understanding of the qualitative interactions within wastewater systems. Therefore, the performance of the secondary clarifiers was not studied in detail. The impact of the temporal storage of sludge in the secondary clarifiers, however, was taken into account.

The analysis of the dominant pollutant removal processes at a wwtp learns that the nitrogen removal generally is the most sensitive process in low loaded wwtps. An assessment of the sensitivity of nitrogen removal to influent fluctuations will therefore give the most information on the necessary knowledge of fluctuations in influent quality.

3.2.2 Selection of a reference wastewater treatment plant

The analysis of the processes and parameters selected in the previous section was performed on a reference wwtp, representing a characteristic Dutch wwtp. In order to be able to detect general trends, this reference wwtp should:

- perform well, i.e. should not have operational problems
- be a (ultra) low loaded activated sludge system (food-to-mass ratio (F/M ratio) ≤ 0.05 kg BOD/kg MLSS/day), as in the Netherlands over 65% of the wwtps is (ultra)low loaded [CBS (2003)] (MLSS = mixed liquor suspended solids)
- have a capacity within the normal range (the average capacity of a Dutch wwtp is 93,000 p.e. [CBS (2003)])
- be loaded according to the design
- have an extensive database of operational and wastewater quality data.

The reference wwtp selected is the wwtp 'Katwoude' of waterboard Hollands Noorderkwartier'. Wwtp 'Katwoude' has a design capacity of 86,300 p.e. and treats the wastewater from the villages Katwoude, Edam, Volendam, Marken and Broek in Waterland. The wastewater is transported to the wwtp by two pressure mains. Wwtp 'Katwoude' consists of a low loaded carrousel (0.042 kg BOD/kg MLSS/day) with a completely mixed non-aerated selector (270 m^3), a completely mixed predenitrification tank (1,760 m^3) and a 5 meter deep aerated carrousel reactor (15,840 m^3), see figure 3.3. The total reactor volume is 17,870 m^3 and the total volume of the secondary clarifiers is 5,588 m^3, see also table 3.5. During the whole year chemical phosphorous removal is applied in the carrousel. The effluent quality of the wwtp meets the requirements, as shown in table 3.2.
The sludge treatment at wwtp 'Katwoude' consists of thickening and subsequent dewatering by centrifugation. The dewatered sludge is transported to wwtp 'Beverwijk' for further treatment.

Table 3.2 Effluent standards and effluent quality of wwtp 'Katwoude' in 2000.

parameter	measured effluent quality	effluent standards[1]
BOD (mg O_2/l)	4.6	10
N_{total} (mg N/l)	5.7	10
P_{total} (mg P/l)	0.6	2
SS (mg SS/l)	6.1	15

[1] Effluent standards are assessed by the moving average over 10 subsequent 24 hour composite samples, collected at least 4 times monthly. N_{total} is assessed by the yearly average. The standards comply with the EU Urban Wastewater Treatment Directive [EEA (1998)]

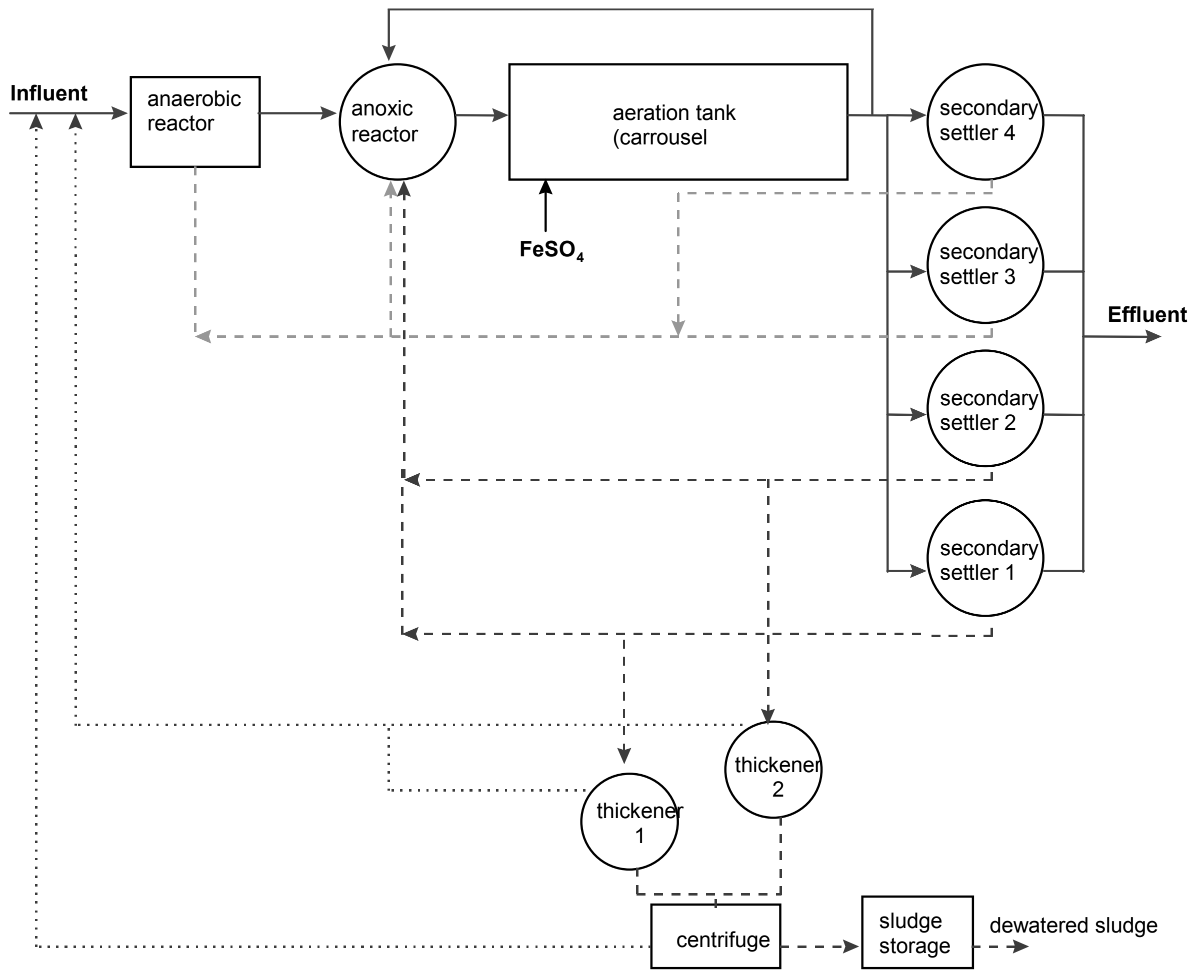


Figure 3.3 Schematic layout of wwtp 'Katwoude'.

Waterboard 'Hollands Noorderkwartier' has planned to install automated process control at the wwtp. As part of the preparations for this process control a number of online Danfoss sensors were installed and tested from September till December 2002. Figure 3.4 shows the location and the measuring range of the sensors. Appendix III gives an overview of the sensor specifications.

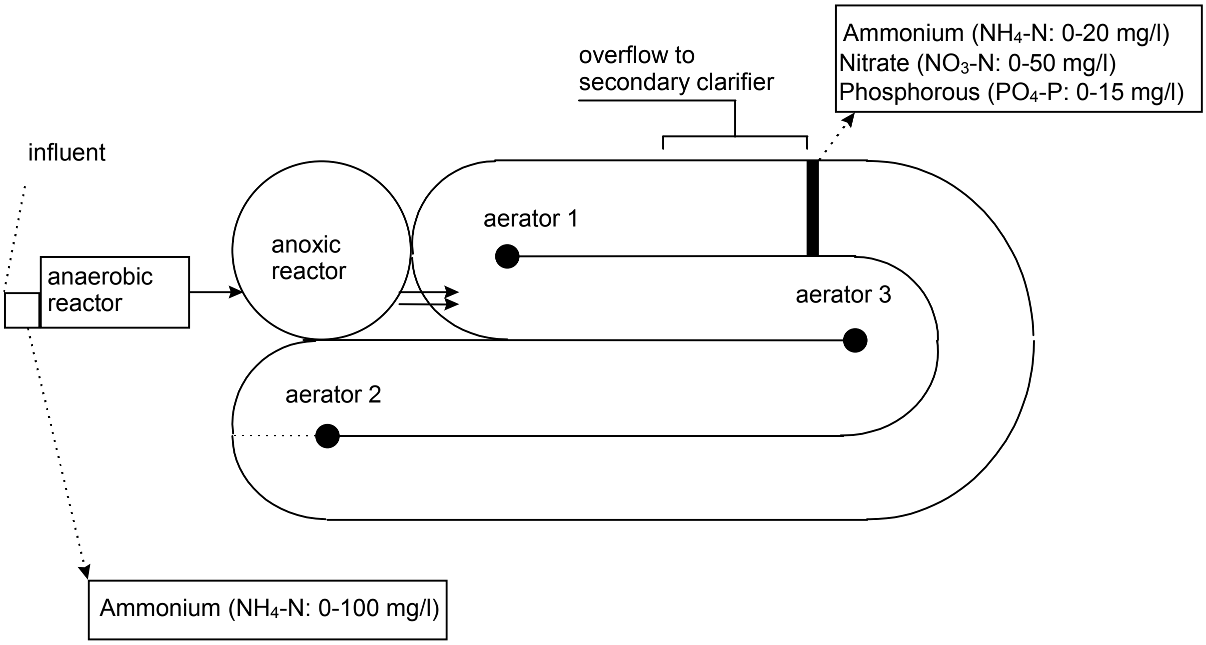


Figure 3.4 Positioning of sensors at wwtp ‘Katwoude’.

3.3 Analysis of the effect of influent fluctuations based on measurement data

The effect of influent fluctuations on the effluent quality of wwtp ‘Katwoude’ was analysed using ‘routine’ measurement data and the data from the installed sensors.
‘Routine’ measurement data stem from the measurements waterboard ‘Hollands Noorderkwartier’ has to take in order to monitor wwtp performance. At wwtp ‘Katwoude’ the routine measurements comprise, apart from influent flow records, 24 hour flow proportional composite samples of the parameters COD, BOD, N_{Kj}, P_{tot} and SS in the influent and N_{Kj}, ammonium, NO_3, N_{tot}, COD, BOD, SS and P_{tot} in the effluent.
The data from the installed sensors was recorded at an interval of 15 minutes.

3.3.1 Analysis of ‘routine’ data

In total 4 years (1999-2002) of available routine data were analysed on the following relations between influent and effluent:

- influent flow – effluent of all nitrogen fractions
- influent COD load – effluent concentrations of all nitrogen fractions
- influent N_{Kj} load – effluent concentrations of all nitrogen fractions

Influent flow – effluent concentrations

In figure 3.5 the effluent concentrations of the nitrogen fractions ammonium, nitrate and Kjeldahl nitrogen were plotted against the influent flow. The figure shows that apart from an increase in the range of effluent concentrations at daily flows of just above dwf (9.500 m^3/d) no relation between influent flow and effluent nitrogen concentrations can be noted.

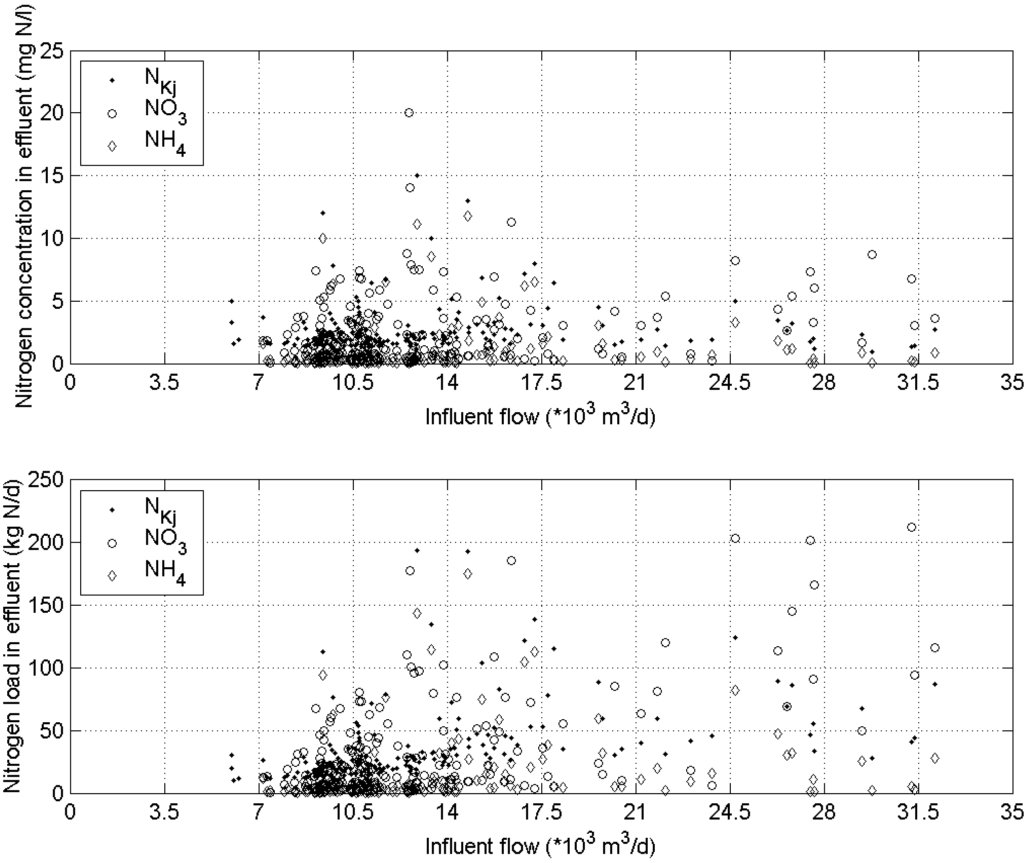


Figure 3.5 Influent flow vs. effluent nitrogen concentrations (upper graph) and effluent nitrogen loads (lower graph). None of the nitrogen concentrations is clearly affected by the influent flow, although a large range is observed. Concentrations are measured as 24 hour flow proportional composite samples.

Influent COD load vs. effluent nitrogen concentrations

Figure 3.6 shows the concentrations of the nitrogen fractions in the effluent plotted against the COD load in the influent. Like for the influent flow, no clear relation exists between the COD load in the influent and the nitrogen concentrations in the effluent.

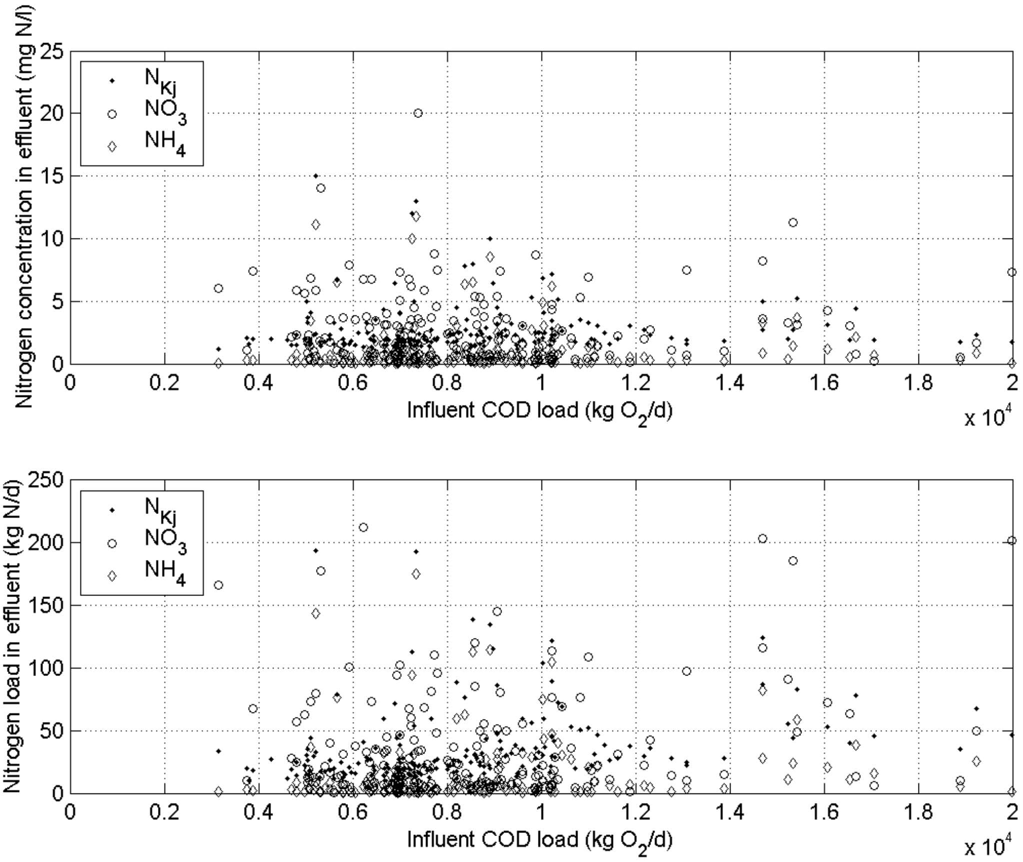


Figure 3.6 Influent COD load vs. effluent nitrogen concentrations (upper graph) and effluent nitrogen loads (lower graph). Concentrations are measured as 24 hour flow proportional composite samples. Again, high nitrogen concentrations in the effluent cannot be explained by a high influent load.

Influent $N_{Kjeldahl}$ load vs. effluent nitrogen concentrations

The results of figure 3.7 illustrate the absence of a relation between the daily $N_{Kjeldahl}$ influent load and the effluent nitrogen concentrations. This lack of correlation may be due to the fact that the adverse effects of a storm event on wwtp performance could be delayed and therefore remain unnoticed as both the underlying influent and effluent data from figure 3.7 are from the same days. In order to take into account the possibly delayed effects the nitrogen concentrations in the effluent averaged over two consecutive days were analysed, with influent data from the first day. Still no clear relation between influent load and effluent quality was found.

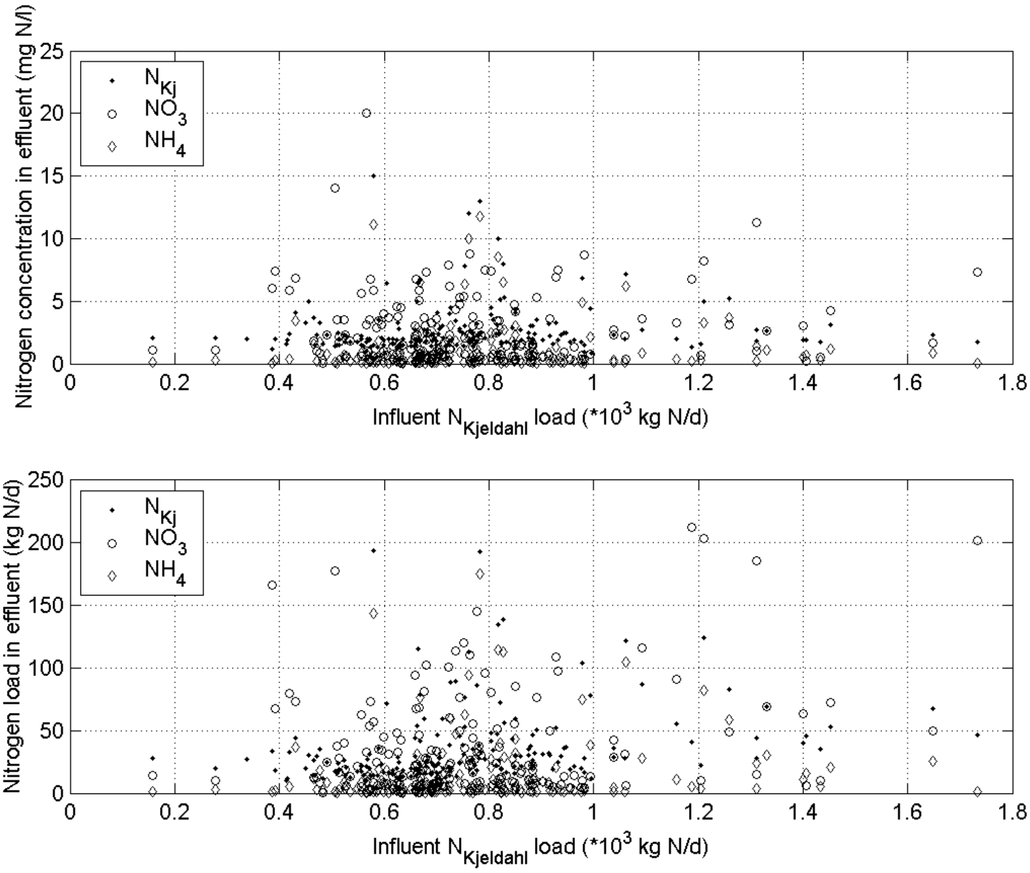


Figure 3.7 Influent $N_{Kjeldahl}$ load vs. effluent nitrogen concentrations (upper graph) and effluent nitrogen loads (lower graph). Concentrations are measured as 24 hour flow proportional composite samples. Like in figure 3.6, high nitrogen concentrations in the effluent do not seem to be related to a high influent load.

N_{total} removal efficiency

A decreasing N_{total} removal efficiency at higher influent flows has been reported in literature [Harremoës *et al.* (1993); Durchschlag *et al.* (1992)]. Even though no decrease in influent quality at higher flows can be noted, see e.g. figure 3.5, an analysis of the same data set reveals that the removal efficiency, calculated as 1 – ratio between the concentration N_{total} in the effluent and influent, decreases with the influent flow (see figure 3.8). As the effluent nitrogen concentration does not increase systematically with the influent flow (see figure 3.5), the dilution of the influent is apparently stronger than the decrease in removal efficiency.

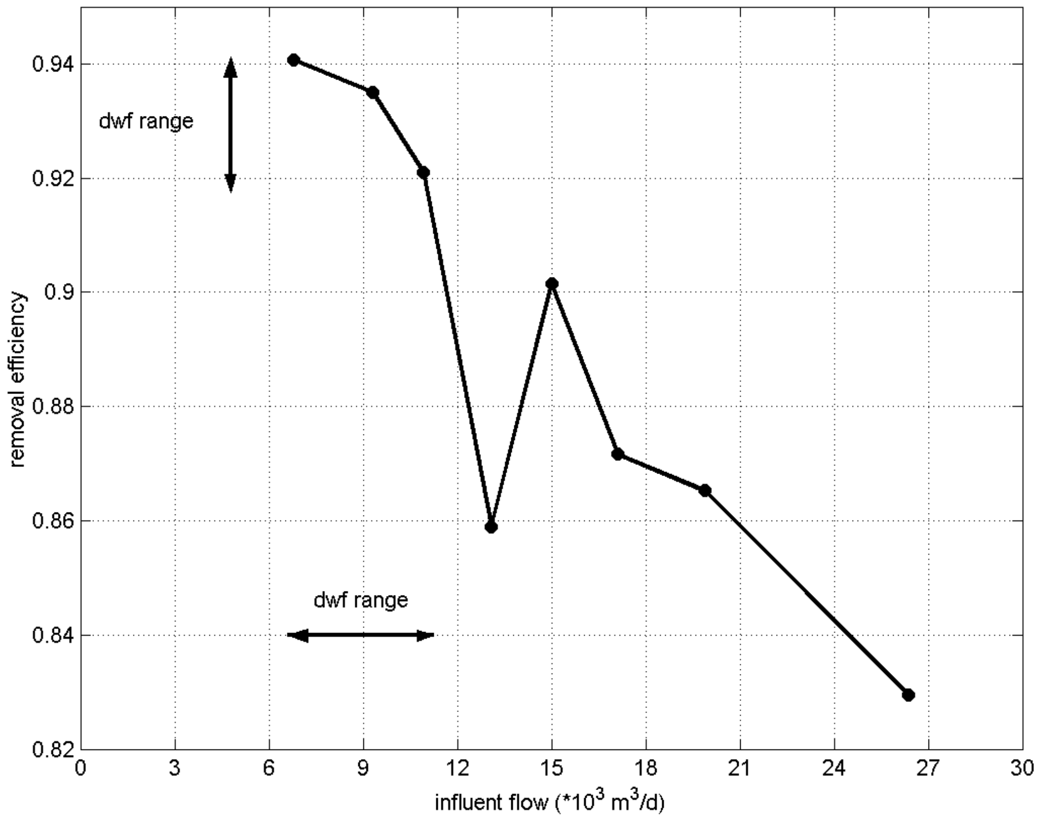


Figure 3.8 N_{total} removal efficiency ($1-N_{total\ effluent}/N_{total\ influent}$). Each point is calculated as the average removal efficiency of the observations within a 2,000 m^3/d interval, except for the final point, which represents the average efficiency of all flows larger than 22,000 m^3/d. The 'dip' in the observed removal efficiency at 13,000 m^3/d is due to the limited number of 20 observations underlying this value with 3 observations being rather low with a removal efficiency of less then 0.65.

Discussion

The analysis of the measured 24 hour composite samples for the influent and the effluent reveals no correlation between influent and effluent values, even though on a number of days the standards are violated and a broad range in effluent concentrations can be noted. The fluctuations in the effluent quality may be due to:

- errors in the measured data, e.g. sampling errors or errors in the chemical analysis
- technical failures, e.g. of pumps, aerators or power supply.
- operational aspects, such as return flow from sludge dewatering. The operational data of wwtp 'Katwoude' reveal that this return flow amounts on average to 6 % and maximum to 25 % of the daily dwf influent load of $N_{Kjeldahl.}$.
- uneven distribution of influent loads over the day. These fluctuations remain unnoticed in the 24 hour composite samples, but may be significant on a smaller scale, as discussed in section 3.3.2.
- temperature effects, as illustrated by figure 3.9.

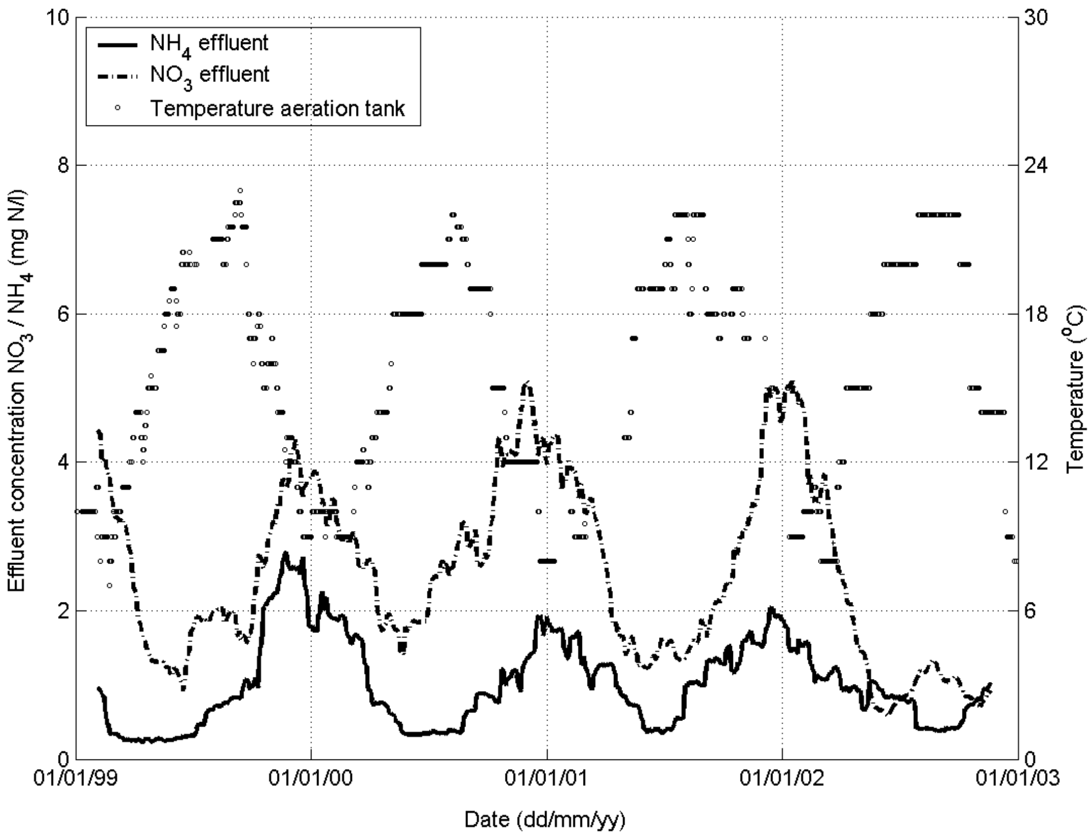


Figure 3.9 Temperature of activated sludge and the moving average of the nitrate and ammonium concentration in the effluent of wwtp ‘Katwoude’ from 1999 to 2002.

3.3.2 Analysis of data from sensors

At wwtp ‘Katwoude’ an intensive measuring campaign took place from 19 September 2002 till 5 December 2002. Apart from the sensors as shown in figure 3.2, also the influent flow and the activated sludge concentration in the carrousel were recorded at a 15 minutes interval. The data from the sensors, as well as operational wwtp data, was recorded by a supervisory control and data acquisition (SCADA) system. Additional measurements were taken to control the performance of the sensors and to gain additional information necessary for modelling.

Preceding the correlation analysis between measured influent and effluent fluctuations, the quality of the data was analysed including checks on completeness and reliability, as shown in table 3.3. Unreliable data was removed from the data set.

Table 3.3 Reduction of data availability as % of total measuring period (AT =aeration tank).

	Flow	NH_4 in influent	NH_4 in AT effluent	NO_3 in AT effluent	activated sludge concentration
missing data due to failures of SCADA system	1.9%	1.9%	1.9%	2.4%	-
missing data due to automatic sensor calibration	-	2.7%	0.7%	-	-
operational problems due to maintenance at wwtp (13-16 Nov)	5.1%	5.1%	5.1%	5.1%	-
low quality data	-	5.1%	-	47.4%	79.1 % (sensor calibrated from 19-11 onwards)
total available	93%	85.2%	92.3%	45.1%	20.9%

Figure 3.10 gives an example of available data from all sensors on 25 and 26 November 2002. All data (concentrations and flow) are presented relative to the maximum measuring range in order to be able to be plotted in one figure.
The wwtp has received an increased influent flow of up to 85% of the maximum pumping capacity during 10 hours, causing the activated sludge concentration to drop from 4.25 to 3.5 g/l.
Approximately 2 hours after the onset of the storm event the ammonium concentration in the influent starts to drop from 30 to 10 mg N/l. The recovery to normal dwf concentrations starts during the emptying phase of the storm event, at 3.00 AM on 26 Nov.
The ammonium concentration in the effluent of the aeration tank increases to 3 mg N/l just after the beginning of the storm event and rapidly decreases during the rest of the event. Finally, the nitrate concentration in the effluent of the aeration tank does not seem to be affected at all by the storm event.

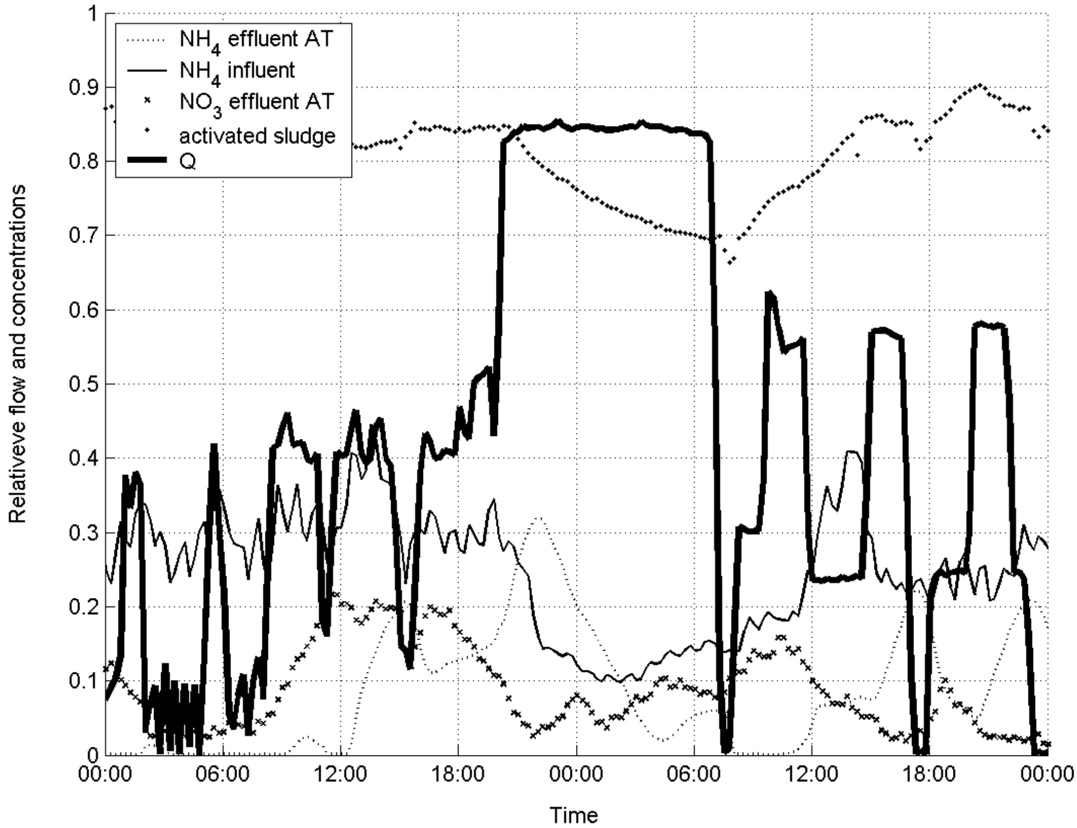


Figure 3.10 Measurements at wwtp 'Katwoude'. Storm event 25-26 November 2002 (9.7 mm). The flow is plotted relative to the maximum pumping capacity. The concentrations are normalised as follows: NH_4 effluent AT divided by 10 mg N/l, NH_4 influent divided by 100 mg N/l, NO_3 effluent AT divided by 10 mg N/l and activated sludge concentration divided by 5 g MLSS/l.

With respect to the interactions within wastewater systems it is important to be able to determine which (combination of) fluctuation(s) in the influent (flow, ammonium, a combination of both or other influences) causes the concentration of ammonium in the effluent to increase. In figures 3.11 to 3.13 the ammonium concentration in the AT effluent was plotted against the influent ammonium concentration, the influent flow and the influent ammonium load respectively. The plotted data comprise the period from 5 to 23 October 2002, with storm events and consequently wwf on 5, 7, 14 and 18 till 20 October.

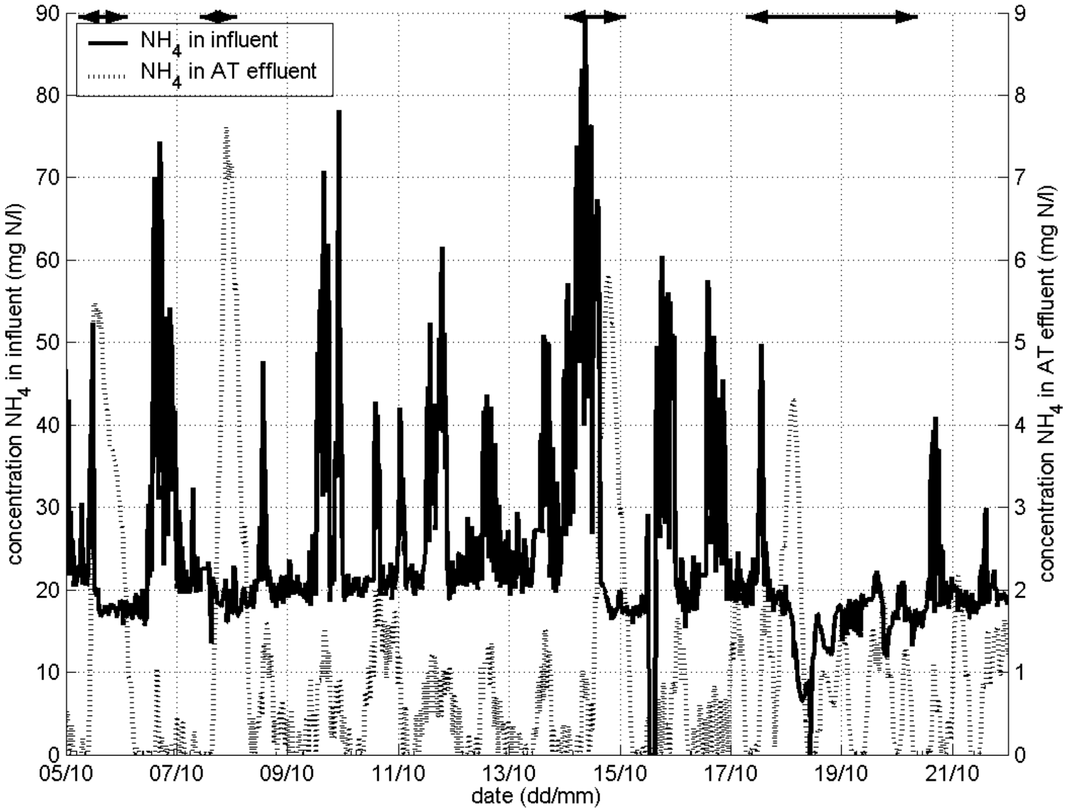


Figure 3.11 Ammonium concentration in the influent and ammonium concentration (mg N/l) in the effluent of the aeration tank (AT). The arrows indicate storm events. The impact of the storm events on the ammonium concentration in the AT effluent (5, 7, 14 and 18 October) is clearly visible. Especially the higher peaks in ammonium concentration of the AT effluent do not seem to correlate with peaks in the ammonium concentration of the influent.

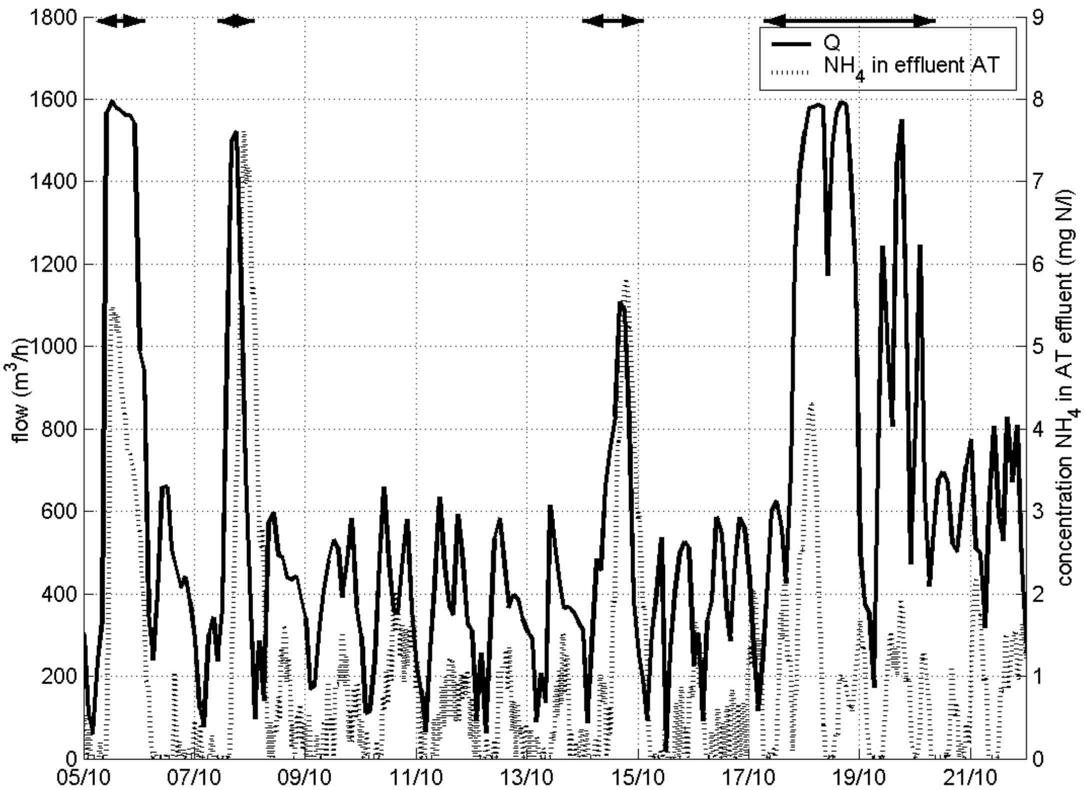


Figure 3.12 Influent flow (m^3/h) and ammonium concentration (mg N/l) in effluent aeration tank (AT). The arrows indicate storm events. Peaks in flow seem to coincide with peaks in AT effluent concentration. However, the decrease in the ammonium concentration in the AT effluent during the large storm event of 18 October can not be explained by flow records only.

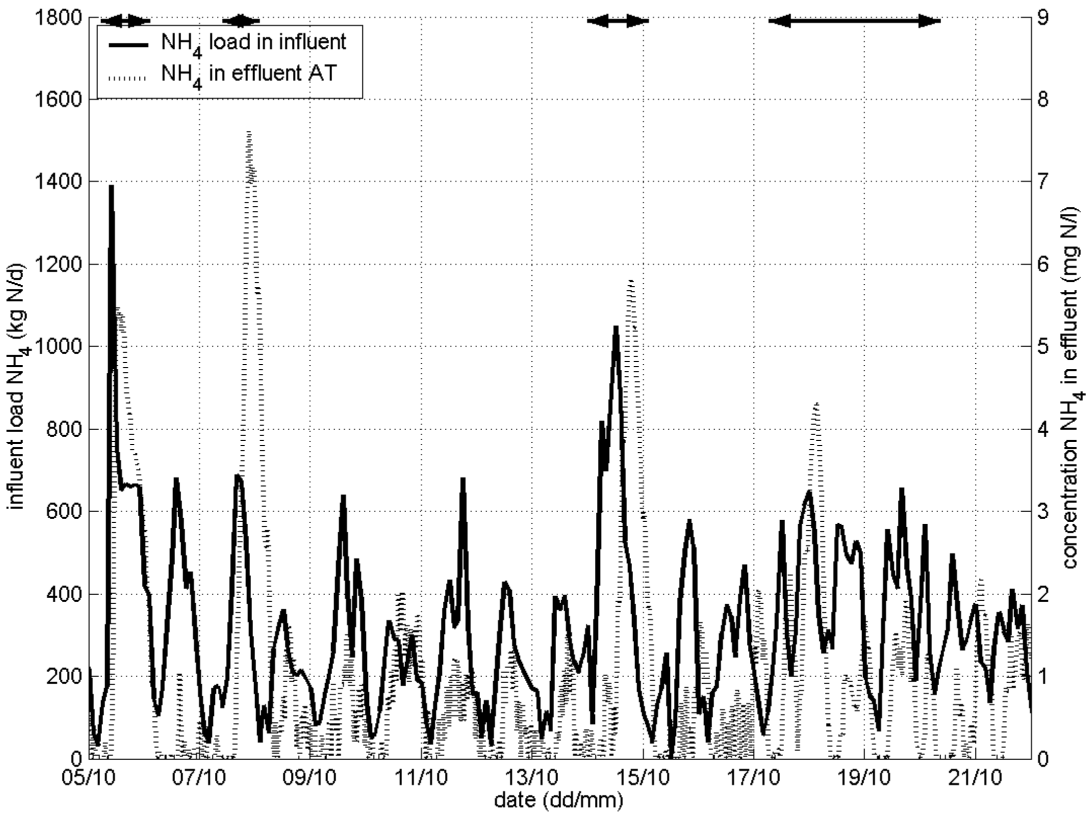


Figure 3.13 Ammonium load in influent (kg N/d) and ammonium concentration (mg N/l) in effluent aeration tank (AT). The arrows indicate storm events. Peaks in ammonium concentration in AT effluent do coincide with peaks in influent ammonium load. However, on some occasions, a high observed peak in the influent load, e.g. on 06/10, apparently does not cause the effluent quality to decrease. This peak in the influent may be due to low quality sensor readings from the ammonium influent sensor.

The ammonium concentration in the AT effluent shows no clear relation with the ammonium concentration in the influent for the period shown in figure 3.11. According to figure 3.12, during dwf and smaller storm events the flow and the ammonium concentration in the AT effluent seem to correlate, but during larger storm events the decrease in the ammonium concentration in the AT effluent can not be explained by the flow.

Figure 3.13 displays the influent load of ammonium plotted against the ammonium concentration in the AT effluent, both showing a clear diurnal profile during dwf and a strong relation during wwf. Based on the data underlying the figures 3.11 to 3.13, the influent load of ammonium is assumed to be an important parameter affecting the effluent quality. This assumption is in accordance with literature as discussed in section 2.5.3. Due to the kinetics of the nitrification process, the nitrification capacity cannot follow swift changes in the influent nitrogen load. Consequently, as soon as the influent load exceeds the momentarily available nitrification capacity, the ammonium concentration in effluent increases. This implies that sewer models should be able to predict both the flow and the concentration of ammonium in order to be able to take into account the effects of the influent fluctuations on wwtp performance.

In order to evaluate the trends observed in figures 3.11 to 3.13, a correlation analysis was performed on the complete available data set. The results of the correlation analyses (figure 3.14) confirms the overall trend as given by figures 3.11 to 3.13. Hardly any correlation exists between the ammonium concentration in the influent and the ammonium concentration in the AT effluent. For both the influent flow and ammonium load in the influent the maximum correlation amounts to approximately 0.4, with peaks at a time shift of approximately 1 and 3 hours.

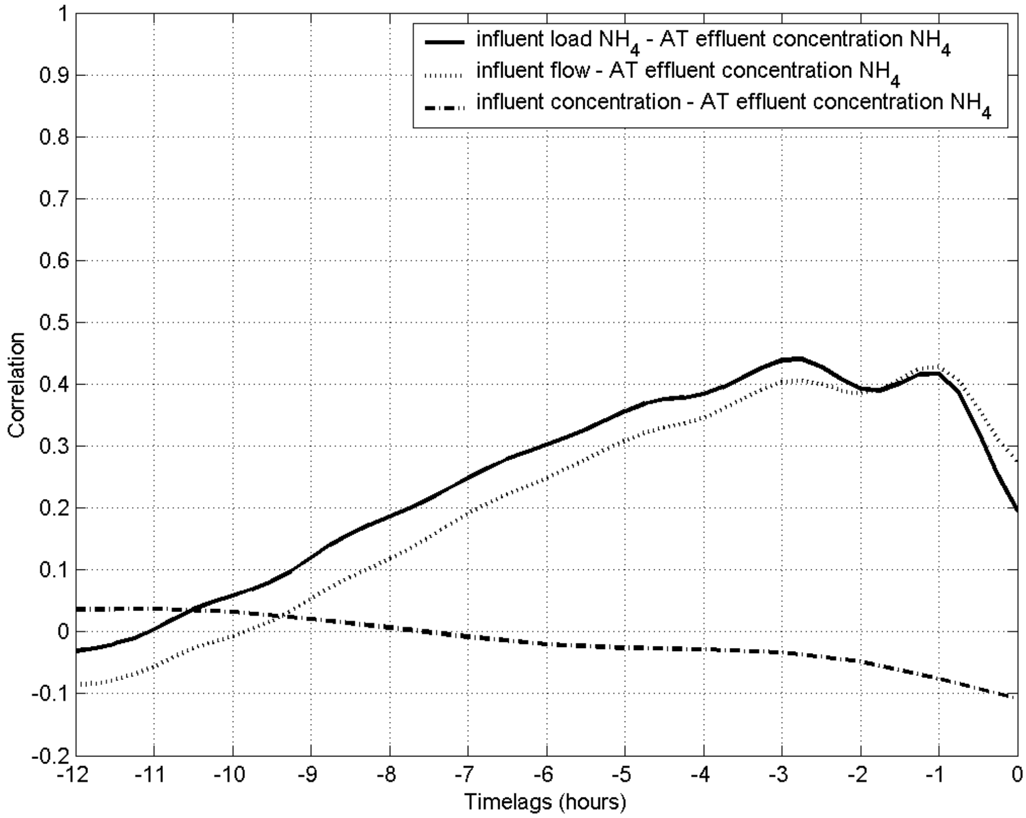


Figure 3.14 Correlation of flow, ammonium load, ammonium concentration in the influent with the concentration ammonium in the AT effluent.

The observed time shift between influent ammonium load and AT effluent ammonium concentration of approximately 1 hour is directly related to the hydraulics of the wwtp. According to the data given in [Meijer *et al.* (2002)], the hydraulic retention time, under average dwf flow, of the selector and denitrification reactor amounts to 40 minutes. The travel time with pure advection through the carrousel till the location of the sensor amounts to 240 m/0.37 m/s = 650 s. Consequently, under dwf conditions the hydraulic travel time is approximately 1 hour. The observed time shift of 3 hours could not be explained directly by available data on the wwtp hydraulics.

The results of the correlation analysis presented in figure 3.14 are based on the complete data set, including dwf and wwf periods. Table 3.4 gives an overview of the correlation of the influent parameters with the ammonium concentration in the effluent for dwf and wwf periods, calculated per day. Again, the ammonium load in the influent has the strongest correlation, in terms of mean correlation coefficient and standard deviation, with the ammonium concentration in the AT effluent.

Table 3.4 Correlation coefficients between influent parameters and ammonium concentration in the AT effluent per day, mean and standard deviation.

	15 min data			2 hour moving average		
	dwf days	wwf days	total period	dwf days	wwf days	total period
concentration NH_4 in influent	0.60 ± 0.15	0.49 ± 0.17	0.51 ± 0.17	0.73 ± 0.20	0.58 ± 0.21	0.62 ± 0.20
flow	0.54 ± 0.07	0.52 ± 0.11	0.53 ± 0.15	0.71 ± 0.12	0.64 ± 0.13	0.65 ± 0.14
load NH_4 in influent	0.61 ± 0.09	0.55 ± 0.11	0.56 ± 0.12	0.80 ± 0.12	0.67 ± 0.12	0.70 ± 0.12

N-removal efficiency
The N_{total} removal efficiency could not be assessed using the sensor data, due to a lack of reliable NO_3 readings. Instead, the nitrification efficiency was calculated, based on the ammonium influent and the ammonium effluent sensor data at 15 minute intervals. Like the N_{total} removal efficiency, as calculated using the 'routine' data, the nitrification efficiency decreases with an increasing flow, as illustrated by figure 3.15.

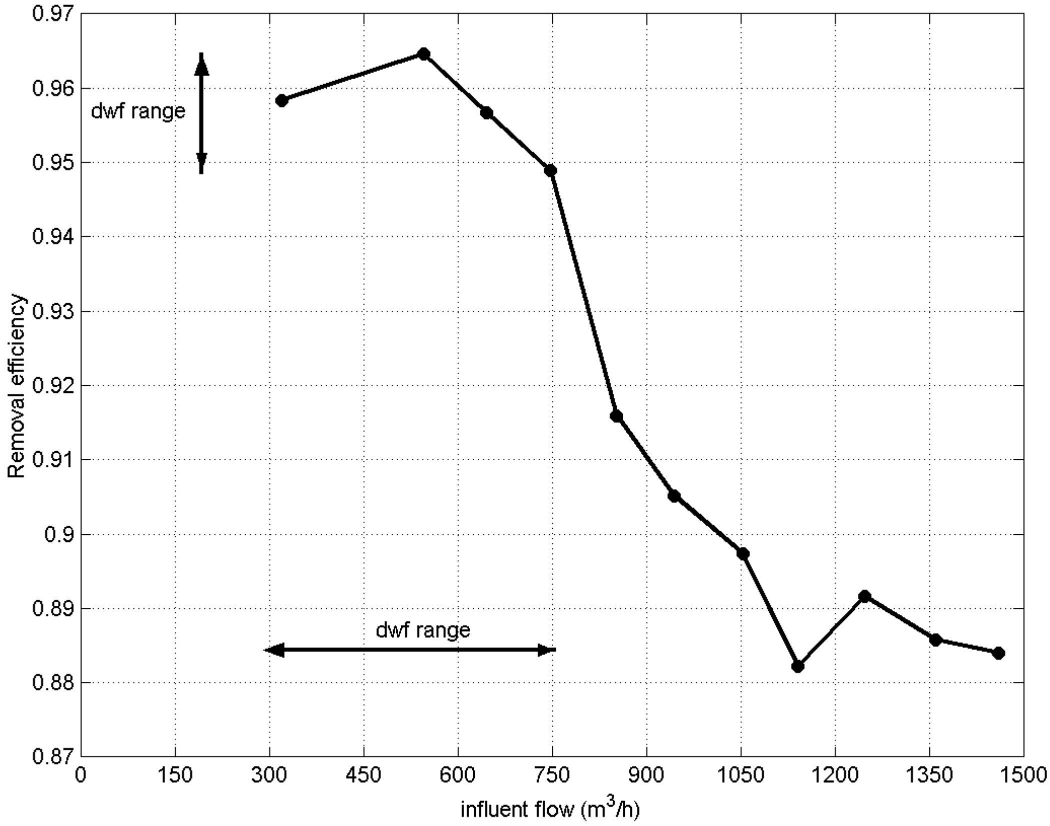


Figure 3.15 Nitrification efficiency ($1-NH_{4\ effluent}/NH_{4\ influent}$).

3.3.3 Discussion
The analysis of the effect of influent fluctuations on effluent quality based on measured data results in the following conclusions:

- the suspended solids concentration in the effluent is not sensitive to high influent loads, indicating that no overloading of the secondary clarifiers occurred.
- the 24 hour averaged samples of the 'routine' data sampling program do not provide insight in the impact of fluctuations in the influent. This phenomenon was observed for other wwtps also, see appendix I.
- the effluent quality of wwtp 'Katwoude' is affected by fluctuations in the influent. Especially the peaks in ammonium concentration in the AT effluent, occurring at the onset of each storm event, give rise to a further analysis of the relevance of the influent fluctuations.
- the peaks in the ammonium concentration in the AT effluent seem to be related to the combination of fluctuations in the influent flow and the influent ammonium concentration. However, this relation was not confirmed strongly by correlation analyses, resulting in moderate correlation coefficients only. The limited correlation is probably due to operational aspects and removal dynamics at the wwtp, not taken into account in the correlation analysis. Moreover, the quality of the data from the ammonium sensor in the influent is poor during a significant length of time, see table 3.3 [Stok (2003)], thus affecting the results from the correlation analysis.

3.4 Analysis of the effect of influent fluctuations based on dynamic modelling

The effect of influent fluctuations on wwtp performance was analysed using a dynamic model. As discussed in section 3.2.1, this analysis focuses on nitrogen and COD removal. Consequently, the original ASM1 model [Henze *et al.* (1987)] or the more recent ASM3 model [Henze *et al.* (2000)] could be applied. Within this thesis, the ASM1 model was selected for reasons of available experience with this model in the Netherlands and data availability. Appendix II gives an overview of the ASM1 process descriptions and model parameters.
As SIMBA is the simulation platform most widely used in the Netherlands (and in e.g. Germany [Leinweber (2002)]), SIMBA (vs. 3.4^{+} [IFAK (1999)]) was selected as simulation platform for practical reasons. This does not affect the quality of the modelling results, as SIMBA is one of the 8 platforms tested within the COST simulation benchmark (COST Action 624 and COST Action 682 [Copp (2000)]), all of them giving reliable results.

3.4.1 Modelling of wwtp 'Katwoude'

Wwtp 'Katwoude', see figure 3.1, was modelled using the SIMBA protocol for dynamic modelling of activated sludge systems [STOWA (2000), Hulsbeek *et al.* (2002)]. The main structure of the SIMBA protocol is shown in figure 3.16. The SIMBA protocol is discussed in more detail in appendix IV. This section briefly describes the development of the model using the SIMBA protocol, starting with the process description.

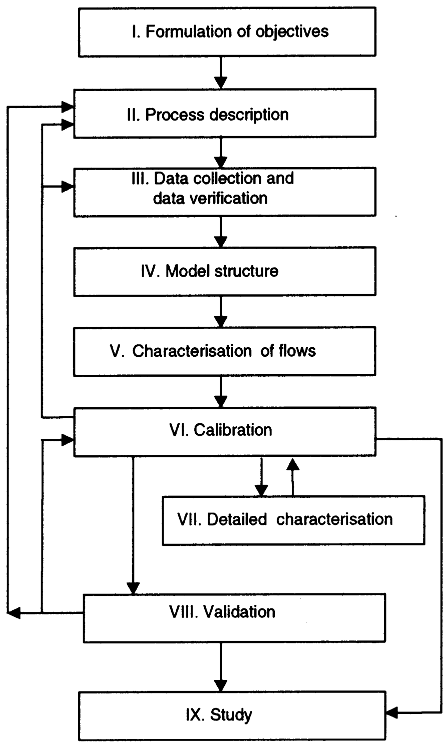


Figure 3.16 Structure of SIMBA protocol (reproduced with permission from [Hulsbeek *et al.* (2002)]).

3.4.2 Phase II. Process description

Wwtp 'Katwoude' comprises only one treatment lane, consisting of a completely mixed non-aerated selector, a completely mixed anoxic reactor, a 5 meter deep aerated carrousel reactor and four secondary clarifiers, as shown in figure 3.17. The dimensions of the reactors and clarifiers are given in table 3.5. The surplus sludge is treated in a thickener and centrifuge. Within this thesis, the sludge treatment will not be taken into account other than in terms of the resulting (mass) flows.

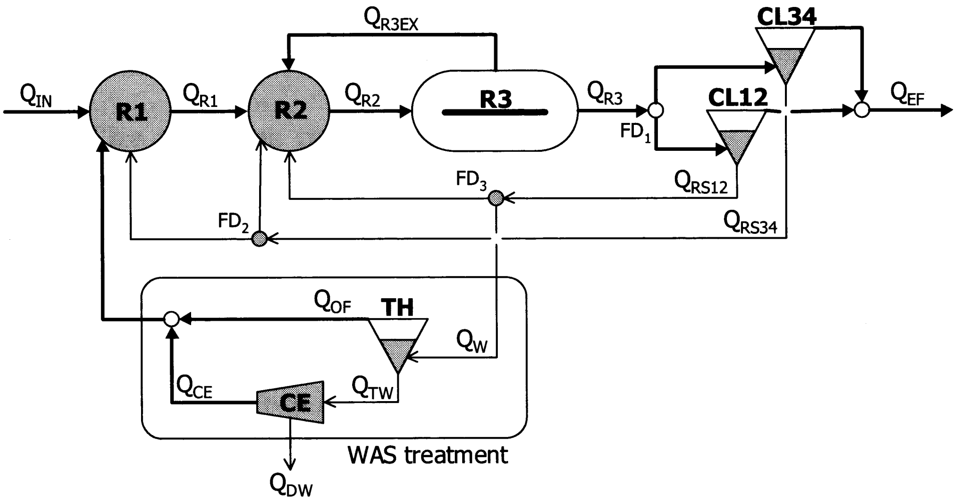


Figure 3.17 Lay-out of wwtp Katwoude. R1: selector reactor, R2: anoxic reactor, R3: aerated carrousel reactor, CL: secondary clarifiers, TH: sludge thickener, CE: sludge dewatering centrifuge, FD: flow dividers. Reproduced with permission from [Meijer (2004)]). Aeration is controlled by oxygen setpoints in the aeration tank.

Table 3.5 Dimensions of wwtp Katwoude.

Reactor	**volume (m^3)**	**depth (m)**	**internal flows**	**flow (m^3/d)**
selector R1	270	3.5	external recycle R3	66,750
pre denitrification tank R2	1,760	5.5	internal recycle R3	1,296,000
aerated carrousel R3	15,840	5	RAS[a] from Cl_{12}	12,144
total reactor volume	17,870		RAS from Cl_{34}	24,960
clarifier, $CL_{1,2}$ (2*1,395 m^3)	2,790	2		
clarifier, $CL_{3,4}$ (1,395, 1,403 m^3)	2,798	1.5		

[a] RAS = return activated sludge

3.4.3 Phase III. Data collection and data verification

In literature, many examples exist of studies describing the calibration of activated sludge model, discussing the adjustment of model parameters only [e.g. Henze (1992); Kappeler and Gujer (1992)]. The correctness of the data used is hardly ever discussed. This is surprising, as according to [Meijer *et al.* (2001)], model results are much more sensitive to operational data than to model parameters. As a result, using erroneous data will lead to unjustified calibration procedures, even though this will often go unnoticed due to the ASM1 model being over-parameterised. Although less likely than in sewer systems, where database errors are a common feature [Clemens (2001a)], also wwtp data may be erroneous. An example of erroneous data are flow dividers not functioning as intended [Meijer (2004)].

[Meijer *et al.* (2002)] describe the use of gross error detection and data reconciliation techniques on mass balances to analyse the correctness of available wwtp data, illustrated by the case of wwtp 'Katwoude'. Based on the aforementioned 'routine' data and an 8 day sampling period (23 February to 2 March 2001) [Meijer *et al.* (2002)] have detected a number of errors in the wwtp plant data. Repairing these errors resulted in a high quality structural and operational database for wwtp 'Katwoude'.

In addition to this data, the installed sensors and additional measurements provided data necessary for accurate modelling.

3.4.4 Phase IV. Model structure

The model structure for the hydraulics is based on the process description. The fully mixed selector and anoxic reactors are each modelled as a Complete mixed Stirred Tank Reactor-model (CSTR). The carrousel is modelled as 15 CSTRs in series with a high recirculation

rate to be able to simulate the oxygen gradients in the carrousel. The secondary clarifier is modelled as a simple model with simplified sludge storage in a completely mixed reactor. The application of the more detailed 1D, 10-layer Otterpohl-Freund model [Otterpohl and Freund (1992)] does in this case not improve the quality of the simulation results [Stok (2003)]. Consequently, the simple model suffices. Denitrification in the secondary clarifier is not taken into account, as no decrease in nitrate concentration in the return sludge was measured during the measuring period of September till December 2002, which is in accordance with the results of [Meijer *et al.* (2002)].
The process control in the model is designed to properly reproduce the actual process control in terms of return sludge rates and aeration.
A detailed description of the model structure is given in appendix V.

3.4.5 Phase V. Characterisation of flows

The most important step within this phase is the characterisation of the influent into the 13 influent fractions of the ASM1 model, see appendix II. In literature, many methods have been described, which can be used for the characterisation of wastewater for modelling of activated sludge processes [see e.g. Sollfrank and Gujer (1991)]. Experience with modelling of full scale treatment plants revealed that not all of these methods result in a reliable and reproducible wastewater characterisation [Roeleveld and Loosdrecht, van (2002)]. Therefore, a guideline for wastewater characterisation has been developed in the Netherlands, based on physical-chemical techniques, thereby enhancing reproducibility and simplicity [Roeleveld and Loosdrecht, van (2002)]. In this thesis, this guideline was adopted, with the exception of the suspended COD fractions X_i and X_s (respectively inert and slowly degradable suspended COD) as shown in table 3.6. The $X_s/(X_s+X_i)$ ratio was used to calibrate the sludge production [Meijer *et al.* (2001)].

Table 3.6 Characterisation of wastewater for wwtp Katwoude, Sept. – Dec. 2002

Parameter (Henze *et al.*, 1987)	value	assumption/calculation
S_i (inert soluble COD)	32.9 mg O_2/l	$COD_{eff,\ mf}$ *0.9
S_s (readily biodegradable substrate)	186.5 mg O_2/l	$COD_{inf,\ mf}$ - S_i
X_i (inert particulate matter)	241.6 mg O_2/l	Xi + Xs = $COD_{inf,\ TSS}$, ratio $X_s/(X_s+X_i)$ based on sludge production
X_s (slowly biodegradable substrate)	215 mg O_2/l	
X_{bh} (active heterotrophic biomass)	0.01 mg O_2/l	assumption
X_{ba} (active autotrophic biomass)	0.01 mg O_2/l	assumption
X_p (particulate decay products)	0 mg O_2/l	assumption
S_O (oxygen)	0 mg O_2/l	assumption
S_{NO} (nitrate and nitrite)	0 mg N/l	assumption
S_{NH} (NH_4^+ and NH_3 nitrogen)	33.7 mg N/l	NH_4-N (measured by sensor)
S_{ND} (soluble biodegradable organic nitrogen)	6.6 mg N/l	0.03*$COD_{inf,\ mf}$
X_{ND} (particulate biodegradable organic nitrogen)	18.3 mg N/l	0.04*$COD_{inf,TSS}$
Alkalinity	7 mg N/l	no alkalinity problems reported
Flow	13,721 m^3/d	measured

in = influent, eff = effluent, mf = filtrated over 0.45 μm-filter, TSS = suspended fraction

The figures of table 3.6 represent the average loading of wwtp Katwoude during the measuring period. The COD values are based on both routinely measured data and additional analyses. The ammonium concentration and the influent flow are available at a 15 minutes interval, as discussed in section 3.2. Simulating wwtp performance during the complete measuring period, however, necessitates knowledge of the dynamics of all influent fractions given in table 3.6. On 4 days, additional data was collected on the fluctuations in influent composition (COD_{mf} and COD_{TSS} and N_{Kj}) during both dwf and wwf. Figure 3.18 gives an example of the results obtained. The measured data show that, especially during wwf, the changes in COD and nitrogen concentrations are quite similar. As a result, it was assumed that all wastewater fractions of the influent have the same dynamics over the complete measuring period.

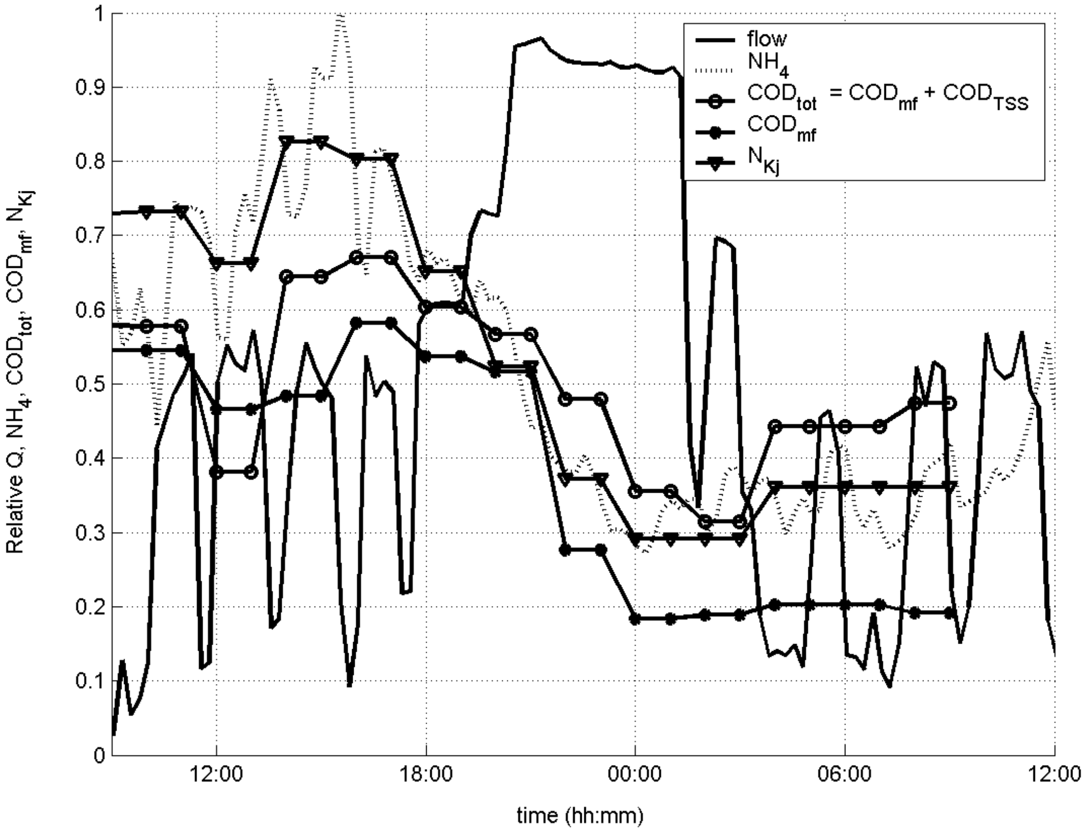


Figure 3.18 Measured profiles of COD and N fractions. Influent measurements wwtp Katwoude, 1 and 2 December 2002. The concentrations are normalised: NH_4 divided by 70 mg N/l, N_{Kj} divided by 90 mg N/l, COD_{tot} divided by 1000 mg O_2/l, COD_{mf} divided by 400 mg O_2/l. The flow is normalised by the maximum pumping capacity.

3.4.6 Phase VI. Calibration and Phase VII. Detailed characterisation

Calibration of the ASM models is not necessarily straightforward given the 24 model parameters and 13 wastewater fractions. In literature many methods have been proposed, ranging from estimating model parameters based on respiration tests [Kappeler and Gujer (1992)] to mathematical methods for automatic parameter estimation [Weijers and Vanrolleghem (1997)]. The estimation of model parameters based on respiration tests is both laborious and may result in erroneous results for full scale treatment plants, as model parameters found to be sensitive in a batch test can be insensitive in a full-scale wwtp. Automatic mathematical parameter estimation will select the parameters numerically most sensitive. However, many of the model parameters, like e.g. the heterotrophic yield are known not to vary significantly in reality. Therefore, [Meijer *et al.* (2001)] advise to focus on a number of model parameters, based on knowledge of the processes. Preferably, only those model parameters, affecting the process to be calibrated only, should be selected. This resulted in the following calibration sequence, [Meijer *et al.* (2001) and Hulsbeek *et al.* (2002)], designed to minimise the number of iterations necessary:

- sludge composition and sludge production
- nitrification
- denitrification

In the case of the calibration of the ASM model for wwtp ‘Katwoude’ the calibration procedure as proposed in the SIMBA-protocol [Hulsbeek *et al.* (2002)] was adopted. The calibration of the Katwoude model is based on the complete data set from the measuring period of September 2002 till December 2002. The subsequent validation of the model is based on the aforementioned 8 day sampling period (23 February to 2 March 2001) [Meijer *et al.* (2002)]. The validation data set comprises the 8 day average of flow and concentration measurements at the influent and the effluent of respectively the selector (R1), the anoxic reactor (R2), the carrousel (R3) and the secondary clarifier.
In order to be able to compare the calibration results with the validation results, a ‘static’ calibration is at first performed on the overall averages of the complete calibration data set, see table 3.7. With the parameters from the ‘static’ calibration the complete measuring period from 19 September 2002 till 5 December 2002 was simulated.

Table 3.7 Averaged values of data sets used for calibration and validation.

	calibration data set (19-09 till 05-12 2002)	**validation data set (23-02 to 02-03 2001) [Meijer *et al.* (2002)]**
flow (m^3/d)	13,721	12,380
temperature (C)	17	9
NH_4 influent (mg N/l)	33.7	38.3
N_{Kj} influent (mg N/l)	55.5	61.5
COD influent (mg O_2/l)	646	773
COD_{TSS} influent (mg O_2/l)	457	380
COD_{mf} influent (mg O_2/l)	219	393
NH_4 AT effluent (mg N/l)	1.1	1.2
NO_x effluent (mg N/l)	1.0	3.3
NO_x AT effluent (mg N/l)	-	3.2
COD effluent (mg O_2/l)	37	43

3.4.7 Calibration results: averaged data

The ‘Katwoude’ model was first calibrated against the averaged data from the complete measuring period

Sludge production

The average activated sludge concentration equals 4.42 g/l, with a sludge age of 19.6 days [Stok (2003)]. The sludge production was calibrated by adjusting the $X_s/(X_s+X_i)$ ratio to 0.47, resulting in a simulated sludge concentration of 4.5 g/l.

Nitrification

The calibration of the nitrification process involves checks on the oxygen concentration and alkalinity and possibly subsequently an adjustment of model parameters. At wwtp Katwoude no problems regarding alkalinity have been reported and the aeration capacity in the model and at the wwtp are in accordance [Meijer *et al.* (2002)]. Therefore, the nitrification was calibrated by adjusting the saturation coefficients for oxygen (K_{O2}) from 0.4 to 0.08 g O_2/m^3 and for ammonium (K_{NH4}) from 1 to 0.5 g NH_4-N/m^3. The resulting ammonium concentration in the AT effluent equals the 1.1 mg N/l given in table 3.5.

Denitrification

The NO_x concentration with the default model parameters is 1 mg N/l. Therefore, no adjustments of the model parameters were made.

3.4.8 Calibration results: measuring period 19 September 2002 till 5 December 2002

The calibration results were obtained by simulation of the complete measuring period from 19 September till 5 December 2002. The monitored operational data, i.e. temperature, oxygen setpoints and surplus sludge rate, was incorporated in the simulations.

Sludge production

The activated sludge concentration varied between 3.0 and 5.3 g MLSS/l, varying as much as 0.5 g MLSS/l per day. These changes could neither be explained by the produced surplus sludge derived from operational wwtp data nor by the measured influent loads. The maximum load of suspended solids measured in the influent was 3,700 kg/d, which could explain an increase of the sludge concentration of only 3,700 kg/d /17870 m^3 reactor volume (see table 3.5) = 0.2 g MLSS/l. The measured influent loads, however, are only available as 24 hour samples on 10 of the 78 days of the measuring period. Therefore, the influent loads cannot be fully excluded as potential disturbing factor. Moreover, the operational data of the wwtp with respect to the surplus sludge rate may be erroneous. Furthermore, the (on site) measured activated sludge concentration may suffer from both sampling and measurement errors. Finally, the applied ASM1 model does not directly predict the measured mixed liquor suspended solids concentration [Gujer *et al.* (2000)]. Given all these uncertainties, it was not attempted to fully describe the changes in activated sludge concentration during the complete measuring period. Similar problems with the simulated activated sludge concentration have been described by [Meijer *et al.* (2001)]. They introduced settling of activated sludge in one of the reactors to overcome the problems of fluctuating activated sludge concentrations.

In this thesis, the problem of largely fluctuating activated sludge concentrations was dealt with by adjusting the pumping capacity regulating the surplus sludge rate. The 3 periods in which the surplus sludge rate was adjusted, compared to the values from the operational data are indicated in figure 3.19.

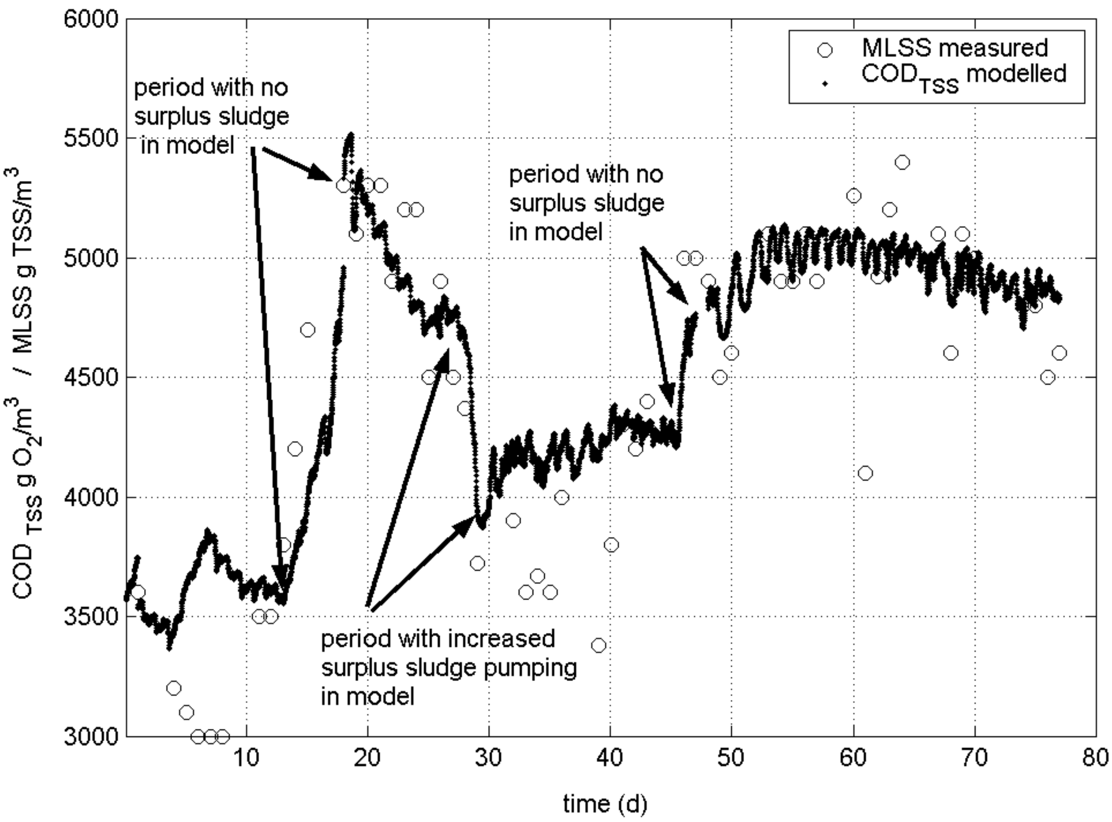


Figure 3.19 Overview of daily measured MLSS concentration compared with simulated COD_{TSS} concentrations.

Figure 3.19 shows the measured MLSS concentration and the simulated COD_{TSS} concentration. A direct comparison of the MLSS and the COD_{TSS} concentrations is in this case allowed due to the characteristics of the activated sludge measured at wwtp Katwoude, with an average measured fixed solids fraction of 27%. This means that the volatile suspended solids (VSS) comprise 73% of the mixed liquor suspended solids (MLSS). COD has generally a constant relation, COD = 1.42*VSS, with the volatile suspended solids [STOWA (2000)]. Consequently, comparing the simulated COD_{TSS} with the MLSS concentration only results in an error of 4 % (COD = 1.42*0.73*MLSS = 1.04*MLSS). This error is small compared to aforementioned uncertainties.

In addition to the daily manually measured activated sludge concentration, a sensor measuring the MLSS concentration in the carrousel was installed during the final phase of the measuring period. In figure 3.20 the simulation results are compared with the data from the suspended solids sensor. The suspended solids sensor systematically underestimates the SS concentration, when compared with the on weekdays routinely taken control samples. Under the assumption that this underestimation is constant, the sensor data were increased by 0.8 kg/m^3. The sensor data show significantly more dynamic changes than the modelled activated sludge concentration. However, applying the Otterpohl – Freund 10-layer secondary clarifier model hardly affected the model results with respect to both nitrification and denitrification [Stok (2003)].

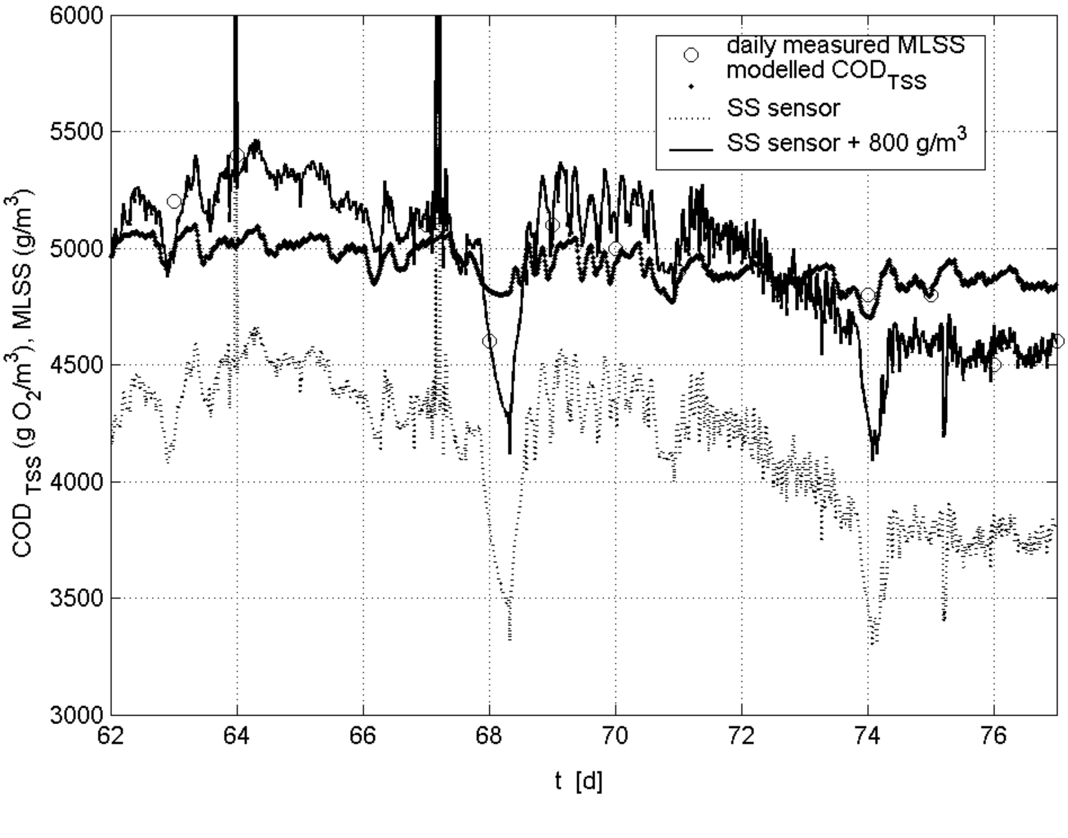


Figure 3.20 Simulated COD_{TSS} concentration and measured activated sludge concentration.

Nitrification

The quality of the simulation results with respect to nitrification could be analysed in more detail, due to the availability of the data from the ammonium sensors located at the inlet of wwtp Katwoude and the effluent of the aeration tank (see figure 3.2). The sensor data was collected with an interval of 15 minutes, therewith enabling a detailed analysis of the nitrification process. Before analysing the quality of the simulation results, erroneous sensor data (see table 3.3) were removed from the data set. Figure 3.21 gives an example of the simulation results from day 60 (18 Nov. 2002) till 77 (05 Dec. 2002).

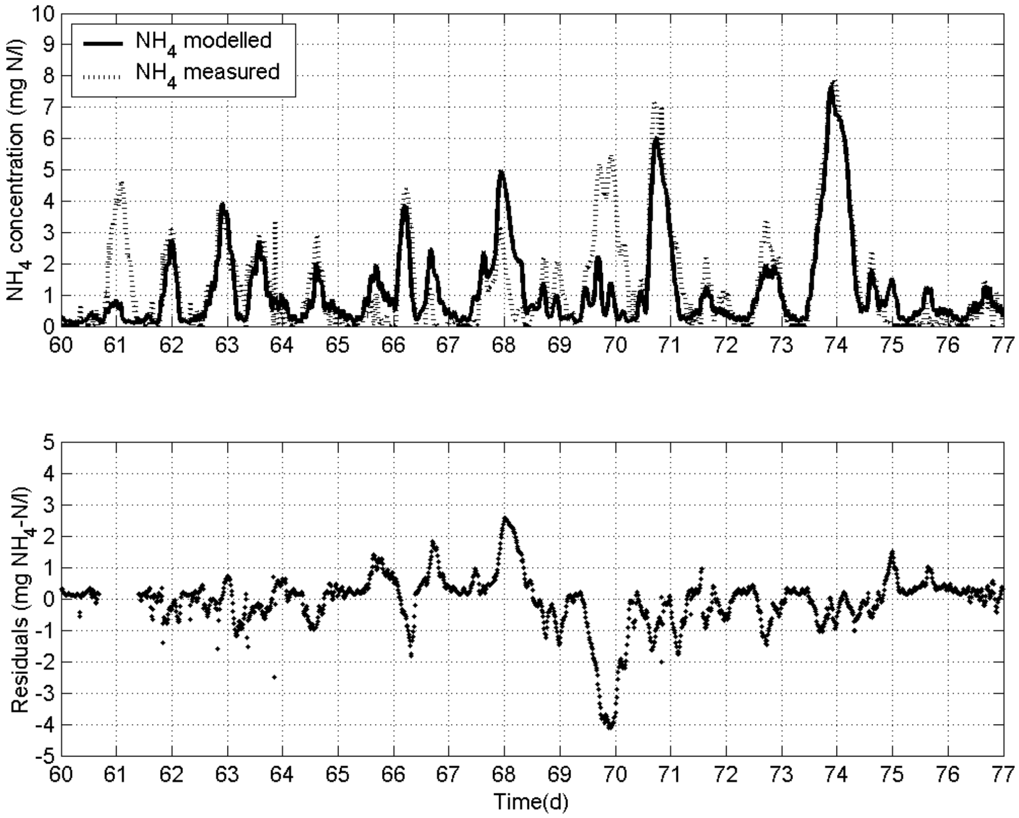


Figure 3.21 Simulation results for nitrification.
(top) Simulated versus measured ammonium concentration in AT effluent.
(bottom) Residuals. At the end of day 60, the sensor measuring the ammonium concentration in the influent was known to give unreliable results. Consequently, the residuals during this period were not taken into account. On day 69 probably the same problem occurred, although this could not be confirmed due to a lack of control measurements.

The quality of the simulation results, i.e. the residuals, was analysed with respect to:

- probability distribution (compared to the normal distribution)
- mean squared error
- relative bias

The probability distribution of the residuals indicates, compared with the normal distribution of the residuals, a high number of low residuals, i.e. the cumulative probability curve is much steeper for small residuals, see figure 3.22. This may be due to the combined effect of the reduced measuring accuracy in the low measuring range (< 0.6 mg/l) and the limitations of the applied Monod kinetics in the ASM1 model for concentration ranges much lower than the Monod constants.

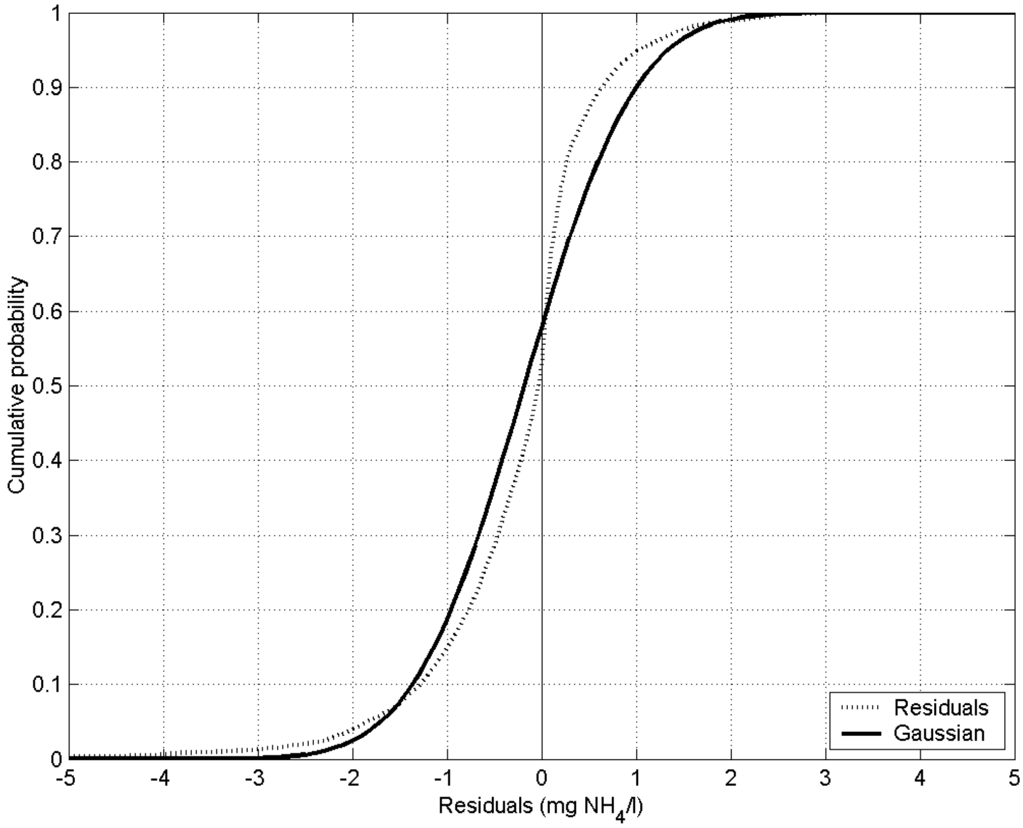


Figure 3.22 Probability distribution for the residuals for the reduced data set compared with a Gaussian distribution based on the residuals.

Overall, the mean squared error (MSE) is 0.87, having the same order of magnitude as the average ammonium concentration of 1.1 mg N/l. However, this relatively large MSE is mainly due to the results on days like day 69 (see figure 3.21), where the input data from the influent sensor can be doubted. For periods without these large residuals the MSE is much lower, e.g. for the period from day 70.5 till day 77 the MSE equals 0.23.

The probability distribution of the residuals and the MSE do not discriminate between residuals due to the model structure and residuals due to measuring errors. Therefore, the relative bias was studied for the complete data set. The relative bias is defined as the ratio between bias and the variance, where the bias is given by [Clemens (2001a)]:

$$bias(k) = \left(\frac{1}{n_w} \sum_{i=1}^{n_w} e(i)^2 \right) - \sigma_w(k)^2 \qquad \text{(eq. 3.1)}$$

where

n_w = number of time steps within shifting time window w

$\sigma_w(k)^2$ = variance of the residuals

$e(i)$ = residual, difference between measurement and model result at time i

$\frac{1}{n_w} \sum_{i}^{n_w} e(i)^2$ = mean squared error (MSE)

w = shifting time window

Figure 3.23 gives an example of the relative bias for the period from day 60 till 77. Peaks in the relative bias occur systematically over the complete measuring period during the periods of low (<0.25 mg N/l) ammonium concentrations. This indicates a systematic error in either the model or the measurements. Moreover, the relative bias peaks irregularly during the ammonium peaks occurring during storm events. As these peaks are not systematic, model errors could be excluded, leaving the known (proven by control measurements [Stok (2003)]) errors introduced by the ammonium sensor in the influent as the most likely explanation for the relatively large residuals.

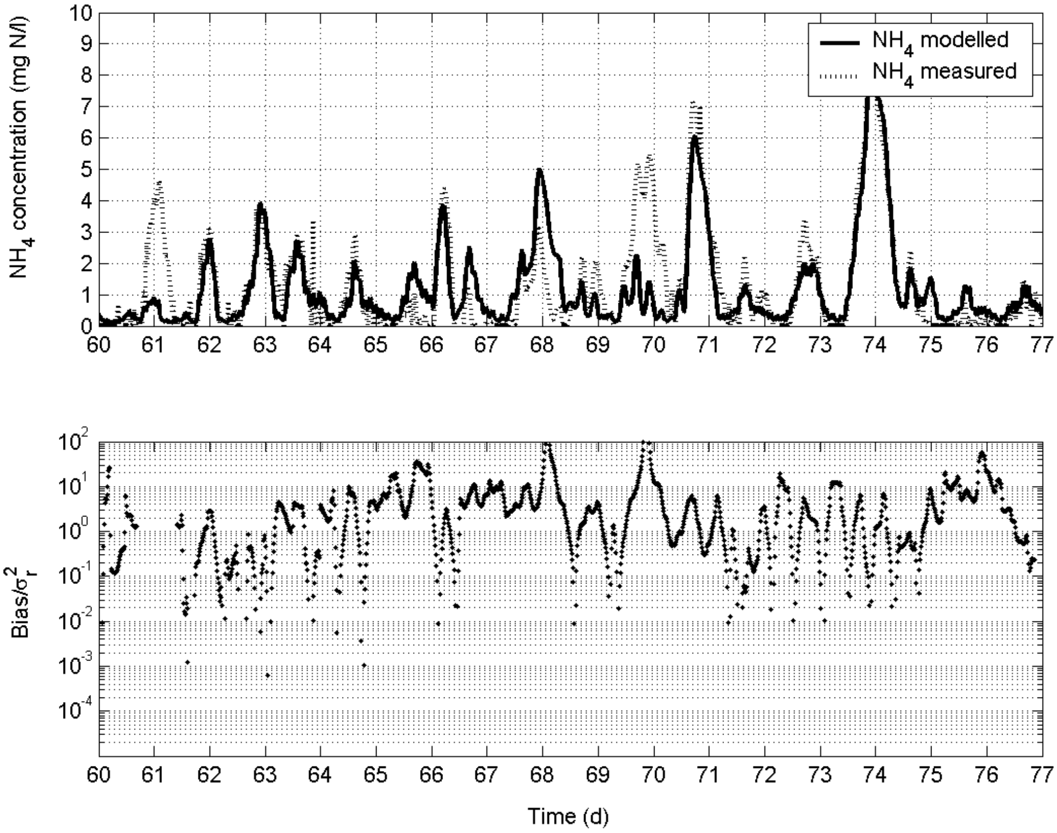


Figure 3.23 Relative bias calculated with a shifting time window of 6 hours.

Based on the analysis of the simulation results and the residuals it is concluded that the dynamics of the nitrification process can be modelled properly with the ASM1 model, with the exception of the very low (<0.25 mg NH_4 N/l) range. This conclusion is in line with results found in literature (see e.g. [Seggelke (2002), Meijer *et al.* (2001)]) Moreover, the values for the model parameters derived for the static situation, i.e. the average of the complete measuring period, proved to give reliable results in a dynamic situation.

Denitrification

The denitrification process could not be analysed as detailed as the nitrification, because the nitrate sensor in the effluent did not give reliable results throughout most of the measuring period. However, the 24 hour samples of the secondary clarifier effluent taken every weekday during the measuring period could fortunately be used to analyse the performance of the denitrification on a larger time scale. As opposed to the nitrification, the model parameters that were selected for the static situation needed adjustment. The mean NO_x concentration in the wwtp effluent is 2.0 mg N/l for the simulation of the complete measuring period, whereas the measured NO_x concentration was 1 mg N/l, see table 3.7. Apparently, the oxygen levels in the carrousel under dynamic conditions in the simulations for the complete measuring period cause the denitrification rate to decrease relative to the static calibration.

After increasing the saturation coefficient for oxygen for heterotrophic biomass (K_{OH}) from 0.2 to 0.7 g O_2/m^3 a mean NO_x concentration in the wwtp effluent of 0.9 was simulated. Figure 3.24 shows the simulated dynamics for the NO_x concentration in the AT effluent compared with the sensor results, illustrating the lack of consistence between the model and the sensor readings.

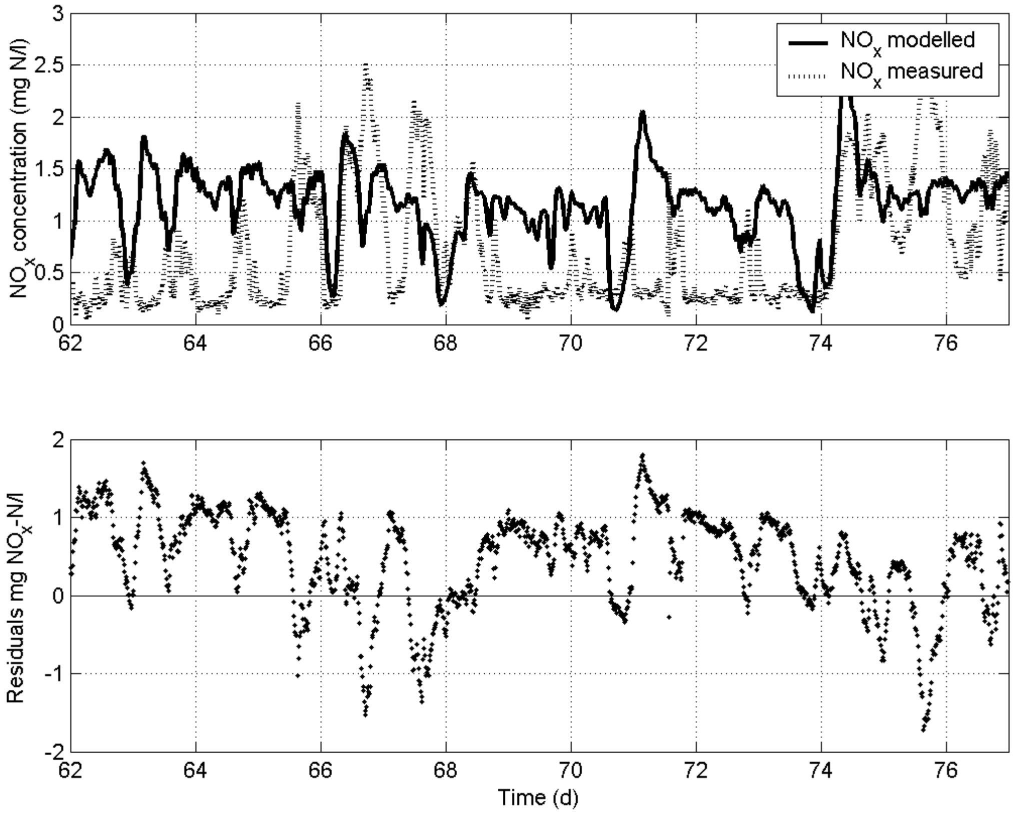


Figure 3.24 NO_x concentration in AT effluent.

However, apart from the NO_x sensor readings also 24 hour samples taken from the effluent of the secondary clarifier are available. A comparison of these data with the simulation results, see figure 3.25, shows that on a 24 hour basis the model reproduces the general trend in the measured NO_x concentration.

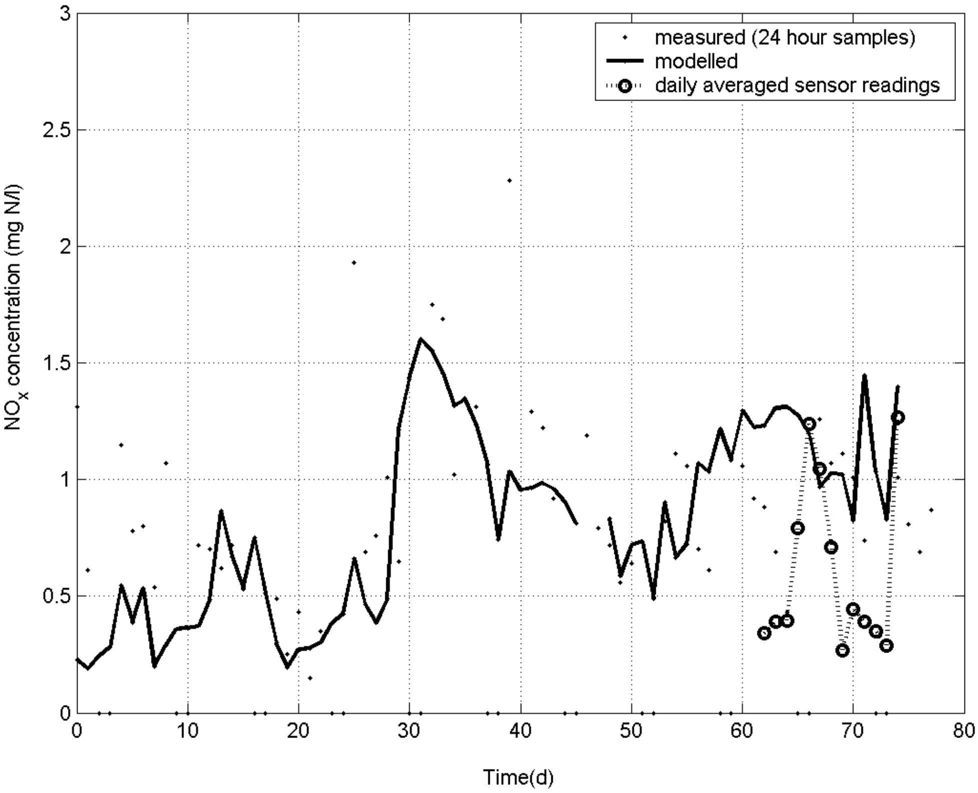


Figure 3.25 24 hour average NO_x concentration in effluent.

Moreover, during the measuring period a number of grab samples were taken to control the performance of the nitrate sensor. These samples provide another opportunity to check the performance of the 'Katwoude' model with respect to the denitrification. In figure 3.26 the simulation results for NO_x in the AT effluent for the complete measuring period and the grab samples are plotted. The figure shows that, apart from large differences on day 39 and 57, the simulated values are in accordance with the grab samples.

Based on the daily averages and the grab samples, it is concluded that the applied ASM1 model is capable of reproducing the general trend in NO_x concentrations properly.

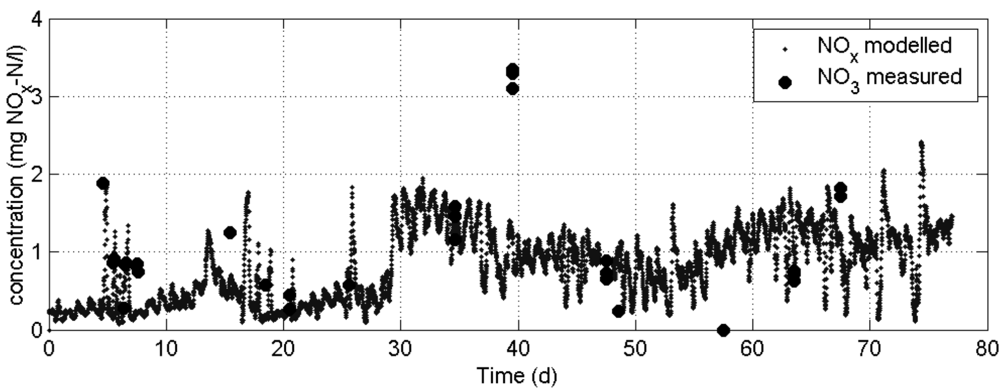


Figure 3.26 Simulated NO_x concentration in AT effluent compared with grab samples.

3.4.9 Phase VIII. Validation

The calibrated model was validated against data from the 8 day sampling period (23 February to 2 March 2001) [Meijer *et al.* (2002)]. With the available daily average values only a validation based on 'static' model results was possible. Therefore, the model parameters derived during the static calibration were used. Table 3.8 summarises the model parameters as applied in this thesis and by [Meijer *et al.* (2002)].

Table 3.8 Model adjustments compared to default values.

process	**Calibration values**	**Meijer *et al.* (2002)**
sludge production	$X_s/(X_s+X_i)$ = 0.47	$X_s/(X_s+X_i)$ = 0.20
nitrification	K_{O2} adjusted from 0.4 to 0.08 g O_2/m^3 K_{NH4} adjusted from 1 to 0.5 g NH_4-N/m^3.	oxygen setpoints in carrousel adjusted from 0.8 and 1.5 g O_2/m^3 respectively to 2 g O_2/m^3
denitrification	no adjustments	K_{OH} adjusted from 0.2 to 0.3 g O_2/m^3

The model settings as given in table 3.6 and the influent data from the 8 day 2001 measuring period were used to try to validate the model, with the surplus sludge rate set within the confidence interval given by [Meijer *et al.* (2002)]. The model results, see table 3.9, are in accordance with the measurements with respect to the sludge production and the nitrification. It is interesting to note that two different approaches, adjusting model parameters as applied in this thesis versus adjusting the environmental conditions, both result in a model representing the measured values properly.

The NO_x concentrations, however, are not in accordance with the measured data. This is probably due to the increased availability of slowly biodegradable substrate due to the increased ratio $X_s/(X_s+X_i)$. A simulation with the $X_s/(X_s+X_i)$ ratio according to Meijer *et al.* (2002) resulted in a NO_x concentration of 3.6 g N/m^3, which is in accordance with the values found by [Meijer *et al.* (2002)].

Table 3.9 Validation results based on averaged measurements (averaged measurements ± standard deviation) recorded during the 8-day sampling period from 23 February to 2 March 2001. The averaged flow was 12,380 m^3/d and the temperature 9 C. Simulation input and results are printed in *Italic*, the simulation results obtained by [Meijer *et al.* (2002)] in **Bold.**

	Influent g/m³		**Anoxic tank (R2) g/m³**			**Aeration tank (carrousel) g/m³**			**effluent g/m³**		
COD_{tot}	772.7±53	*773*	-	**4869**	*4930*	4840± 1263	**4845**	*4905*	43.7± 8.3	**43**	*48.5*
COD_{TSS}	379±93	*380*	-	**4806**	*4860*	4795± 1263	**4805**	*4863*	0± 8.9	**4.1**	*6.5*
COD_{mf}	393±41	*393*	-	**63**	*70*	44.3± 0.6	**40**	*42*	44± 8.3	**40**	*42*
NH_4	39.9±4.9	*38.3*	5.3±1.5	**5.1**	*5.6*	1.0± 0.6	**1.2**	*1.2*	0.7± 0.6	**1.2**	*1.2*
NO_x	0±0	*0*	0.8±0.6	**0.4**	*0.3*	3.3± 1.0	**3.3**	*2.0*	3.2± 0.7	**3.5**	*2*

Based on the validation results it is concluded that the model could be validated well with respect to sludge production and nitrification. With respect to the denitrification the validation results are less satisfying.

3.5 Sensitivity to influent fluctuations

The sensitivity of wwtp performance to influent fluctuations gives, in a problem oriented approach [Rauch *et al.* (1998)], the requirements for the quality of the influent data to be provided by either measurements or a sewer (process) model. The quality of the influent data can be specified by the types of errors in the data [(Haller (2002); Clemens (2001a)]:

- systematic errors or bias: deviation between the measured or simulated value and the actual value. Systematic errors affect the reliability of a measurement or simulation.

- random errors: random errors are related to the accuracy of the measures or the simulation. This accuracy is usually assessed by a 95% confidence interval. Gross errors are errors outside of the confidence interval and could be identified with an analysis of the outliers [(Haller (2002)].

The quality of measured influent data as well as methods to enhance this quality are described in detail by [Haller (2002)]. Within this thesis, the focus is on the requirements for sewer process models.
The results of simulations with sewer models have a limited accuracy and reliability due to shortcomings in e.g. the process descriptions, the numerical implementation of the process descriptions, the available input data and the database describing the system. The accuracy of simulations, measured by the confidence interval, could be increased to a certain extent by improving the quality of the input data [Meijer *et al.* (2002)] or the underlying database [Clemens (2001a)]. However, even after making every possible effort, uncertainties in model results will remain [Korving (2004)]. This is in analogy with measuring results, which also could theoretically be at best a very good approximation of the actual values.
With respect to the requirements for knowledge of sewer processes, the reliability of the influent data to be provided by sewer process models is the first issue to be addressed. Given the state of the art in knowledge of sewer processes, see chapter 2 and e.g. [Ashley *et al.* (1999)], sewer process model results are expected to contain systematic errors.

In this thesis, these systematic errors are considered small enough as long as the wwtp influent data provided by sewer models do not cause the pollutant levels in the wwtp effluent to deviate significantly from the levels reached without systematic error.

3.5.1 Method for assessing the impact of systematic errors

The influence of systematic errors in the influent was assessed by simulations with the fully calibrated and validated wwtp ‘Katwoude’ model. The calibrated model is assumed to represent the situation without systematic errors in the influent data (which is in reality not true, as shown by the analysis of the bias of the model residuals in section 3.4.7). The systematic errors are introduced by *varying a number of relevant parameters* in the influent data with a *constant multiple* during a *certain period*.

The parameters are selected based on a preliminary sensitivity analysis [Langeveld *et al.* (2003)], a sensitivity analysis performed by [Leinweber (2002)] and data from literature on the fluctuations in wastewater composition during storm events [e.g. Bertrand-Krajewski *et al.* (1995)]:

- flow (the main process driver)
- ammonium concentration (representing the effect of dilution of the nitrogen load)
- suspended COD, comprising X_i (inert suspended COD) and X_s (slowly biodegradable COD) (representing the effect of the release of sewer sediment during a storm event).

The selected parameters were varied in the range from – 50% to + 100% for 2 storm events, see figure 3.27.

The duration of the period of introducing systematic errors is rather important, as shown by [Langeveld *et al.* (2003)]. Short term fluctuations are easily buffered in a wwtp, especially in the modern low loaded wwtps dominantly applied in the Netherlands [CBS (2003)]. A distinction was made between the phases where the ammonium concentration of the influent falls and rises. For both events three situations were analysed: a systematic error in the aforementioned influent parameters during the whole storm event, and a systematic error during either the ‘dilution’ or the ‘recovery’ phase.

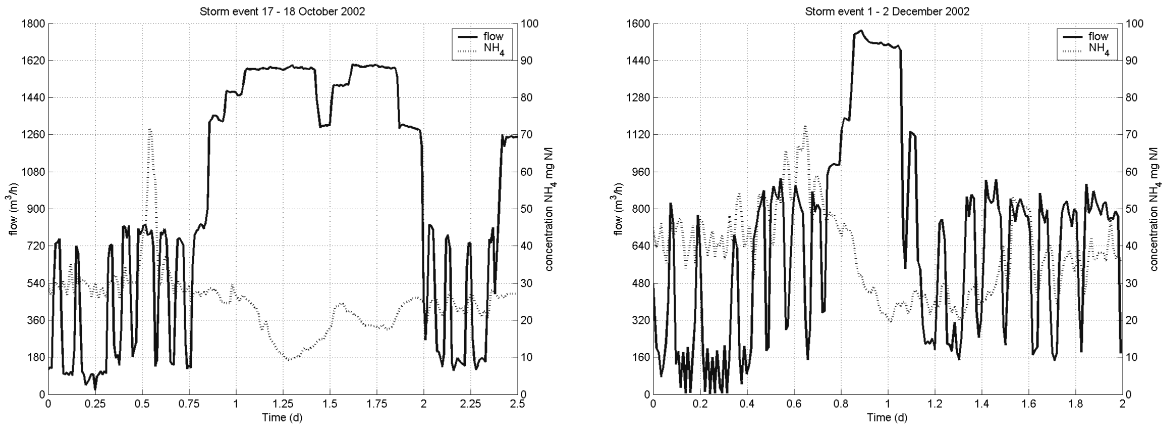


Figure 3.27 Storm events used in sensitivity analysis. The temperature of the activated sludge was 19°C on 17-18 October and 14°C on 1-2 December.

The sensitivity to the defined systematic errors was quantified by simulating the wwtp 'Katwoude' model with the influent containing systematic errors. In total 36 simulations (3 parameters, 3 periods and 12 intermediate values within the –50 to + 100 % range) were performed for both storm events.

The simulation results with the added systematic errors, as exemplified by figure 3.28, were compared with the original situation on the deviation in daily averaged ammonium concentration in the AT effluent. This deviation was accepted as long as it does not exceed plus or minus 0.5 mg N/l. As soon as an added systematic error causes the daily averaged ammonium concentration in the AT effluent to deviate more than this acceptable deviation, the systematic error is not to be accepted.
The level of 0.5 mg N/l was selected based on expert judgement, as no levels of acceptable deviations has been found in literature. The 0.5 mg N/l value was chosen in between the most stringent effluent standard of 1 mg N/l and the measuring accuracy of 0.1 mg N/l achievable with today's ammonium sensors [Rieger *et al.* (2002)]. A sensitivity analysis showed the final result not to be very sensitive to the exact value of 0.5 mg N/l of this criterion, see appendix VII.

In addition to this criterion of deviation from the daily average concentration of ammonium in the effluent 3 other criteria were tested, see appendix VII. Again, the final result of the analysis did not vary much compared with the applied criterion of deviation from the daily average ammonium concentration in the effluent.

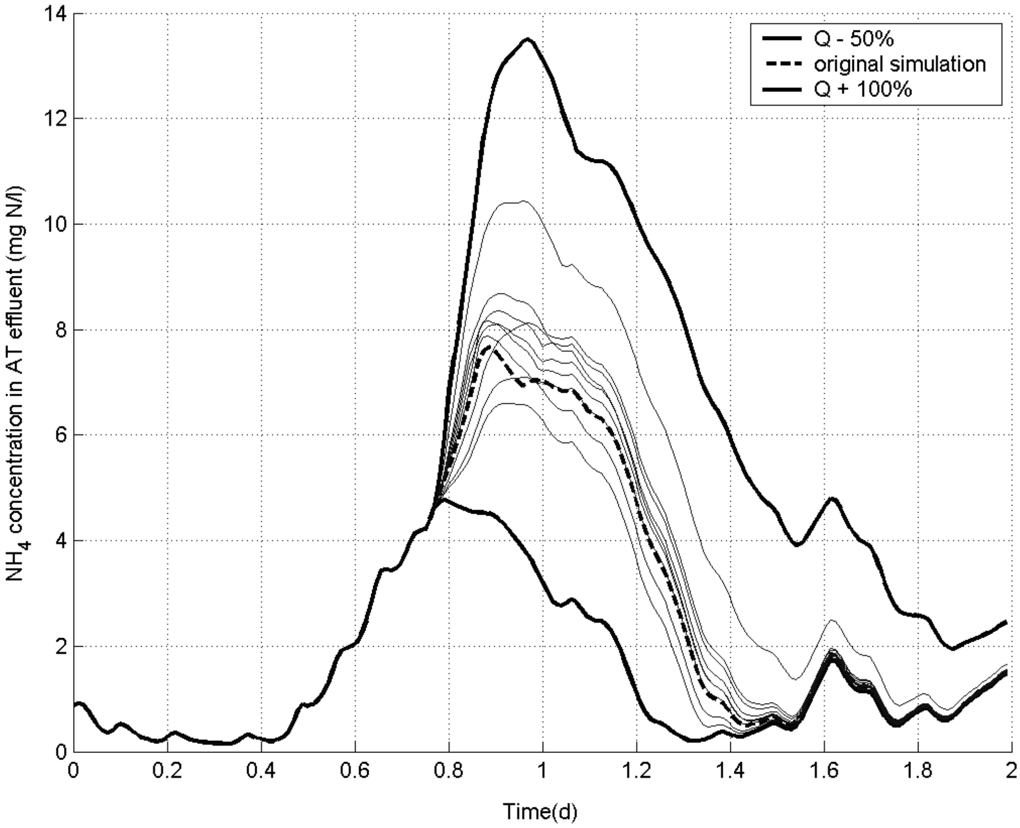


Figure 3.28 Effect of systematic errors in influent flow during the dilution phase of the storm event (time from 0.77 to 1.03) on ammonium concentration in the AT effluent. The thick lines represent the outer ranges (-50% and + 100% respectively), the thin lines represent simulation results for systematic errors within this interval.

3.5.2 Results of assessing the impact of systematic errors

Table 3.11 shows the results of the analysis of the effect of systematic errors in the aforementioned influent parameters for the storm of 1 December 2002. The figures in the table represent the systematic errors that cause the simulation result to deviate more than ±0.5 mg NH_4-N/l. E.g. if the flow during the dilution phase of the storm event contains a systematic error larger than 25% the deviation of the daily ammonium concentration in the effluent just exceeds the value of 0.5 mg N/l.

Table 3.11 Systematic errors, rounded to 5% values, causing significant deviations in simulation results (in %). Storm 1 December 2002.

		flow			**NH_4 in influent**			**COD_{susp} in influent**		
criterion		dilution phase	recovery phase	total storm event	dilution phase	recovery phase	total storm event	dilution phase	recovery phase	total storm event
Averaged NH_4	-0.5 mg N/l	-25	-	-25	-25	-	-20	-	-	-
	+ 0.5 mg N/l	25	30	10	25	40	15	-	-	-

The results have the same order of magnitude for the parameters flow and ammonium, confirming the statement from section 3.3 that the influent load of ammonium is the important factor. Fluctuations in the influent concentration of COD within the –50 to + 100% do not affect the ammonium concentration in the effluent significantly.

The period during which the systematic errors were introduced, however, affects the level of significant fluctuations. Apparently, systematic errors during the ‘dilution’ phase of the storm event have a stronger impact on the effluent quality than systematic errors introduced in the ‘recovery’ phase.

The results for the storm of 1 December 2002 (as shown in table 3.11) and the results for the storm event of 17 October (see table 3.12) have the same order of magnitude, even though the characteristics of the storm events differ significantly. This enables the formulation of requirements for wwtp influent data, taking the most stringent requirements from table 3.11 and 3.12 into account, as shown in table 3.13.

Table 3.12 Systematic errors causing significant deviations in simulation results (in %). Storm 17 October 2002.

		flow			NH_4 in influent			COD_{susp} in influent		
criterion		dilution phase	recovery phase	total storm event	dilution phase	recovery phase	total storm event	dilution phase	recovery phase	total storm event
Averaged NH_4	-0.5 mg N/l	-50	-	-10	-50	-	-20	-	-	-
	+ 0.5 mg N/l	25	50	10	25	50	15	-	-	-

The boundaries for allowable systematic errors in influent data of table 3.13 are in accordance with the results obtained for a theoretical ‘average’ Dutch wwtp (100.000 i.e., low loaded, with primary clarifier) [Langeveld *et al.* (2003)]. In addition, literature [i.e. Bruns (1999); Rauch *et al.* (1998); Leinweber (2002); Seggelke (2002)] confirms the sensitivity of the nitrification process at wwtps to fluctuations in influent loads, even though none of the authors quantifies this sensitivity.

Table 3.13 Boundaries for allowable systematic errors in influent data (in %).

	flow			NH_4 in influent			COD_{susp} in influent		
criterion	dilution phase	recovery phase	total storm event	dilution phase	recovery phase	total storm event	dilution phase	recovery phase	total storm event
lower boundary	-25	-50	-10	-50	-50	-25	-	-	-
upper boundary	25	30	10	25	40	15	-	-	-

3.6 Conclusions

Based on the analysis of the impact of influent fluctuations on the quality of wwtp effluent, the following conclusions have been drawn:

Data analysis: relation between influent and effluent

- influent and effluent data from ‘routine’ measurements at Dutch wwtps do not hold much information on the sensitivity of wwtp effluent quality to influent fluctuations. The only noticeable effect is the reduction of nitrogen removal efficiency due to increased wastewater flows.
- data measured at a 15 minutes interval provides more information with respect to the impact of influent fluctuations on effluent quality, although a correlation analysis showed only moderate correlation coefficients. A correlation was found between:
    - influent flow and ammonium concentration in the AT effluent
    - concentration ammonium in the influent and ammonium concentration in the AT effluent

  The sensor data also shows a decrease in nitrification efficiency due to increased wastewater flows.

Modelling

- the ASM 1 'Katwoude' model could be calibrated with one single set of model parameter values for the complete measuring period from 19 September 2002 to 5 December 2002. Validation of the model with data from 23 February 2001 to 2 March 2001 showed that the model could be validated well with respect to sludge production and nitrification. Consequently, a well calibrated and validated ASM 1 model for wwtp Katwoude was obtained, which properly reproduces the dynamic response of the wwtp to transient loadings with regard to the final effluent quality.

Requirements for sewer process models

- the sensitivity analysis with the 'Katwoude' wwtp model resulted in minimum requirements for sewer process models to be used within an integrated approach. The results given in table 3.14 show that the flow and the ammonium concentration in the influent are sensitive parameters with respect to wwtp effluent quality. The COD concentration is less important and therefore larger errors with respect to the quality of COD influent data can be accepted.

Table 3.14 Boundaries for allowable systematic errors in sewer model results to be used in an analysis of wwtp performance (in %).

	flow			**NH_4 in influent**			**COD_{susp} in influent**		
criterion	dilution phase	recovery phase	total storm event	dilution phase	recovery phase	total storm event	dilution phase	recovery phase	total storm event
lower boundary	-25	-50	-10	-50	-50	-25	-	-	-
upper boundary	25	30	10	25	40	15	-	-	-

Chapter 4 Sewer process modelling

4.1 Introduction

The sensitivity of the performance of a wwtp, in terms of effluent quality, to fluctuations in the influent composition and flow was analysed in chapter 3. This analysis resulted in minimum requirements for influent data to be used to assess the performance of a wwtp under transient conditions, see table 3.14. These influent data can originate from either measurements or sewer models. In this chapter, sewer models as implemented in current software products and knowledge of relevant sewer processes are confronted with these minimum requirements in order to analyse their applicability for studying the interactions within wastewater systems.

Table 4.1 shows the main wastewater components, the associated sewer processes and their relative relevance with respect to the interactions within wastewater systems. The flow and the soluble and fine suspended fractions of the influent are important with respect to wwtp effluent quality, see chapter 3. The larger suspended fractions and their associated pollutant load of especially COD [Ristenpart *et al.* (1995)] are less important with respect to the wwtp. The biodegradability of the wastewater may be relevant with respect to the wwtp effluent quality. The sewer sediments, however, are not much related to wwtp effluent quality, as demonstrated in chapter 3. Even the organic part of the sewer sediment will easily be removed by a wwtp, as the typical design load of a secondary clarifier of 0.7 m/h suffices to retain organic sediment with a diameter of 50 μm, assuming free settling. In reality even smaller particles will be retained in the sludge blanket. Consequently, in this chapter only the hydrodynamics, solute transport, suspended solids transport and biotransformations are studied.

Table 4.1 Wastewater components, associated processes and their relevance with respect to the interactions within wastewater systems.

Parameter	sewer process involved	relevance
Flow	hydrodynamics	high
soluble/fine suspended fractions (< 63 μm)	solute transport	high
susp./settleable fractions (63 μm –100 μm)	sedimentation/resuspension	low
sediment fraction (> 100 μm)	sediment transport	none
biodegradability	biotransformations	probably

4.2 Hydrodynamic modelling of sewer systems

Current hydrodynamic models are widely applied to simulate the hydrodynamics within sewer systems [Clemens (2001a)]. In the Netherlands, the predominant, commercially available, software packages are Hydroworks/Infoworks, Mouse and SOBEK. All these models are based on the De Saint Venant equations (equation 2.1 and 2.2). In principle, the models are capable of properly describing the hydrodynamics [Clemens (2001a)]. However, a good description of the hydrodynamic transport process does not guarantee the model results to be reliable and accurate. [Clemens (2001a)] lists the following difficulties in using hydrodynamic models:

- acquiring a correct database containing all geometrical information of the sewer network
- acquiring a good description of the contributing area in terms of sizes and types of surface as well as the points of discharge into the sewer system
- selecting a suitable runoff process with correct runoff parameters
- obtaining well-calibrated models with a quantification of the quality of the calibration results

[Clemens (2001a)] describes means to deal with each of the aforementioned difficulties, illustrated by the case The Hoven, a small and flat catchment near Deventer, the Netherlands. The main characteristics of the sewer system are given in table 4.2.

Table 4.2 Sewer system 'De Hoven' [Clemens (2001a)].

contributing area	12.69 ha
number of inhabitants	2,200
storage volume	865 m^3 (6.8 mm)
pumping capacity	119 m^3/h (0.9 mm/h)
number of CSO structures	3

Based on the quantification and subsequent analysis of the quality of the calibration results of 5 storm events [Clemens (2001a)] concludes:

- it is possible to obtain a well-calibrated hydrodynamic model with residuals (differences between model results and measurements, in this case water levels) with an order of magnitude of 5 to 10 centimetres, provided that the database describing the sewer system and contributing areas has a high quality;
- the runoff process is the 'weakest link' in hydrodynamic modelling;
- the parameters describing the runoff process have a low portability.

In this thesis, the general validity of Clemens' conclusions has been tested for the mildly sloping catchment of Loenen, having characteristics as shown in table 4.3.

Table 4.3 Sewer system 'Loenen'.

contributing area	in total 56.5 ha, with 23.4 ha impervious
number of inhabitants	2,100
storage volume	900 m^3 (3.8 mm)
pumping capacity	209 m^3/h, of which 141 m^3/h available for runoff i.e. the 'pump over capacity' is 0.6 mm/h
number of CSO structures	2

4.2.1 Material and methods

The sewer system of Loenen was equipped with a monitoring network especially designed for the calibration of a hydrodynamic model [Witteveen+Bos (2000)] using the design method for monitoring networks developed by Clemens [Clemens (2001a, 2001b, 2002)]. The monitoring system, as shown in figure 4.1, comprises 2 automatic rain gauges, 7 water level sensors installed at invert level, 4 water level sensors installed somewhat higher in the manhole and 1 water level sensor in the pond receiving the CSO discharge. The water level sensors have an absolute accuracy interval of 0.02 m (95% confidence interval). Moreover, the pumped volumes have been registered at a 5 minutes interval.
The monitoring network has been functioning well from 28/08/01 till 26/12/01. Table 4.4 gives some statistics of the measuring period. The total precipitation during the measuring period amounts 593 mm, which is equivalent to approximately 1200 mm/a. Compared to the average annual precipitation of 800 mm in the Netherlands, the measured precipitation is relatively high. A comparison with the average number of storms with a precipitation of over 1 mm/d and 5 mm/d for De Bilt, the location of the Dutch meteorological institute KNMI, shows that the total number of storm events is not unusual. The relatively high amount of precipitation in the measuring period is due to higher volumes per storm event.

Table 4.4 Statistics of measuring period 28/06/01-26/12/01/ in Loenen.

	Loenen, measuring period	**Extrapolated to annual values**	**Average annual values (De Bilt, 1955-1979)**
total number of days	180		
number of days with precipitation > 1 mm	80	160	129
number of days with precipitation > 5 mm	40	80	54
number of CSO events	16	32	
total precipitation	593 mm	1200 mm	800 mm

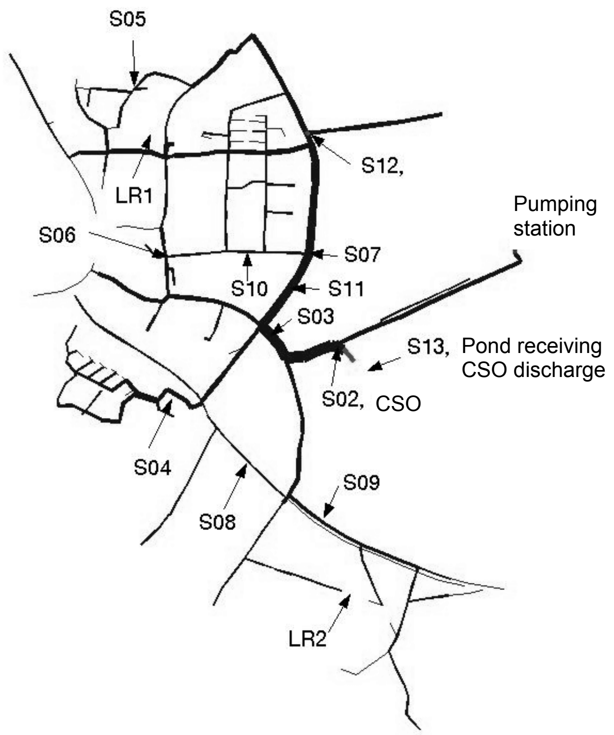


Figure 4.1 Monitoring network Loenen (schematic) with locations of the equipment: rain gauges LR1 and LR2, level sensors in the sewer system S02 to S12 and level sensor S13 in the pond. The pond discharges to a small brook through a trash rack, which showed to be vulnerable to blocking, causing the water level in the pond to rise above the weir level. The water level sensor planned at the pumping station has not been installed due to the site being inaccessible during the foot-and-mouth disease crisis during the summer of 2001.

The measured data provided by the monitoring network was used to calibrate the hydrodynamic model, applying the calibration procedure developed by [Clemens (2001a)]. The applied event based calibration procedure, which could be iterative from step 2 till 5, consists of the following steps:

check quality of structural database of sewer system
select storm event for calibration
select set of parameters for calibration
try to find ‘optimal’ value for model parameters
analyse residuals

The hydrodynamic model used in this thesis is Hydroworks™, vs 6.0.

4.2.2 Results and discussion

The calibration of the hydrodynamic model of Loenen has resulted in 15 storm events being fully calibrated. A detailed discussion of the calibration results is given for the storm event of 18/07/01, followed by a summary of the results from all calibrated storm events.

Quality of structural database of sewer system

The structural database of the sewer system of Loenen already had a rather high quality at the beginning of the study. Nevertheless, a number of database errors were present, of which the height of the weir of the CSO construction was most noticeable. During the 1980s the CSO of the sewer system of Loenen was intensively monitored as part of the NWRW research project [NWRW (1989)]. Within the NWRW project, the original concrete weir has been lowered and a sharp weir was installed, in order to be able to determine the overflow volume with some accuracy. At the end of the NWRW project, the sharp weir has been removed, however, without restoring the original overflow structure. As a result, an error in the database existed, only to be noticed after the first trials for calibration being rather unsuccessful above the level of 17.75 m^+ NAP, see figure 4.2.

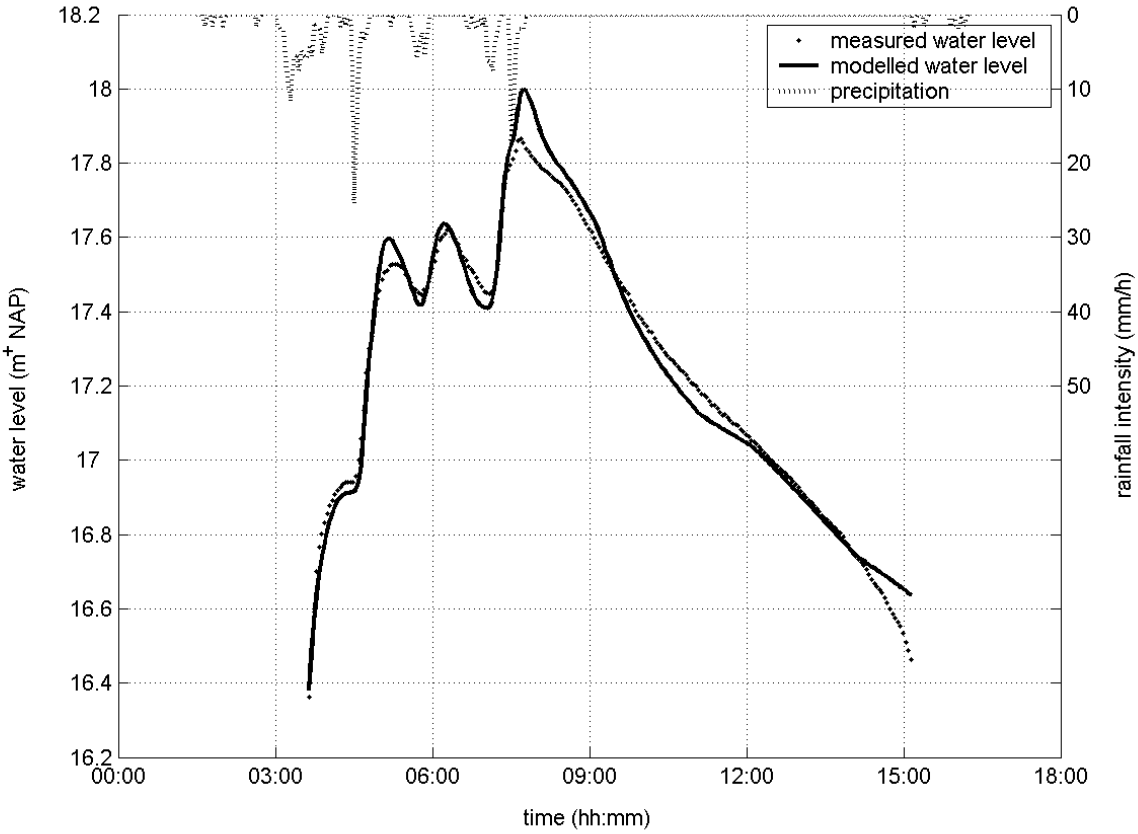


Figure 4.2 Modelling results with incorrect crest level of overflow weir. Loenen, storm event 18 July 2001, location S02.

Apart from an incorrect weir level, another interesting phenomenon occurred during the measuring period: the rise of the water level of the pond receiving the CSO discharges above the CSO weir, causing interference of the pond with the sewer system. During 11 of the 16 storm events causing the CSO to discharge the water level in the pond rose above the crest level, as shown in figure 4.3. Fortunately, a water level sensor, S13, was installed in the receiving pond, enabling modelling of these storm events with an additional boundary condition. Without this measured boundary condition, the storm events with interference with the receiving waters could not be modelled properly.

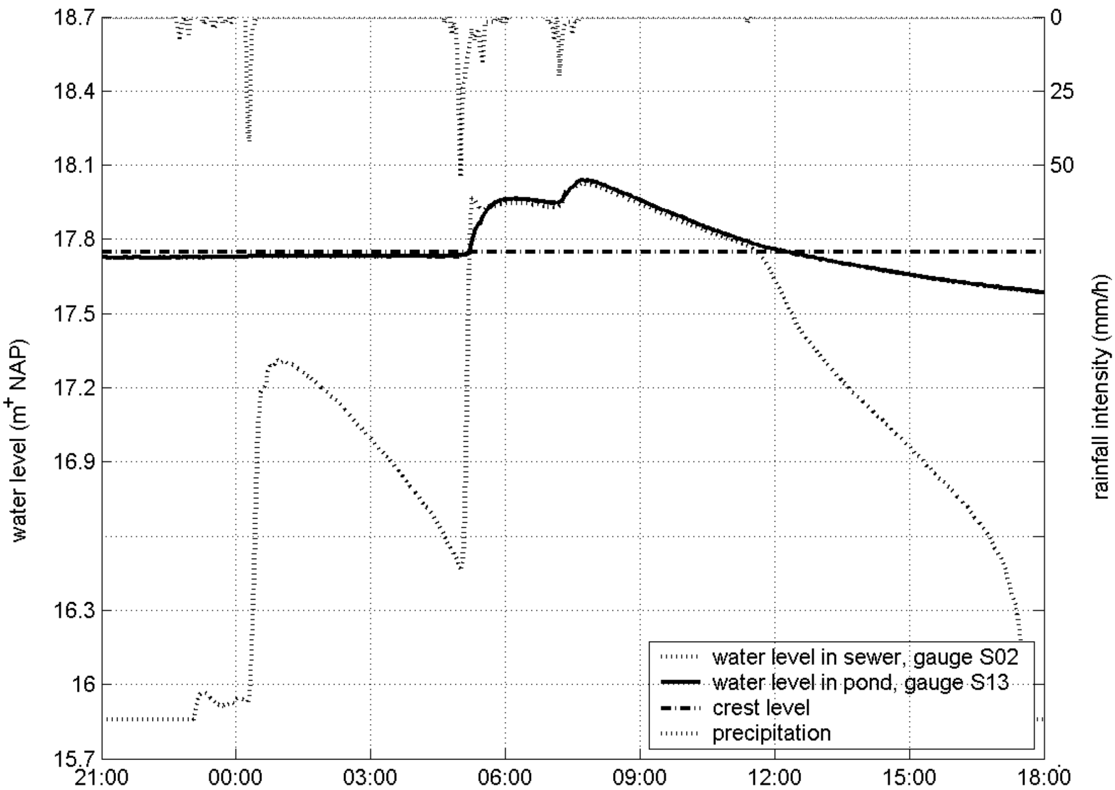


Figure 4.3 Water level in CSO structure (S02) and in the receiving water (i.e. pond: S13) during storm event 16/08/01. At the onset of the storm event, the water level in the pond is just below the crest level of the weir. Just before 06:00 hours, the sewer system starts to overflow and as a result, the water level in the pond starts to rise. After 08:00 no more precipitation was recorded and the pumping station starts emptying the sewer system. At 12:00, as the water level in the pond and the sewer system reaches the level of the overflow weir, the water level in the sewer system starts dropping more rapidly, as the pond is no longer discharging into the sewer system. The continuing decrease of the water level in the pond after 12:00 is due to the fact that the outlet of the pond into a local stream was cleaned by the storm water entering the pond through the CSO.

Selection of storm events for calibration

40 days with a total precipitation of over 5 mm have been recorded, see table 4.4. However, on many of these days the water level in the sewer system hardly rose and, consequently, the measured data does not contain much information suitable for calibration. Moreover, during the final weeks of the measuring period typical winter conditions with snow and snow melt were observed, which are not taken into account at all in the applied (Desbordes) runoff model. Consequently, all storms during this period could not be calibrated [Henckens *et al.* (2003)]. Therefore, only 13 of the 16 CSO events have been calibrated, supplemented with 2 storm events where the water levels in the sewer system rose to a level just below crest level. Table 4.5 gives an overview of return periods of the calibrated storm events. The return periods are based on the intensity-duration-frequency (IDF) curves for De Bilt, which are based on 72 years of rainfall data [Buishand and Velds (1980), Bouwknegt and Gelok (1988)]. The table illustrates that storms with a wide range of return periods were evaluated, thus representing a wide range of operational conditions.

Table 4.5 Return periods of calibrated storm events.

return period (year^{-1})	storm event
> 2.5	03/08/01
1 – 2	19/07/01; 07/08/01; 25/09/01
0.5 –1	16/08/01; 02/10/01
0.25-0.5	30/06/01; 27/08/01; 17/09/01; 07/10/01
0.2 – 0.25	23/07/01
0.1 –0.2	18/07/01; 29/11/01
0.05 – 0.1	07/11/01
< 0.05	23/10/01

Selection of a set of parameters for calibration

The selection of a limited set of parameters for calibration of the hydrodynamic model of Loenen is not a trivial task. Two types of parameters have been distinguished:

- physical characteristics:
    - hydraulic roughness of the 367 conduits (and 354 manholes)
    - overflow coefficient for the weir in the CSO structure
- inflow parameters:
    - dry weather flow, consisting of wastewater and infiltration/inflow
    - storm runoff. In the Netherlands, the NWRW 4.3 inflow model is normally applied, identifying as many as 12 types of contributing surfaces [Stichting Rioned (1999)], see appendix IX. In Loenen, only four types of contributing areas are distinguished: flat surfaces, either impervious or semi-pervious, and flat and inclining roofs. All types of surfaces are characterised by their routing coefficient and the initial storage losses, whereas the Horton infiltration model, comprising 4 parameters [Horton (1940)], only applies to the flat semi-pervious surfaces. In total, this results in 12 parameters for calibration.

In the case of Loenen, not all of the aforementioned parameters were taken into account. The hydraulic roughness was set for all conduits to a fixed value of 4 mm. This assumption is valid as flow velocities in Dutch sewer systems are generally rather low, i.e. less than 1 m/s, and consequently, the hydraulic roughness is not a sensitive parameter [Clemens (2001a)]. Besides, in the Horton infiltration model only the maximum infiltration capacity was taken into account, as no data is available on the decline and recovery of the infiltration capacity. As a result, the set of parameters to be used in the calibration is limited to 11 parameters, as shown in table 4.6.

Table 4.6 Set of parameters for calibration of hydrodynamic model for Loenen. Default values before calibration are given in annex IX

parameter	description	unit
N1	dwf	m^3/h.ha
B2	initial losses on flat, impervious areas	mm
B5	initial losses on flat, semi-pervious areas	mm
B7	initial losses on inclining roofs	mm
B8	initial losses on flat roofs	mm
F2	routing coefficient for flat, impervious areas	s
F5	routing coefficient for flat, semi-pervious areas	s
F7	routing coefficient for inclining roofs	s
F8	routing coefficient for flat roofs	s
I5	maximum infiltration capacity semi-pervious areas	mm/h
CC	overflow coefficient	$m^{0.5}$/s

Optimisation of model parameter values

The search for the combination of parameter values giving the 'best' fit is the phase of the calibration receiving most attention in literature [Clemens (2001a)]. In this case, the software and methodology as developed by [Clemens (2001a)] were applied. More information can be found in [Boomgaard *et al.* (2002a) and Clemens (2001c)]. Based on the Maximum Likelihood Estimates (MLE) method, a genetic algorithm was applied to identify a promising parameter set, followed by the Levenberg-Marquart algorithm for fine-tuning of the parameter values of the promising parameter set.

The result of this phase of the calibration procedure is a set of model parameters, their identifiability and cross correlation. The information on the model parameters obtained in this phase can be used to try to further reduce the parameter set while maintaining the achieved level of mean squared error (MSE) [Clemens, (2001a); Korving (2004)]. A detailed discussion of the possibilities of further reducing the number of parameters is considered to be beyond the scope of this thesis.

Analysis of residuals
The analysis of the residuals gives information on the quality of the calibration results, which are shown in figure 4.4 and 4.5. The residuals have an order of magnitude of ± 5 cm, which is in accordance with the order of magnitude found for sewer system De Hoven [Clemens (2001a)]. The residuals are, like the residuals of the ASM model 'Katwoude' in section 3.4.7, analysed with respect to:

- probability density function (compared to the normal distribution)
- mean squared error
- relative bias

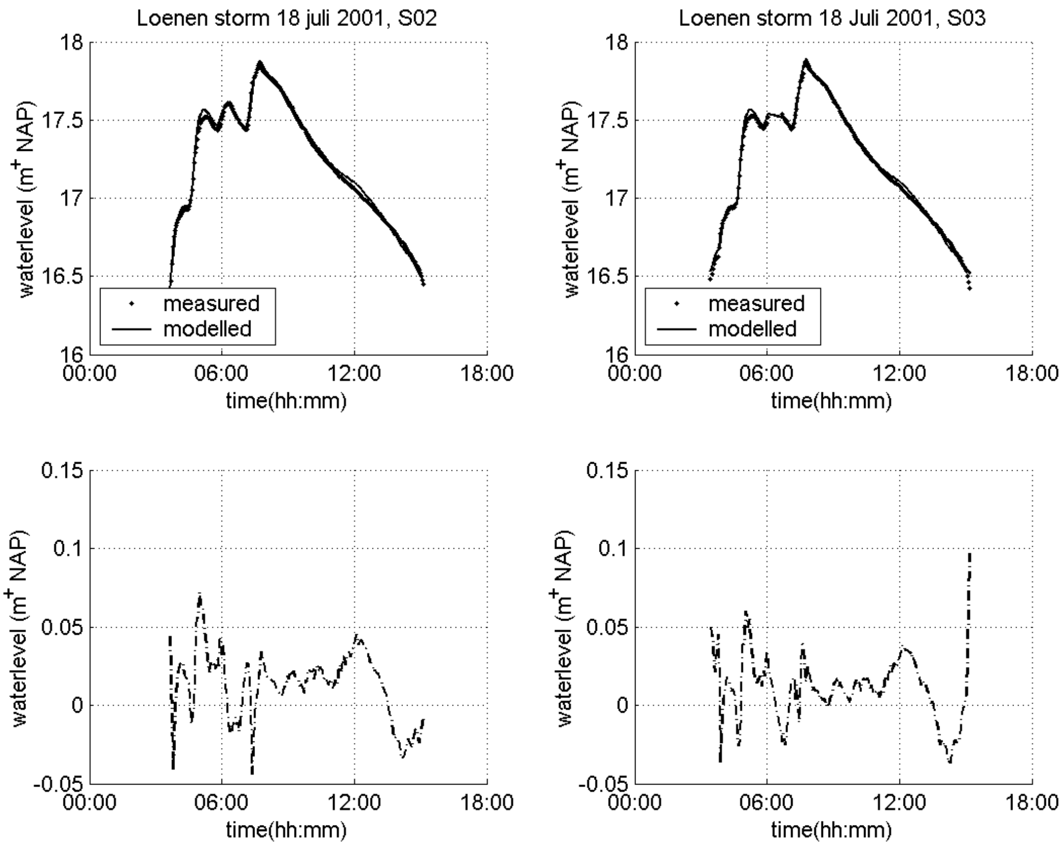


Figure 4.4 Measured and modelled water levels during the storm of 18/07/01 in Loenen. The numbers of the measuring locations refer to the sites shown in figure 4.1.

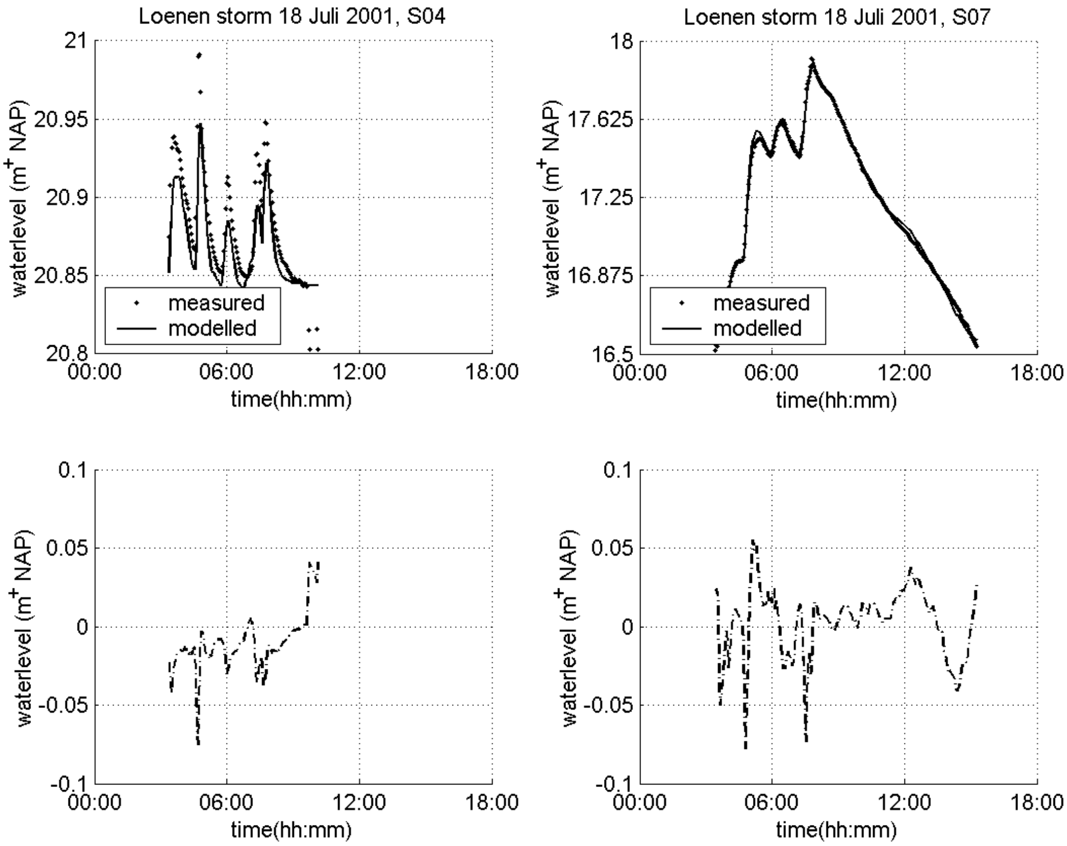


Figure 4.5 Measured and modelled water levels during the storm of 18/07/01 in Loenen. The numbers of the measuring locations refer to the sites shown in figure 4.1.

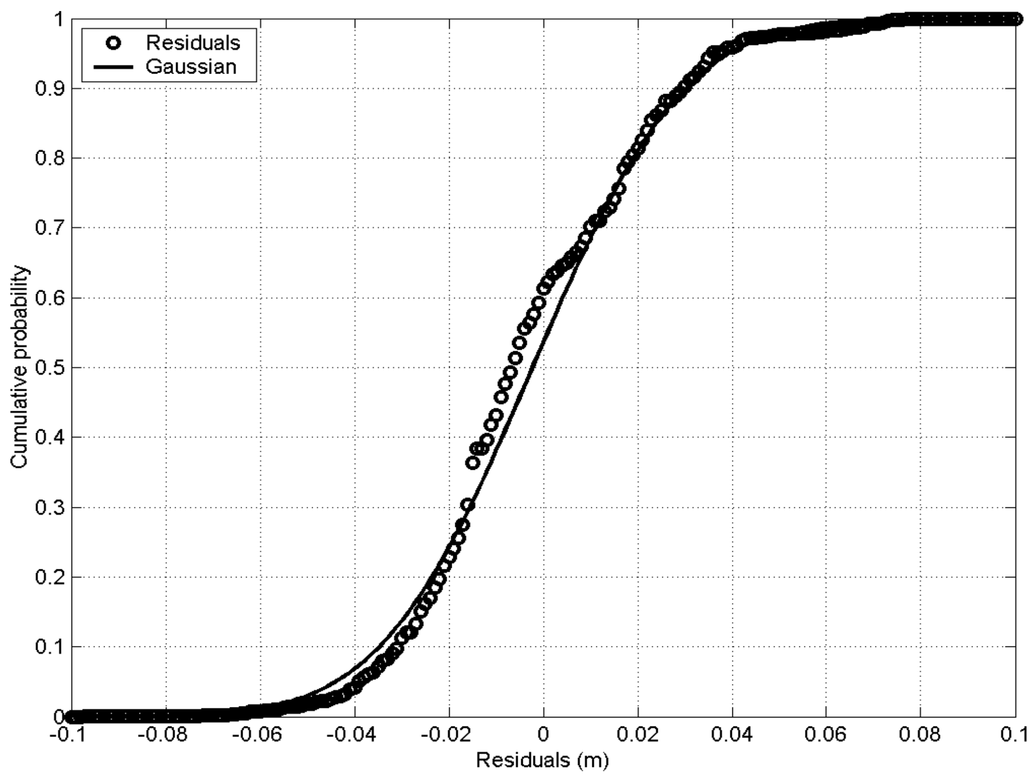


Figure 4.6 Cumulative probability density function of residuals for all measuring locations of storm 18/07/01.

The probability density of the residuals for storm 18/07/01, shown in figure 4.6, deviates slightly from Gaussian, indicating that systematic errors are limited. The statistic key-values for the residuals, in terms of mean squared error (MSE), variation (VAR) and standard deviation (STD), given in table 4.7, are equivalent to the values found for De Hoven [Clemens (2001a)].

Table 4.7 Statistics of residuals for storm 18/07/01.

	S02	S03	S04	S07	S12	Total
MSE	$6.22*10^{-4}$	$5.27*10^{-4}$	$4.17*10^{-4}$	$4.98*10^{-4}$	$2.37*10^{-3}$	$6.45*10^{-4}$
STD	$2.15*10^{-2}$	$2.04*10^{-2}$	$1.52*10^{-2}$	$2.23*10^{-2}$	$2.15*10^{-2}$	$2.53*10^{-2}$
VAR	$4.61*10^{-4}$	$4.14*10^{-4}$	$2.31*10^{-4}$	$4.96*10^{-4}$	$4.63*10^{-4}$	$6.42*10^{-4}$

The relative bias, defined as bias/σ_r^2 and calculated for a shifting time window, is shown in figure 4.7 for four gauges. The relative bias is substantial for all gauges, i.e. bias > 0.2 σ_r^2 [Clemens (2001a)], especially during the emptying phase of the storm event. This phenomenon was also observed in the case De Hoven [Clemens (2001a)].

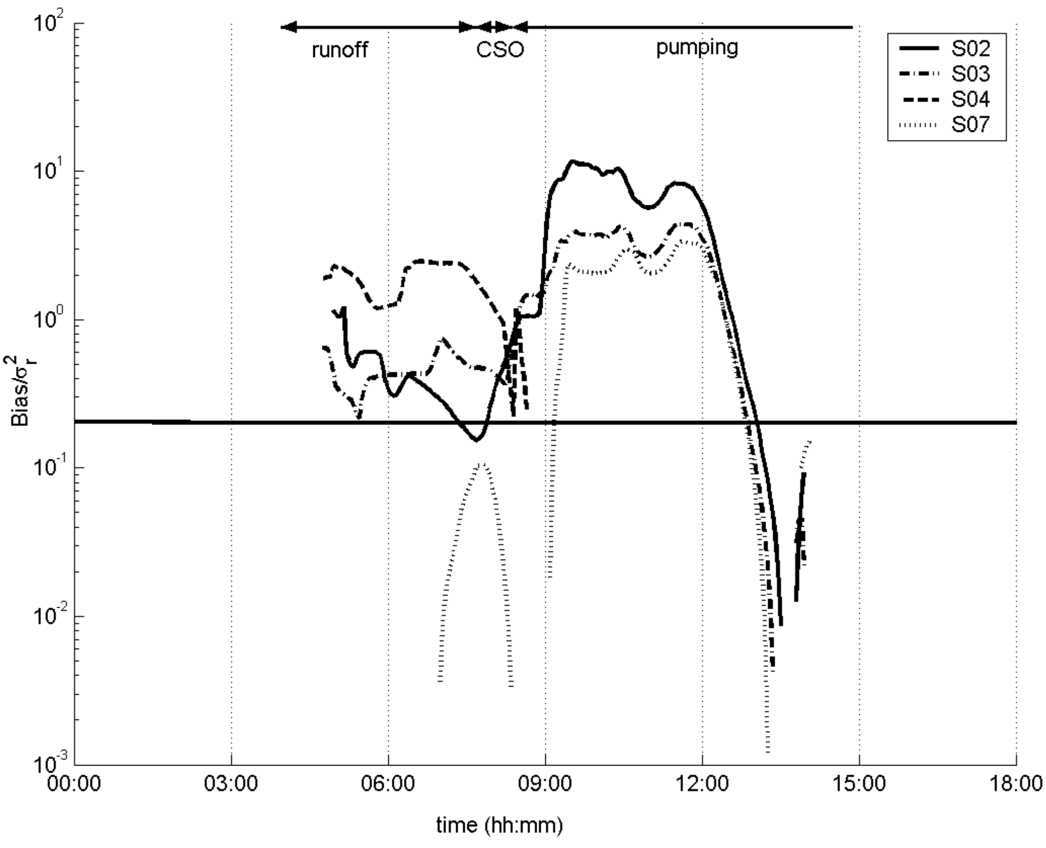


Figure 4.7 Relative bias for storm 18/07/01 in a shifting time window of 180 minutes for 4 gauges. Gauge S12 is not shown as for this gauge not enough measurements were available. All gauges show considerable bias, especially during the falling limb of the storm event (after 08:00 hour).

As mentioned before, apart from storm 18/07/01 14 other storms were calibrated and fully analysed. A detailed presentation of all storms is given in appendix X. In this section, only a brief overview of the quality of the calibrations in terms of the mean squared error and the probability distributions is given, see figures 4.8 to 4.11. The results show that the mean squared error for all storms but 30/06/01 is one order of magnitude larger than for the storm of 18/07/01. In addition, the probability distribution of all storms deviates, although in a varying extent from the Gaussian distribution. This indicates the presence of systematic errors, which are likely due to:

- the initial condition of the contributing areas implicitly accounted for by the value of the runoff parameters from the set of calibration parameters given in table 4.6. The initial losses depend on the temperature and humidity of the contributing surface at the beginning of the storm event. As soon as the storm event comprises more than 1 sub-event, it is impossible to account for the initial losses of all subevents properly. Consequently, the model results will contain systematic errors;
- sediment deposited in and transported through the sewer system not accounted for in the simulations;
- interference of the water level in the receiving pond with the water level in the sewer system;
- the spatial variability of the rainfall is for the case Loenen not relevant given the catchment size of approximately 2 km^2 [Willems (2000)].

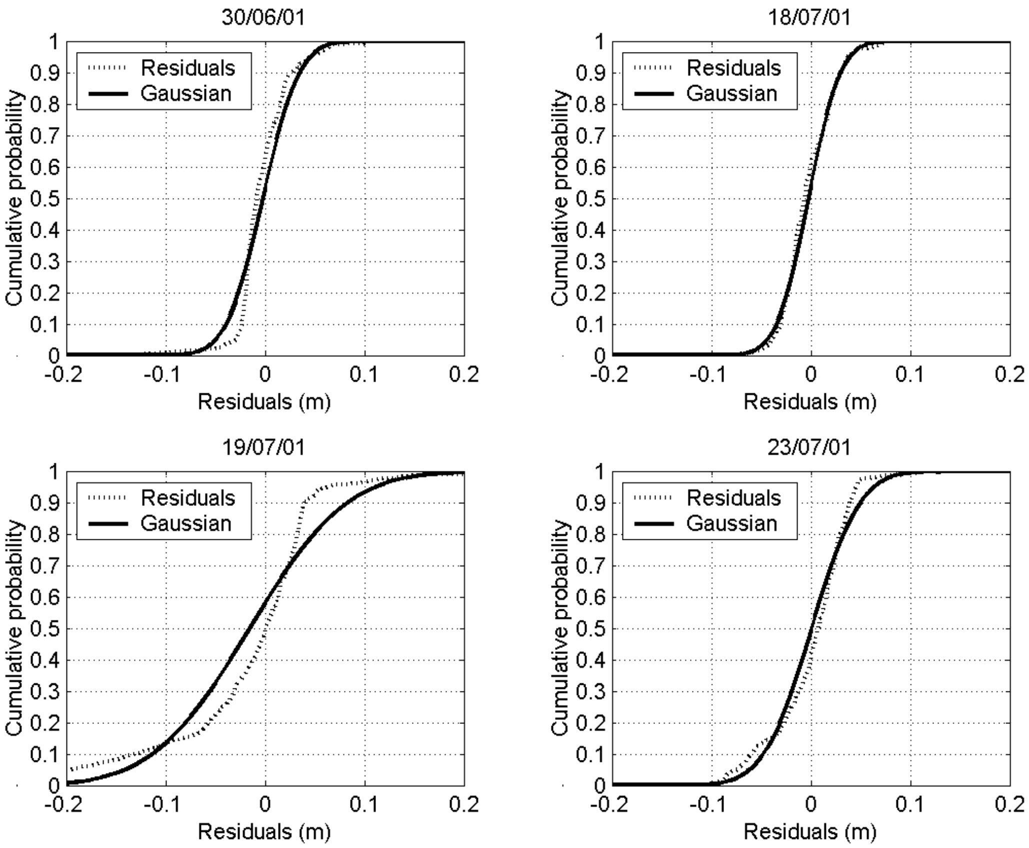


Figure 4.8 Cumulative probability distribution of residuals for storm 30/06/01 (MSE = $8.44*10^{-4}$ m^2), 18/07/01 (MSE = $6.45*10^{-4}$ m^2), 19/07/01 (MSE = $6.11*10^{-3}$ m^2) and 23/07/01 (MSE = $1.46*10^{-3}$ m^2).

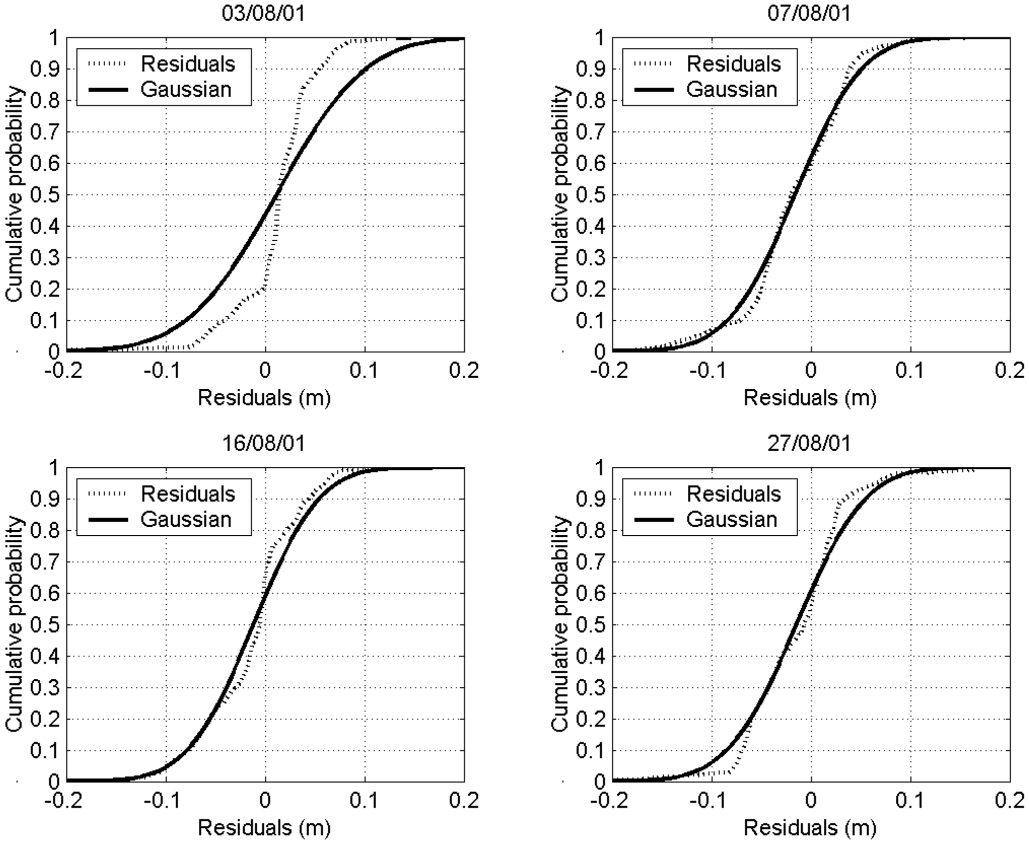


Figure 4.9 Cumulative probability distribution of residuals for four storms in August 2001: 03/08/01 (MSE = $5.13*10^{-3}$ m^2), 07/08/01 (MSE = $3.02*10^{-3}$ m^2), 16/08/01 (MSE = $2.82*10^{-3}$ m^2) and 27/08/01 (MSE = $3.18*10^{-3}$ m^2).

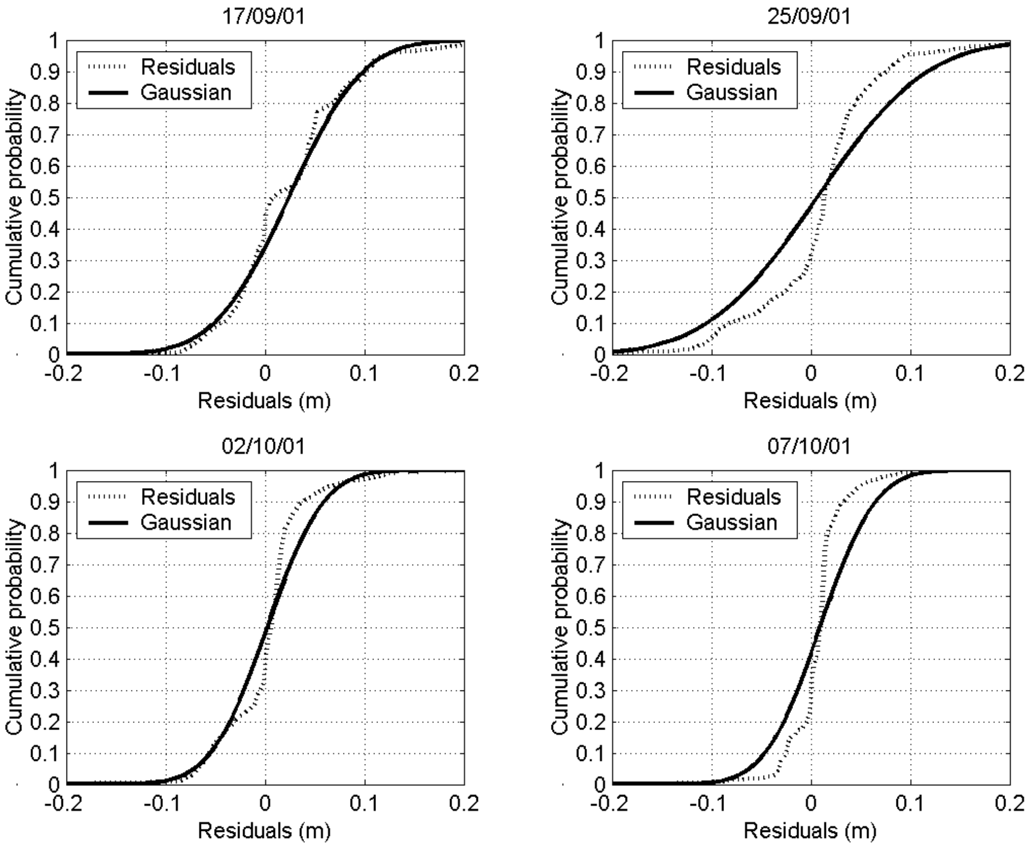


Figure 4.10 Cumulative probability distribution of residuals for storms: 17/09/01 (MSE = $4.03*10^{-3}$ m^2), 25/09/01 (MSE = $7.55*10^{-3}$ m^2), 02/10/01 (MSE = $2.01*10^{-3}$) and 07/10/01 (MSE = $1.99*10^{-3}$).

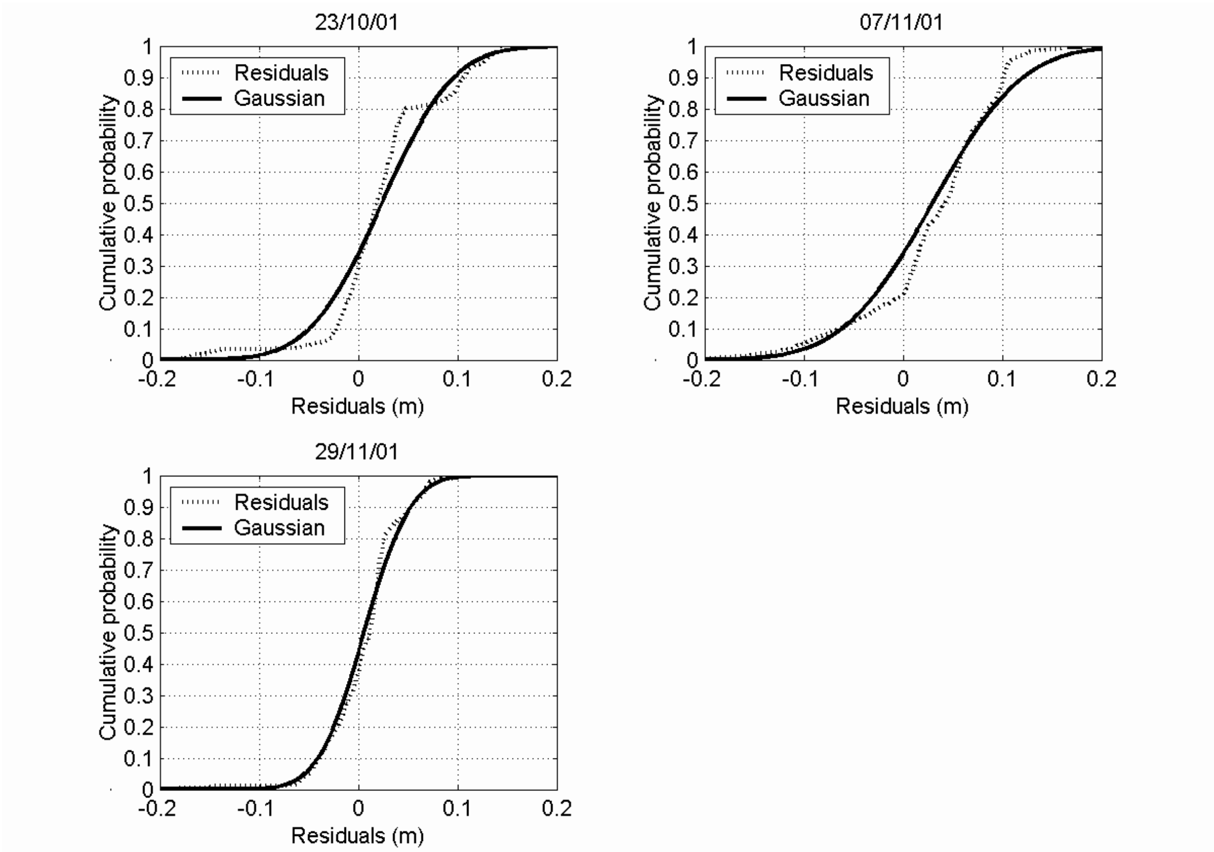


Figure 4.11 Cumulative probability distribution of residuals for storms 23/10/01 (MSE = $3.80*10^{-3}$), 07/11/01 (MSE = $6.00*10^{-3}$ m^2) and 29/11/01 (MSE = $1.38*10^{-3}$ m^2).

The range of values found for the model parameters in the 15 calibrations is shown in table 4.8, illustrating the variation in model parameter values. Especially the parameters involved in the runoff process show a considerable variation. The value for the overflow coefficient is rather constant for storm events without an influence of the water level in the receiving pond.

Table 4.8 Range of parameters values found during the calibration of the hydrodynamic model of Loenen

parameter	Description	Range
N1 (m^3/d.ha)	dwf	218-984
B2 (mm)	initial losses on flat, impervious areas	0.1-5.4
B5 (mm)	initial losses on flat, semi-pervious areas	0.1-5.3
B7 (mm)	initial losses on inclining roofs	0.2-3.3
B8 (mm)	initial losses on flat roofs	0.2-4.9
F2 (s)	routing coefficient for flat, impervious areas	18-905
F5 (s)	routing coefficient for flat, semi-pervious areas	18-833
F7 (s)	routing coefficient for inclining roofs	11-965
F8 (s)	routing coefficient for flat roofs	40-947
I5 (mm/h)	maximum infiltration capacity semi-pervious areas	0.4-8.9
CC ($m^{0.5}$/s)	overflow coefficient	0.67-0.76[a]

[a] The range for the overflow coefficient is based on the 4 storms causing a CSO event without interference with the water level in the receiving pond. The fact that no constant value for the overflow coefficient was found is likely due to the applied Kindsvater and Carter equation [Wallingford software (2000)] using a fixed power of 1.5 to the water level above the weir. Calibration of full-scale overflow structures [Veldkamp and Clemens (2001)] showed that both the weir constant a and the power b of the general equation for flow over a weir $Q = a*h^b$ have to be adjusted.

4.2.3 Quality of simulation of flow

With respect to the performance of the wwtp, influent flow is one of the main drivers. The quality of the simulation of the flows was not taken into account explicitly in the calibration routine. However, the quality of the simulation results with respect to the volume of wastewater transported to the wwtp is strongly linked to the quality of the simulation results with respect to water levels.

In the Loenen case, the discharge of the pumps is related to the water level in the pump sump. Figure 4.12 gives the measured pumped volumes for storm 18/07/01 compared with the volumes calculated with the calibrated model. During the storm event, the water levels are constantly well above the switch on levels of the pump. The minor difference between the simulated and measured pumped volumes indicate that the pumping pattern and capacity is well implemented in the hydrodynamic model. After the storm event, the pumped volumes start to deviate. This is due to the dwf implemented in the model being one of the calibration parameters. The value for this parameter is set to give the best fit during the storm event. As a result, the dwf introduced in the model is only valid in a numerical sense during the storm event. After the storm event, the real dwf and the dwf as implemented in the model may deviate significantly.

A simulation of the total measuring period shows that the average dwf (wastewater including infiltration/inflow) was 740 m^3/d, see figure 4.13. This daily volume has the same order of magnitude as the average daily drinking water consumption, which was 730 m^3/d for the year 2001. This indicates that the average amount of infiltration and inflow in the sewer system of Loenen contributes only to approximately 10 % of the total dwf, assuming that 10% of the consumed drinking water does not enter the sewer system.

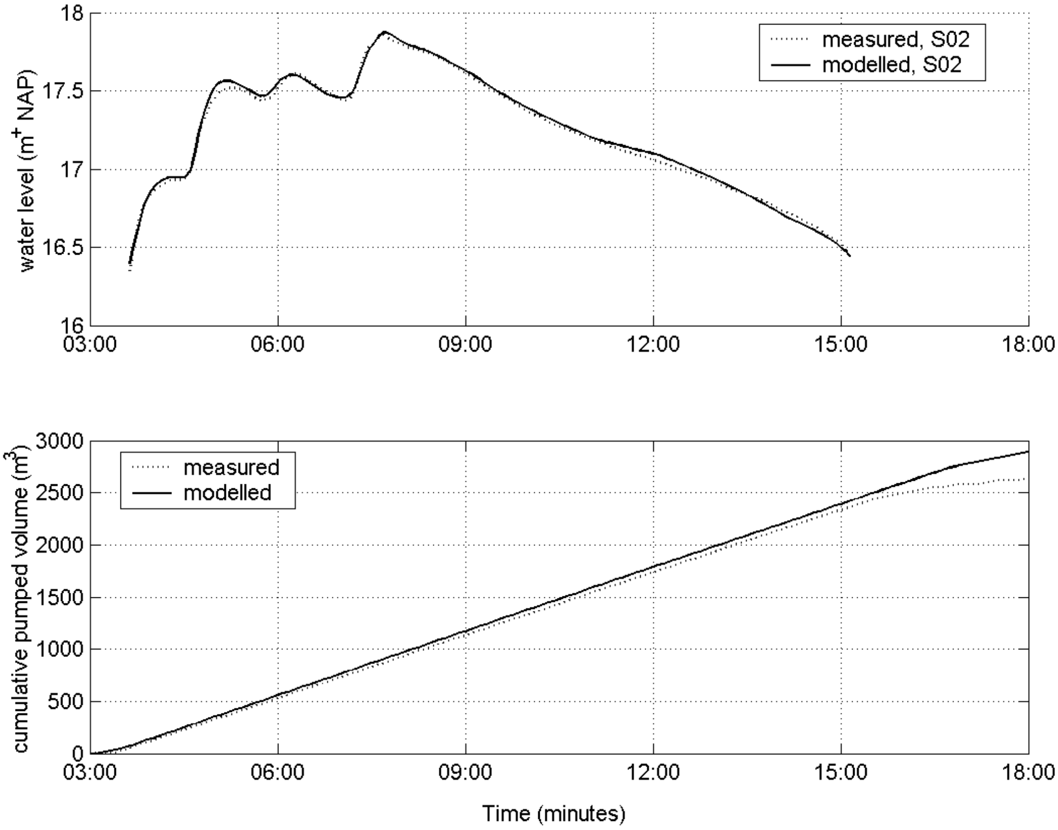


Figure 4.12 Cumulative pumped volumes for storm 18/07/01. The upper graph shows the measured and simulated water levels for gauge S02, the level gauge nearest to the pumping station. Just after 15:00 hour, the water level dropped below the level gauge. The lower graph shows the measured and modelled pumped volumes.

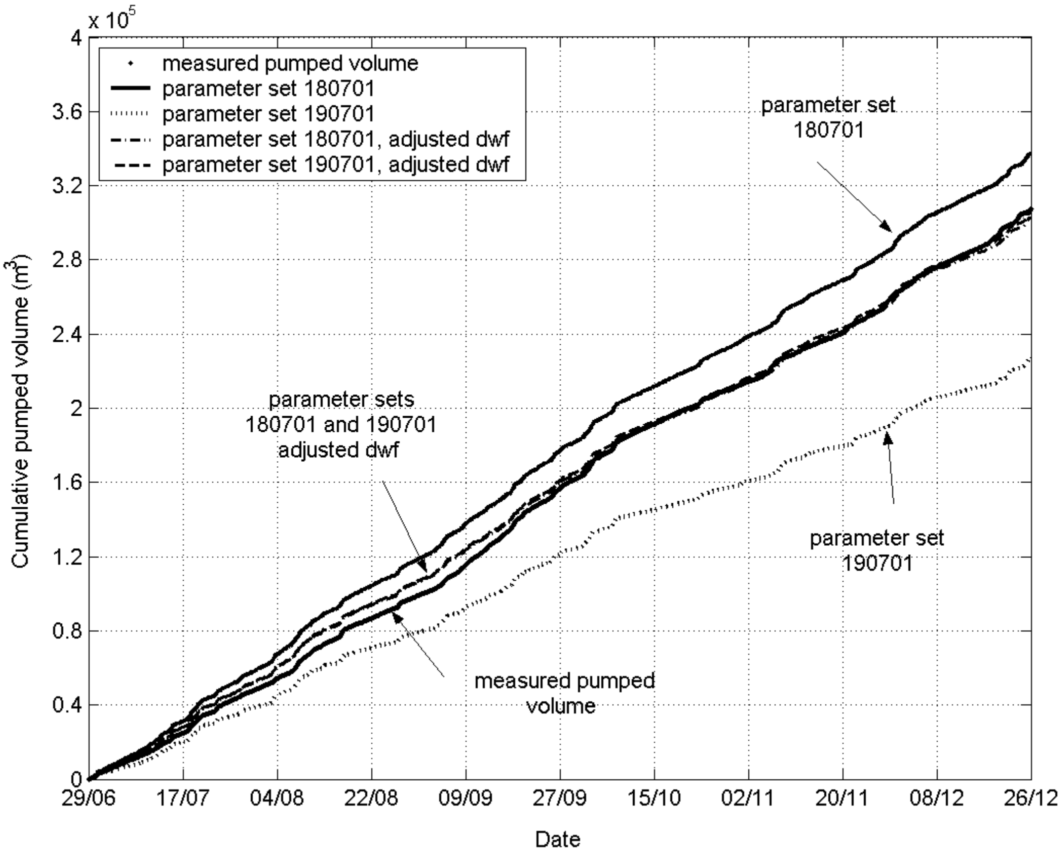


Figure 4.13 Measured and simulated pumped volumes for the measuring period in Loenen. The complete measuring period was simulated using the parameter sets from the calibration of storm 18/07/01 and 19/07/01. The parameter set for storm 180701 has a dwf of 984 m^3/d, whereas the parameter set for storm 190701 has a dwf of 290 m^3/d. After adjusting the dwf in the parameter set for both storms to 740 m^3/d, the measured pumped volumes could be represented properly.

The total pumped volumes are mainly determined by the dwf and it can be concluded that the runoff parameters are of minor interest only. However, this conclusion does not hold with respect to the total CSO volume. In this case, the runoff parameters strongly affect the total overflow volumes, as illustrated in figure 4.14. The figure shows the cumulative overflow volumes. The decrease in the cumulative overflow volume during a number of storms is due to the water level of the pond receiving the CSO discharge being above the crest level of the CSO weir. Consequently, water from the pond enters the sewer system. As it is to be expected that the water from the pond flowing over the weir into the sewer system consists mainly of discharged CSO volume, the total volume passing the weir in both directions was totalled.

The dotted lines represent the simulations for the parameter sets of storm 18/07/01 and 19/07/01 with the dwf set to 740 m^3/d. The total difference in overflow volume for the measuring period between the simulation results using these two parameters sets is 30 mm, which is significant given the average annual CSO volume of 42.7 mm, calculated using 10 years of rainfall of De Bilt (1955-1964) [Korving (2004)].

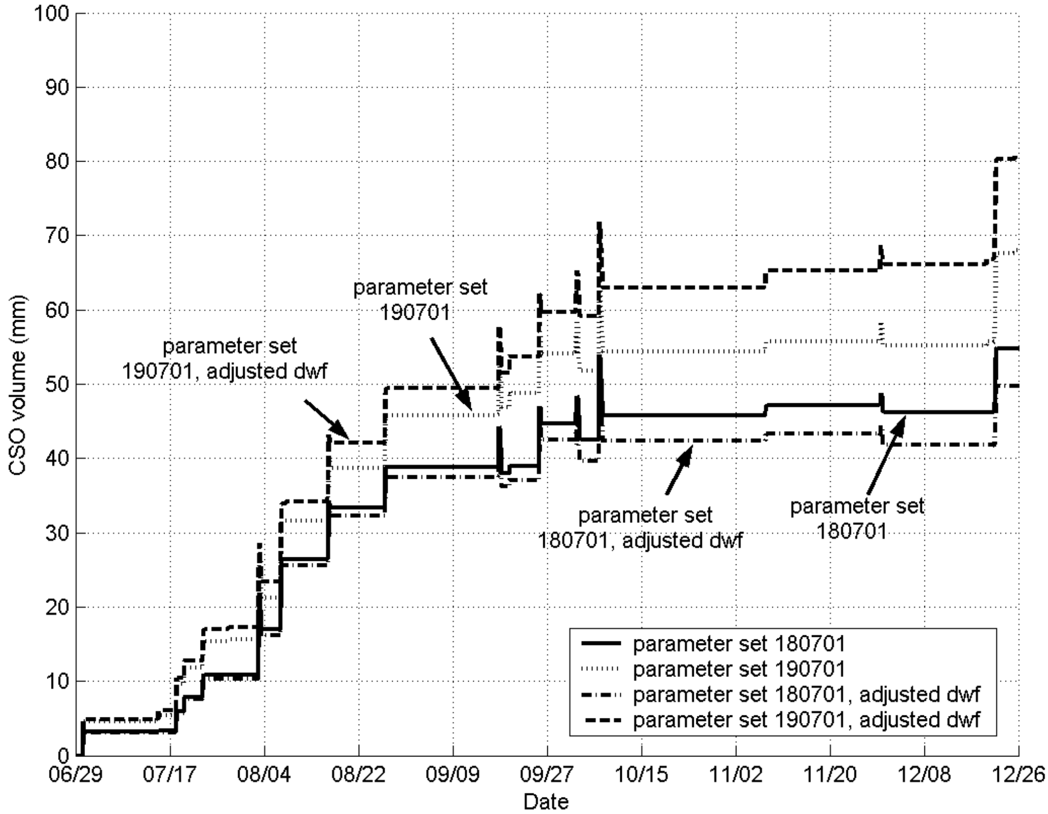


Figure 4.14 Simulated cumulative overflow volumes in Loenen for the period 29/06/01 to 26/12/01 using the original parameter sets for storms 18/07/01 and 19/07/01 and the parameter sets with the adjusted dwf. The drops in the cumulative volume are caused by water from the pond entering the sewer via the CSO. For some storm events, e.g. the storm event of 02/10/01, see also Appendix X, the total inflow from the pond into the sewer system is larger than the volume discharged through the CSO.

4.2.4 Conclusions on hydrodynamic modelling

The results of the hydrodynamic modelling of the sewer system of Loenen show that hydrodynamic modelling of sewer systems is possible with a high accuracy and reliability, although this requires a high quality database describing the sewer system and the contributing surfaces. The calibration results for Loenen are in accordance with the results found for the sewer system De Hoven, having different characteristics [Clemens (2001a)]. Consequently, the conclusion of Clemens [Clemens (2001a)] that hydrodynamic modelling of sewer systems is possible with a high accuracy could be confirmed. Moreover, the shortcomings of the runoff model currently used in the Netherlands, as identified by [Clemens (2001a)] were confirmed. For event-based calibration, the applied runoff model is detailed enough for a close representation of the measured water levels. However, with respect to calibration of rain series it is concluded that the runoff model is not able to take the time varying characteristics of the contributing areas into account. E.g. wetting losses on streets pavement vary significantly with time.

With respect to the wwtp, this limitation is not significant, as the flow arriving at the wwtp in rain series calculations depends to a large extent on the dry weather flow. With respect to the calculated overflow volumes, however, this limitation is significant.

4.3 Modelling of solute transport

Solute transport in sewer systems is the result of the combined effects of advection with the flow and dispersion in all directions. Commercially available sewer models, like MOUSE, SOBEK and Hydroworks, have implemented solute transport by means of the 1-dimensional advection dispersion description given in equation 2.4. The sewer models, however, differ in the numerical methods for solving the advection dispersion equation. Consequently, the quality of the model results differs significantly for the various models [Flamink *et al.* (2003)]. Moreover, the simplification to a 1-dimensional approach may not be valid for situations where the flow field is 3-dimensional rather than 1-dimensional. This situation occurs e.g. when manholes surcharge and the solutes become 'trapped' in the surcharged manhole, only to be released gradually. [Guymer *et al.* (1998) and Boxall *et al.* (2003)] take this effect into account by the use of an aggregated dead zone model, a solution also implemented by [Mazijk, van (1996)] for the description of the effect of dead zones near the river banks on the solute transport in the river Rhine. Although the aggregated dead zone models result in a rather good description of the solute transport, the tracer experiments, necessary to determine the effect of the dead zones [Boxall *et al.* (2003)], hamper widespread application in practice.
This section discusses the validity of the 1-dimensional advection dispersion approach, as implemented in current sewer models, for modelling solute transport in sewer systems.

4.3.1 Material and methods

The validity of the 1-dimensional advection dispersion approach was tested by field experiments and subsequent modelling. The field experiments consisted of:
- tracer experiments in the sewer systems of Loenen, as described in section 4.2, and Beekbergen;
- measuring wastewater concentration profiles in the sewer system of Ulvenhout during dwf and wwf.

The modelling consisted of:
- determining the dispersion coefficient with an advection dispersion model based on equation 2.4 and implemented in Matlab®. The Matlab® model consists of a first order upwind scheme in combination with the van Leer Limiter [Vreugdenhil en Koren (1993)], see appendix XIII. The advantages of this calculation scheme are the capability of dealing with steep gradients and the negligible numerical diffusion.
- determining the possibility of describing solute transport in sewers with the 1-dimensional equation 2.4. This will reveal whether more complex models, like the aggregated dead zone models as suggested by [Guymer *et al.* (1996)], are necessary.
- trying to reproduce the results from the tracer experiments in Loenen and Beekbergen and the measuring period in Ulvenhout, using the commercially available sewer models Hydroworks [Wallingford Software (2000)] and SOBEK [WLDelft (1998); Dhondia and Stelling (2002)].

Tracer experiments in Loenen and Beekbergen

The well-described sewer system of Loenen, as shown in figure 4.1, provided an excellent opportunity for performing tracer experiments. The tracer experiments in Loenen consisted of dosing a sodium chloride solution (87.5 g NaCl/l) in a manhole and measuring conductivity (WTW Tetracon 325, accuracy interval ± 0.5 % of value) every 3 seconds at 2 locations downstream. This method has also been successfully applied by [Rieckermann and Gujer (2002) and Huisman *et al.* (2000)].
The tracer experiments took place during dwf in 2 sewer reaches without side connections, as shown in the longitudinal profiles of figure 4.15 and 4.17. The measuring results for the 3 tracer experiments in the 454 m sewer reach and the tracer experiment in the 233 m sewer reach are shown in figures 4.16 and 4.18 respectively.

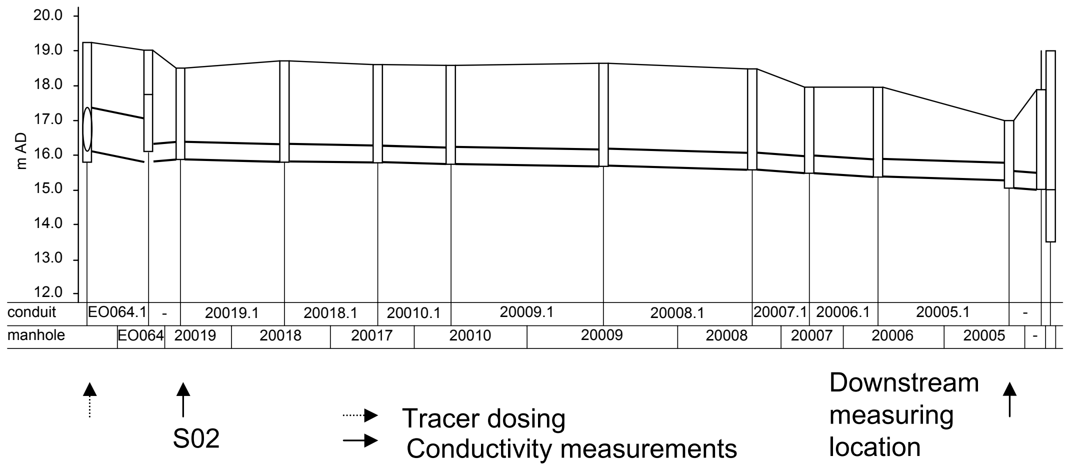


Figure 4.15 Loenen, 454 m reach (distance between measuring locations), ∅ 0.5 m. The numbers of the gauges refer to figure 4.1.

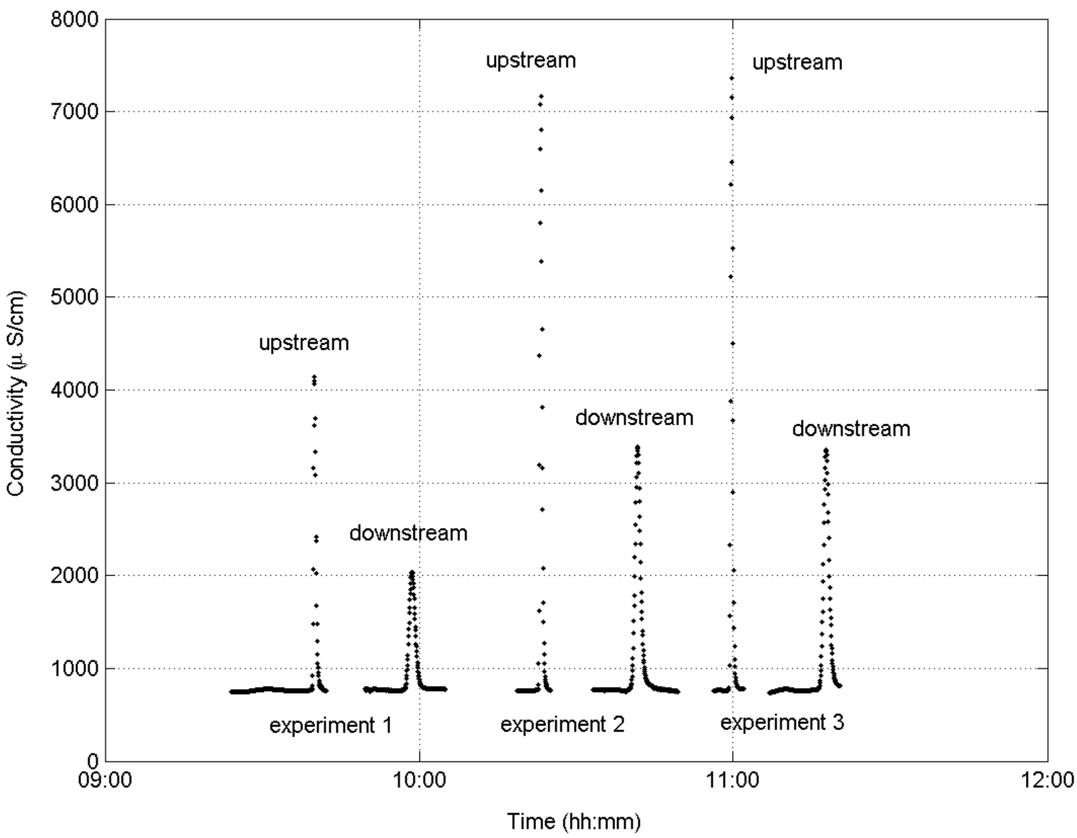


Figure 4.16 Measured conductivity tracer experiments Loenen 14 December 2001, reach 454 m. The dosed NaCl solution causes distinctive peaks above the background conductivity of approximately 700 µS/cm

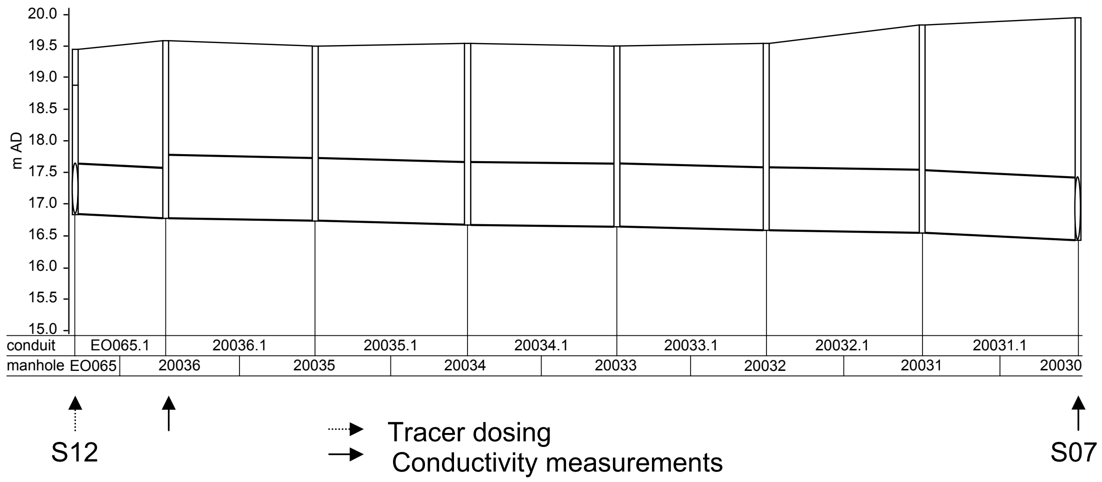


Figure 4.17 Loenen, 233 m reach (distance between measuring locations), ∅ 1 m. The numbers of the gauges refer to figure 4.1.

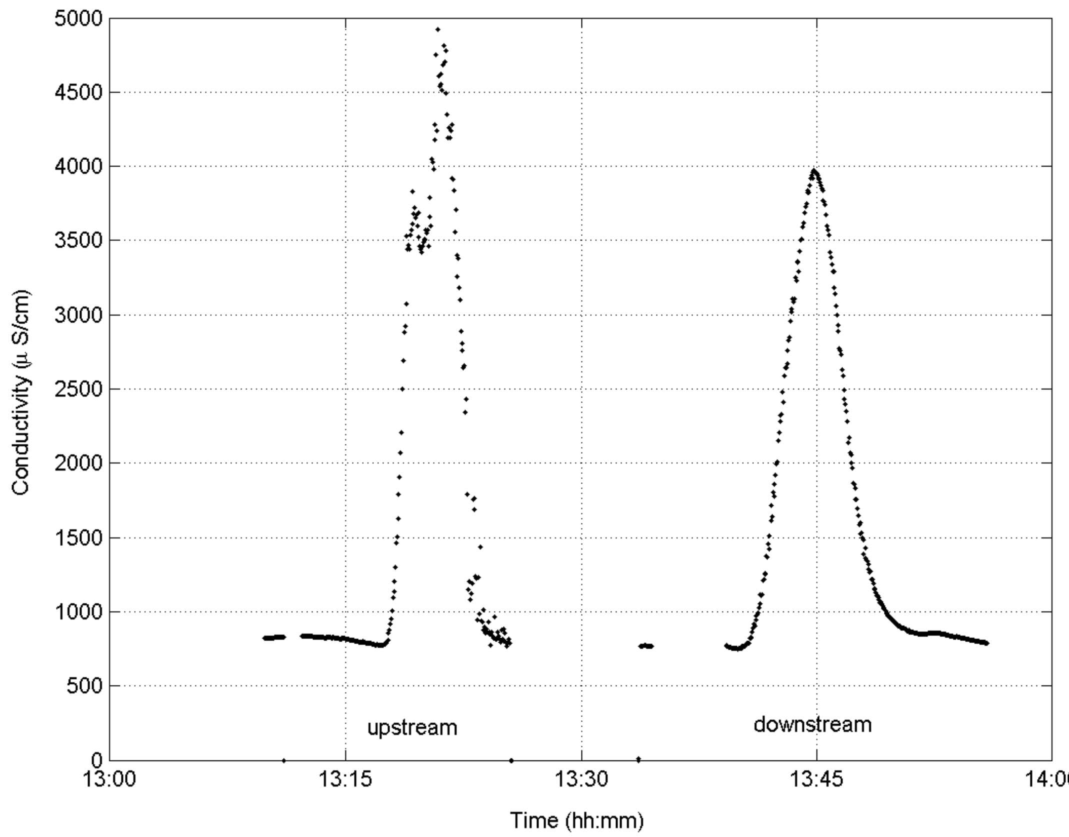


Figure 4.18 Measured conductivity tracer experiment Loenen 14 December 2001, reach 233 m. In the upstream measurements two peaks can be distinguished. This is due to the fact that the tracer was dosed two times within 3 minutes time. At the downstream measuring location the two peaks are not distinguishable anymore.

The measuring results given in figure 4.16 and 4.18 show that in both sewer reaches dispersion occurs. The measured data also shows the background conductivity level to vary.

This could be observed in figure 4.18, where the conductivity of the raw sewage decreases in the 10 minutes just before the arrival of the sodium chloride around 13:18 hours. The varying background level hampers establishing a proper background level. Consequently, the mass balances in terms of conductivity are not fully balanced, as shown in table 4.9.

Table 4.9 Mass balances for the tracer experiments in Loenen, 14 December 2001. The balances are calculated by determining the surface of the conductivity peaks above the dwf base conductivity of 770 µS/cm.

Experiment	upstream (µS/cm.s)	downstream (µS/cm.s)	difference (%)
454 m reach, experiment 1	1.28	1.25	-2.1%
454 m reach, experiment 2	2.41	2.37	-1.7%
454 m reach, experiment 3	2.27	2.35	+3.5%
233 m reach, experiment 1	9.15	9.72	+6.3%

The sewer system of Beekbergen has, like the sewer system of Loenen and De Hoven, been equipped with an intensive monitoring network, see figure 4.19, for calibrating a hydrodynamic model. Combined with the accurate database of Beekbergen, this provided again an opportunity experiment in a well-described sewer system. Table 4.10 shows some characteristics of the sewer system 'Beekbergen'.

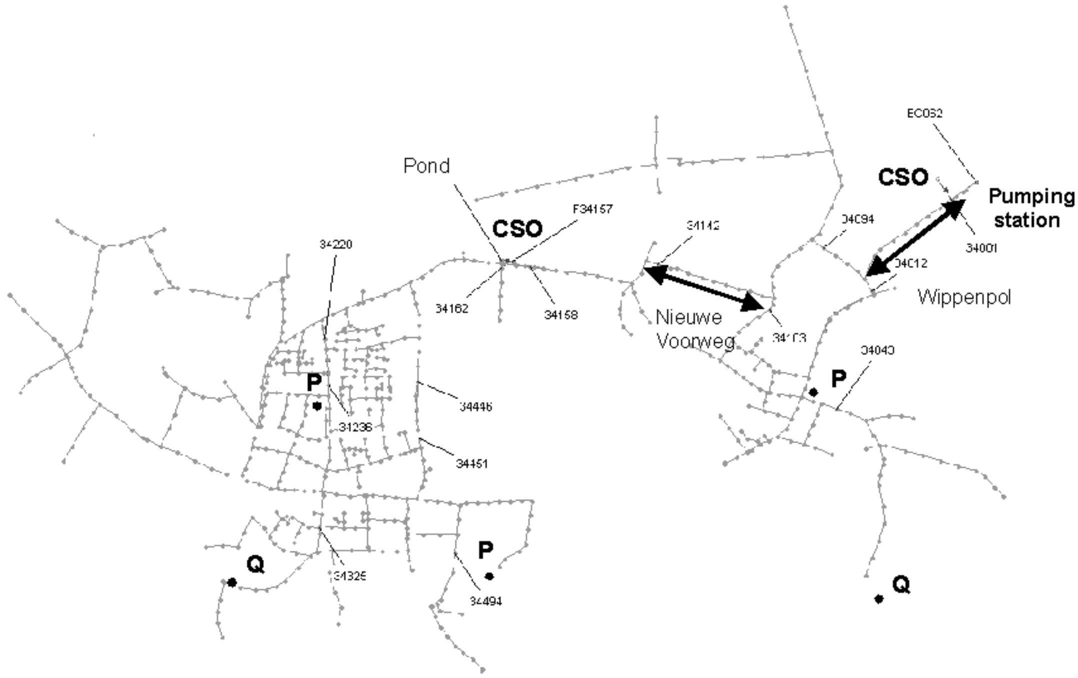


Figure 4.19 Sewer system 'Beekbergen' (schematic). The indicated manholes have been equipped with a level sensor. The **Q** indicates measurement of the flow from an injecting pressure main, the **P** indicates measuring locations for precipitation. The arrows indicate the two reaches for the tracer experiments, Nieuwe Voorweg and Wippenpol.

Table 4.10 Sewer system 'Beekbergen'.

contributing area	29.5 ha
number of inhabitants	14011
storage volume	1211 m^3 (4.1 mm)
pumping capacity	608 m^3/h, of which 230 m^3/h available for runoff i.e. the 'pump over capacity' is 0.8 mm/h
number of CSO structures	2

In the sewer system of Beekbergen, the sewer reaches Nieuwe Voorweg and Wippenpol were selected for tracer experiments. Both reaches have no side connections, as shown in the longitudinal profiles in figures 4.20 and 4.22. The tracer used in these experiments was Rhodamine WT dye and the measuring equipment consisted of three SCUFA® fluorimeters (http://www.turnerdesigns.com/t2/instruments/scufa.html). The SCUFA® fluorimeters were each calibrated in the lab to known Rhodamine WT solutions, as shown in the calibration curves in appendix VIII. In total 8 tracer experiments were performed successfully. The measurement results of the experiments are shown in figures 4.21 and 4.23.

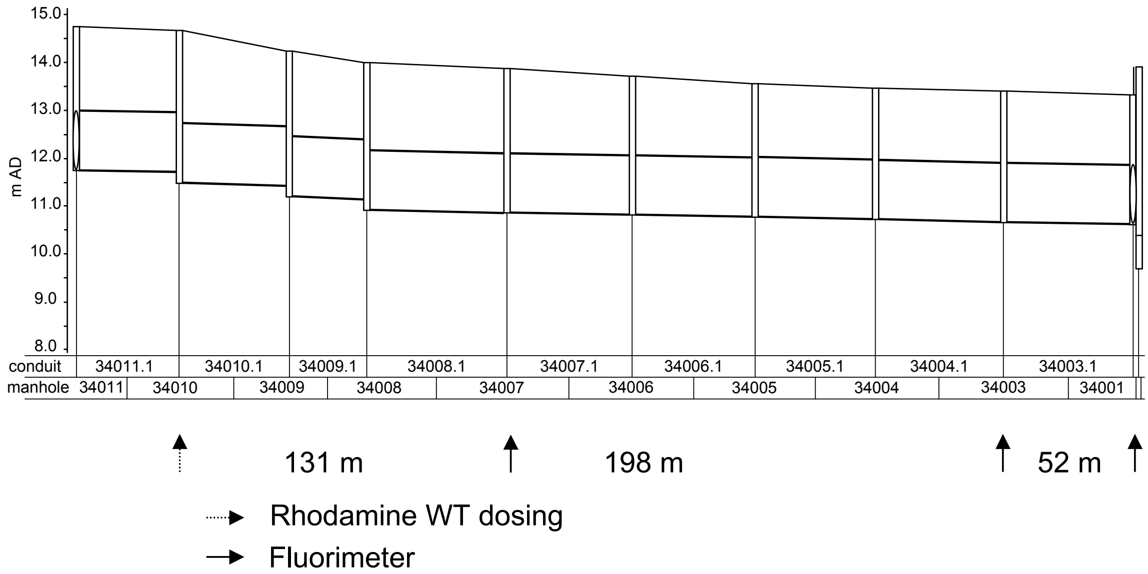


Figure 4.20 Tracer experiment Beekbergen, reach 'Wippenpol', ∅ 1.25 m.

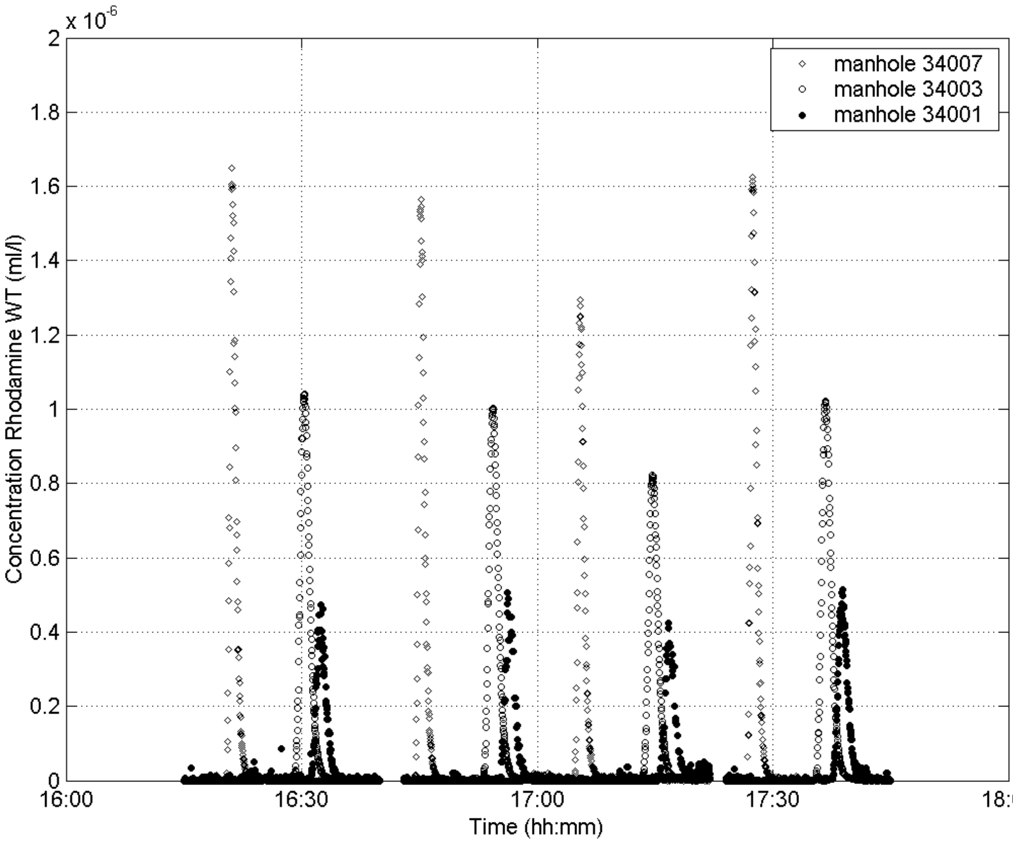


Figure 4.21 Measured Rhodamine WT levels, experiment 16 September 2003, reach 'Wippenpol'.

Figure 4.21 shows the measured Rhodamine WT levels in the reach ‘Wippenpol’ to sharply decrease between the manholes 34003 and 34001. Mass balances, given in table 4.11, and control measurements in the laboratory learned that the measured levels of downstream manhole 34001 could not be trusted. During the subsequent experiments on the 18th of September 2003 the same sensor failed again and did not record data. As a result, figure 4.23 only shows the data measured in manholes 34139 and 34133.

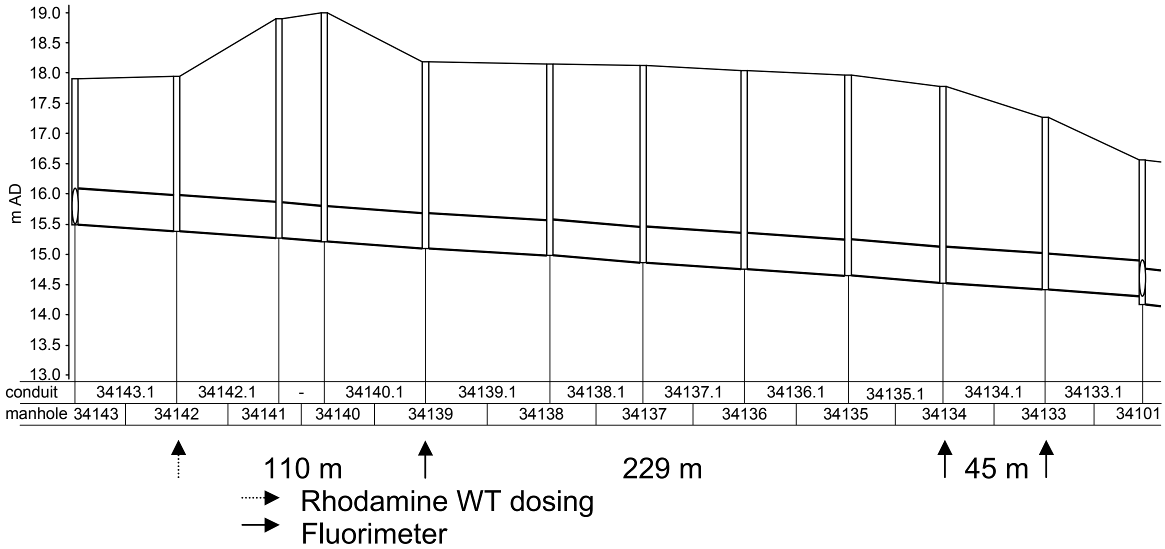


Figure 4.22 Tracer experiment Beekbergen, reach ‘Nieuwe Voorweg’, ∅ 0.6 m.

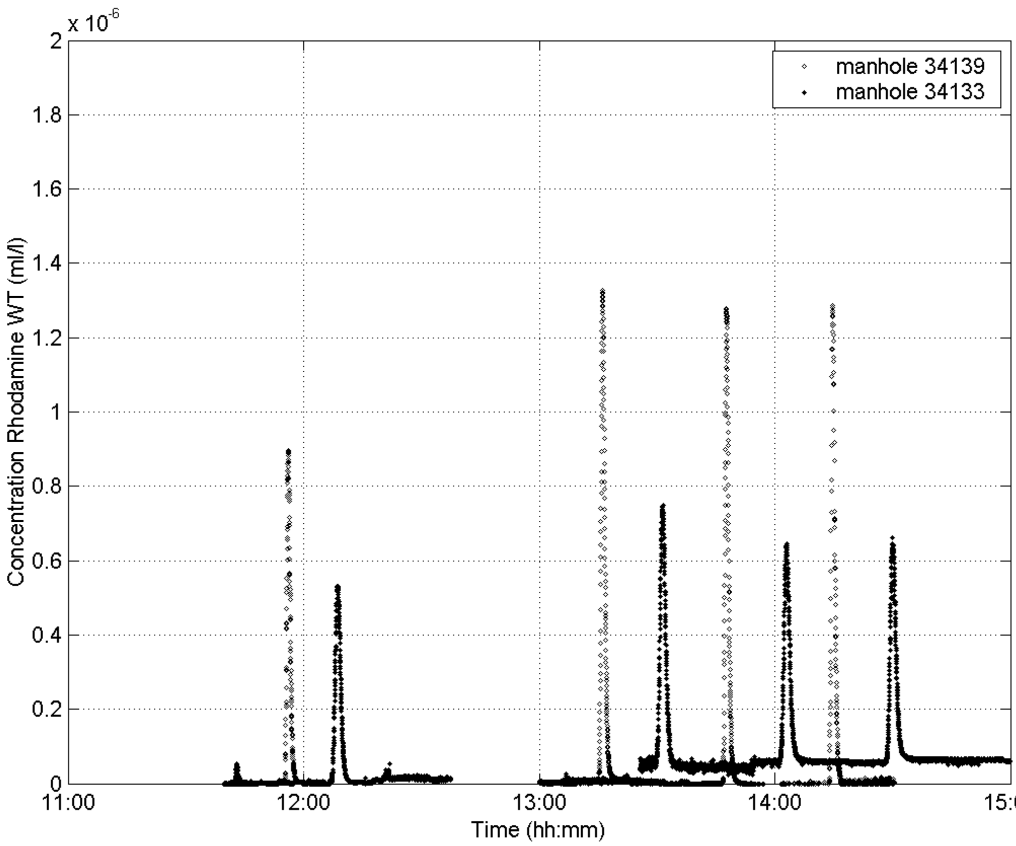


Figure 4.23 Measured Rhodamine WT levels, experiment 18 September 2003, reach ‘Nieuwe Voorweg’.

The measured Rhodamine WT levels in manhole 34133 on the 18th of September 2003 show a sudden increase around 13:30 hours. This increase could be attributed to vegetables blocking the SCUFA® sensor, as observed at the end of the experiments. Consequently, the mass balances for the final three experiments are incorrect and these experiments were not used in the analysis of the dispersion in this sewer reach.

Table 4.11 Balances for the tracer experiments in Beekbergen (ml Rhodamine WT/l.s).

Wippenpol	**mass in manhole 34007**	**mass in manhole 34003**	**mass in manhole 34001**
experiment 1	$8.9234*10^{-5}$ (100%)	$8.9171*10^{-5}$ (99.9%)	$3.7389*10^{-5}$ (41.9%)
experiment 2	$8.5122*10^{-5}$ (100%)	$8.5510*10^{-5}$ (100.5%)	$3.8929*10^{-5}$ (45.7%)
experiment 3	$7.1620*10^{-5}$ (100%)	$6.9973*10^{-5}$ (97.7%)	$3.5630*10^{-5}$ (49.7%)
experiment 4	$8.9753*10^{-5}$ (100%)	$8.7702*10^{-5}$ (97.7%)	$4.2190*10^{-5}$ (47.0%)
Nieuwe Voorweg	**mass in manhole 34139**		**mass in manhole 34133**
experiment 1	$5.4806*10^{-5}$ (100%)		$5.2717*10^{-5}$ (96.1%)
experiment 2	$8.9271*10^{-5}$ (100%)		$6.6684*10^{-5}$ (74.7%)
experiment 3	$8.2222*10^{-5}$ (100%)		$6.0218*10^{-5}$ (73.2%)
experiment 4	$8.5388*10^{-5}$ (100%)		$6.2270*10^{-5}$ (72.9%)

Measuring concentration profiles in Ulvenhout

In the sewer system of Ulvenhout, located near Breda, an intensive measuring campaign started at the end of 2002. The measuring set up was designed according to the design method for monitoring networks developed by [Clemens (2001a); Clemens (2002)] and the monitoring network should provide sufficient information to be able to calibrate a hydrodynamic model, like the ones mentioned before in Loenen and Beekbergen. As such, Ulvenhout provided another location for experiments in a well described sewer system. Figure 4.24 shows the lay out of the sewer system of Ulvenhout. Characteristics of the sewer system of Ulvenhout are given in table 4.12.

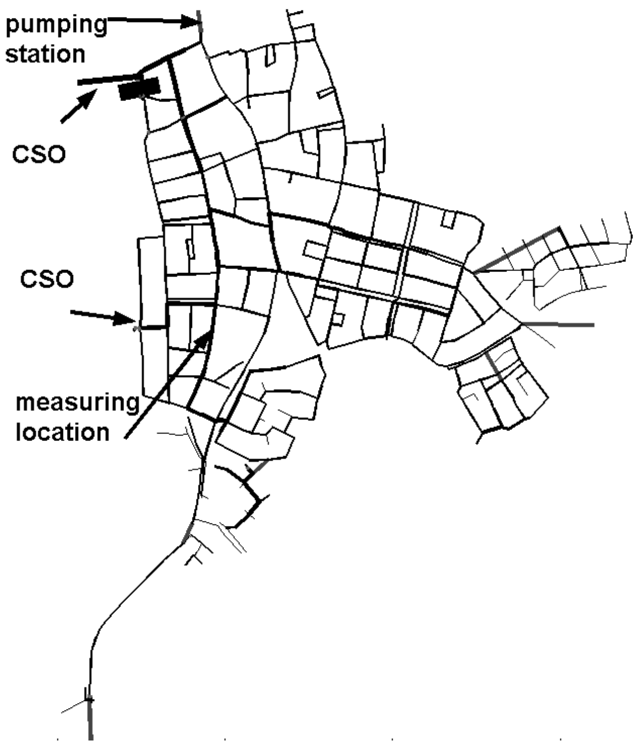


Figure 4.24 Sewer system (schematic) of Ulvenhout with the measuring location.

Table 4.12 Characteristics of combined sewer system 'Ulvenhout', exclusive of 4.6 ha of improved separate sewers.

contributing area	52.2 ha
number of inhabitants	4316
storage volume	2322 m^3 (4.5 mm)
pumping capacity	380 m^3/h, of which 301 m^3/h available for runoff i.e. the 'pump over capacity' is 0.6 mm/h
number of CSO structures	4

At the in figure 4.24 indicated measuring location, wastewater samples were taken every hour in the period from 31 March 2003 to 8 April 2003 with an automatic vacuum sampler. the samples were taken in the manhole at 6 cm from the invert level. During the storm event of 1 April 2003 additional samples were taken manually every 10 minutes, to enable a more detailed study of the fluctuations in wastewater quality.
The samples were filtered using a 0.45 μm filter and subsequently analysed on ammonium using Merck test no. 1.14559 and dissolved COD using Merck test no. 1.14541. Moreover, during the onset of the storm event the conductivity and the temperature of the wastewater were recorded using the WTW TetraCon 325®.
The storm event of 1 April 2003 has a total volume of 20.6 mm, with 13.2 mm within the first 4 hours of the event. The storm event occurred after a long dry period and as a result the initial losses were significant. Only after 2 hours after the beginning of the storm event runoff from the streets entering the gully pots was observed visually. Figure 4.25 shows the rainfall intensities and the water level in the sewer, measured in the manhole just downstream of the sampling location. The figure also shows the water levels, simulated with a calibrated Hydroworks model. The quality of the calibration results is in accordance with the quality obtained for Loenen (MSE (mean squared error) for this storm event is $5.2 \cdot 10^{-3}$ m^2), see section 4.2 and De Hoven [Clemens (2001a)].

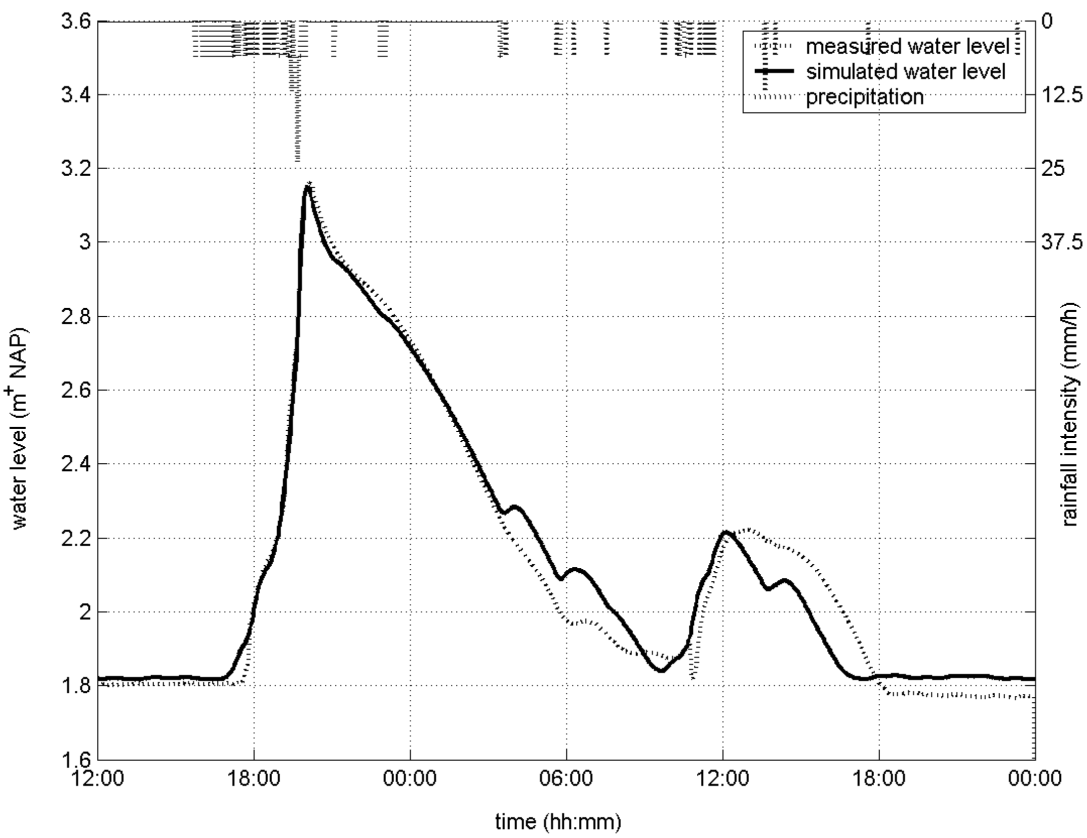


Figure 4.25 Water level and precipitation, storm event 1 April 2003, Ulvenhout. The hydrodynamic calibration was performed as part of a study by Witteveen+Bos by Lennard Stigter.

Figure 4.26 shows the water quality parameters measured during the storm event of 1 April 2003. The missing data between 22:00 and 13:00 hours are due to clogging of the automatic vacuum sampler. However, the initial phase of the storm event and the recovery phase to dwf conditions were covered by measurements. The water quality parameters measured are temperature (on-line), conductivity (on-line and in samples), ammonium (samples) and dissolved COD (samples).
The temperature of the wastewater, measured during the filling phase only, decreases slightly: from 11.3 °C to 10.0 °C. The conductivity, ammonium concentration and dissolved COD all show the same trend during the storm: a decrease during the filling phase of the storm event and an increase during the tail of the storm event.
Similar results have been described by [Krebs *et al.* (1999)], who measured temperature, conductivity and ammonium for a number of storm events in Dresden. Besides, [Bertrand-Krajewski *et al.* (1995)] found a similar relation between ammonium and conductivity in a measuring project in Boran-sur-Oise.
Given the consistency of the measurement data from Ulvenhout with data found in literature it was concluded that the measured data can be used to analyse the potential of current sewer models to describe the ammonium concentration and the dilution rate during storm events.

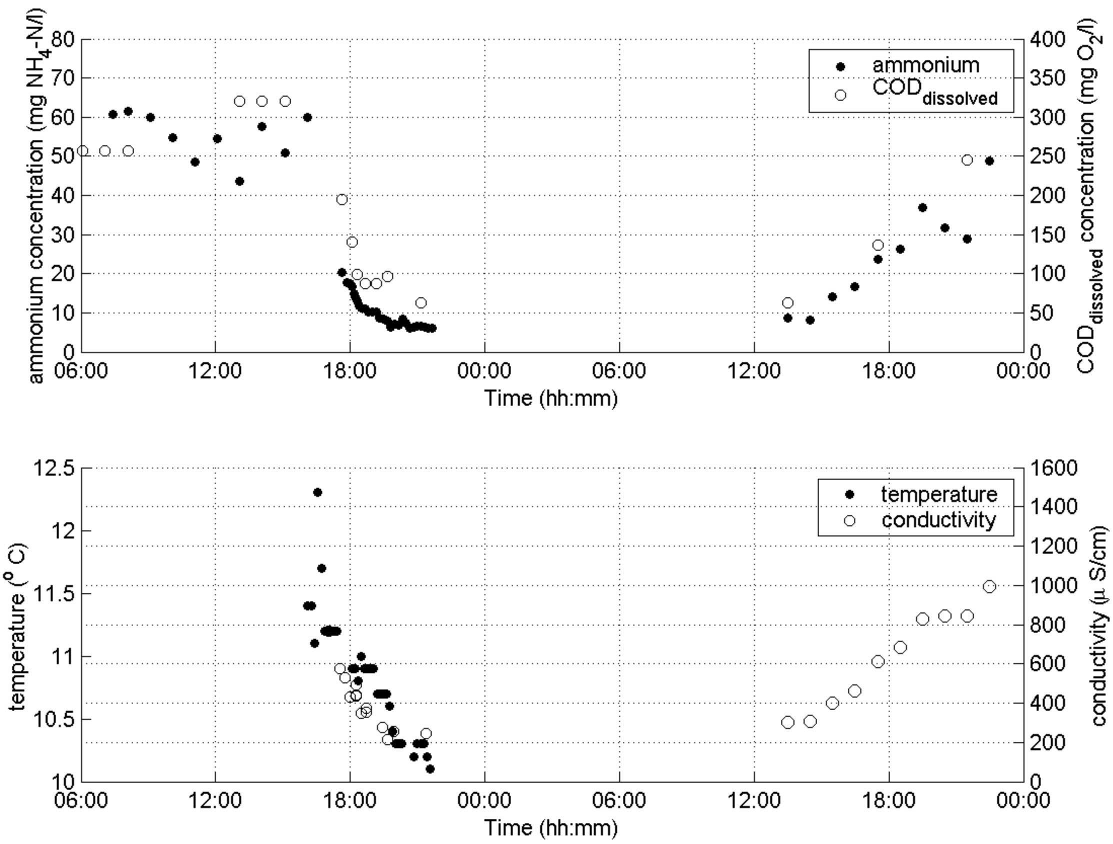


Figure 4.26 Ammonium, dissolved COD, temperature and conductivity, 1 and 2 April 2003, Ulvenhout.

4.3.2 Results and discussion

The data from the tracer experiments were used to determine dispersion coefficients and to research whether advection-dispersion can be described with a 1-dimensional model. Figures 4.27 to 4.30 show the results for one tracer experiment per sewer reach in Loenen and Beekbergen. The upper graphs show the measured conductivity in the upstream and downstream manhole and the modelled, using the Matlab model, conductivity downstream.

The lower graphs shows the residuals after trying to fit the model to the data by adjusting the dispersion coefficient. In the Matlab simulations, the flow velocity was derived from the measured travel time of the tracer. As the experiments lasted only 10-15 minutes, the changes in dwf could be neglected. The velocities and the calculated dispersion coefficients are given in table 4.13.

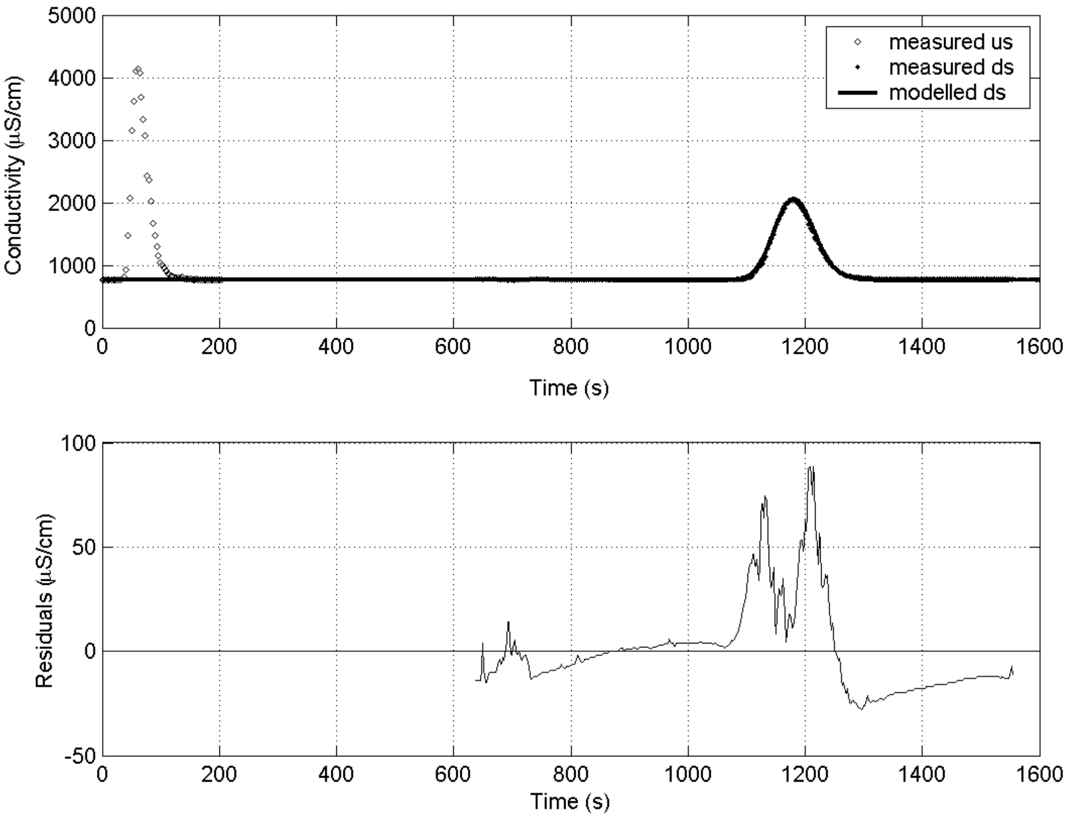


Figure 4.27 Tracer experiment 1, 14/12/01. Loenen, 454 m sewer reach, dispersion coefficient = 0.15 m^2/s.

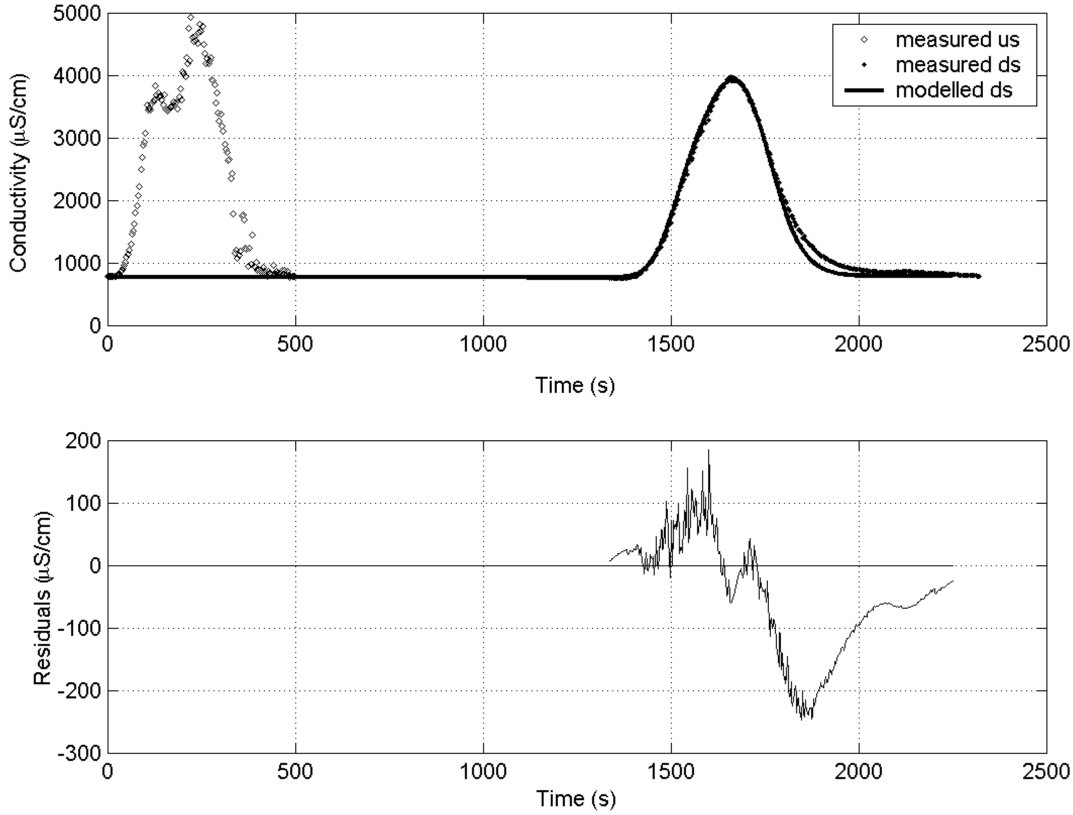


Figure 4.28 Tracer experiment 4, 14/12/01. Loenen, 233 m sewer reach, dispersion coefficient = 0.04 m^2/s.

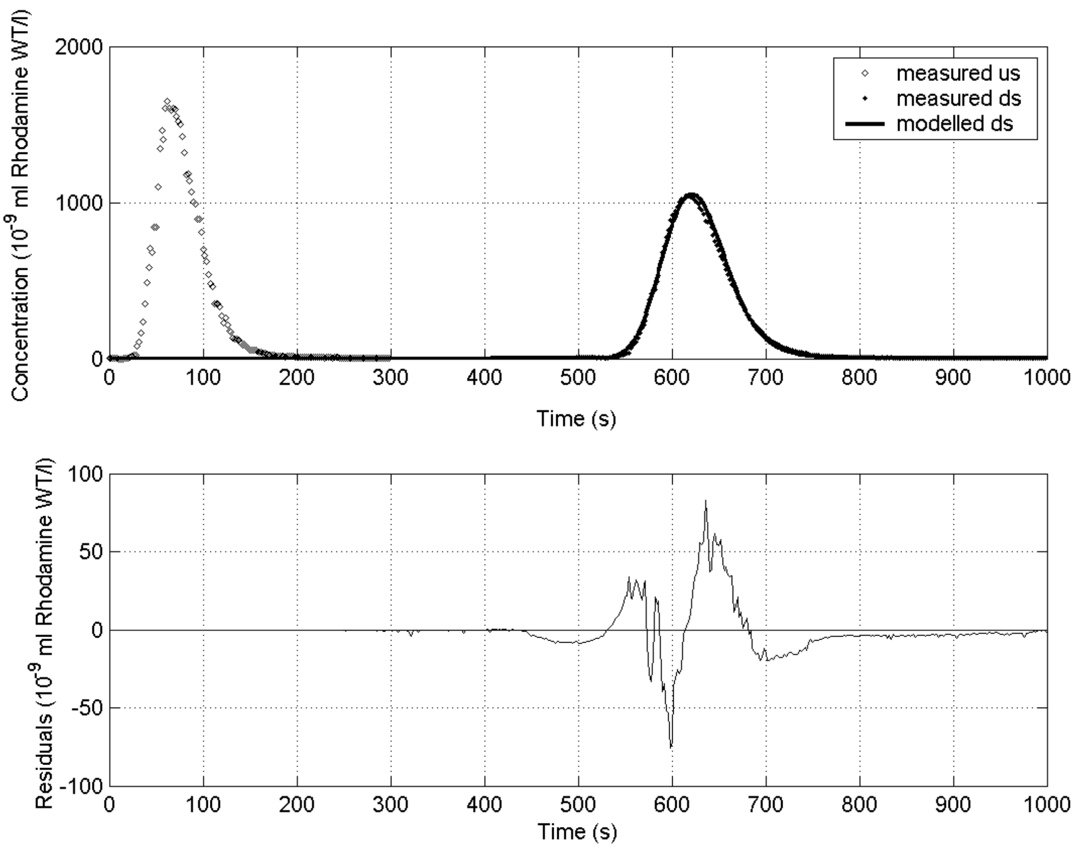


Figure 4.29 Tracer experiment 1, 16/09/03. Beekbergen, reach Wippenpol, dispersion coefficient = 0.13 m^2/s.

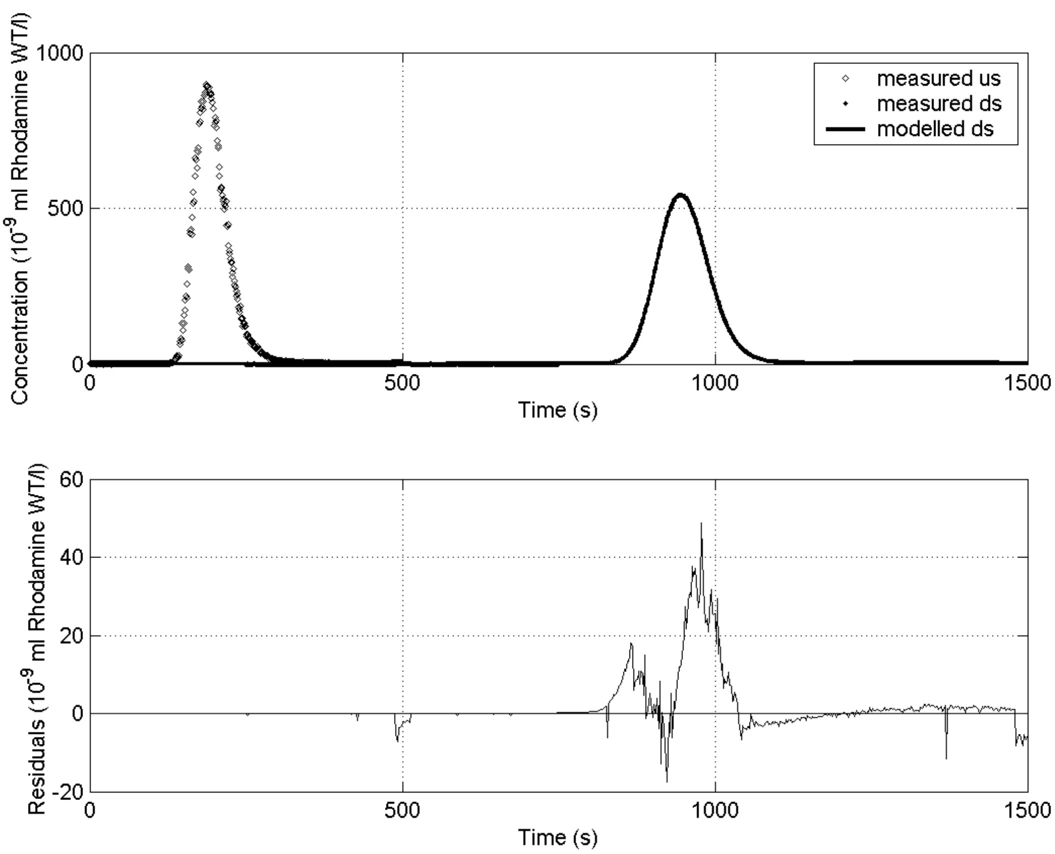


Figure 4.30 Tracer experiment 5, Beekbergen, 16/09/03. Reach Nieuwe Voorweg, dispersion coefficient = 0.145 m^2/s.

Table 4.13 gives an overview of the dispersion coefficients found with the tracer experiments results. The 95% confidence interval is based on a sensitivity analysis using equation 4.1, assuming a Gaussian distribution of the residuals.

$$\sigma_K^2 = \sigma_r^2 (J^T J)^{-1} \qquad \text{(eq. 4.1)}$$

where

σ_K standard deviation dispersion coefficient

σ_r standard deviation residuals

$\underline{J}$ Jacobean matrix, given by equation 4.2

$$\underline{J} = \begin{bmatrix} \frac{\partial r_i}{\partial K} \\ \cdot \\ \cdot \\ \frac{\partial r_n}{\partial K} \end{bmatrix} \qquad \text{(eq. 4.2)}$$

where

r residual

K dispersion coefficient

The values of the dispersion coefficient are consistent for the experiments done in the same sewer reaches. Moreover, the dispersion coefficients have the same order of magnitude as values found under similar (dwf) conditions. [Huisman *et al.* (2000)] reported a dispersion coefficient of 0.05 m^2/s and [Boxall *et al.* (2003)] reported a mean dispersion coefficient of 0.06 m^2/s with a standard deviation of 0.05 m^2/s.

Table 4.13 Dispersion coefficients

experiment	**flow velocity (m/s)**	**dispersion coefficient (m^2/s) (95% confidence interval)**
Loenen 454 m reach, experiment 1	0.41	0.15 (±0.0020)
Loenen 454 m reach, experiment 2	0.41	0.14 (±0.0018)
Loenen 454 m reach, experiment 3	0.42	0.145 (±0.0028)
Loenen 233 m reach, experiment 4	0.16	0.04 (±0.0001)
Beekbergen, Wippenpol, experiment 1	0.36	0.13 (±0.0024)
Beekbergen, Wippenpol, experiment 2	0.36	0.13 (±0.0030)
Beekbergen, Wippenpol, experiment 3	0.36	0.13 (±0.0030)
Beekbergen, Wippenpol, experiment 4	0.36	0.135 (±0.0034)
Beekbergen, Nieuwe Voorweg, experiment 5	0.36	0.145 (±0.0020)

The tracer experiment data were also used to test the numerical implementation of the advection dispersion equation in SOBEK and Hydroworks. Figure 4.31 gives the modelling results for experiment 1 in the 454 m reach in Loenen. The default model of SOBEK, using the SOBEK modules ‘Rainfall-runoff’, ‘Sewer flow’ and ‘Water quality’, assumes a space step dx equal to the length of the conduit in the water quality modules. Consequently, a very high numerical dispersion could be observed.
However, water quality simulations in sewers could also be implemented in SOBEK using the three aforementioned modules combined with the ‘Channel flow’ module.
The Channel flow module enables determining the space step manually. In previous versions of SOBEK, however, the travel time depended on the space step. This was due to the addition of calculation nodes with physical dimensions, thus increasing the total length of the sewer reach in the model. Consequently, the finer the calculation grid, the more delay occurred [Flamink *et al.* (2003)]. In SOBEK test version 20600039zgß, this problem has nearly been solved. The time difference between the arrival of the centroids is now only 20 seconds. The results of simulations using the tracer data and a space step of 1 m and 5 m are shown in figure 4.31. Numerical dispersion is still present, as the measured conductivity could not be reproduced. The numerical dispersion, as observed in the SOBEK model results, is equal to using a dispersion coefficient in the Matlab model of 0.4 m^2/s for a space step of 1 m and 1.1 m^2/s for a space step of 5 m. As such, the numerical dispersion is still significant.

Hydroworks uses a simplified version of the advection dispersion equation 2.4, as in Hydroworks physical dispersion is not accounted for. The space step in Hydroworks is set automatically. Nonetheless, numerical dispersion is also present in Hydroworks. The model results of Hydroworks are almost identical to the results obtained with SOBEK with a space step of 5 m. Consequently, it is concluded that Hydroworks shows significant numerical dispersion.

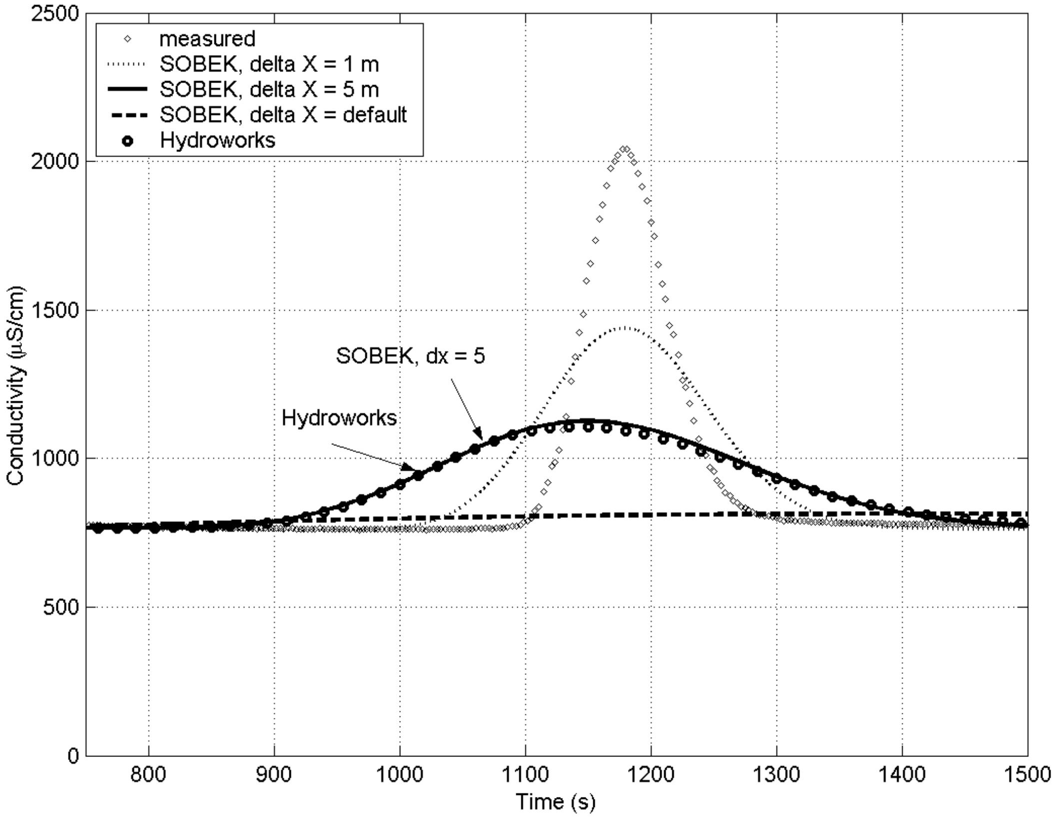


Figure 4.31 Measured conductivity in downstream node in experiment 1 in Loenen (14/12/01, experiment 1, 454 m reach) compared with modelling results obtained with Hydroworks and 3 versions of SOBEK.

The tracer experiments showed that, at least during dwf, the 1 dimensional advection dispersion equation 2.4 can be used to describe solute transport in sewers. Moreover, the tracer experiments revealed that current commercial sewer models have shortcomings in a numerical sense, as they are not capable of reproducing the results from the tracer experiments. However, the data from the tracer experiments show extremely high concentration gradients, not observed under normal conditions in sewer systems. Therefore, it was analysed whether current commercially available sewer models are capable of reproducing the normal concentration gradients observed in wastewater composition.

The data measured in Ulvenhout were used to analyse the potential of current sewer models to reproduce variations in the concentrations of dissolved compounds. Based on the measured data, a dwf ammonium profile was determined. This profile, combined with the calibrated hydrodynamic (Hydroworks) model, has been used to simulate the ammonium concentration during the storm event of the 1st of April 2003.

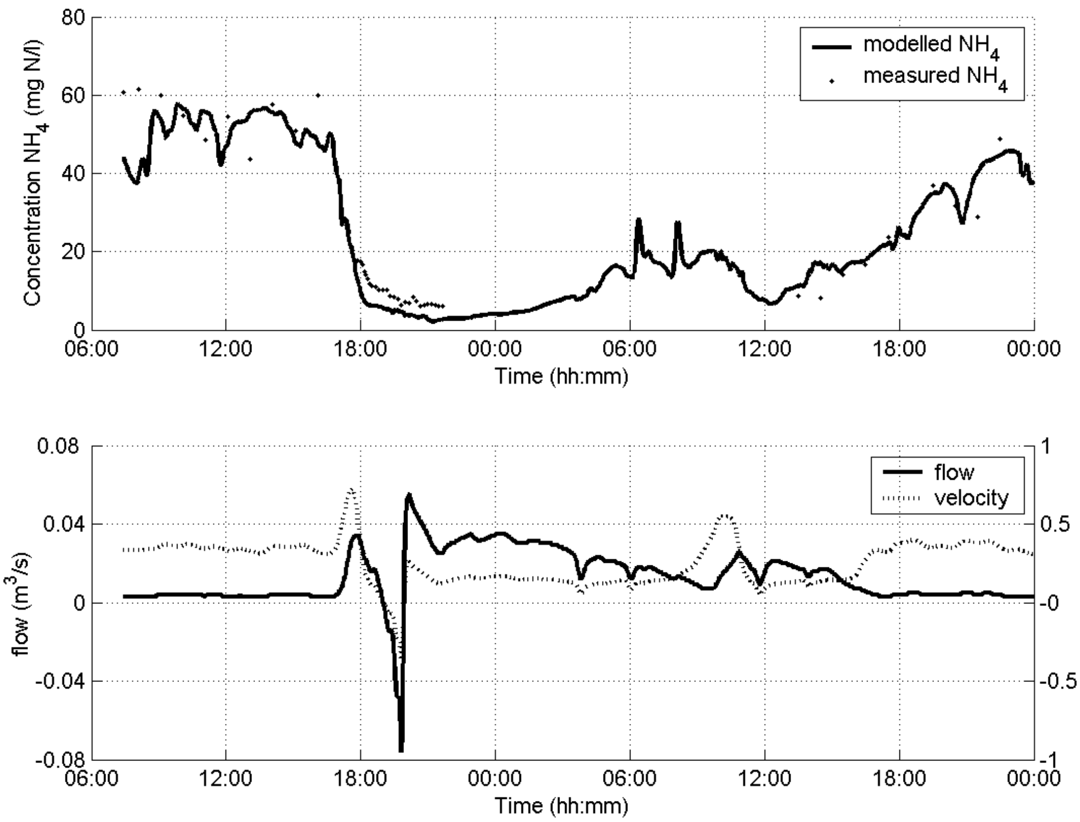


Figure 4.32 Storm 01/04/03, Ulvenhout. Measured and modelled ammonium concentration (upper graph) and flow and velocity (lower graph). It is interesting to note that during the storm event the flow reversed towards the nearest CSO, see figure 4.24. The measured and calculated water levels for this storm event are given in figure 4.25.

The simulated and measured ammonia concentration of figure 4.32 show the same trend, which is in accordance with the results of [Bouteligier *et al.* (2001)], see figure 2.8. This indicates that problems with numerical dispersion are not significant during wwf. This can be due to the actual dispersion in sewers during wwf having the same order of magnitude as the observed numerical dispersion. [Boxall *et al.* (2003)] found a mean value for the dispersion coefficient in a sewer during wwf of 0.6 m^2/s with a standard deviation of 1 m^2/s.
In addition, the concentration gradients in sewer systems with respect to ammonium during storm events are small enough not to cause significant numerical problems. Moreover, due to the small gradients an aggregated dead zone model does not seem to be necessary.
Nevertheless, the model systematically underestimates the ammonium concentration during the dilution phase of the storm event. This may be due to the fact that in Hydroworks the ammonium concentration in storm runoff is set to zero, while ammonium levels in runoff normally range between 1 mg N/l (average concentration in rainwater in the Netherlands [Stolk (2001)] and 5 mg N/l [Ashley and Crabtree, 1992; Bertrand-Krajewski *et al.*, 1995)].

Figure 4.33 shows the same data as figure 4.32, but also gives the difference between the measured and modelled ammonium concentration. The difference between model and measurement varies between +27% to –67% during the dilution phase of the storm event and from +110% to –16% during the recovery phase of the storm event. This range exceeds the boundaries for acceptable systematic errors in the sewer model results established in chapter 3. The values for the acceptable systematic errors are, however, based on the average error over the complete dilution and recovery phases. The average error over the dilution phase is –17%, which is within the acceptable boundary of –50%. Moreover, the average error over the recovery phase is 17%, where also +50% is acceptable according to table 3.14. Calculating the average error over the complete event is unfortunately not possible due to missing data between 22:00 and 13:00.

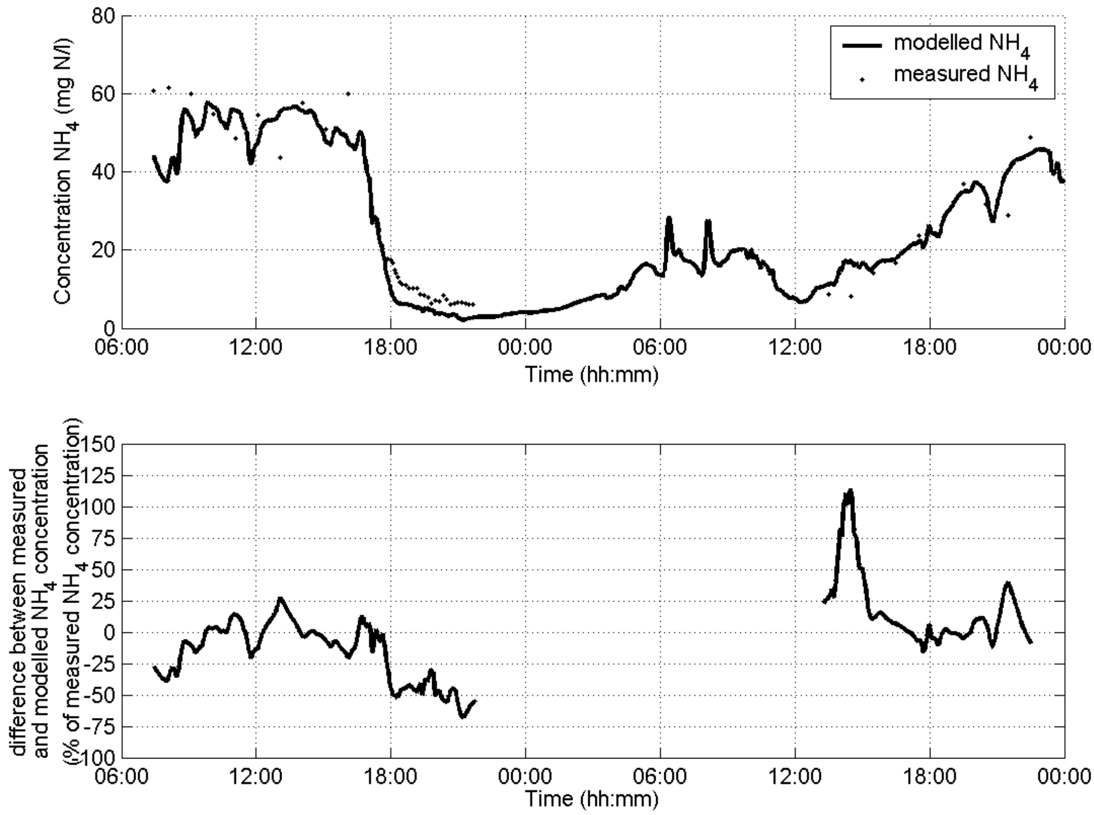


Figure 4.33 Quality of modelling, storm event 01/04/03, Ulvenhout.

4.3.3 Conclusion

The available Hydroworks model systematically underestimates the concentration during the dilution phase of a storm event. This is due to the fact that the runoff is assumed to contain no ammonium, which is not true in reality. In addition, the results of the tracer experiments revealed that Hydroworks and SOBEK have a numerical dispersion exceeding the observed physical dispersion. As the concentration gradients in sewer system are small in comparison with the gradients introduced during the tracer experiments, this numerical dispersion is not hampering practical application of the models for simulating solute concentrations in wwtp influent.

Based on a comparison of the boundaries for acceptable systematic errors in wwtp influent data, it is concluded that today's sewer models suffice with respect to the simulation of the ammonium concentration during a storm event. As this conclusion is based on 1 storm and literature results only, further research is recommended to support this conclusion.

4.4 Suspended solids

Suspended solids typically comprise up to 70% of the total mass of solids in transport [Ashley *et al.* (1999)]. Moreover, a relatively high proportion of the total pollution load is associated with the suspended solids [Nieuwenhuijzen, van (2002); Bertrand-Krajewski *et al.* (1993)].

Suspended solids are typically 40 µm in size during both dwf and wwf and primarily attributed to sanitary solids [Butler *et al.* (2003)]. Settling velocities are usually less than 10 mm/s [Crabtree (1989)]. This general definition is in accordance with the threshold values for suspended solids transport in a sewer system, calculated by the empirical relation 4.3 [Ashley *et al.* (1999); Ashley and Verbanck (1996)]:

$$\eta = \frac{w_s}{\kappa u_*} \leq 3 \qquad \text{(eq. 4.3)}$$

where

η	sedimentation parameter	(-)
w_s	particle settling velocity, for heterogeneous suspension	(mm/s)
u_*	fluid bed shear velocity	(mm/s)
κ	von Karman's constant ≈ 0.4	(-)

The results of simulations with the calibrated hydrodynamic model of Loenen were used to calculate the shear velocity to be at least 0.01 m/s under dwf conditions, whereas the shear velocity ranges between 0 (due to backwater effects) and 0.07 m/s during wwf. Consequently, according to equation 4.3, full suspension will normally be reached under both dwf and wwf conditions for suspended solids with a settling velocity of less than 10 mm/s.

[Saul *et al.* (2003)] describe an empirical and deterministic approach to the transport of suspended solids in sewers. The empirical approach is an extension of the earlier work of [Gupta and Saul (1996)], with the total suspended solids concentration calculated based on the antecedent dry weather period and the increase in discharge. The deterministic approach is based on the assumption of the existence of a weak sediment layer with an increasing critical shear stress with erosion depth [Skipworth *et al.* (1999)]. The Skipworth approach has been reported to be able to predict suspended solids concentrations with an accuracy of 60% of the event mean concentration [Tait *et al.* (2003)].
A limitation of both approaches, however, is the need for reliable measurement data. Acquiring reliable data on suspended solids transport is rather complicated, due to the highly spatial and temporal variability of the characteristics of suspended sewer solids [Jack *et al.* (1996), Ristenpart (1995); Ashley *et al.* (1994)].

Current sewer models provide suspended solids transport models as part of the water quality modelling. However, these models rely upon erosion-deposition criteria developed in fluvial environments, therewith oversimplifying the sewer sediment characteristics [Tait *et al.* (2003)]. Consequently, the performance of these models is poor from a theoretical point of view, as illustrated by the example of Hydroworks/Infoworks [Bouteligier *et al.* (2002a); (2002b)].
Hydroworks/Infoworks provide an erosion-deposition criterion based on the Ackers-White sediment transport theory, which was developed for transport of non-cohesive sediment particles in open channels assuming steady uniform flow. As sewer sediment normally is cohesive [Berlamont and Torfs (1996)], the Ackers-White equation has been modified in the software in order to be more applicable to sewer conditions. Nonetheless, the Ackers-White theory is used out of its application range, which is highly questionable according to [Bouteligier *et al.* (2002a); (2002b)]. Moreover, the interaction between the hydraulics and deposited and eroding sediments has not been taken into account, further limiting the applicability of the model [Margetts (2000)]. Yet, apparently successful applications of the Hydroworks quality module for modelling total suspended solids and COD have been reported [Zug *et al.* (1998), David (2002)].

In order to get an improved understanding of the temporal and spatial variations in suspended solids transport, a measuring network was installed in the sewer system of Loenen in conjunction with the hydraulic measuring network described in section 4.2. In this case only turbidity was measured, as turbidity is relatively easy to measure with a high frequency. Moreover, recent research suggests that turbidity measurements could replace traditional samples for the estimation of the TSS concentration in sewers once the tubidity is calibrated against TSS measurements [Bertrand-Krajewski (2004)]. Without calibration, turbidity cannot be related directly to the actual concentration of suspended solids due to the high variability of the suspended solids characteristics [Jack *et al.* (1996), Ristenpart (1995)]. Nonetheless, variations in suspended solids concentrations will possibly be reflected by variations in the turbidity. Consequently, the turbidity measurements were only used to investigate the dynamics of the transport of suspended solids and to analyse whether these dynamics could be related to the hydrodynamics.

4.4.1 Material and methods

In the well described sewer system of Loenen 8 turbidity sensors (Staiger Mohilo, 7000 SWN4(-T)) were installed in the same period as the aforementioned hydraulic measuring network. The turbidity sensors have been designed according to the ISO 7027 standard for measuring light scattering under a 90 ° angle. The measuring range is 0 – 500 NTU (normalised turbidity units).
Figure 4.34 shows the hydraulic monitoring network and the locations of the turbidity sensors.

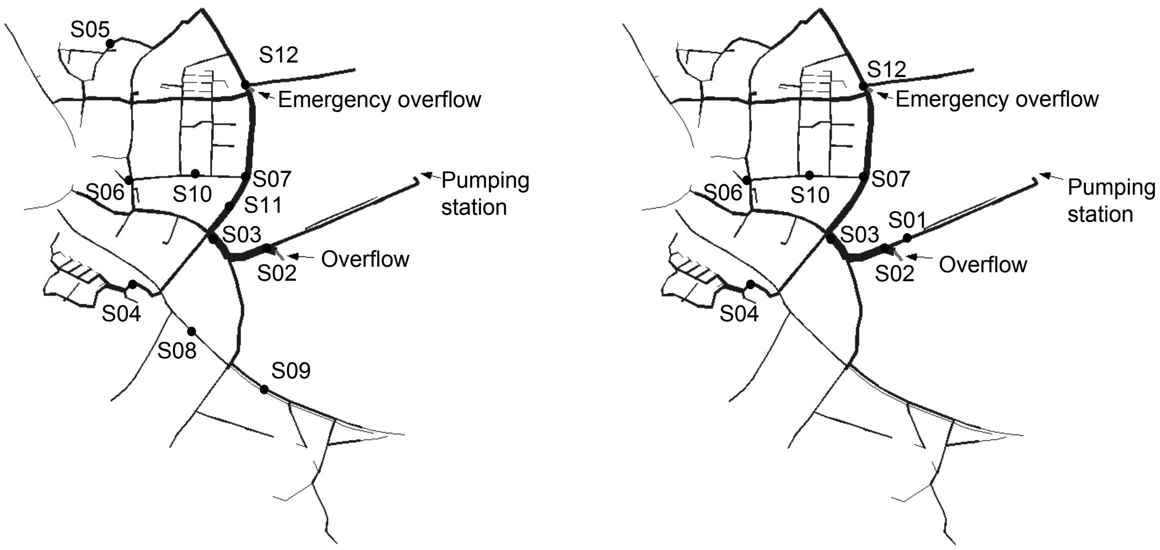


Figure 4.34 Monitoring network Loenen (schematic). Left hand side: locations of water level gauges. Right hand side: locations of turbidity sensors.

The turbidity was recorded with a time interval of 60 seconds. This frequency is selected based on the swift fluctuations in hydraulic conditions in sewer systems [Henckens and Schuit (2002)]. The 8 turbidity probes were installed at a height of 10 cm above the manhole bottom. Although this is sufficient under storm conditions, hardly any recordings are available under dwf conditions. Due to the harsh environment in the sewer system, failure of the measuring equipment occurred frequently. Especially turbidity sensors located in the upstream reaches of the sewer system proved to be vulnerable to clogging [Henckens and Schuit (2002)].
Figure 4.35 shows the measured turbidity for the storm of 18 July 2001, described in detail in section 4.2.

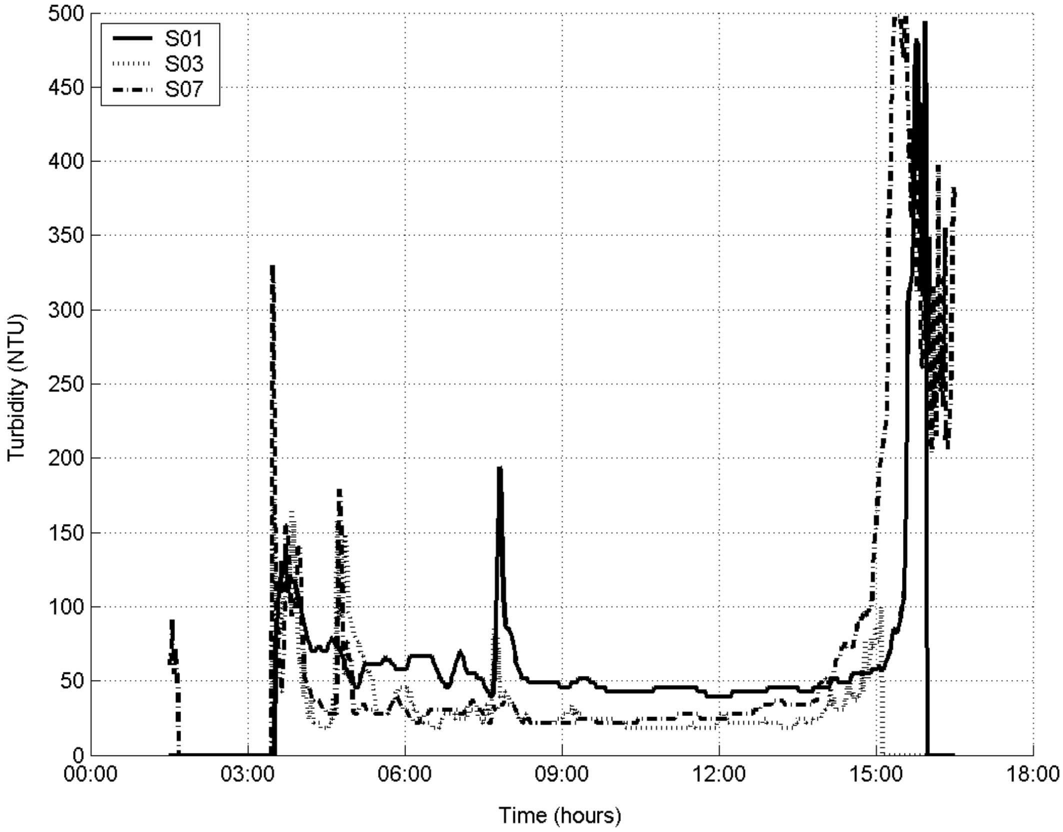


Figure 4.35 Turbidity measured during storm 18 July 2001. The numbers of the locations refer to the right hand map of figure 4.34. The water levels and rainfall intensity are given in figures 4.2, 4.4 and 4.5

The measuring results shown in figure 4.35 are representative for turbidity profiles measured in Loenen during 15 storm events. At the beginning and at the end of the storm event clear peaks in turbidity were measured at all locations. Moreover, during the event a number of turbidity peaks were recorded. Henckens (2001) and Schellart (2002) unsuccessfully tried to link the measured turbidity to the measured water level and rainfall intensity. Moreover, the turbidity does not seem to travel through the sewer system. Turbidity peaks measured at locations downstream could not be related to turbidity peaks measured a few minutes earlier at a location immediately upstream [Veldkamp *et al.* (2002)]. Apparently, local conditions seem to affect the turbidity levels.

As described in section 4.2, 15 storm events have been calibrated. The calibrated storm events comprise most of the available turbidity data. The 15 storm events were used to evaluate the main drivers for suspended solids transport.

4.4.2 Results and discussion

The relation between hydraulic drivers, such as flow velocity and shear stress, derived from the calibrated hydrodynamic model of Loenen and the measured shear stress is discussed and exemplified for only one of the 15 storm events. The storm event selected is the same storm event, 18 July 2001, as presented in section 4.2.

Figure 4.36 shows the turbidity measured at location S01 and the flow velocity, calculated using the calibrated model. At the initial stage of the storm event during the filling of the system the flow velocity and the turbidity both show a peak. At approximately 4:30 hour the sewer reach is completely filled and the flow velocity reduced to 0.2 m/s due to backwater effects. At almost the same moment also the turbidity decreases. At 07:30 (see figure 4.2) it starts raining again and the turbidity increases again, although the flow velocity remains at a level of 0.2 m/s as the sewer system is still filled. At the end of the storm event, during the emptying of the system, the flow velocity increases to 0.35 m/s and at the same time the turbidity peaks at almost 500 NTU.

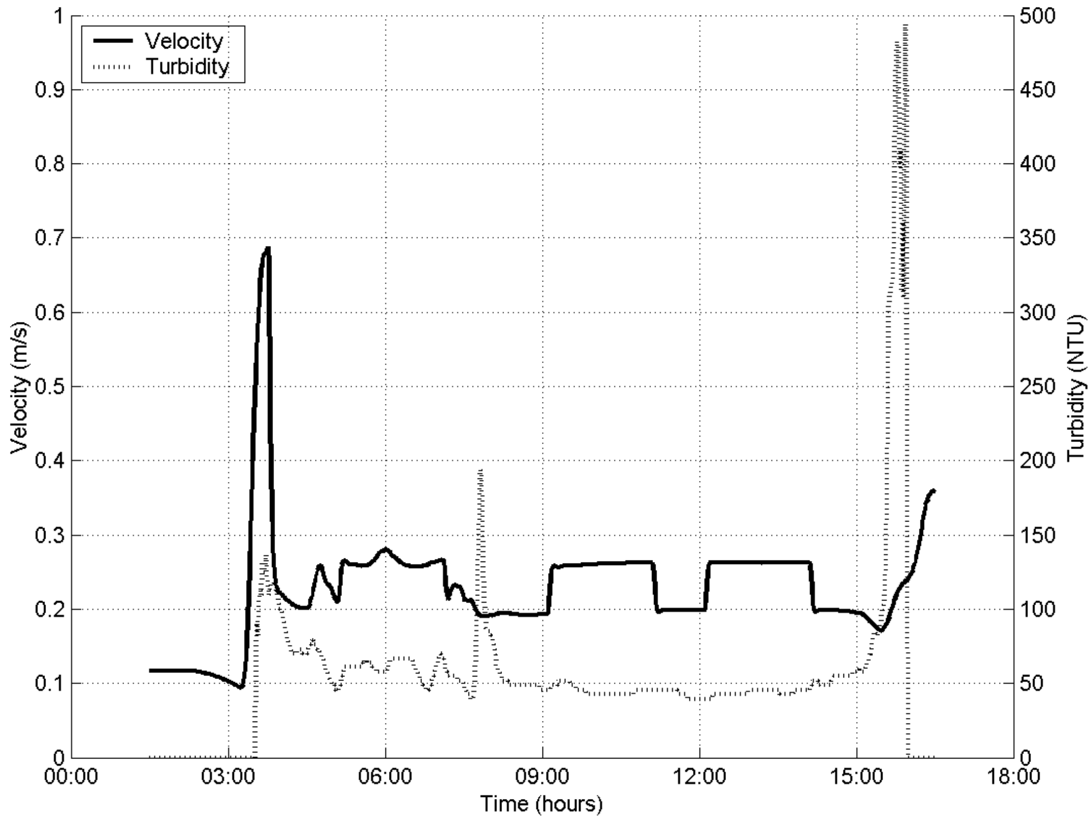


Figure 4.36 Turbidity and flow velocity. Loenen, storm 18 July 2001, location S01.

Figure 4.37 shows the measured turbidity at the same measuring location plotted against the shear stress, which is, according to literature [e.g. Saul *et al.* (2003)], the main driver for suspended solids transport. The shear stress was calculated using the calibrated model and equation 4.4.

$\tau = \rho gRi$ (eq. 4.4)

where

τ	shear stress	(N/m^2)
ρ	fluid density	(kg/m^3)
g	gravity acceleration	(m/s^2)
R	hydraulic radius (calculated each timestep using the calibrated model)	(m)
i	hydraulic gradient (calculated each timestep using the calibrated model)	(-)

The turbidity peaks at the beginning and the end of the storm event seem to be related to the shear stress, with shear stresses up to 2.7 N/m^2 at the beginning of the storm event. The intermediate turbidity peak, however, does not seem to be related to the local shear stress.

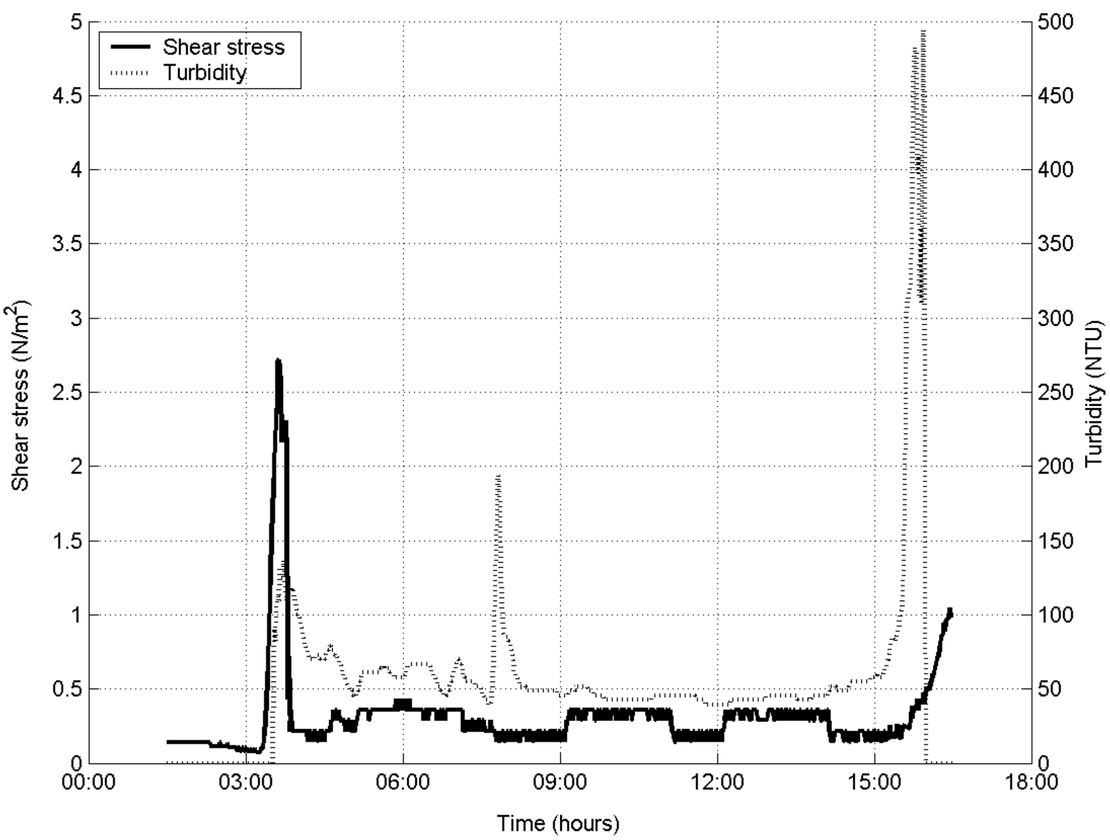


Figure 4.37 Turbidity and local shear stress. Loenen, storm 18 July 2001, location S01.

Figure 4.38 shows the flow velocity and turbidity for the same event for location S07. Like at location S01, both the flow velocity and the turbidity show an initial peak. In this case also the intermediate peak in the turbidity at 04:30 hour seems to correspond with a change in the flow velocity. The shear stress, however, does not reflect this intermediate peak, see figure 4.39. At the end of the storm event the turbidity peaks again at 500 NTU while both the flow velocity and shear stress increase towards the end of the storm event.

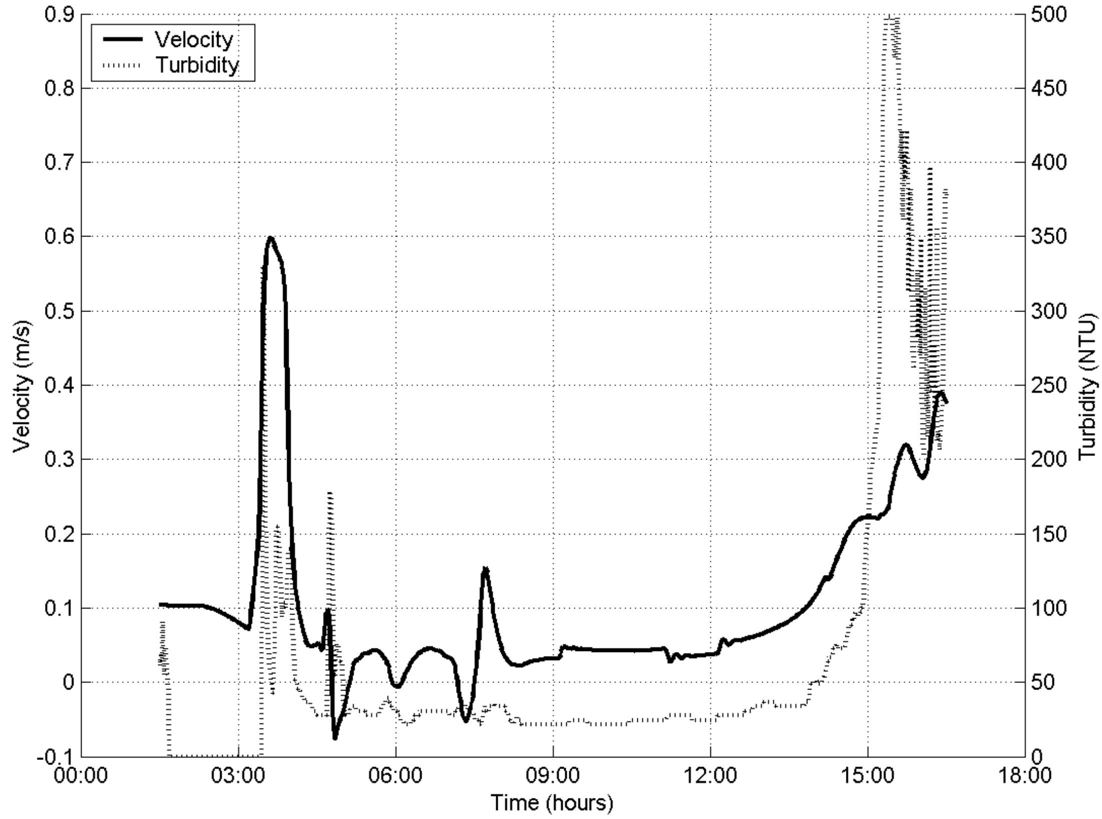


Figure 4.38 Turbidity and flow velocity. The flow direction reversed two times: at 04:30 and 07:30 hour. Loenen, storm 18 July 2001, location S07.

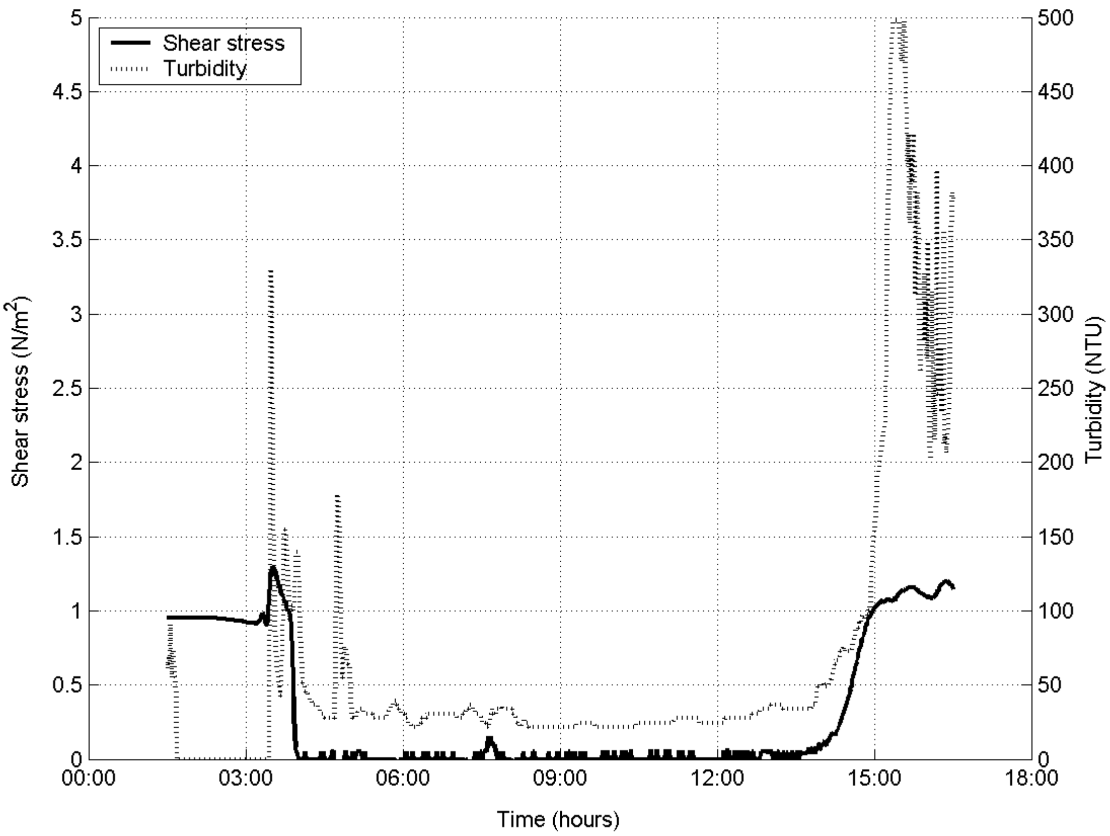


Figure 4.39 Turbidity and local shear stress. Loenen, storm 18 July 2001, location S07.

An analysis of figures 4.37, 4.39 and figure 4.2 shows that intermediate turbidity peaks seem to correspond with periods of rainfall. This phenomenon was studied for all available events, showing that storms consisting of only one continuous period of precipitation do not show intermediate turbidity peaks and storms comprising several periods of rainfall do show these peaks. It has be noted, however, that not all intermediate peaks in turbidity are related to periods of rainfall. Apparently, the filling of the sewer system is a period associated with peaks in the measured turbidity. This effect is illustrated by figure 4.40, showing the calculated inflow and measured turbidity at location S07. The additional inflow during the storm event does not affect the calculated shear stress in the downstream locations affected by backwater effects, as shown in figures 4.37 and 4.39. However, at the upstream measuring location S04 peaks in the shear stress and flow velocity can be noted coinciding with peaks in the measured turbidity at the downstream locations, as illustrated by figures 4.41 and 4.42.

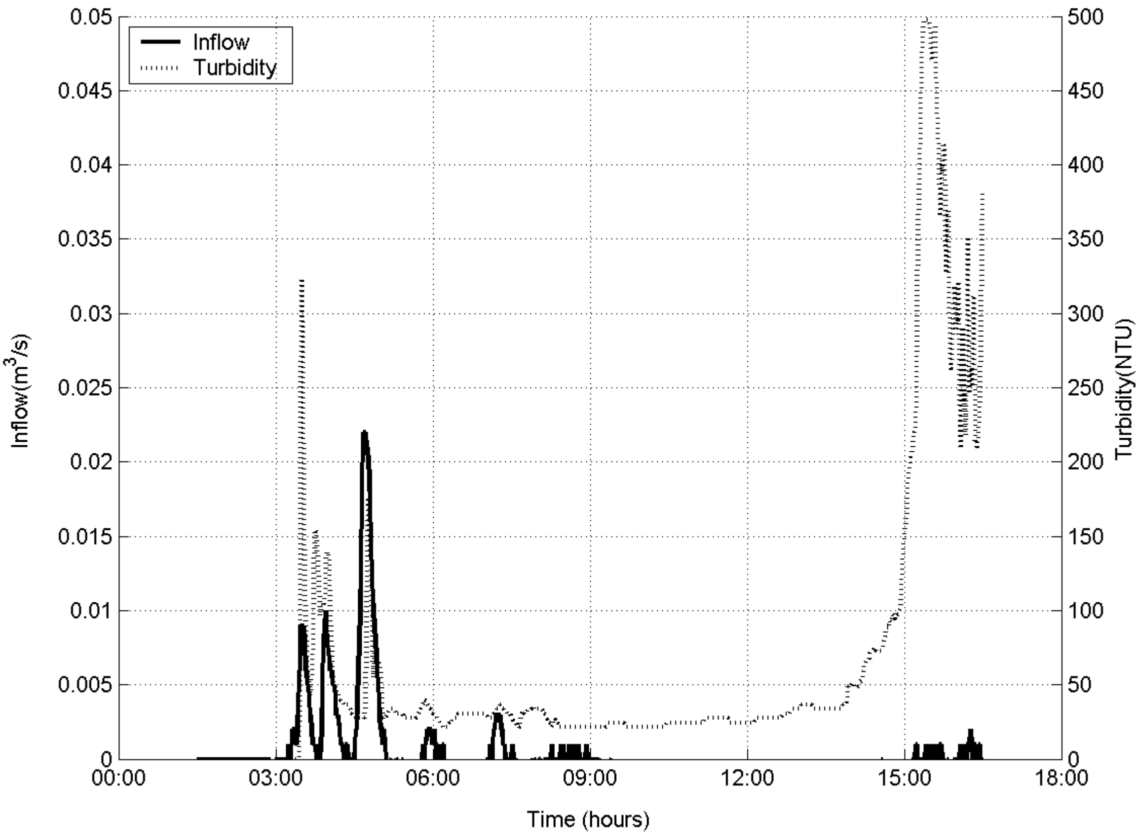


Figure 4.40 Inflow and turbidity for storm 18 July 2001, location S07.

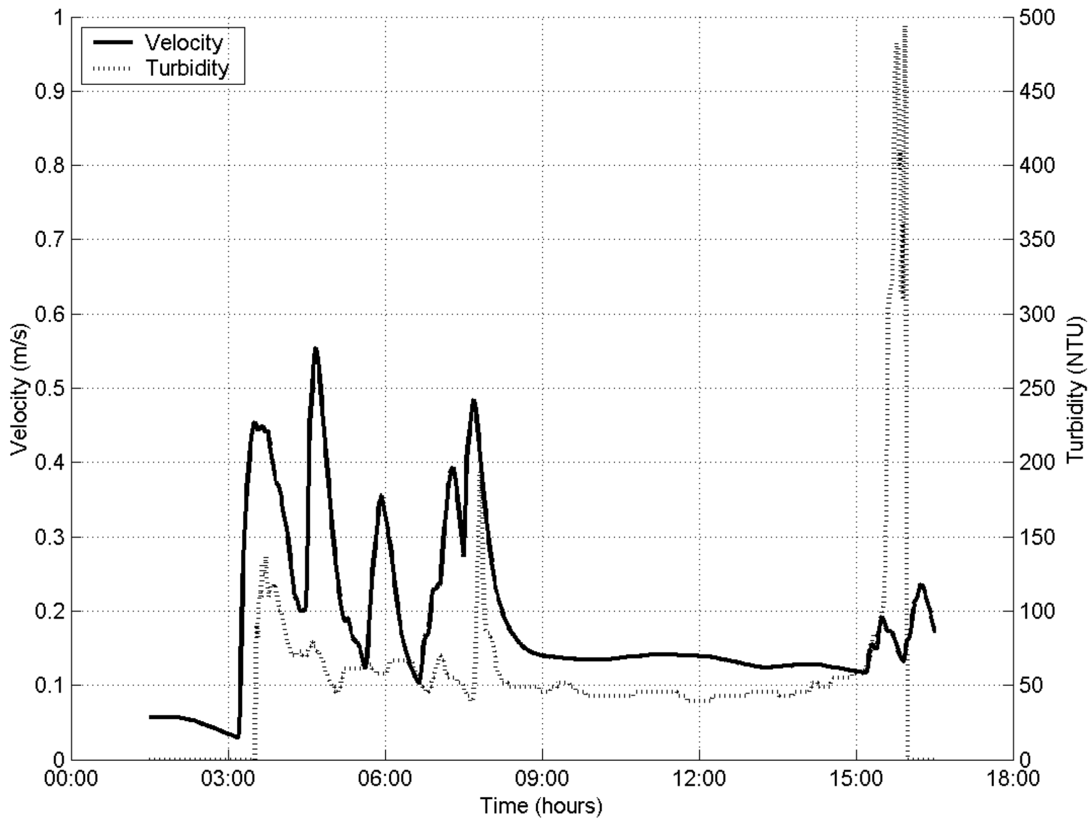


Figure 4.41 Turbidity measured at location S01 and velocity calculated for location S04.

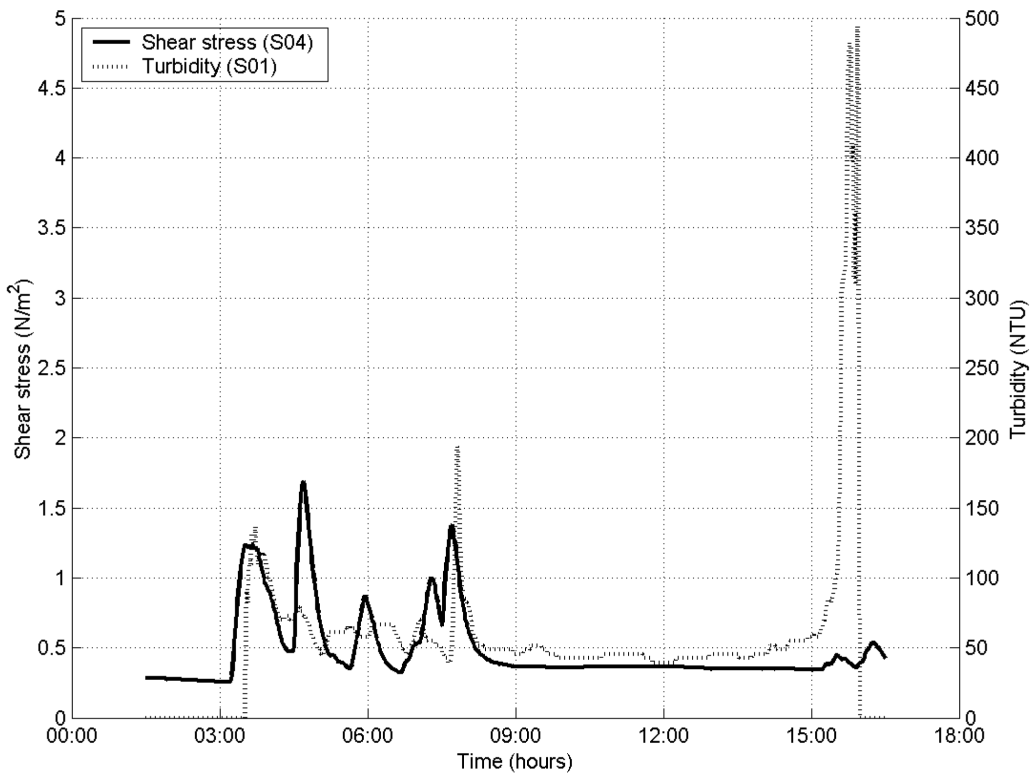


Figure 4.42 Turbidity measured at location S01 and shear stress calculated for location S04.

The afore described analysis was performed for all 15 storm events.
The initial peaks in turbidity always coincide with a peak in the local shear stress and flow velocity. The peak values for this local shear stress vary per location and range from 1.8 - 5 N/m^2 for S01, 0.5 - 4 N/m^2 for S03 and 1.2 - 1.5 N/m^2 for S07. These ranges overlap to a high extent the 0.7 N/m^2– 7 N/m^2 range recorded for critical shear stress [Ristenpart and Uhl (1993)]. Nevertheless, the value of the calculated shear stress and of the flow velocity does not seem to be related with the actual value of the measured turbidity. This may be due to the time varying characteristics of the suspended solids [Jack *et al.* (1996), Ristenpart (1995)].
Consequently, the hydraulic conditions, represented by shear stress and flow velocity, can be regarded as an important driver for the first peak in turbidity but not as a sole explanatory factor. This result supports both the empirical Gupta and Saul approach and the deterministic Skipworth approach, as described by [Saul *et al.* (2003)].
The intermediate peaks in turbidity coincide in 25% of the cases with the calculated local shear stress and approximately 50% of the intermediate turbidity peaks correlates with periods of additional inflow, which is reflected in both the shear stress and flow velocity in the upstream parts of the sewer system not affected by backwaters.

The final peaks in turbidity do not significantly coincide with a peak in the calculated shear stress or flow velocity, with the exception of location S07. An analysis of the calculated shear stress showed that at all measuring locations the shear stress returns to dwf values at the end of the event. At location S01, this dwf shear stress normally does not exceed 0.5 N/m^2, whereas for location S07 (see figure 4.38) this dwf shear stress is always approximately 1 N/m^2. Consequently, at location S07 an increase in the shear stress can always be noted, in contrast to the other locations.

4.4.3 Conclusions on suspended solids transport

Modelling of suspended solids transport has been and will be one of the challenges in the field of urban drainage modelling. The results from the 6 month measuring period in Loenen have shown that the dynamic fluctuations in turbidity, measured at an interval of 60 seconds, seem to be correlated to the dynamics in hydraulics, whether represented by shear stress or flow velocity. A direct relation of either shear stress or flow velocity with turbidity could not be found, likely due to the time varying characteristics of the suspended solids.
Nevertheless, the results of the measuring period indicate that during storm events swift changes in turbidity exist, which are likely to reflect swift changes in the suspended solids concentrations. Therefore, it is concluded that modelling approaches for suspended solids transport should be capable of dealing with these swift fluctuations.

4.5 Transformations in sewer systems

In-sewer transformation processes affect wastewater quality. Especially during dwf the transformation processes can exert a significant influence on wastewater quality [Hvitved-Jacobsen *et al.* (2002)]. The sewer system is dominated by heterotrophic micro-organisms that degrade and transform wastewater components. Easily biodegradable substrate is removed and biomass is produced. For a small catchment in Switzerland, [Huisman (2001)] calculated the total conversions of dissolved COD to be 30 % under aerobic conditions. Dutch sewer systems are normally well aerated. Therefore, aerobic conditions will prevail. This section describes the analysis of the transformation rates to be found in a typical Dutch sewer.

4.5.1 Material and methods

The transformation rates in a sewer under aerobic conditions cannot be measured easily, as especially the transformation rate in the sewer biofilm is hard to measure without advanced equipment [Huisman (2001)]. Therefore, the transformation rates under aerobic conditions were estimated from an oxygen mass balance over a sewer reach. Equation 4.5 gives a mass balance, assuming that no accumulation of oxygen within the sewer reach takes place, [Huisman (2001)]:

$$\overbrace{\mathit{inflow} + \mathit{reaeration}}^{\mathit{in}} = \overbrace{\mathit{outflow}}^{\mathit{out}} + \overbrace{\mathit{conversion\ in\ biofilm} + \mathit{conversion\ in\ suspension}}^{\mathit{conversion}} \quad \text{(eq. 4.5)}$$

In this case, sewer reach 'Wippenpol' in the sewer system of Beekbergen, shown in figure 4.43 and described previously in section 4.3, served again as measuring location. The hydraulic measuring network of Beekbergen provided basic data on water levels, whereas the simultaneous tracer experiments give very accurate estimates of travel times and flows. In addition to the tracer experiments, oxygen probes were installed at the upstream (WTW Oxi 325) and downstream (WTW Oxi 340i) end of the sewer reach, see figure 4.40. Moreover, 14 wastewater samples, taken at the downstream end of the sewer reach, were used to measure oxygen uptake rates (OUR) and the water quality parameters COD_{total}, $COD_{dissolved}$ using Merck test no. 1.14541 and ammonium using Merck test no. 1.14559. In addition, the oxygen content of the sewer atmosphere was measured. The oxygen uptake rate was measured on site in a completely stirred reactor by measuring the oxygen concentration and temperature after aerating the sample for 60 seconds. This resulted in an initial oxygen concentration at the start of the OUR measurements of approximately 4 mg O_2/l. The OUR measurements were finished as soon as the oxygen concentration was below 0.2 mg O_2/l.

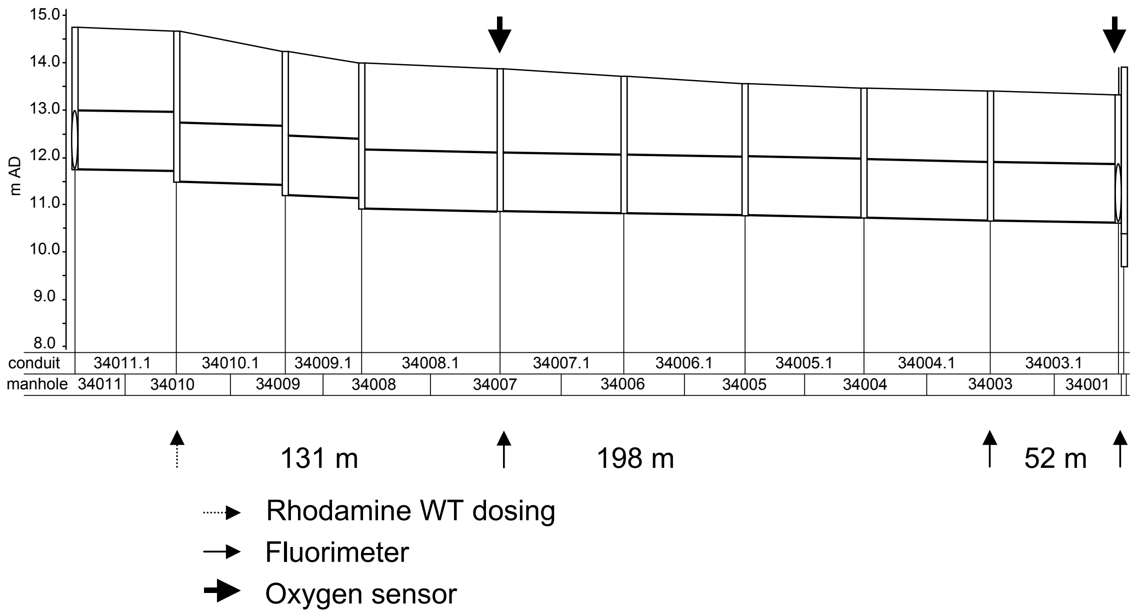


Figure 4.43 Measuring locations sewer system Beekbergen, reach 'Wippenpol'.

The measured hydraulic and water quality data provide information on all items but the conversions in the biofilm from the mass balance of equation 4.5:

inflow — the upstream oxygen level was recorded at a 15 seconds interval

reaeration — the reaeration was estimated using available reaeration equations, which is acceptable as the hydraulic conditions are well-known

outflow — the downstream oxygen level was recorded at a 15 seconds interval

conversion in biofilm — no information available

conversion in suspension — the conversion rate in the suspension was estimated using OUR measurements

The aerobic transformations in sewer systems can also be estimated using available models. In this case, the performance of the ASM1 model adapted for sewer conditions [Vollertsen and Hvitved-Jacobsen (2000)] was tested.

4.5.2 Results and discussion

The results of the tracer experiments of 16 September 2003 were used to calculate the travel time and the wastewater flow over sewer reach 'Wippenpol', as shown in table 4.14. Although the flow varied between 22.4 l/s and 27.9 l/s, the travel time was rather constant with a mean of 680 s. The average flow velocity in the 250 m sewer reach was 0.37 m/s.

Table 4.14 Travel times and flow derived from tracer experiments in sewer reach 'Wippenpol'.

time (HH:MM)	travel time (s)	flow (l/s)
15:08	680	22.4
16:21	674	22.4
16:45	677	23.4
17:05	685	27.9
17:27	682	22.3

Figure 4.44 shows the measured oxygen levels and the measured OUR of the sewage. The differences in downstream and upstream oxygen levels seem to be correlated to the OUR, indicating that the biological activity of suspended biomass is significant. The decrease in oxygen level over the sewer reach does not have a clear relation with the DO concentration of the sewage at the upstream manhole.

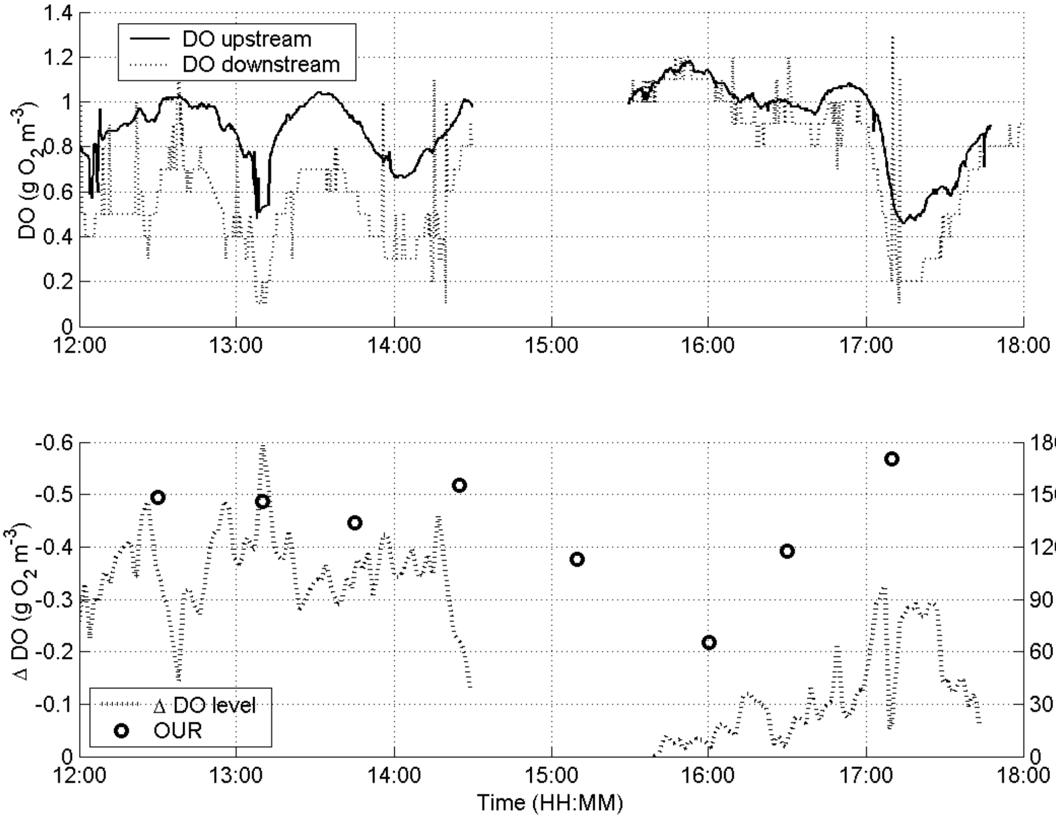


Figure 4.44 Experiment Beekbergen, reach Wippenpol, 16/09/03. Oxygen levels in upstream and downstream manhole (top) and the decrease in oxygen level and the OUR of the sewage (down). The measured values of the downstream manhole were plotted with a time shift equal to the calculated mean travel time. The horizontal lines of the downstream dissolved oxygen concentration are due to the downstream oxygen sensor saving the oxygen concentration with only one decimal.

The ASM [Henze *et al.* (1987)] based model concept for transformations in sewer systems described by [Hvitved-Jacobsen *et al.* (1998b)] was used to study the applicability of the model for Dutch sewer conditions. In this case, the wastewater was aerobic during the experiments. Consequently, the model outlined in table 4.15 is limited to aerobic conversions. The model was used to simulate the measured decrease in oxygen level over the sewer reach in Beekbergen with the model parameters derived from literature [Hvitved-Jacobsen *et al.* (1998a); Hvitved-Jacobsen *et al.* (1998b)], given in table 4.18.

Table 4.15 Aerobic transformation processes (after: [Hvitved-Jacobsen *et al.* 1998a)]. Wastewater fractions and model parameters are given in table 4.16 and 4.18 respectively.

	S_s	X_{Bw}	X_{S1}	X_{S2}	$-S_O$	process rate
aerobic growth in bulk water	$-1/Y_{Hw}$	1			$(1-Y_{Hw})/Y_{Hw}$	Eq. 4.6a
aerobic growth in biofilm	$-1/Y_{Hf}$	1			$(1-Y_{Hf})/Y_{Hf}$	Eq. 4.6b
maintenance energy requirement	-1				1	Eq. 4.6c
aerobic hydrolysis, fast	1		-1			Eq. 4.6d, n =1
aerobic hydrolysis, slow	1			-1		Eq. 4.6d, n = 2
reaeration					-1	Eq. 4.6e

$\mu_H\, S_S/(K_S+S_S)\, S_O/(K_O+S_O)\, X_{Bw}\, \alpha_w^{(T-20)}$ (eq. 4.6a)

$k_{1/2}\, S_O^{0.5}\, Y_{Hf}/(1-Y_{Hf})\, A/V\, S_S/(K_{Sf}+S_S)\, \alpha_f^{(T-20)}$ (eq. 4.6b)

$q_m\, S_O/(K_O+S_O)\, X_{Bw}\, \alpha_w^{(T-20)}$ (eq. 4.6c)

$k_{hn}\, (X_{Sn}/X_{Bw})/(K_{Xn}+X_{Sn}/X_{Bw})\, S_O/(K_O+S_O)\, (X_{Bw} + \varepsilon\, X_{Bf}\, A/V)\alpha_w^{(T-20)}$ (eq. 4.6d)

$K_La\, (S_{OS}-S_O)$, where S_{OS} is the dissolved oxygen saturation concentration (g/m^3), derived from empirical equation 4.6f) (eq. 4.6e)

$S_{OS} = 468/(31.6+T)$, with T in °C (eq. 4.6f)

The reaeration coefficient K_La, given in equation 4.7, used by [Hvitved-Jacobsen *et al.* (1998a)], is based on results from radiotracer experiments [Jensen and Hvitved-Jacobsen (1991)] and the original work of [Parkhurst and Pomeroy (1972)]. Equation 4.8 gives the reaeration equation derived by [Huisman (2001)] based on gas exchange measurements using sulphur hexafluoride (SF_6). For the conditions in the Beekbergen sewer reach, hardly any difference exist in the calculated reaeration coefficients, with a K_La of 0.547 h^{-1} for equation 4.7 and 0.532 h^{-1} for equation 4.8. However, for pipes with a higher gradient and higher flow velocities equation 4.5 and 4.6 start to deviate significantly. E.g. if the Wippenpol sewer reach would have had a sewer slope of 9‰ rather than 1‰ the difference in calculated reaeration coefficient would have been 22% instead of 3%.

$$K_La = 0.86\,\frac{(1+0.2Fr^2)(su)^{3/8}}{d_{mean}} \qquad \text{(eq. 4.7)}$$

$$K_La = 1.375\,\frac{u^*(1+0.41Fr^2)}{d_{mean}} \qquad \text{(eq. 4.8)}$$

where

K_La	reaeration coefficient	(h^{-1})
Fr	Froude number $(u/(g\, d_m)^{0.5})$	(-)
g	gravitational acceleration	(m/s^2)
u	flow velocity	(m/s)
s	sewer slope	(m/m)
d_{mean}	hydraulic mean depth (= w/A)	(m)
w	width of water surface	(m)
A	cross sectional area of flow	(m)
u^*	shear velocity	(m/s)

Table 4.16 COD components and dissolved oxygen of wastewater in upstream manhole. The fractionation is based on [Hvitved-Jacobsen *et al.* (1998b)] and measured COD fractions given in table 4.17

Component		value used in model	unit
X_{Bw}	heterotrophic active biomass	60 (adjusted for each simulation)	g COD/m^3
X_{Bf}	heterotrophic active biomass	-	g COD/m^3
X_{S1}	hydrolysable substrate, fast biodegradable	40	g COD/m^3
X_{S2}	hydrolysable substrate, slowly biodegradable	40	g COD/m^3
S_S	readily biodegradable substrate	50	g COD/m^3
S_i	inert soluble COD	250	g COD/m^3
S_O	dissolved oxygen	measured upstream	g O_2/m^3
COD	total COD	440 (mean of 14 samples)	g COD/m^3

w: water phase, f: biofilm

Table 4.17 Measured COD concentrations and upstream dissolved oxygen levels.

Time (HH:MM)	COD_{total}	$COD_{dissolved}$	$COD_{suspended}$	DO measured upstream (g O_2/m^3)
8:45	520	230	290	
11:30	530	360	170	
10:40	640	330	310	
12:30	440	280	160	1.0
13:10	420	320	100	0.5
13:45	430	310	120	0.9
14:25	380	310	70	1.0
16:00	390	300	90	1.2
16:30	420	310	110	1.0
17:10	400	310	90	0.5

Table 4.18 Model parameters used in the sewer process model given in table 4.15. Only the value of KO was adjusted compared to the values given by [Hvitved-Jacobsen *et al.* (1998b)]

symbol and definition		value (range in literature)	Unit
μ_H	maximum specific growth rate for heterotrophic biomass	3.25 (3.25[b] – 7[a])	d^{-1}
Y_{Hw}	suspended biomass yield constant for heterotrophics	0.55	gCOD/gCOD
K_S	saturation constant for readily biodegradable substrate	1.0	$gCOD/m^3$
K_O	saturation constant for DO	**0.1** (0.05[cd] -0.5 [ab])	gO_2/m^3
α_w	temperature coefficient in the water phase	1.07	-
q_m	maintenance energy requirement rate constant	1.0	d^{-1}
$k_{½}$	½ order rate constant	4.8 (2.5[a] - 4.8[b])	$gO_2^{0.5}m^{-0.5}d^{-1}$
Y_{Hf}	biofilm yield constant for readily heterotrophic biomass	0.55	gCOD/gCOD
K_{Sf}	saturation constant for readily biodegradable substrate	1.0	$gCOD/m^3$
ε	efficiency constant for the biofilm biomass	-	-
α_f	temperature coefficient in the biofilm	1.03	-
k_{h1}	hydrolysis rate constant, fraction 1 (fast)	4.0	d^{-1}
k_{h2}	hydrolysis rate constant, fraction 2 (slow)	1.0	d^{-1}
K_{X1}	saturation constant for hydrolysis, fraction 1	0.5	gCOD/gCOD
K_{X2}	saturation constant for hydrolysis, fraction 2	0.2	gCOD/gCOD

[a] [Hvitved-Jacobsen *et al.* (1998a)]
[b] [Hvitved-Jacobsen *et al.* (1998b)]
[c] [Gudjonsson *et al.* (2002)]
[d] [Vollertsen and Hvitved-Jacobsen (2000)]

The measured OUR fluctuated significantly over the measuring period, as shown in figure 4.44, indicating that the biological activity of the wastewater varies. The measured OUR values were used to determine the suspended heterotrophic biomass concentration X_{Bw} of

each sample with equation 4.6a, assuming a μ_H of 3.25 d^{-1} [Hvitved-Jacobsen *et al.* (1998)]. This assumption is valid, as the process rate of aerobic growth in bulk water has a linear relation with both the specific growth rate μ_H and the suspended heterotrophic biomass and in addition the hydraulic retention time was only 680 seconds.
The derived heterotrophic biomass concentration was subsequently used in the simulations. The only model parameter adjusted compared to the literature values given in table 4.18 was the saturation constant for dissolved oxygen, K_O. The K_O was set to 0.1 g O_2/m^3 for all experiments. Table 4.19 gives the results of the simulation of the dissolved oxygen concentration for sewer reach Wippenpol.

Table 4.19 Measured and simulated dissolved oxygen levels.

experiment (time)	X_{Bw} (g COD/m^3)	DO measured upstream (g O_2/m^3)	DO measured downstream (g O_2/m^3)	DO simulated downstream (g O_2/m^3)	simulated – measured DO (g O_2/m^3)	% contribution of biofilm to aerobic conversions
12:30	62.2	1.0	0.6	0.7	0.1	19
13:10	61.1	0.5	0.1	0.4	0.3	16
13:45	56.2	0.9	0.6	0.7	0.1	20
14:25	65.2	1.0	0.7	0.7	0.0	19
16:00	27.3	1.2	1.1	1.2	0.1	37
16:30	49.3	1.0	1.0	0.9	-0.1	23
17:10	71.4	0.5	0.3	0.3	0.0	14

The difference between the measured and simulated values in table 4.19 is rather low for the range of upstream dissolved oxygen and COD_{total}, $COD_{dissolved}$ and $COD_{suspended}$ levels, see table 4.17. Therefore, it is concluded that the ASM1 based sewer model as shown in table 4.15 properly describes the changes in dissolved oxygen level in an aerobic sewer reach, thus confirming literature results [Hvitved-Jacobsen *et al.* (1998a, 1998b)].
Furthermore, the observed reaeration rate, OUR of the sewage and transformation rates are comparable with values found in literature for sewers with comparable slopes [e.g. Huisman (2001)]. This indicates that the results obtained for this sewer reach can be considered to be representative for conditions typically observed in Dutch sewer systems (especially the low slope of the sewers).

The relative contribution of the biofilm to the total aerobic COD conversions, based on model results, was on average only 21 %. Therefore, it is concluded that the aerobic conversions in this sewer reach were dominated by the conversions by suspended biomass. Consequently, the OUR of the sewage gives a good indication of the total aerobic activity in the sewer.

In addition, the total aerobic conversions in a sewer system are limited by the reaeration capacity. Therefore, as long as the reaeration capacity is less than the total aerobic activity of the biofilm and the wastewater, the potential for aerobic conversions in a typical Dutch sewer system can best be estimated by the available reaeration capacity.
The reaeration can be calculated using equation 4.8. For the sewer reach in Beekbergen, which is representative for many sewers in the Netherlands, an average reaeration rate of 100 g $O_2m^{-3}d^{-1}$ was calculated. Combining this reaeration rate with the average aerobic hydraulic retention time of a typical Dutch sewer system of 2 hours (u = 0.25 m/s, transport distance to pumping station = 1,800 m), the total amount of available oxygen is 8.3 g O_2/m^3. With a yield coefficient of 0.55 d^{-1} (see table 4.18) to 0.67 d^{-1} [Huisman (2001)], this results in a maximum aerobic conversion within sewer systems between 18 to 25 g COD/m^3. As this aerobic conversion is an order of magnitude less than the fluctuations in COD concentrations observed in sewer systems (the COD varied between 380 and 640 g COD/m^3 during the experiments on the 16th of September in Beekbergen, see table 4.19), the aerobic transformations in a typical Dutch sewer do not significantly contribute to the fluctuations in the influent concentration.

4.5.3 Conclusion on aerobic transformations in sewer systems

The rate of fluctuations in COD concentrations in sewer systems is an order of magnitude higher than the aerobic transformation rate. Consequently, it is concluded that the aerobic transformations in sewer systems are generally not relevant with respect to the influent fluctuations for Dutch wastewater systems. However, in situations with very long aerobic transport times, the aerobic conversions can be significant, as illustrated by [Hvitved-Jacobsen *et al.* (2002) and Ashley *et al.* (2002b)]. This conclusion confirms research by [Koch and Zandi (1973)], who found that an aerobic length of pipe of 42.7 km was necessary to be able to remove 30% of the BOD.

4.6 Conclusion

In chapter 3 requirements for the quality of wwtp influent data, to be provided by sewer process models, were established. In this chapter, the quality of simulation results of current sewer process models was compared with these requirements. The main conclusions in this respect are:

- the modelling of the hydrodynamics with state of the art sewer models is sufficiently reliable to provide the influent data necessary to be able to assess the impact of the dynamic interactions within wastewater systems;
- the quality of the modelling of solute transport is just meeting the requirements. As this conclusion is based on only one experiment (and confirmed by literature) it is recommended to verify the conclusions with additional experiments;
- the aerobic biotransformations in sewer systems are of limited interest with respect to the fluctuations in the influent composition unless there are long travel times.

In addition to these conclusions with respect to the quality of influent data, it is concluded with respect to CSO discharges that:

- current knowledge of the hydrodynamics in sewer systems allows a realistic simulation of CSO volumes for event-based calibrated models. As the portability of the model parameters is low, the accuracy of predicted CSO volumes for rain series is significantly less than can be achieved by event-based calibrated models;
- the knowledge on suspended solids transport is still insufficient to reliably describe suspended solids concentrations discharged by CSOs.

Chapter 5 Optimisation of wastewater systems: optimisation techniques

5.1 Introduction

Wastewater systems usually develop over several decades and have an even longer lifetime. As with any part of the public infrastructure, wastewater systems are subject to changes during their lifetime [Korving *et al.* (2001)]. These changes can be manifold, ranging from simple physical demands such as the number of connected dwellings or a change in water use to tightening of environmental standards, as highlighted in chapter 1. Consequently, wastewater systems often need improvements during their lifetime in order to be able to meet the requirements. Each time a wastewater system needs to be improved, a potential optimisation problem arises, in which e.g. the investment (or whole life) costs have to be minimised while meeting the requirements. The introduction of the interactions between sewer system and wwtp to wastewater system optimisation studies generally complicates the optimisation problem. This is due to the fact that, compared with a traditional volume based approach, the number of possible measures to be taken into account increases and the assessment of wastewater system performance is more complicated.

The application of heuristic search is a possible solution to the increasingly complicated wastewater system optimisation studies. This chapter discusses the applicability of heuristic search algorithms for wastewater system optimisation.

5.2 Wastewater system optimisation

Wastewater system optimisation studies are increasingly applied in the Netherlands [Mameren, van (2001)] and elsewhere [Gill *et al.* (2001)] to adjust wastewater system performance to meet the (new) requirements. Although wastewater system optimisation studies are rather diverse in their incorporation of technical, political, social, financial and economic aspects, all optimisation studies have a number of components in common.

Essentially, wastewater system optimisation comes down to identifying the configuration and/or operation strategy of the wastewater system that best meets the combination of objective(s) and constraints. In practice, this implies:

- searching options for improvement.
- qualifying these options.

Searching options for improvement

The types of measures to be taken into account to improve wastewater system performance are related to the approach adopted. Figure 5.1 gives a schematic overview of a wastewater system which is to be optimised using a volume based approach, as e.g. applied in the Netherlands. The interaction between sewer system and wwtp is in this case reduced to a boundary condition (being the hydraulic capacity of the wwtp). In a volume based approach, only measures affecting the CSO volume are taken into account, reducing the types of potential measures to additional storage, pumping capacity or reducing the impervious area discharging to the sewer system. Even in a volume based approach with only 3 types of measures, the number of possible combinations for optimisation of the wastewater system can be extremely high. Assuming that each type of measure can be implemented in 3 ways (additional storage capacity of 1, 2 or 3 mm, additional pumping capacity of 0.1, 0.2 or 0.3 mm/h and reducing the impervious area with 10, 20 or 30%) the simple case with 1 contributing sewer system has already 3^3 possible solutions. In reality, most wastewater systems comprise (much) more than 1 contributing sewer system. This enlarges the optimisation problem exponentially with the number of catchments, assuming that each catchment has the same number of possible solutions.

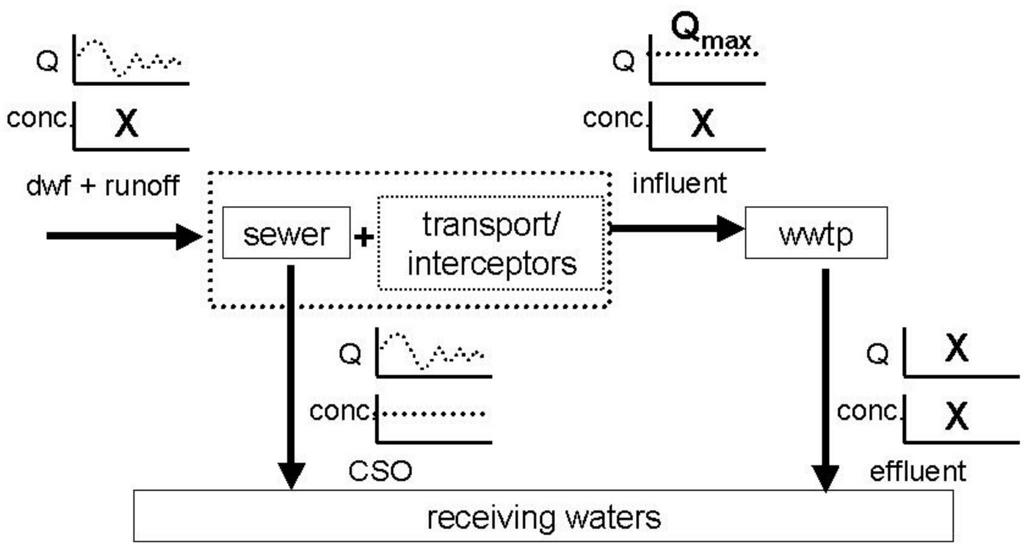


Figure 5.1 Assessment of dynamics within the wastewater system in a volume based approach. A **X** indicates that a parameter (Q (flow) or conc. (concentration)) is not taken into account, a horizontal dotted line that a parameter is assumed constant and a fluctuating dotted line that the fluctuations of a parameter are assessed.

Figure 5.2 illustrates schematically the impact of applying a water quality based approach to wastewater system optimisation. In this case, the dynamics of the inflow to the sewer system, the wwtp influent and effluent, and the CSO discharge in terms of flow and water quality are taken into account. Consequently, measures like water quality based RTC or routing sewer sediment through a sewer system, which are not options in a volume based approach, become available. As a result, the optimisation problem becomes even more complicated as the number of options increases.

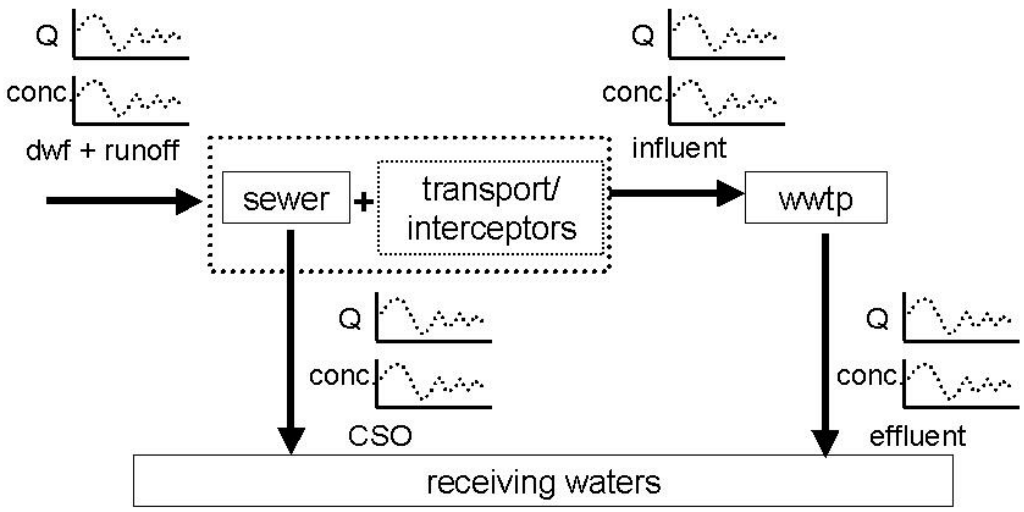


Figure 5.2 Assessment of dynamics within the wastewater system in a water quality based approach.

Theoretically, the search for the optimal combination of measures can be performed:

- manually; due to practical limitations such as available time and money budget, only a limited number of combinations of measures can manually be selected and evaluated. The best option is supposed to represent the ‘optimal solution’.
- automatically; an algorithm searches automatically for the ‘best’ solution. In general, two families of search algorithms can be distinguished:
  - gradient based methods
  - heuristic methods

The search methodology to be considered most appropriate strongly depends on the optimisation problem. Manual optimisation is limited to clear optimisation problems with a rather limited number of variables, while automatic optimisation is more suited for larger and more complicated optimisation problems. Wastewater system optimisation problems typically show multiple optima and discontinuities, limiting the applicability of many search algorithms such as classical gradient based techniques [Rauch and Harremoës (1999b)]. Heuristic methods have been reported to be capable of dealing with the type of optimisation problem encountered in wastewater system optimisation [Gill *et al.* (2001)]. Section 5.3 discusses the applicability of two heuristic search algorithms to be used within wastewater system optimisation studies.

Assessing the quality of solutions to wastewater system optimisation problems
Usually, the objective(s) and constraints for a wastewater system optimisation study are described in general terms. In order to be able to take these into account within an optimisation study, quantifiable objectives and constraints need to be formulated [Rauch and Harremoës (1999a), (1999b)]. In general, wastewater system optimisation is a trade off between costs and performance, which means that both costs and performance have to be quantified.

The inclusion of costs within wastewater system optimisation studies is in the Netherlands normally limited to provision costs [Boomgaard *et al.* (2001)], although sometimes also operational costs are included [e.g. Willemsen (2000)].
In the UK, a whole life costing approach is being developed for sewer asset management. Within this approach, the emphasis is on including all relevant costs, including provision, replacement, maintenance and operation [Cashman *et al.* (2002)]. The level of detail of the assessment of costs should be defined in an early stage of the optimisation process by the stakeholders.

The performance of a wastewater system is to be assessed by performance indicators, which depend on the adopted approach (e.g. volume or water quality based). Theoretically, all aspects involved in wastewater system performance should be taken into account by the use of measurable performance indicators [Matos *et al.* (2003); Ashley and Hopkinson (2002)]. However, a sewer system benchmark study in the Netherlands revealed that the data, necessary to assess the performance of sewer systems to other topics than directly related to the standards and regulations, is very hard to get [Stichting Rioned (2003b)].

Costs are normally incorporated in an objective function, shown in equation 5.1, as they are relatively easy to quantify. In addition, a penalty function can be included to embed the performance of the wastewater system in an objective function. The latter requires capitalising wastewater system performance, potentially introducing subjectivity in the objective function, as e.g. no universal value for the discharge of 1 m^3 wastewater through a CSO exists. This topic is addressed in more detail by [Korving (2004)]. However, wastewater performance can also be addressed separately as additional constraints [Boomgaard *et al.* (2001)].

$$S_{(costs)} = \sum_{i=1}^{n} \alpha_i (M_i) + \alpha_p (P_{required} - P) \qquad \text{(eq. 5.1)}$$

where:

S	= objective function
α_i	= specific cost function
M_i	= specific measure
α_p	= penalty function
P	= performance level

5.3 Algorithms for optimisation of wastewater systems

The search for the optimal solution can be performed manually or automatically with the use of suitable gradient based or heuristic methods, as mentioned before. The latter option is promising given the typical characteristics of wastewater system optimisation studies:

- wastewater systems show non linear behaviour, making wastewater system performance hard to predict;
- measures taken in one part of the wastewater system may have detrimental effects on the performance of other parts of the wastewater system. A well known example of the latter is the impact of large storage volumes in the sewer system on wwtp performance [Durchschlag *et al.* (1992)], see also chapter 6;
- objective functions for wastewater systems may be discontinuous;
- wastewater system optimisation may be subject to a wide variety of boundary conditions, such as political, administrative and social constraints;
- the potentially could be a myriad of possible solutions.

Heuristic methods have been reported to be able to deal with optimisation problems like wastewater system optimisation [Rauch and Harremoës (1999a); (1999b), Goldberg (1989), Muschalla (2002)]. However, examples in literature are limited to single objective optimisation studies, whereas wastewater system optimisation studies typically comprise multiple objectives and constraints. Therefore, the potential of the use of two heuristic methods, a genetic algorithm and simulated annealing, for dealing with wastewater system optimisation has been analysed. These two algorithms were selected as at Delft University of Technology these algorithms were already used for calibration of hydrodynamic models [Boomgaard *et al.* (2002a)].

5.3.1 Genetic algorithms

The term ‘genetic algorithm’ and its underlying principles were introduced in the late 1970’s by Holland (1975). The basic concept is to build an analogue optimisation process as it is postulated to be applied by living nature: ‘survival of the fittest’, resulting in a kind of statistical trial and error process. The terms and specific jargon typically used when working with genetic algorithms is somewhat peculiar and uses terms like ‘population, genes, mutate and cross-over’. The general structure of a genetic algorithm is illustrated by a pseudo code, shown in figure 5.3.

```
Procedure Genetic Algorithm
Begin
t:= 0
    Initialise population (t)
    Evaluate population (t)
    While criterion not satisfied do:
        Begin
            t:= t+1
            Select population (t)
            Crossover population (t)
            Mutate population (t)
            Evaluate population (t)
        End
End
```

Figure 5.3 Pseudo code of a genetic algorithm (after [Veltri and Pecora (2001)].

Each solution to the wastewater optimisation problem consists of a combination of a number of measures, like a chromosome consists of a number of genes. When using a genetic algorithm, each solution is coded as a combination of genes. At the start of an optimisation with a genetic algorithm, a number of possible solutions is generated, forming the individuals of the initial population.

Each individual of this initial population is evaluated by the results of the objective function. The results of this evaluation will be used to rank the individuals according to their ‘fitness’.

The 'fittest' individuals (or 'best' solutions) are selected to generate a new population (offspring) by techniques called 'crossover' and 'mutation'. Crossover recombines parts of the most promising individuals, forming new individuals. As such, crossover does not introduce new information, or in this case measures, but combines existing genes. In order to introduce new information, the newly formed individuals are subject to mutation, which in this case takes place at the level of individual measures, the 'genes' of the solutions with a selected probability. After mutation, the new generation is ready to be evaluated against the objective function.
This procedure continues until predefined optimisation criteria are met. In cases where it is not possible to define a proper optimisation criterion, e.g. when no knowledge on the objective function exists, the procedure can be stopped after a predefined number of generations.

A genetic algorithm can be designed according to the characteristics of the optimisation problem to be solved. The three main parameters to be selected are:
- population size;
- type of crossover: exchange of individual or packages of genes (measures);
- probability of mutation and mutation rate.

5.3.2 Simulated annealing

Simulated annealing (SA) is a stochastic computational technique derived from statistical mechanics for finding near globally solutions to large optimisation problems [Kirkpatrick *et al.* (1983)]. [Metropolis *et al.*, (1953)] first introduced this principle into numerical minimisation. The method itself has a direct analogy with thermodynamics. In a fluid, atoms are moving freely, whereas in a solid atoms are ordered. In the intermediate phase, when a fluid solidifies, the number of options for ordering available to the atoms decreases. If freezing occurs quickly, atoms can get stuck in a sub-optimal ordering. If freezing occurs slowly and the transition phase takes longer, the atoms have more time to end up in the optimal configuration. The temperature of the system, also referred to as specific heat, determines the possible rate of change in a thermodynamic system.
An optimisation process can also be designed as a process of annealing. In the beginning of the process, many different measures can be evaluated and the search area is explored wildly. After a while, promising results have been obtained, which need to be explored a little further or, in other words, fine-tuning is required. By decreasing the 'temperature' of the system, the freedom to explore the search area decreases. Figure 5.4 shows, analogue to figure 5.3, a pseudo code illustrating the general structure of a simulated annealing process.

```
Procedure Simulated Annealing
Begin
T:= 0
  Initialise set of possible solutions (t)
  Evaluate objective function S (t)
  While criterion not satisfied do:
    Begin
      t:= t+1
      Mutation of solutions (t)
      Evaluate objective function S (t)
          Accept or reject new set of solutions , depending on probability function of equation 5.2
    End
End
```

Figure 5.4 Pseudo code for simulated annealing.

The first steps in the procedure for simulated annealing are similar to the ones for a genetic algorithm, differing only in the number of sets of possible solutions. After preparing and evaluating the first set of solutions, a new set of solutions is formed by mutating the original set randomly. This new set of solutions is evaluated and subsequently the acceptance or rejection of this new set of solutions takes place. This is the step in the simulated annealing procedure that has an analogy with annealing of a thermodynamic system.

A thermodynamical system may change its configuration from energy level ε_0 to energy level ε_1. This change only occurs if [Kirkpatrick *et al.* (1983)]:

- $\varepsilon_1 < \varepsilon_0$, in this case the new configuration has a lower energy state than the old one and this change will always be accepted.
- $\varepsilon_1 > \varepsilon_0$, then the new configuration has a higher energy state than the old one and is accepted with a certain probability. This probability is given by the Boltzmann distribution, $e^{-\beta(\varepsilon_1 - \varepsilon_0)}$, with $\beta = \frac{1}{kT}$, where k is Boltzmann's constant and T is temperature.

The Boltzmann distribution shows that the probability of accepting a worse solution (i.e. accepting a higher energy state) decreases with temperature. When applying simulated annealing to wastewater optimisation studies, this temperature can be represented by the iteration number. The probability of accepting a new solution is given in equation 5.2.

$$P_{acc} = \begin{cases} 1 & if\ S_{new} < S_{old} \\ e^{-(S_{new} - S_{old})\,\beta\, i} & if\ S_{new} \geq S_{old} \end{cases} \qquad \text{(eq. 5.2)}$$

where

S objective function value

β Boltzmann constant

i iteration number

The general scheme of always accepting improvements while sometimes accepting worse solutions with decreasing probability is known as the Metropolis Algorithm. The interesting feature of this scheme is the possibility of escaping from a local optimum, even at the end of an optimisation and the fact that simulated annealing is able to deal with multiple, possibly conflicting constraints [Kirkpatrick *et al.* (1983)].

Like the genetic algorithm, simulated annealing can be tailored to a specific optimisation problem, although simulated annealing offers only two operators for tuning the algorithm:

- the rate of mutation and
- the Boltzmann's constant.

5.4 Implementation of GA and SA in wastewater system optimisation

The applicability of both algorithms for wastewater system optimisation has been tested in a simplified version of an existing wastewater system optimisation study [Witteveen+Bos (2001)]. This simplified example of a volume based wastewater system optimisation study was selected to illustrate the performance of the algorithms, rather than to illustrate the impact of the dynamic interactions within wastewater systems on wastewater system performance. The latter is discussed in chapter 6.

5.4.1 Case description

The wastewater system consists of 8 small combined sewer systems, linked to a wwtp by pressure mains, see figure 5.5. The performance of the sewer system is judged by the annual CSO volume, which has to be equal to or less than the annual CSO volume of the 'reference' system, discussed in table 1.2. The performance of the wwtp is measured by the effluent standards, in this case COD (50 mg/l), N_{tot} (10 mg/l) and P-removal (1 mg/l). However, like in figure 5.1, in this example of a typical volume based wastewater system optimisation study, the wwtp serves as a boundary condition in terms of an hydraulic treatment capacity.

The incentive to start a wastewater system optimisation study was in this case non-compliance with the standards of the sewer system. The water board and the municipalities involved initiated a wastewater system optimisation study with the objective of complying with current standards at minimal provision costs.

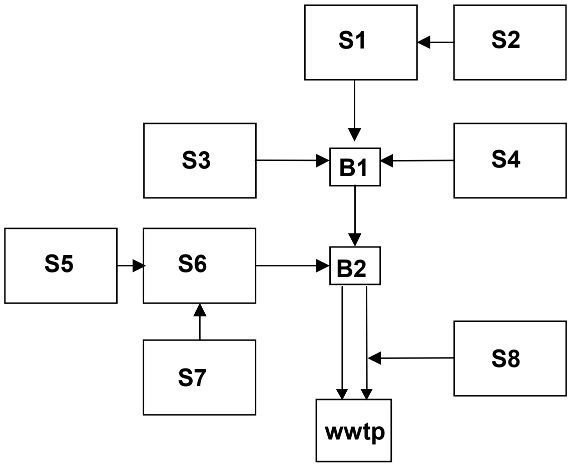


Figure 5.5 Lay out of the waste water system of combined sewer systems, interceptor sewers, booster pumps and WWTP. Table 5.1 lists the most important system characteristics.

Table 5.1 Wastewater system characteristics.

reference	Type	storage capacity (mm)	pump capacity (m^3/h)	connected surface area (ha)
S1	combined sewer	7.4	218	13.00
S2	combined sewer	9.2	14	1.32
S3	combined sewer	9.6	110	12.10
S4	combined sewer	13	55	1.95
S5	combined sewer	8.5	50	5.31
S6	combined sewer	8.8	236	7.47
S7	combined sewer	5.9	28	2.24
S8	combined sewer	6.6	68	4.44
B1	booster station	-	514	
B2	booster station	-	750	
WWTP	-	-	810	

A quick scan revealed the wwtp to perform well and identified 16 options for improvement of the sewer system, i.e. increasing the storage or pumping capacity for each of the 8 combined systems. As a result, a 16 dimensional combinatory problem has to be solved.

Modelling of wastewater system performance

Wastewater system performance is evaluated according to the approach given in figure 5.1. This implies modelling the CSO volumes and a static evaluation of wwtp performance. The model applied in this case was developed during the original optimisation study [Witteveen+Bos (2001)]. In this study, it was concluded that the interceptor sewers and booster stations are dominant factors with respect to overall system performance. Consequently, no detailed hydrodynamic model has been applied for each sewer system, but each sewer system is modelled in Hydroworks as a storage tank with a variable volume. This approach is valid, as the contributing catchments are relatively flat and therefore the error of simplifying the sewer system as a reservoir model will be acceptable [Korving (2004)].

The interceptor sewers are described by their head-discharge relationships. The wwtp is not modelled, as the original optimisation study [Witteveen+Bos (2001)] revealed that with respect to the wwtp no problems were to be expected. The wwtp is included in the optimisation study by assuming that as soon as the hydraulic design loading exceeds the hydraulic capacity of the secondary clarifiers, an additional secondary clarifier will be necessary.

5.4.2 Objective function and constraints

The definition of the objective function is a crucial step in the optimisation procedure. The performance of the wastewater system is introduced in the optimisation procedure as a constraint, as in this case the responsible authorities requested full compliance with the standards. Introducing wastewater system performance in the objective function, by using a penalty function or costing of environmental damage, was therefore in this case not possible, as this could result in a solution which does not fully comply with the standards, but is a trade-off between provision and penalty costs.

Consequently, the objective function consisted of cost functions only. This is of course a rather simple approach, which, however, gives insight in the potential of the studied optimisation algorithms. As the authorities were only interested in provision costs, the cost function given by equation 5.3 was used:

$$S_{(\cos ts)} = \sum_{i=1}^{n} \alpha_i (\Delta V_i) + \sum_{j=1}^{m} \alpha_j (\Delta P_j) + \sum_{k=1}^{l} \alpha_k (\Delta Cap_{Int,k}) + \alpha_{wwtp} \Delta Cap_{WWTP} \qquad \text{(eq. 5.3)}$$

where

α_x	specific cost function
V	storage basin volume
P	pump capacity
Cap_{Int}	capacity interceptor sewer (pressure main)
Cap_{wwtp}	hydraulic capacity wwtp

The cost functions α_x are of major importance for the optimisation process. Figure 5.6 shows the general cost functions used. The cost functions are designed to represent approximate values of provision costs, including scale effects. The values of the costs are based on general figures given by [Stichting RIONED (1997)], except for the costs of increasing the capacity of pressure mains. The latter costs are taken into account as a fixed price as soon as the current design capacity is exceeded. These costs were derived from the original optimisation study [Witteveen+Bos (2001)].

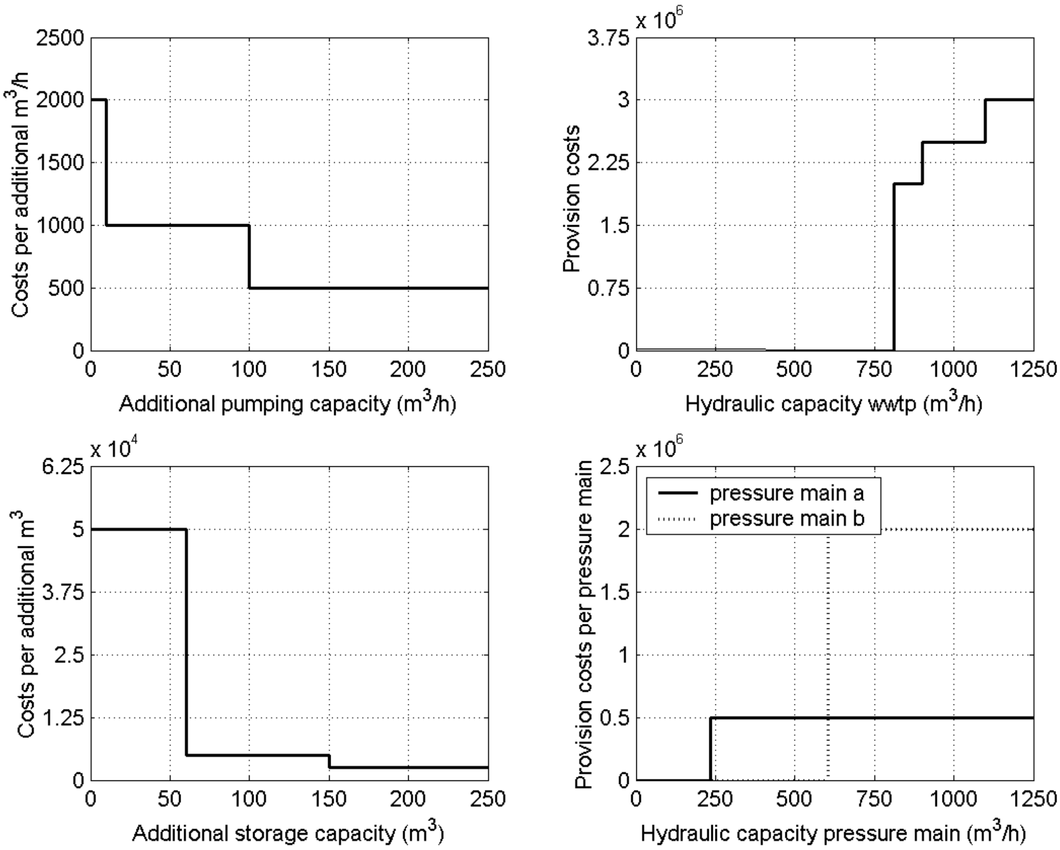


Figure 5.6 Cost functions (costs in Euro) used in the wastewater system optimisation study. The costs for building small (< 60 m^3) storage volumes have been made extraordinarily high to prevent the optimisation routine from resulting in impractical solutions. An impractical, but numerically valid, solution could be to install storage tanks of less than 60 m^3 at each CSO. Pressure mains have to be replaced as soon as the hydraulic capacity exceeds the design capacity. It is assumed that the provision costs of a pressure main depend on site specific conditions, rather than on the new capacity to be installed [Witteveen+Bos (2001)].

In the optimisation routine, at first the value of the objective (cost) function is assessed. Secondly, wastewater performance is assessed by introducing the simulated overflow volume as a constraint. This constraint is implemented in the optimisation procedure by accepting each solution that complies with the standards and also accepting solutions that do not comply with a certain probability. This probability, calculated by $e^{-cv*\alpha}$, with cv as the level of constraint violation, decreases for an increasing constraint violation. The factor α determines how strict the constraint is imposed. This type of constraint implementation may help the optimisation routine explore solutions rather close to the required performance level.

Applied software
The hydrodynamic model Hydroworks™ vs. 6.0 has been used to calculate influent flows at the WWTP as well as overflow volumes. Furthermore, Compaq Visual Fortran, version 6.1, has been used to program the optimisation algorithms and the Macro Scheduler of MJT net Ltd for running these applications.

5.4.3 Optimisation results

Figure 5.7 gives the results of the use of a genetic algorithm to solve the 16 dimensional optimisation problem. The main characteristics of the genetic algorithm, the population size and the mutation rate, have been adjusted to try to tune the genetic algorithm to the specific optimisation problem. The results given in figure 5.7 illustrate that the characteristics of the genetic algorithm do not dominantly affect the solution to the optimisation problem after 1000 function evaluations, as all solutions found have the same order of magnitude. Moreover, the settings of the algorithm do not seem to have a dominating impact on the convergence.

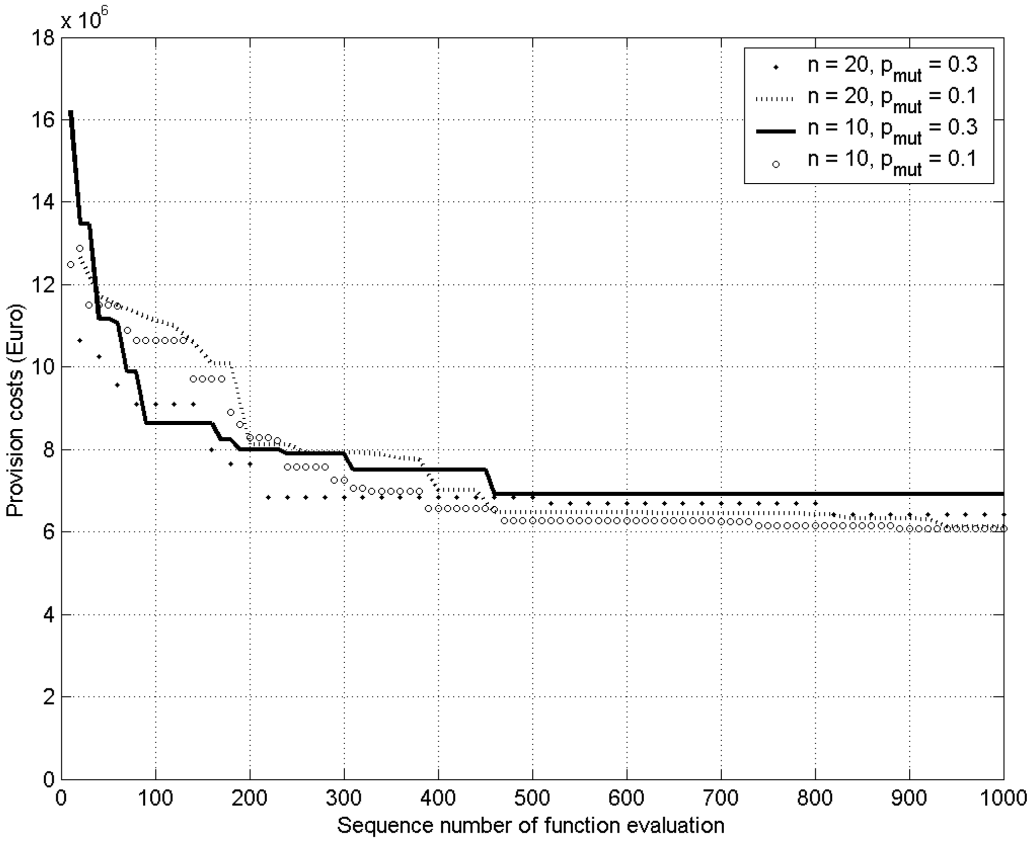


Figure 5.7 Optimisation results genetic algorithm. The algorithm is able to find cheaper solutions complying with the standards by combining the available measures (additional storage capacity or adjustment of the pumping capacity) at the various locations. The main cost reductions are in this case achieved by building relatively large storage tanks (larger than 60 m^3, as the unit price decreases with the storage capacity build) and minimising additional pumping capacity. Especially the latter aspect is important in this case with respect to cost reductions, as increasing the hydraulic capacity of the wwtp or the pressure mains is relatively expensive, see figure 5.6.

Figure 5.8 shows the results for the optimisation using simulated annealing. In this case, both the optimisation process and the final optimisation result depends on the applied Boltzmann constant. Apparently, for simulated annealing, the performance of the algorithm depends more on the settings of the algorithm than for a genetic algorithm. This result is in accordance with literature. According to [Kalivas (1992)], simulated annealing performs best when approximately 80% of the uphill moves, i.e. more expensive solutions, are accepted. From the statistics of 4000 function evaluations for the first optimisation with a β value of 10^{-8}, it appears that in this particular optimisation process only 36% of the uphill moves were accepted. Based on the results from the optimisation with a β value of 10^{-8} it has been deduced that a β value of 10^{-9} should suffice. An optimisation using a β value of 10^{-9} shows that this β value of 10^{-9} indeed generated lower cost solutions. This simulated annealing results are comparable with the genetic algorithm optimisation results with a population size of 20 and a probability of mutation of 0.3, as shown in figure 5.9.

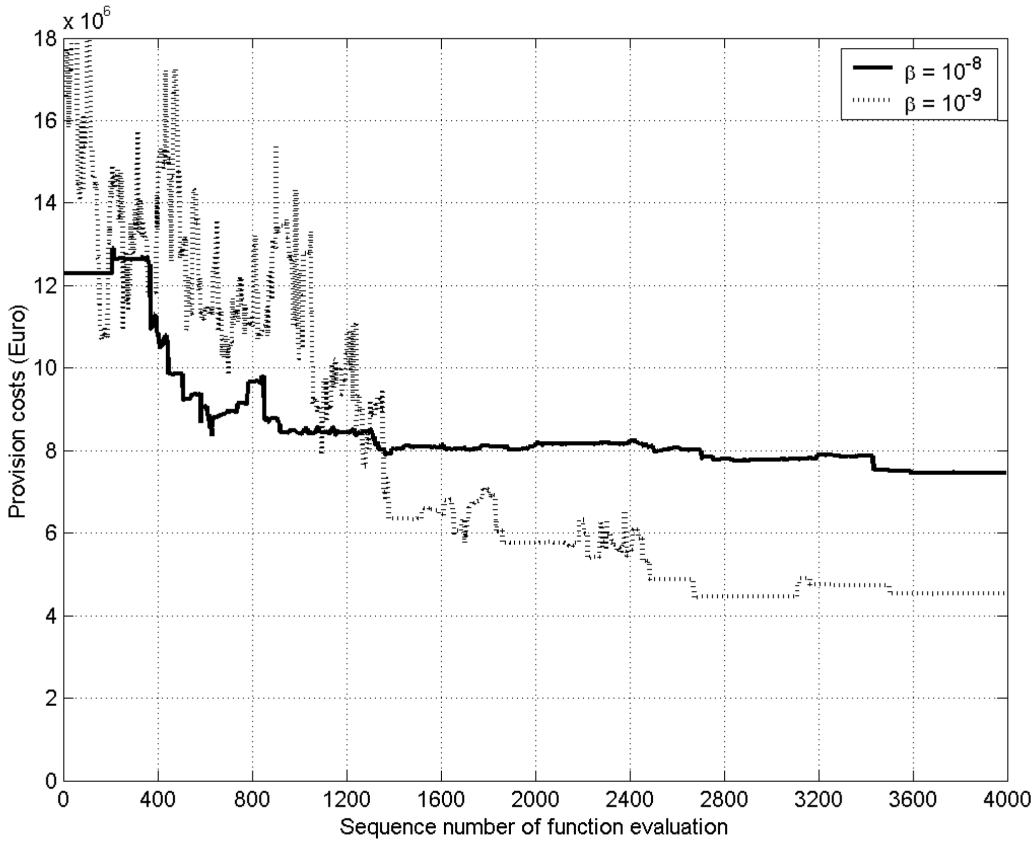


Figure 5.8 Optimisation results simulated annealing.

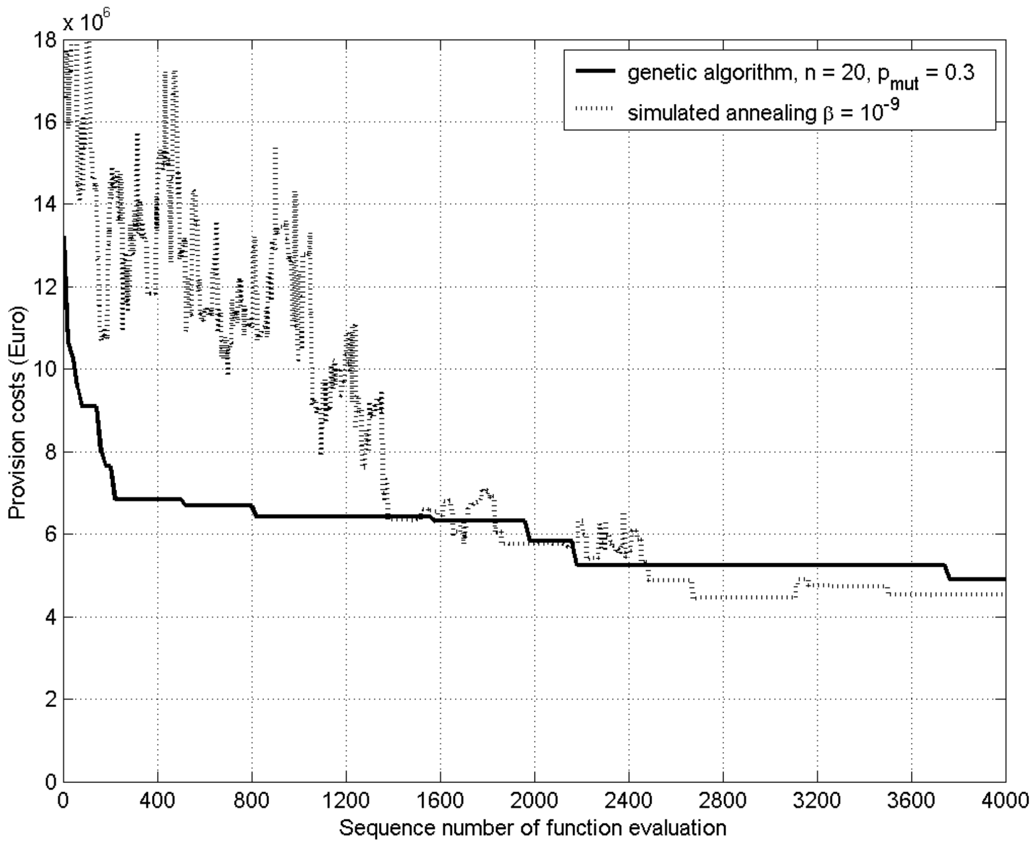


Figure 5.9 Comparison of optimisation results using a genetic algorithm and simulated annealing.

Adjusting a genetic algorithm and simulated annealing to a specific optimisation problem is inevitably a trial and error process, according to the ‘No free lunch’ theorem [Wolpert and Macready (1996)]. This theorem implies that it is theoretically impossible to determine in advance what the optimal settings will be of a genetic algorithm or simulated annealing. Especially the simulated annealing results show that a balance has to be found between a too rapid convergence and a too wide exploration of the optimisation problem.
Another general characteristic of the use of heuristic algorithms is the large number of function evaluations necessary. This limits the practical application of these algorithms to optimisations where the function evaluations can be performed fast. The application of simplified models, such as SIMPOL [FWR (1998)], illustrated by [Gill *et al.* (2001) and Savic and Walters (2002)] is a possible solution to this practical problem. Optimisation studies where the standards require detailed modelling and model results in the form of amplitude/duration/frequency relationships are at the moment computationally too demanding. Of course, current practical problems with simulation times will not hamper the application of optimisation algorithms like a genetic algorithm or simulated annealing in the long run.

Sensitivity of optimisation results to the quality of model results
In this example the wastewater system had to comply fully with CSO discharge standards, as current Dutch practice demands full compliance. In this case the standard was the annual CSO volume, which was calculated using an uncalibrated simplified sewer model. Although this approach is common in the Netherlands, the quality of the simulation results in terms of accuracy and reliability can be questioned [Clemens (2001a)]. Moreover, the analysis of the quality of the simulation results of a fully calibrated hydrodynamic model in section 4.2 shows that even a fully calibrated model does not exactly represent the measurements. A certain deviation will always exist and, consequently, model results should not be seen as hard figures.
The uncertainty of model results could be introduced in the optimisation procedure in a number of ways. Most straightforward is accepting a certain uncertainty interval around the model results. Figure 5.10 shows the impact on the optimisation result of accepting an uncertainty interval of the model results of 5 % and 10 %.

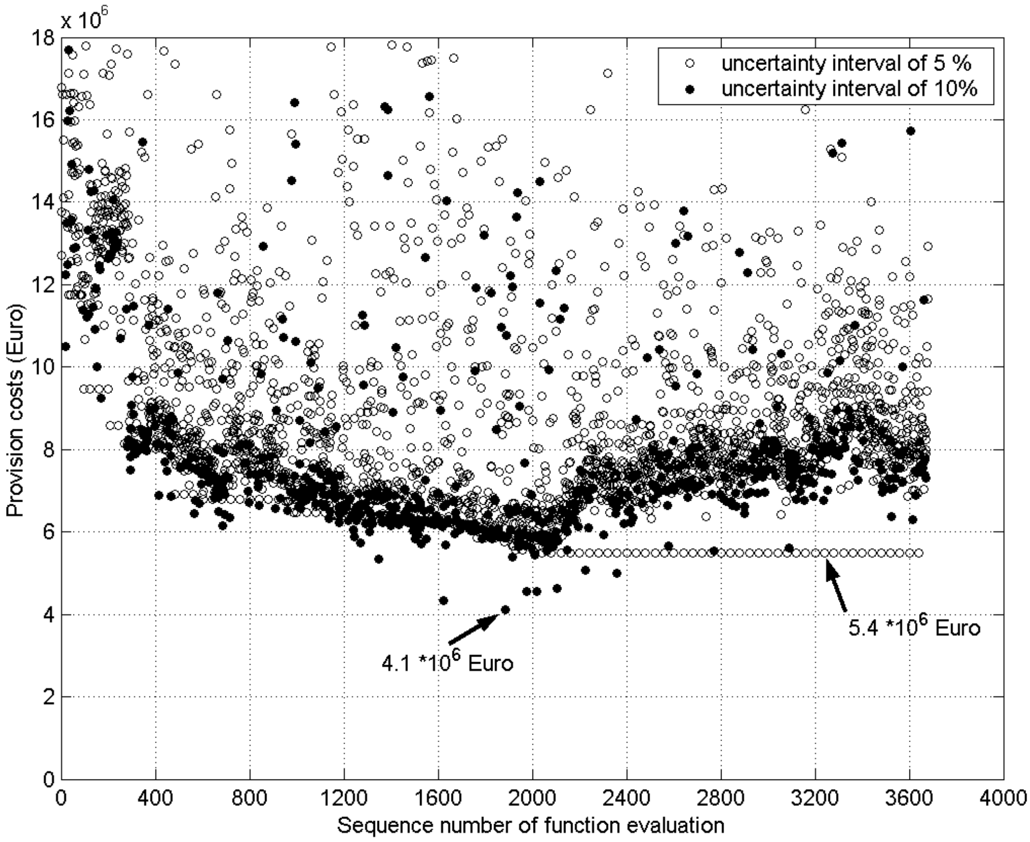


Figure 5.10 Results of optimisation with a genetic algorithm (n = 20, p_{mut} = 0.3) for an accepted model accuracy of 5% and 10%.

Allowing an uncertainty interval of 5% resulted in a solution with a total provision cost of 5.4 million Euro. Accepting an uncertainty interval of 10% resulted in a solution costing only 4.1 million Euro, a cost reduction of 24% (1.3 million Euro). This result illustrates that accepting an uncertainty interval of 10% (which is to be considered minor given the uncertainties related to CSO discharge modelling [Korving (2004)] results in this example in a major saving in provision costs. Therefore, it is concluded that optimisation results, irrespective of the optimisation method, should be subjected to a sensitivity analysis to assess the robustness of the final solution found to uncertainties associated with modelling and costing. In this respect, it could be worthwhile to analyse whether modelling or costing contributes most to the associated uncertainties.

5.5 Conclusion

Wastewater system optimisation studies can be categorised as multidimensional combinatorial problems with multiple objectives and possibly conflicting constraints. In order to enhance wastewater system optimisation, two heuristic methods have been tested with respect to their applicability to wastewater system optimisation. Both a genetic algorithm and simulated annealing proved to be capable of dealing with a typical wastewater system optimisation problem.

The case study illustrates that using a genetic algorithm or simulated annealing results in a large number of simulations. The results of these simulations contain a lot of information on the sensitivity of the perceived optimal solution to e.g. the imposed standards. As such, performing a wastewater system optimisation study results in a better understanding of the performance of the wastewater system, which in itself may help improving wastewater system performance in daily engineering practice.

Chapter 6 Wastewater system performance and interactions

6.1 Introduction

The main focus of this thesis is the sensitivity of wwtp performance to fluctuations in the influent under transient conditions and the possibilities of predicting these influent fluctuations with current sewer models.
Nitrogen removal at the wwtp, especially nitrification, proved to be very sensitive to fluctuations in the nitrogen load in the influent. The nitrogen in the influent is associated with the dissolved and fine suspended wastewater fractions [Nieuwenhuijzen, van (2002)], see also table 2.10 and origins mainly from the wastewater rather than from runoff or in-sewer sediment stocks, see table 2.13. Consequently, a sewer model describing the transport of solutes properly is sufficient to enable the study of the effect of storm events on wwtp performance. Chapter 4 shows that current sewer models are capable of meeting the requirements for sewer process modelling with respect to solute transport.
As a result, optimisation of wastewater systems taking into account the interactions within wastewater systems seems feasible.
This chapter discusses the potential of optimisation of wastewater systems, taking into account the knowledge of the interactions within wastewater systems. In addition, the relation between wastewater system characteristics and the influent pollutograph under transient conditions is discussed.
Furthermore, a semi-hypothetical case study illustrates that the dynamic interactions within wastewater systems have different consequences for different parameters, such as ammonium, total nitrogen and COD. Optimising a wastewater system with respect to the total COD or ammonium load discharged to the receiving waters renders completely different results. The role of the interactions with respect to total wastewater system performance is discussed for a number of storm events and wastewater system characteristics.

6.2 Impact of sewer and wastewater transport system characteristics

In chapter 4 the processes in the sewer system affecting the influent pollutograph were discussed, with the hydrodynamics and the transport of solutes as the key processes with respect to the interactions within wastewater systems.
The impact of these processes on the influent pollutograph is to a large extent determined by the characteristics of the contributing sewer and wastewater transport systems. Main characteristics of sewer systems are the interceptor or pumping capacity and the (dynamic) in-sewer storage. Wastewater transport systems can be pressure or gravity mains, although in the Netherlands they are mostly pressurised due to the low available natural gradients. Pressure mains are normally characterised by their dry weather hydraulic retention time.

The sewer system characteristics determine to what extent wastewater is present in the sewer system at the onset of a storm event and the mixing of this wastewater with the runoff. Wastewater can especially be present in ‘lost storage’ or pressure mains. ‘Lost storage’ is defined by [Clemens (2001a)] as the amount of volume held by a given drainage system that cannot be emptied either by gravitationally induced discharge or by pumping stations. ‘Lost storage’ can be due to subsidence of sewers or water level control devices maintaining a certain sewage level in the sewer system, as exemplified in figure 6.1. As such, sewer system operation and maintenance can exert a strong influence on the pollution potential of sewer systems [Korving (2004)].

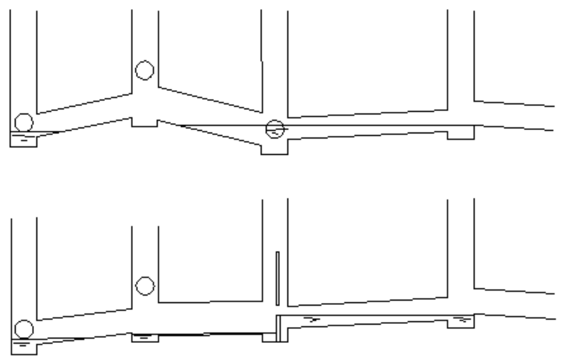
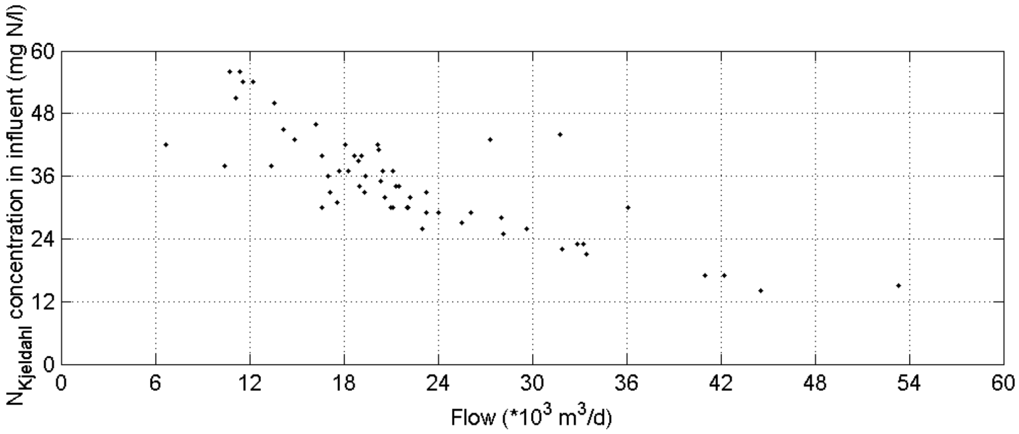


Figure 6.1 'Lost storage' in sewer systems due to subsidence (top) and control structures (bottom) (reproduced with permission from [Clemens (2001a)]).

During the last decades large-scale, centralised wwtps serving large urban areas were increasingly applied in the Netherlands [Graaf, van der (1992)]. As a result, extensive networks of wastewater transport systems have been installed to convey the wastewater to centralised wwtps. Especially pressure mains with a long hydraulic retention time can exert a significant impact on the loads arriving at a wwtp [Graaf, van der (1992)]. This impact is attributable to the volume of wastewater stored in these pressure mains, which arrives during storm events at the wwtp with wwf flow rate while still having a dwf concentration. Consequently, large peaks in the influent load can occur.

The impact of wastewater stored in sewer systems or pressure mains on the influent pollutant loads was analysed for three wastewater systems. Theoretically, the daily nitrogen load in the wwtp influent of a wastewater system without large in-sewer stocks of dwf is independent from the total flow during storm events. This is due to the fact that storm runoff normally contains a low concentration of nitrogen, see table 2.4.

Figure 6.2 shows the Kjeldahl nitrogen concentration and load of the wwtp influent versus the influent flow for wastewater system 'Schalkwijk', situated in Haarlem. This wastewater system has no pressure mains and an insignificant amount of 'lost storage' [Herbergs (2001)]. The upper graph shows that, apart from a limited number of outliers, the Kjeldahl nitrogen concentration decreases with the influent flow. The lower graph illustrates that the influent load of Kjeldahl nitrogen does not depend on the influent flow, as also during storm runoff the influent load stays within the normal dwf range. The outliers may be due to the operation of pumping stations in the contributing sewer system or to sampling or measurement errors.



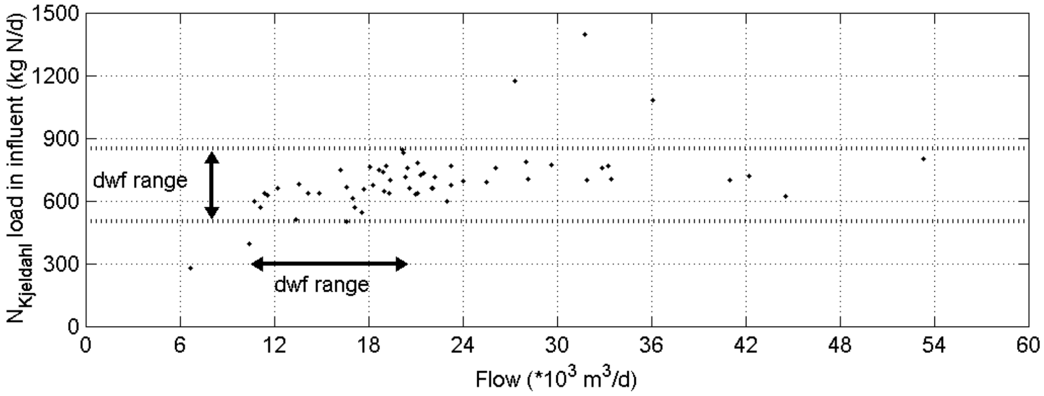


Figure 6.2 Influent flow vs. $N_{Kjeldahl}$ concentration (upper graph) and $N_{Kjeldahl}$ load (lower graph) for wwtp Schalkwijk, Haarlem, 1999. [Herbergs (2001)]. The 'dots' show measured data. The $N_{Kjeldahl}$ concentration decreases with the influent flow and the $N_{Kjeldahl}$ load stays within the dwf range during storm events. This indicates that dilution is the dominant process related to $N_{Kjeldahl}$ during a storm event and that in the sewer system of Haarlem during dwf no large stocks of wastewater are available in e.g. lost storage.

Figure 6.3 shows the daily ammonium concentration and load plotted against the influent flow for wwtp Katwoude. During wwf, the decrease of the ammonium concentration is not proportional with the flow. This is illustrated especially by the lower graph of figure 6.3, showing that the loads arriving at the wwtp during storm events exceed the normal dwf range in ammonium load in the influent. This can only partly be explained by the ammonium load stored in the pressure mains, as the maximum load to be expected is the value based on the dwf range plus the volume stored in the pressure main, as indicated in figure 6.3. Apparently, the sewer systems discharging to wwtp Katwoude can easily contain up to 80 kg ammonium, which is, with a mean dwf concentration of 40 mg ammonium N/l, equivalent to a volume of 2,000 m^3. This volume has the same order of magnitude as the total volume of 1,926 m^3 of the pressure mains [Stok (2003)]. This volume can be stored in lost storage, but may also be due to the operation of the pumping stations [Korving (2004)].

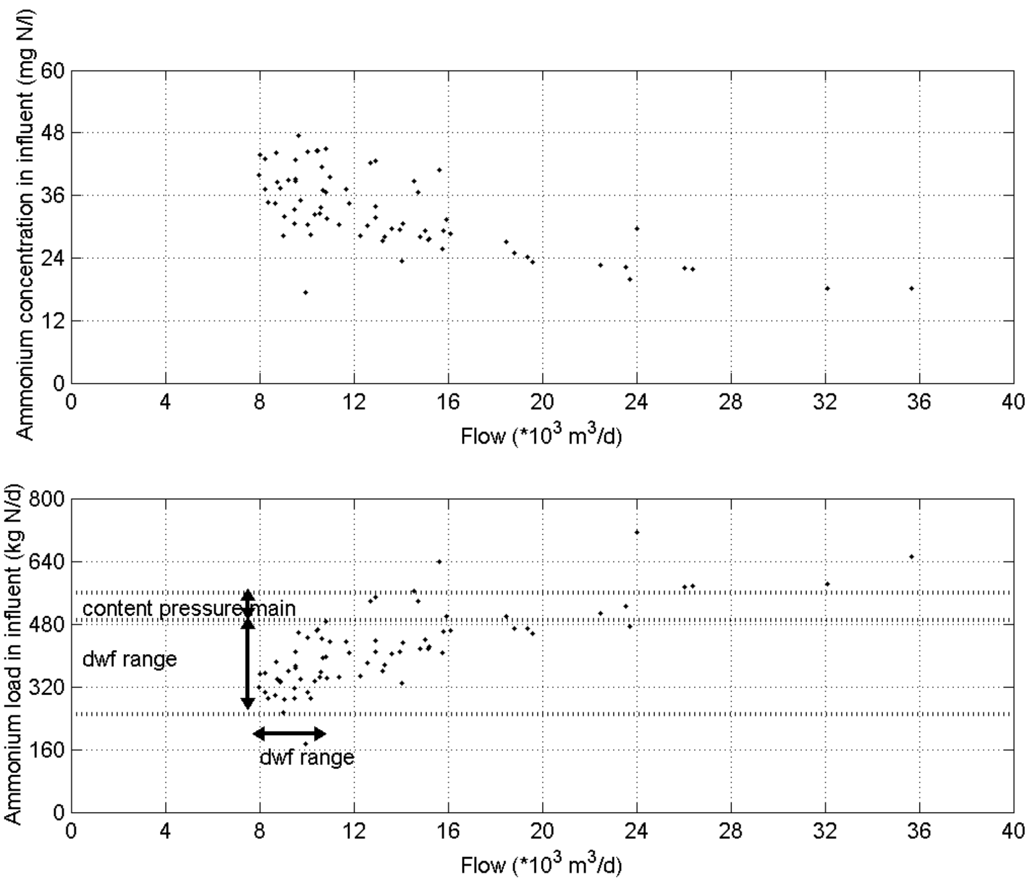


Figure 6.3 Influent flow vs. ammonium concentration (upper graph) and ammonium load (lower graph) for wwtp Katwoude, September – December 2002 [Stok (2003)]. The ammonium concentration decreases with the influent flow, although not proportional as the influent load during wwf exceeds the dwf range. This exceedance can partly be explained by the volume of wastewater stored in the pressure mains, as indicated in the lower graph. Data points exceeding the dwf range plus the load contained in the pressure main can either be attributed to lost storage in the contributing sewer systems, releasing an additional load of ammonium during storm events or to measurement errors.

In addition, figure 6.4 shows the dwf influent profile for wwtp Katwoude. The morning peak in flow and ammonium concentration, which normally arrive at the same time [Krebs et al. (1999)], have been separated in time by the pressure main. Consequently, the peak in ammonium load arriving at the wwtp occurs almost 3 hours later than the peak in influent flow. This phenomenon is important when optimising wwtp performance under dwf conditions. I.e., at wwtp 'Dokhaven' in Rotterdam, the daily dwf peak load exceeds the nitrification capacity during a few hours. One of the options to improve the performance of wwtp 'Dokhaven' is to buffer the peak load. In this case, it is important to know whether the peak in influent flow coincides with the peak in influent load, as the former would imply the application of volume based RTC and the latter of water quality based RTC.

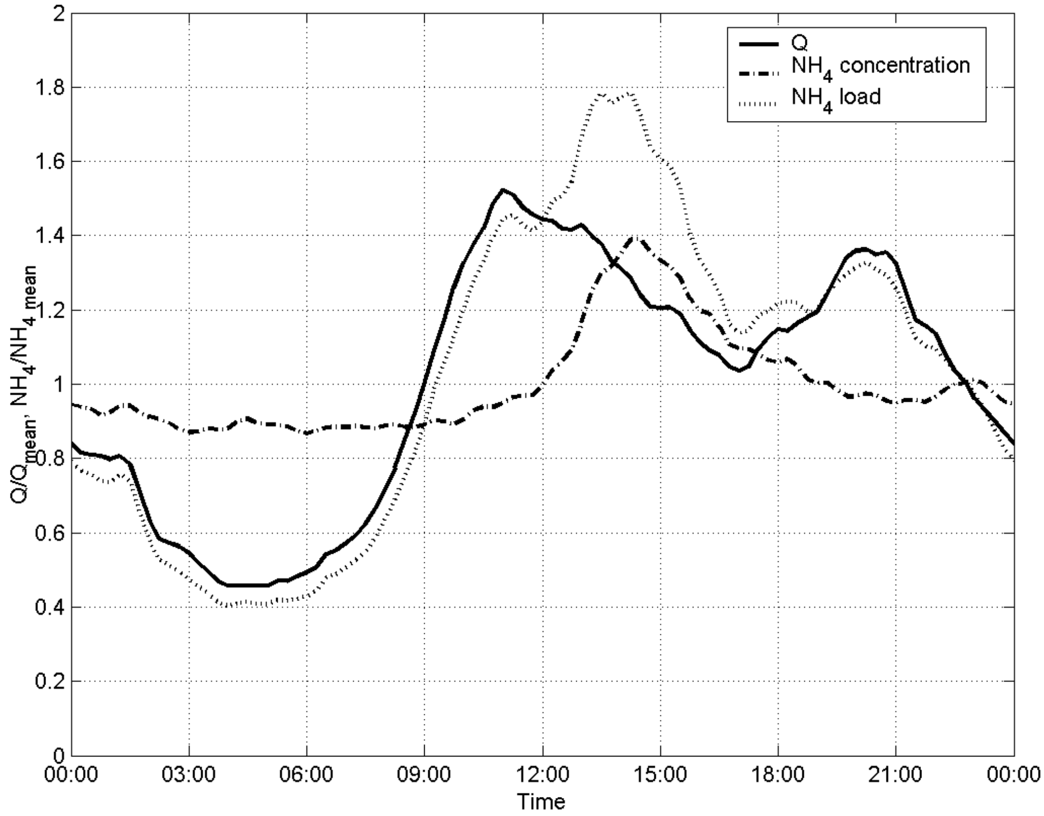


Figure 6.4 Dwf profile for Katwoude: influent flow, ammonium concentration and ammonium load. The peak in ammonium load arrives almost 3 hours later than the peak in the influent flow due to the transport through the pressure mains.

The impact of the pressure mains in wastewater system 'Katwoude' can also be observed during wet weather periods. Figure 6.5 shows the influent flow, ammonium concentration and ammonium load for the storm event of 25 and 26 November 2002. After the onset of the storm event the flow increases to wwf level, whereas the influent ammonium concentration remains at dwf level for a few hours, causing a peak in the influent load of ammonium around 20:00 hour on 25 November 2002.

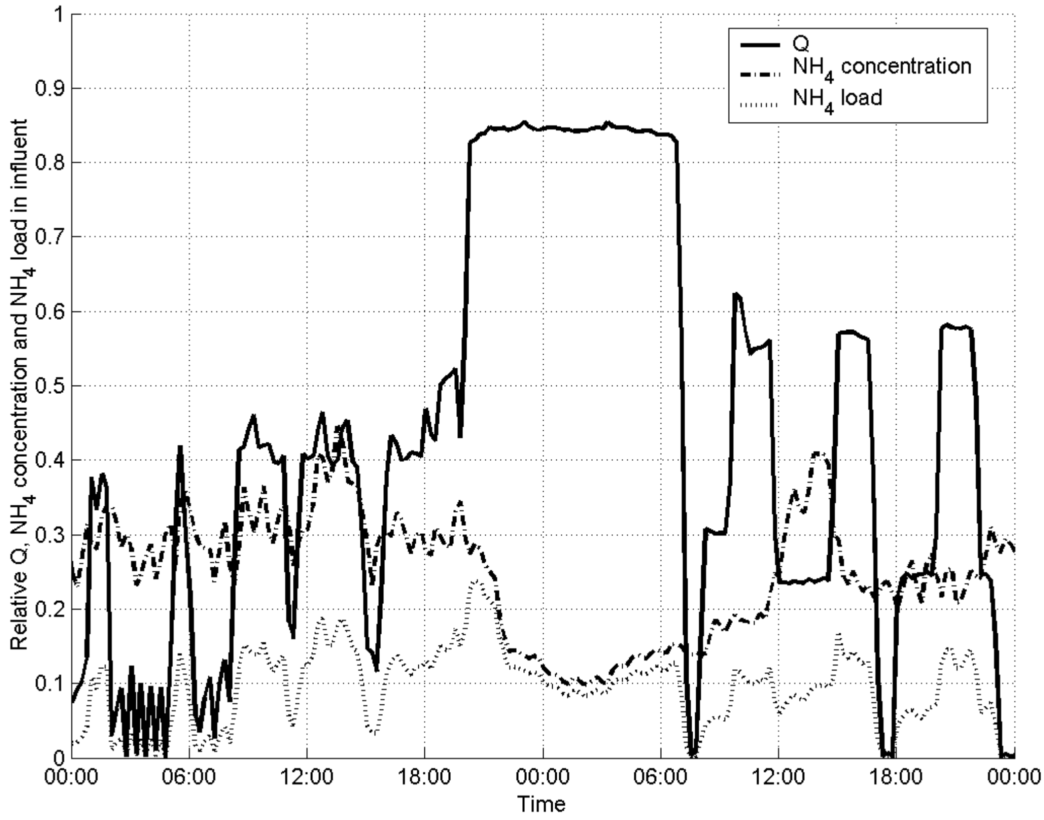


Figure 6.5 Typical wwf profile for Katwoude: influent flow, ammonium concentration and ammonium load. The pressure mains cause a noticeable peak in the influent load of ammonium. The NH_4 concentration is normalised by dividing by 100 mg N/l and the flow by dividing by the installed pumping capacity. The plotted NH_4 load in the graph is the normalised flow times the normalised NH_4 concentration.

Figure 6.6 shows the daily Kjeldahl nitrogen concentration (upper graph) and load (lower graph) for wwtp Zaandam-Oost. The sewer system of Zaandam-Oost has been equipped with internal weirs to maintain a relatively high water table in the sewer system during dwf in order to prevent groundwater infiltration. In addition, a number of pressure mains discharge to the wwtp. As a result, the daily Kjeldahl nitrogen load during wwf reaches values up to 2.5 times the average Kjeldahl nitrogen load under dwf conditions, which may cause the effluent quality of the wwtp to deteriorate. Consequently, it should be examined whether the overall effect of the internal weirs on the total wwtp effluent load is still positive, given the large fluctuations in the influent. A detailed discussion of this particular case, however, is considered to be beyond the scope of this thesis.

Nonetheless, the examples given in this section illustrate the impact of the sewer system characteristics, such as the size of pressure mains or the amount of dwf volume stored in a sewer system at the onset of a storm event.

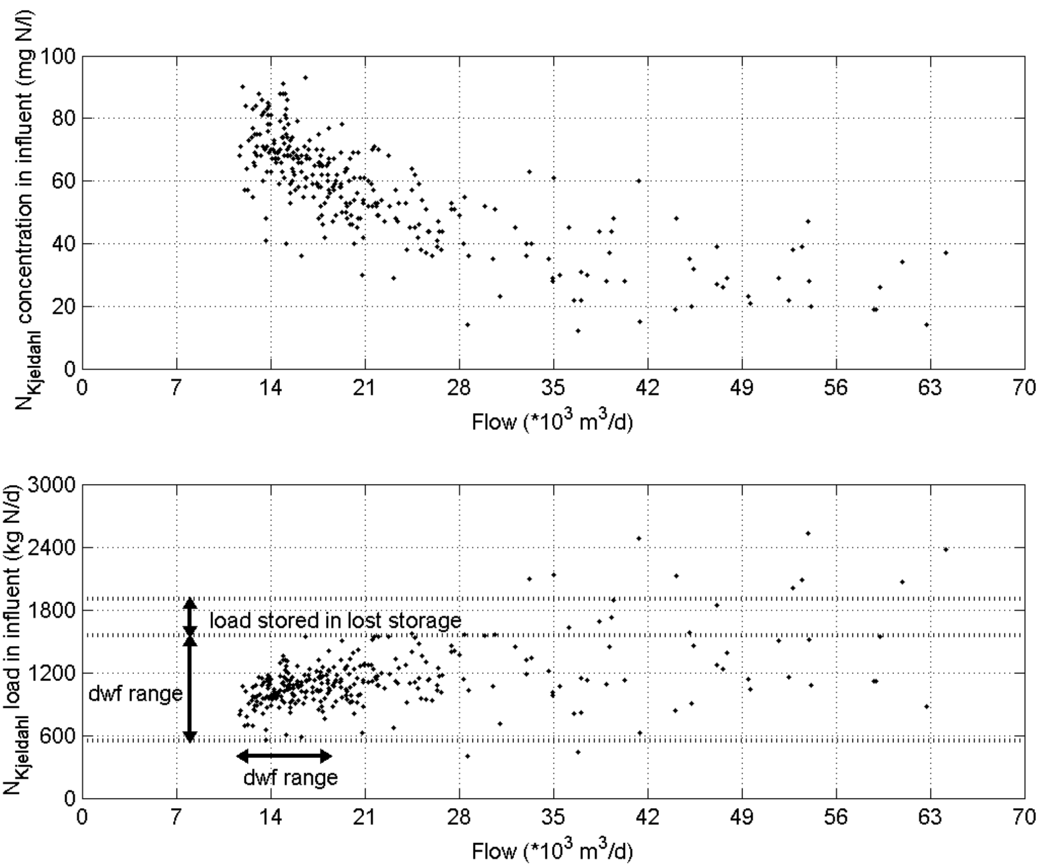


Figure 6.6 Influent flow vs. $N_{Kjeldahl}$ concentration (top) and load (bottom) for wwtp Zaandam, 1997-2000 [Herbergs (2001)]. The data used are based on the routine sampling programme at the wwtp, giving 24 hour flow proportional values. The concentration of $N_{Kjeldahl}$ shows a large fluctuation. Nevertheless, a trend of decreasing concentrations with increasing flow can be observed. The $N_{Kjeldahl}$ load in the influent shows a large fluctuation during days with wwf, reaching values from 500 to 2500 kg $N_{Kjeldahl}$. This large fluctuation is partly due to a volume of wastewater stored in the sewer system of Zaamdam due to internal weirs. The volume of lost storage is, according to available data, approximately 0.7 mm [Herbergs (2001)], which is equivalent to 30% of the daily dwf volume with 200 inhabitants per hectare and 120 litre per person and day. Consequently, this volume possibly explains a small proportion of the data with an influent load exceeding the dwf range. The influent loads which cannot be explained by the known lost storage are probably due to an unknown volume of lost storage, the volume of wastewater kept in upstream pressure mains or operational aspects.

6.3 Case study

This case study discusses the importance of sewer system characteristics on wastewater system performance, taking the interactions within wastewater systems into account. Wastewater system performance is assessed by the following parameters:

- total overflow volume, representing a volume based approach to sewer system performance, like adopted in the Netherlands since 1992, see table 1.2.
- Ammonium, a parameter often applied to analyse overall wastewater system performance as it is an indicator for acute receiving water quality problems [Jack (1999)].
- Total nitrogen, used e.g. by [Harremoës and Rauch (1996)] to assess accumulative pollution from the wastewater system.
- COD, which is a frequently used parameter. This parameter was the first parameter discussed in relation with the total pollutant discharge from the wastewater system to

the receiving waters by the early 1990s German 'Gesammtemmissionsgruppe' [Durchschlag et al. (1991); (1992)]. COD, however, has the disadvantage of being a general parameter with no direct relation with the receiving water quality. In addition, COD discharged via a CSO has completely different characteristics than COD discharged in wwtp effluent [Sakrabani et al. (2004); Servais et al. (1999)]. As COD_{total} has no direct relation with the receiving water quality, in this case only the biodegradable COD is taken into account in order to be able to compare the loads discharged by the wwtp and the CSO.

6.3.1 Materials and methods

The wastewater system used in this case study is semi-hypothetical as no 'real' wastewater system was available. The wastewater system is based on the combination of the sewer system of Loenen, described in chapter 4, with wwtp Katwoude, described in chapter 3, both discharging at the same location in the receiving waters. The receiving waters were not modelled, as this is considered to be beyond the scope of the thesis.

Modelling approach

The wastewater system was modelled using the available Hydroworks model for Loenen and the available SIMBA model for Katwoude sequentially, as shown in figure 6.7. Consequently, the dynamic system performance of both the sewer system and the wwtp under transient conditions could be accounted for.

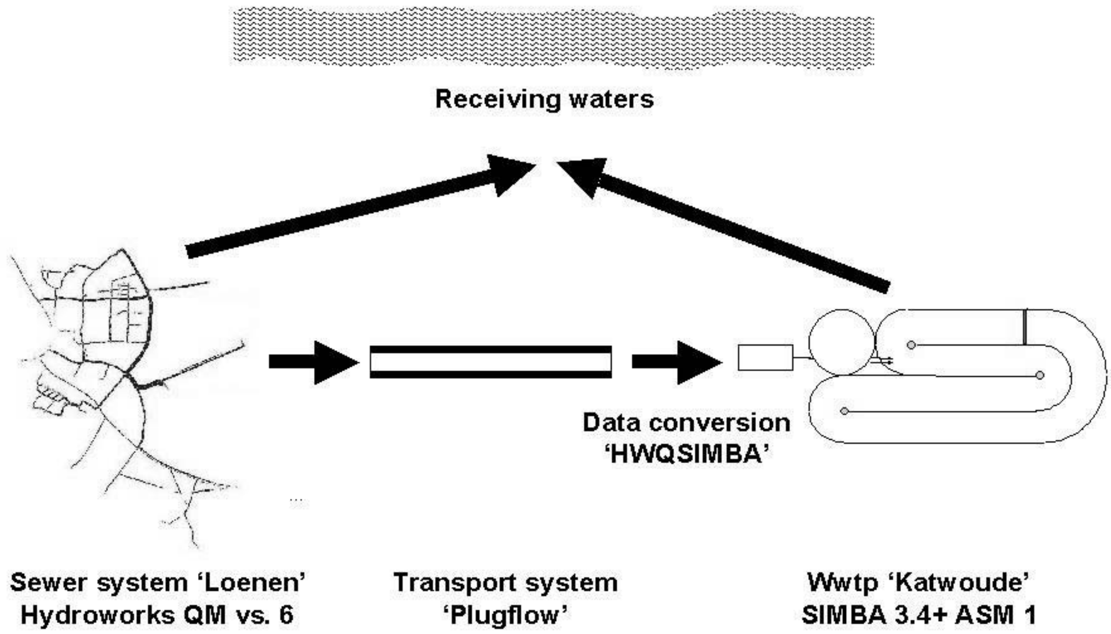


Figure 6.7 Semi-hypothetical urban water system. The program HWQSIMBA was developed together with Marcel Boomgaard and the program plugflow was developed during the research project.

The Hydroworks model for Loenen was used to calculate the flows and the dilution rate. In this semi-hypothetical case study, the sewer system of Loenen was either discharged directly to the wwtp or via a pressure main. The transport through the optional pressure main was modelled by the developed program Plugflow. No transformations are modelled in the pressure main. SIMBA input files were generated by the program HWQSIMBA, a conversion program developed in cooperation with Marcel Boomgaard. HWQSIMBA transforms the Hydroworks output files into SIMBA input files by upscaling the flow from the 6,500 p.e. Loenen to the 86,300 p.e. Katwoude and by applying the dilution rate to the average dwf wastewater characteristics, given in table 3.6. In addition, the program HWQSIMBA was used to calculate the concentration and fractionation of the wastewater discharged via the CSO, again by applying the simulated dilution rate at the location of the CSO to the average dwf wastewater characteristics. In both cases, the fractionation of the wastewater was kept constant as chapter 3 proved this to be a realistic assumption. Consequently, it is possible to compare the pollutant load discharged by the wwtp effluent and the CSO, as the same ASM1 variables are used. The implications of simplifying sewer system performance by only taking the hydrodynamics and the transport of solutes into account are discussed in the sections discussing the results of this case study.

Sewer and transport system characteristics
The impact of sewer and transport system characteristics on wastewater system performance was evaluated by simulating the wastewater system for a number of wastewater system configurations. The following properties were adjusted, as shown in table 6.1:

- interceptor capacity (as most Dutch sewer systems contain pumping stations, the interceptor capacity is sometimes also referred to as pumping capacity)
- volume of a storm water settling tank
- hydraulic dwf retention time of the pressure main

Table 6.1 Wastewater system properties.

Parameter	Value
Additional storage	0 and 6 mm[a]
Interceptor capacity	0.5, 0.6 and 0.7 mm/h
Pressure main dwf retention time	0, 6 and 24 h

[a] The value of 6 mm was derived from a study performed by Witteveen+Bos [Henckens (2004)], aiming at meeting the volume based requirements of the ‘reference system’, given in table 1.2.

Storm events
In this case study, it is the objective to illustrate the importance of the interactions within wastewater systems on wastewater system performance. As simulating rain series was practically not possible due to the required simulation times, two storm events with a different return period were used. Storm event A has a return period of 1 year^{-1} and event B a return period of 0.1 year^{-1}. Figure 6.8 gives the cumulative rainfall for both events. Storm event A represents storm events causing a significant CSO event. Storm event B represents storm events not causing a CSO event, but causing a substantial increase in the volume of wastewater to be dealt with at the wwtp. As shown in table 4.4, the latter category comprises a large number of storm events (on average 54 annually) compared with the on average 10 storm events per year causing a CSO event. Therefore, this category of storm events is considered relevant with respect to overall wastewater system performance.
By selecting two storm events only, the impact of sequential storm events is not addressed specifically, as suggested e.g. by [Jack (1999)]. Nonetheless, section 6.3.3 discusses the relevance of the state of the wastewater system at the onset of a storm event.

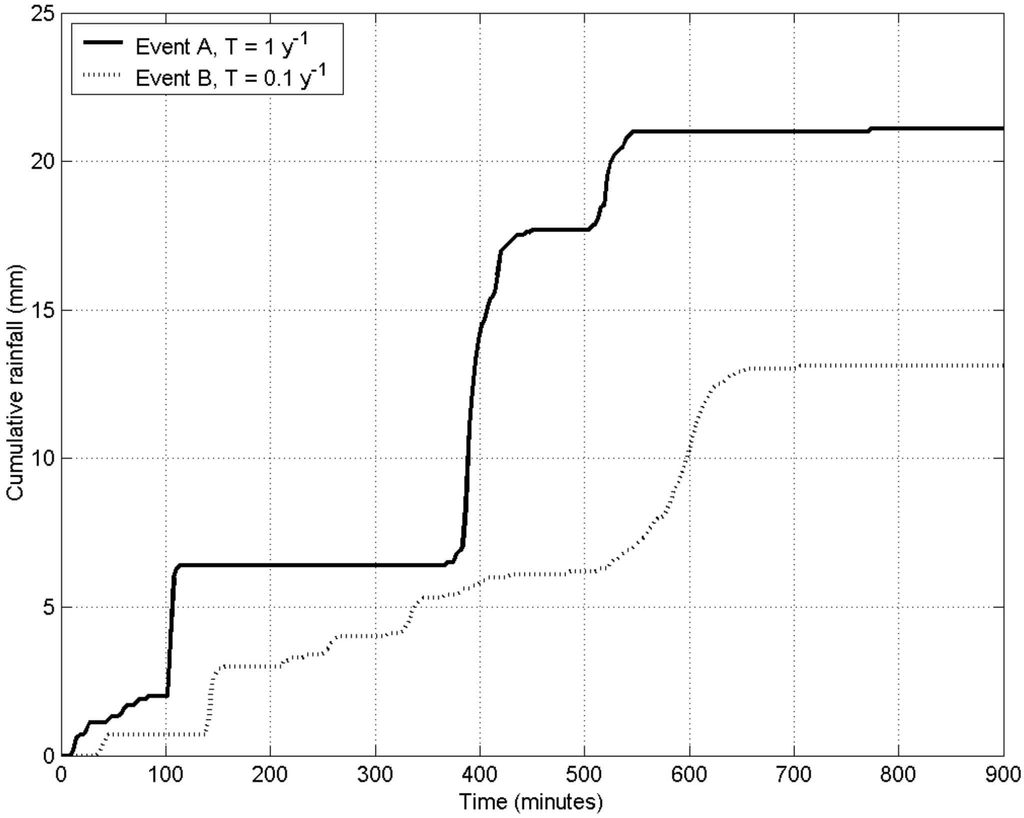


Figure 6.8 Cumulative rainfall for storm events A and B. The return periods are derived from the IDF curves for De Bilt [Bouwknecht and Gelok (1988)] by assessing the maximum intensity for intervals of 15, 30, 60, 120, 240 and 480 minutes. The interval with the highest return period determines the return period of the storm event.

6.3.2 Wastewater system performance: reference system

The reference for assessing wastewater system performance is the performance of the wastewater system having the characteristics of sewer system Loenen, see section 4.2 and wwtp ‘Katwoude described in chapter 3:

- no additional external storage capacity
- pumping capacity of 0.7 mm/h
- no pressure main

while assuming:

- a temperature of the activated sludge of 15 ^{o}C (mean annual value wwtp Katwoude)
- a diurnal profile in the dwf in terms of flow and constant concentrations over the day.

The performance of the reference system was analysed for dwf conditions, storm event A and storm event B. All results shown in this case study are simulation results.

Storm event A causes a CSO event with a total volume of 7 mm, while storm event B does not cause the CSO to spill.
Figure 6.9 shows the ammonium concentration in the wwtp effluent during dwf and storm event A. The ammonium concentration increases to 2.5 mg N/l during the storm event and decreases rapidly after the event to dwf levels.

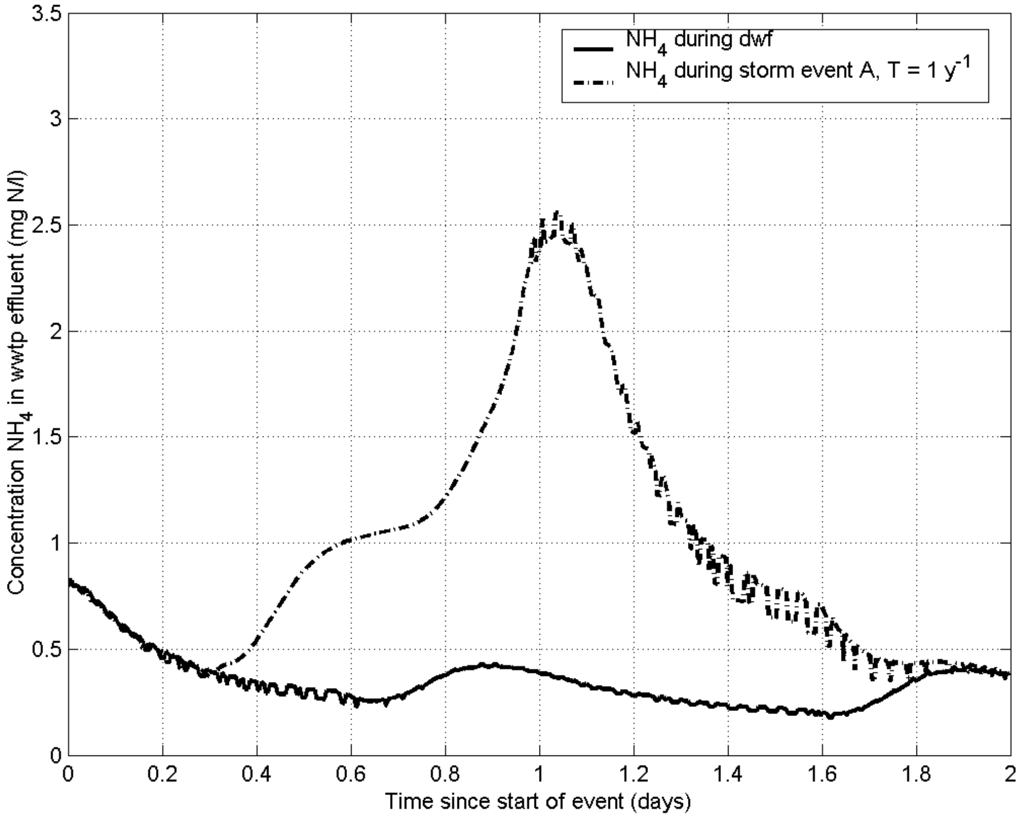


Figure 6.9 Simulated ammonium concentration in wwtp effluent during storm event A (T = 1 y^{-1}). After the storm event, the ammonium concentration returns to normal dwf values. The time on the x-axis is relative to the start of the simulation time at 17:30 hours. The storm event started at 22:30 hours.

In addition to the ammonium concentration shown in figure 6.9, figures 6.10 and 6.11 give an illustration of the impact of storm event A on the wwtp effluent quality for the parameters total nitrogen (N_{total}) and $COD_{biodegradable}$ (comprising the ASM1 fractions S_s and X_s). The quality of the simulation results in terms of properly reproducing the dynamics for these parameters was not analysed in detail in chapter 3 due to a lack of reliable data. Therefore, the results with respect to the parameters total nitrogen and $COD_{biodegradable}$ are to be considered as an indication only with respect to wwtp performance. Moreover, the Hydroworks model applied only takes the hydrodynamics and the solute transport into account. In reality, however, sewer sediment also contributes to the total load of biodegradable COD in CSO volumes. [Chebbo et al. (2003); Vollertsen (1998)]. Consequently, the simulated COD values for the CSO likely underestimate the total load of biodegradable COD discharged.
In the remainder of this chapter total nitrogen and $COD_{biodegradable}$ are only addressed as total loads, like in table 6.2.

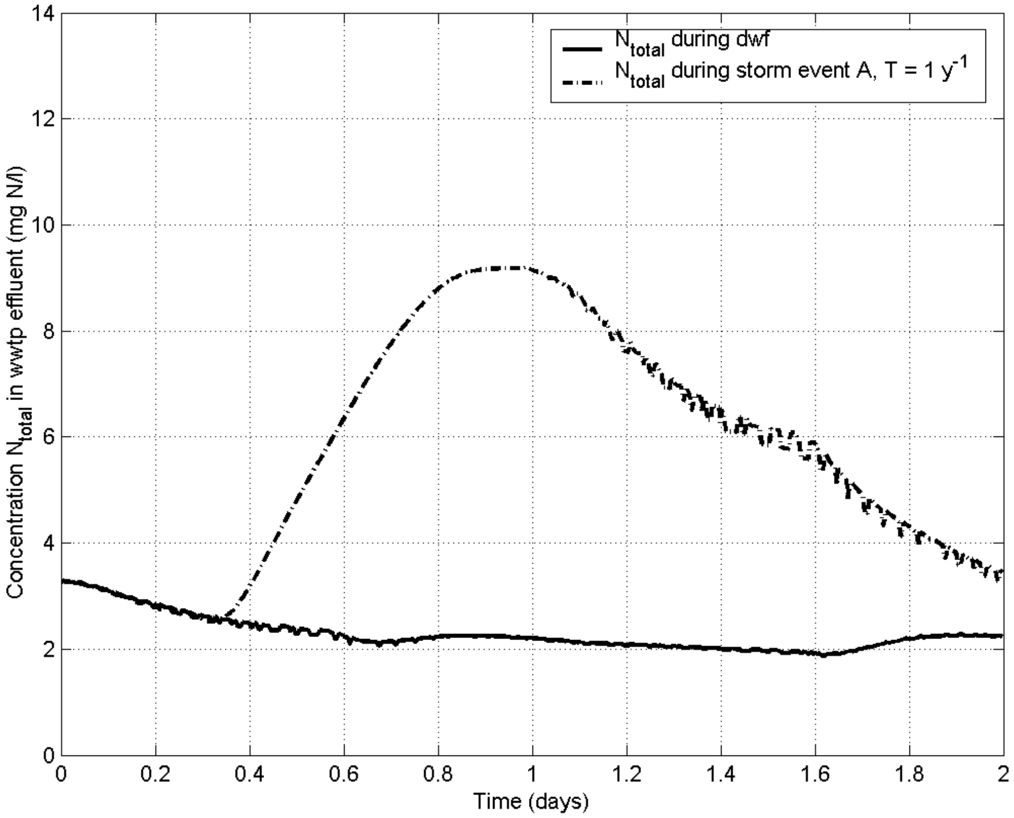


Figure 6.10 N_{total} concentration in wwtp effluent during storm event A (T = 1 y^{-1}). The N_{total} concentration increases significantly during the storm event, to decrease to dwf values after the storm event.

The total nitrogen concentration in the wwtp effluent increases significantly during storm event A. However, the values are still well below the 10 mg N/l required in the Netherlands, see table 1.1. The concentration of $COD_{biodegradable}$, shown in figure 6.11, is hardly affected by the storm event.

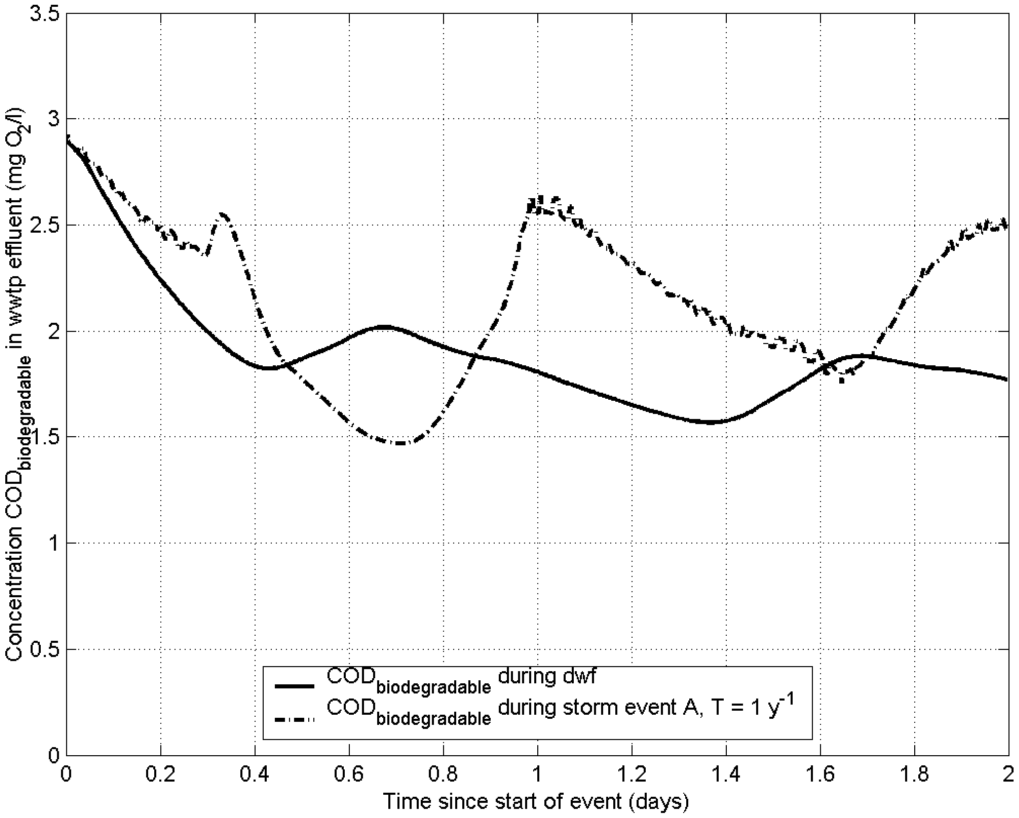


Figure 6.11 $COD_{biodegradable}$ concentration in wwtp effluent during storm event A (T = 1 y^{-1}). The deviation from the normal dwf concentration is negligible.

Figures 6.12, 6.13 and 6.14 show the reference situation for storm event B (T = 0.1 y^{-1}). Although storm event B differs strongly from storm event A in terms of precipitation depth and intensity, the impact on wwtp performance in terms of the effluent concentration ammonium, total nitrogen and $COD_{biodegradable}$, is comparable.

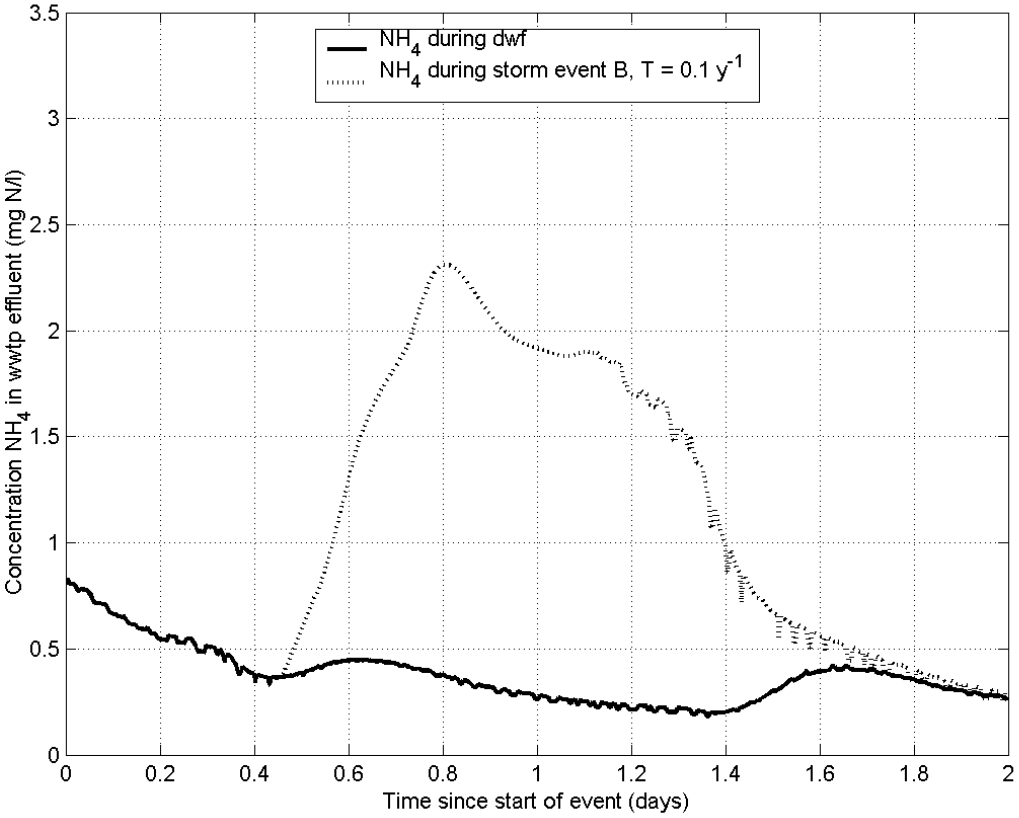


Figure 6.12 Ammonium concentration in wwtp effluent during storm event B ($T = 0.1\ y^{-1}$). After the storm event, the ammonium concentration returns to normal dwf values. The time on the x-axis is relative to the start of the simulation time at 00:00 hours. The storm event started at 08:00 hours.

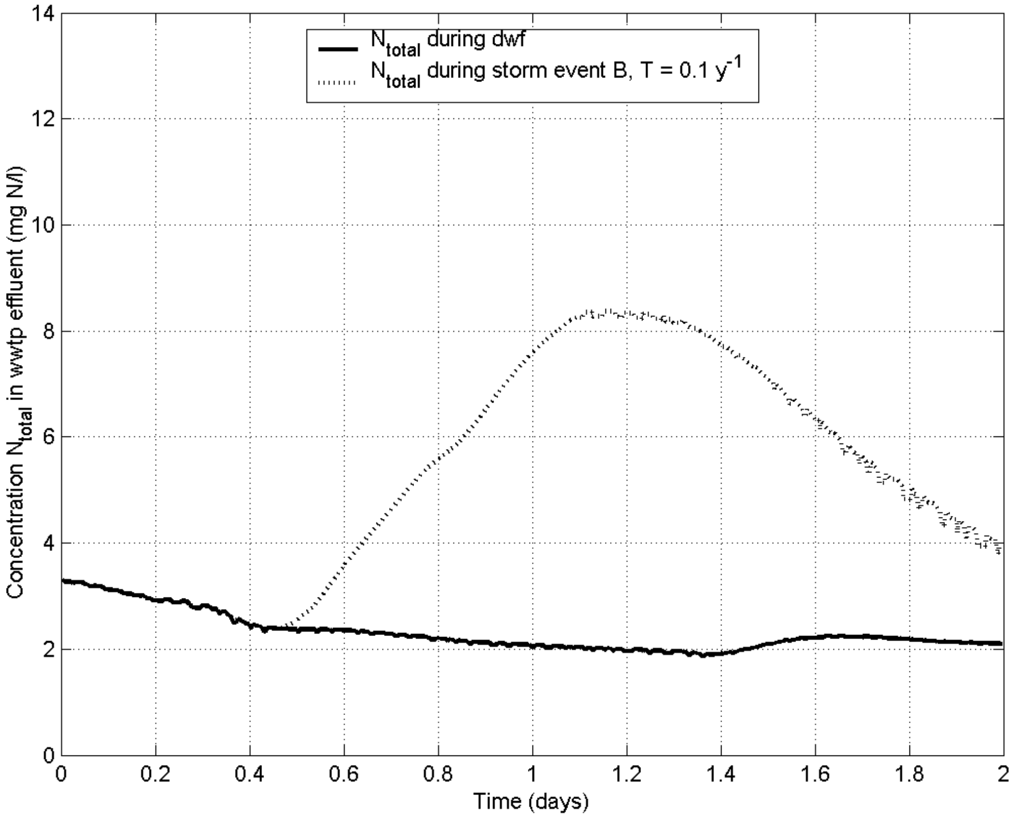


Figure 6.13 N_{total} concentration in wwtp effluent during storm event B (T = 1 y^{-1}). The response of wwtp effluent concentration to this storm event is comparable with the response to storm event A.

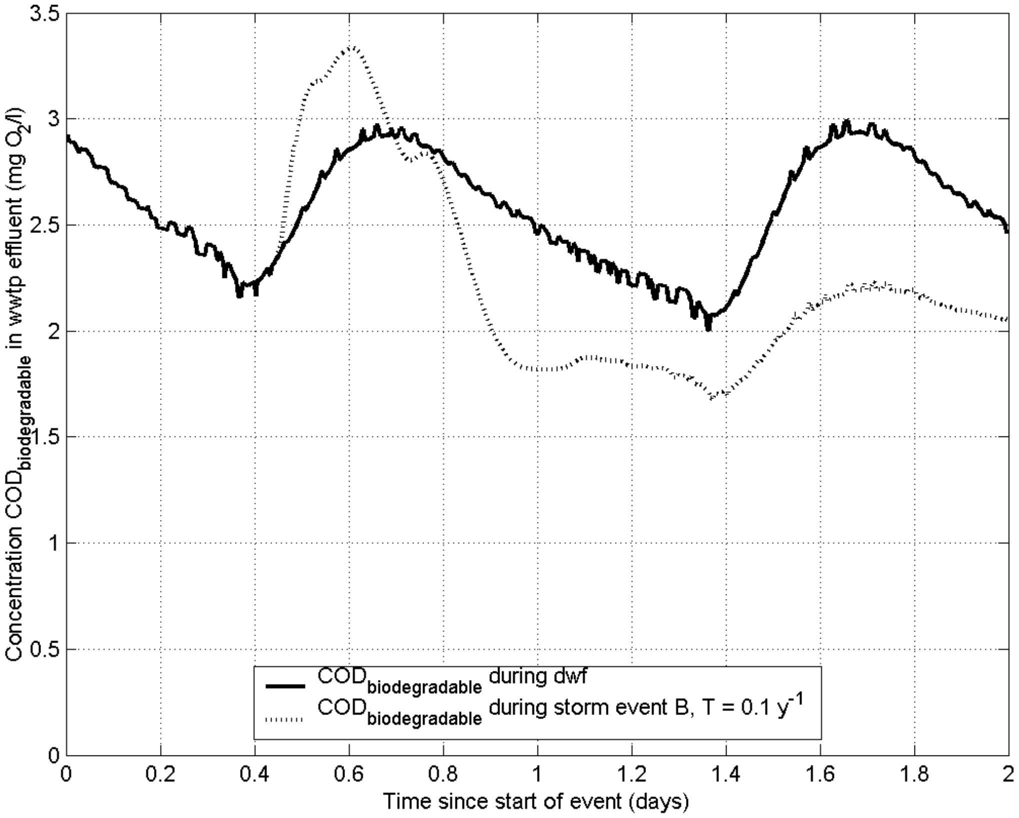


Figure 6.14 $COD_{biodegradable}$ concentration in wwtp effluent during storm event B (T = 0.1 y^{-1}). Like in storm event A, the deviation from the normal dwf concentration is negligible.

Table 6.2 gives the loads discharged via the wwtp effluent and the CSO for the dwf situation and both storm events. The loads are calculated for the total period of two days shown in the figures in this section. For both storm events, all parameters show a significant increase in the total load discharged by the wwtp relative to the dwf loads. For ammonium and total nitrogen this is due to a combination of increased concentrations due to a reduced removal efficiency at the wwtp, like illustrated in figure 3.8 and 3.15, and elevated flows. The total volume discharged increases relative to the dwf volume with 2.3 for storm event A and 2.2 for storm event B.
For $COD_{biodegradable}$, this increase in effluent load is mainly due to the higher effluent volume, as the concentration of $COD_{biodegradable}$ in the effluent hardly increases during the storm events, see figure 6.11 and 6.14.

The relative contribution of the CSO to the total load discharged differs strongly per parameter. For ammonium and total nitrogen, the contribution of the CSO to the total load discharged to the receiving waters is relatively small. However, the load discharged by the sewer system is discharged during the spilling period, which is significantly shorter than the 48 hours over which the loads of the wwtp are calculated. An assessment of the impacts of the discharged loads on the receiving water quality is necessary to be able to take this aspect into account. However, as stated earlier in this chapter, modelling the receiving waters is considered to be beyond the scope of this thesis. Therefore, in this chapter wastewater system performance is only assessed from and emission point of view.
For $COD_{biodegradable}$, however, the CSO discharges almost twice the load of the wwtp. This indicates that for COD CSO abatement is much more effective overall than for the nitrogen fractions. This effect is in reality even stronger, as the contribution of the in sewer stocks of pollutants to the total $COD_{biodegradable}$ load was neglected.

Table 6.2 Loads discharged during dwf, storm event A (T= 1 y^{-1}) and storm event B (T= 0.1 y^{-1})

Point of discharge	Ammonium load ($kg\ NH_4$-N)			N_{total} load (kg N)			$COD_{biodegradable}$ load (kg COD)		
	dwf	storm A	storm B	dwf	storm A	storm B	dwf	storm A	storm B
CSO load	0	13	0	0	23	0	0	154	0
wwtp load	9	39	52	36	239	197	79	79	91
total load	9	52	52	36	262	197	79	233	91

6.3.3 Seasonal and diurnal variations

Section 6.3.2 illustrated that both assessed storm events have a significant impact on wwtp effluent quality and the total discharged pollutant load. The results for the reference system were obtained while assuming a temperature of 15°C. Normally, the temperature of the activated sludge shows a seasonal variation. The temperature has a clear impact on the wwtp effluent quality, as illustrated in figure 3.9. In addition, the dwf typically shows a diurnal profile [e.g. Krebs et al. (1999)]. This section discusses the impact of the seasonal variation in activated sludge temperature and the diurnal dwf profile on the total load discharged by the wastewater system.

Impact of seasonal variations: wastewater temperature

The impact of the seasonal variation in activated sludge temperature is illustrated for storm event A for a temperature of 10, 15 and 20 °C respectively. Figure 6.15 shows the impact of storm event A on the effluent quality to vary with the temperature of the activated sludge. Relative to the reference situation with a temperature of the activated sludge of 15°C, a temperature of 10°C causes a significant increase in the ammonium concentration in the wwtp effluent, whereas a temperature of 20°C hardly affects the ammonium concentration. Consequently, the total ammonium load discharged varies significantly, as summarised in table 6.3, illustrating that the temperature effect is strongly non-linear.

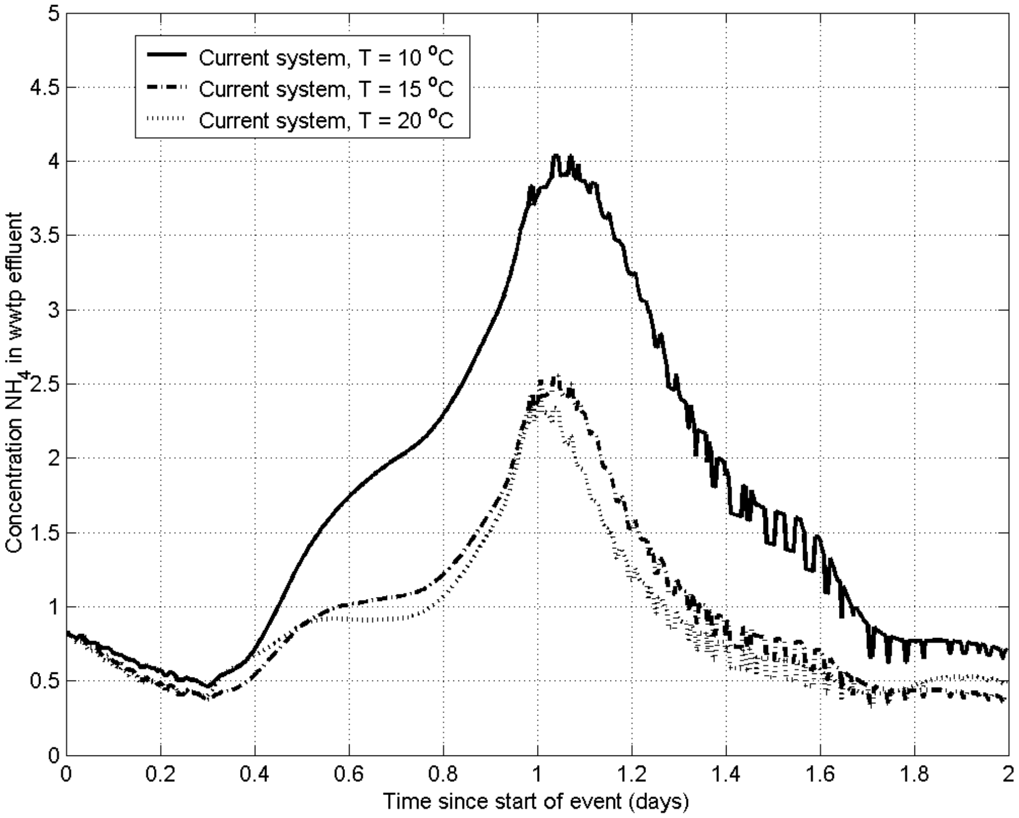


Figure 6.15 Effect of activated sludge temperature on ammonium concentration in wwtp effluent for storm event A (T = 1 y^{-1}). A decrease in temperature to 10°C is clearly detrimental, whereas an increase in temperature from 15 to 20°C hardly affects the simulation result.

Table 6.3 also gives the total loads of total nitrogen and $COD_{biodegradable}$ discharged during storm event A for a temperature of 10, 15 and 20°C. With respect to total nitrogen, the same trend was observed as for ammonium. The wwtp effluent load of $COD_{biodegradable}$, however, decreases with a lower temperature.

A consequence of the temperature dependent dynamic response of the wwtp to a storm event is the fact that the proportion of effluent and CSO loads changes with the wastewater temperature. Within wastewater optimisation studies, the temperature selected may therefore have an effect on the optimal solution found.

Table 6.3 Loads discharged during storm event A at a temperature of 10, 15 and 20°C.

Point of discharge	**Ammonium load (kg NH_4-N)**			**N_{total} load (kg N)**			**$COD_{biodegradable}$ load (kg COD)**		
	10°C	15°C	20°C	10°C	15°C	20°C	10°C	15°C	20°C
CSO load	13	13	13	23	23	23	154	154	154
wwtp load	67	39	37	320	239	147	73	79	122
total load	80	52	50	343	262	170	227	233	276

Impact of diurnal profile on response to transient conditions

The dwf typically shows a diurnal profile, see e.g. figure 6.4. Consequently, the impact of a storm event on the total pollutant load discharged by the wastewater system varies with the time at which a storm event takes place. Figure 6.16 gives an illustration of the impact of storm event A, starting at 08:00, 16:00 and 22:30 hour respectively, on the ammonium concentration in the wwtp effluent. The three situations, varying only in the starting time of the storm event, show a considerably different response to the storm event. Table 6.4 gives the loads of ammonium, total nitrogen and $COD_{biodegradable}$ for the three situations.

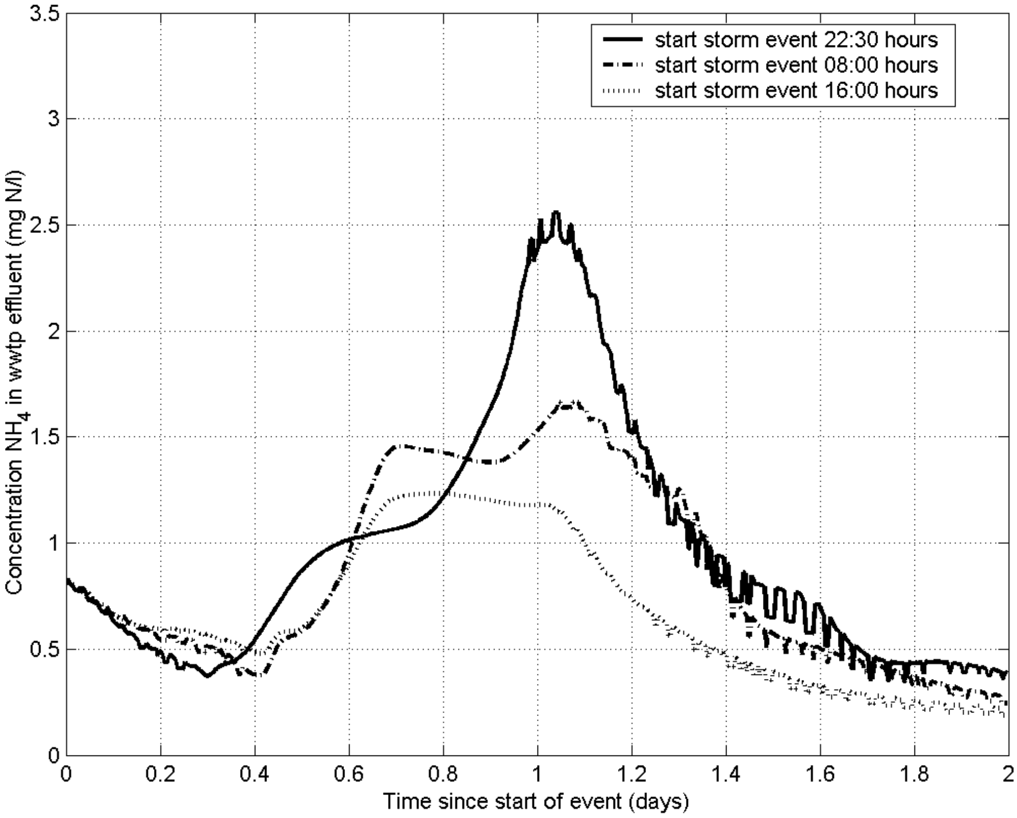


Figure 6.16 Effect of time of start of storm event A (T = 1 y^{-1}) on ammonium concentration in wwtp effluent for a simulation temperature of 15 °C. The time on the x-axis is relative to the start of the simulation time for the three situations. Clearly, the moment a storm event takes place is significant with respect to the ammonium concentration in the wwtp effluent.

Table 6.4 Loads discharged during storm event A, starting at 22:30, 08:00 and 16:00 hour.

Point of discharge	**Ammonium load (kg NH_4-N)**			**N_{total} load (kg N)**			**$COD_{biodegradable}$ load (kg COD)**		
	22:30	08:00	16:00	22:30	08:00	16:00	22:30	08:00	16:00
CSO load	13	62	32	23	107	56	154	735	386
wwtp load	39	37	34	239	212	220	79	76	82
total load	52	99	66	262	319	277	233	812	468

Table 6.4 shows that the wwtp effluent load of ammonium does not vary strongly with the starting time of the storm event, even though figure 6.16 gives the impression that the wwtp effluent quality and therewith the effluent load of ammonium varies significantly with the point in time of the start of the storm event. The same conclusion holds for total nitrogen and $COD_{biodegradable}$. The load discharged by the CSO, however, varies strongly for the three situations. This is due to the fact that the volume of dwf produced during the storm event varies for the three situations and consequently, both the dilution and the overflow volumes, summarised in table 6.5, are different. The variation in CSO loads is much higher than the variation in CSO volume. E.g., the ammonium load discharged by the CSO for storm event A starting at 08:00 hour is 4.8 times higher than the CSO load discharged by storm event A starting at 22:30 hour. The overflow volume, on the other hand, is only 17% higher.

Consequently, the dilution rate for the three storms starting at a varying point in time is the main factor determining the total pollution load discharged via the CSO.

The results obtained in the analysis of the impact of the diurnal profile on the response to transient conditions show that the actual state of both the sewer system and the wwtp at the onset of a storm event is very important. This state depends on the antecedent period. In this section only the variation in this state during dwf were addressed. It should be noted, however, that preceding storm events could also exert a significant effect on the capability of the wastewater system to deal with subsequent storms. An example of this effect is given by [Jack (1999) and Jack et al. (1999)].

Table 6.5 Overflow volumes during storm event A, starting at 22:30, 08:00 and 16:00 hour.

Point of discharge	**starting time of event**		
	22:30	08:00	16:00
CSO volume (mm)	6.9	8.1	7.3

The results shown in table 6.4 are very important with respect to wastewater system optimisation studies. As the proportion between pollution load discharged via the CSO and wwtp effluent varies strongly, also the relative effectiveness of measures taken in the sewer system or at the wwtp varies. Section 6.3.4 and 6.3.5 elaborate further on this important aspect with examples discussing the impact of additional storage and changes in the pumping capacity.

6.3.4 Impact of additional storage capacity

The impact of additional storage capacity on wastewater system performance is discussed for storm A, starting at 22:30 and 08:00 hour respectively.
For storm A starting at 22:30 hour, the additional storage capacity reduces the overflow volume from 7 mm to 1 mm. The additional storage, however, has detrimental effects on wwtp performance in terms of ammonium concentration in the effluent. Figure 6.17 shows the ammonium concentration in the wwtp effluent for the situation with and without additional storage capacity. The ammonium concentration increases due to the prolonged loading of the wwtp.

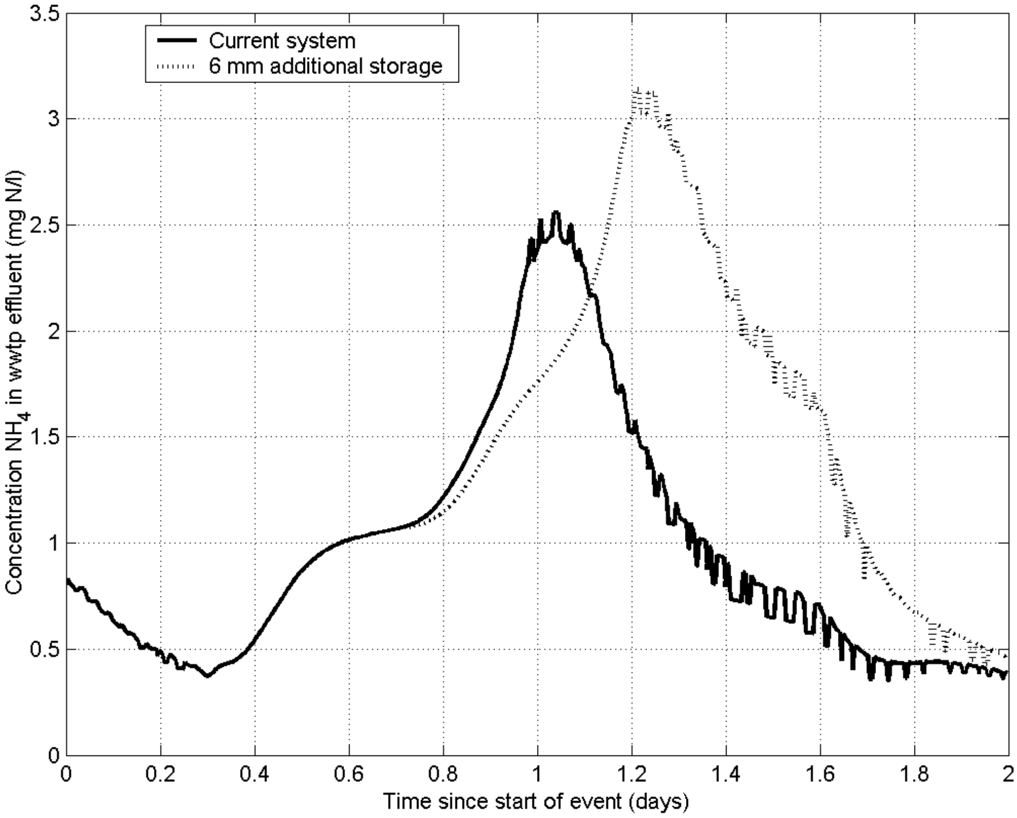


Figure 6.17 Effect of 6 mm of additional external storage on the ammonium concentration in wwtp effluent for storm event A (T = 1 y^{-1}) starting at 22:30 hours. The additional storage causes a deterioration of the effluent quality.

Table 6.6 shows the impact of the additional storage on the total pollutant loads discharged to the receiving waters. Although the additional storage causes the load discharged via the CSO to decrease significantly for all parameters, its impact is detrimental with respect to the ammonium and total nitrogen load discharged via the wwtp effluent. In this case, the detrimental effects on effluent quality dominate the positive effects on CSO load for the parameters ammonium and total nitrogen. With respect to COD, however, additional storage capacity is beneficial overall.

Table 6.6 Impact of additional external storage capacity on loads discharged during storm event A, starting at 22:30 hour.

Point of discharge	Ammonium load (kg NH_4-N)		N_{total} load (kg N)		$COD_{biodegradable}$ load (kg COD)	
	current system	6 mm additional storage	current system	6 mm additional storage	current system	6 mm additional storage
CSO load	13	2	23	4	154	25
wwtp load	39	58	239	326	79	95
total load	52	60	262	330	233	120

Figure 6.18 shows the ammonium concentration in the wwtp effluent for storm event A starting at 08:00 instead of 22:30 hour. The impact of the additional storage is again detrimental with respect to the ammonium concentration in the effluent. However, the impact of the additional storage on the total ammonium load discharged is in this case beneficial due to the relatively large reduction in the load discharged via the CSO, see table 6.7.

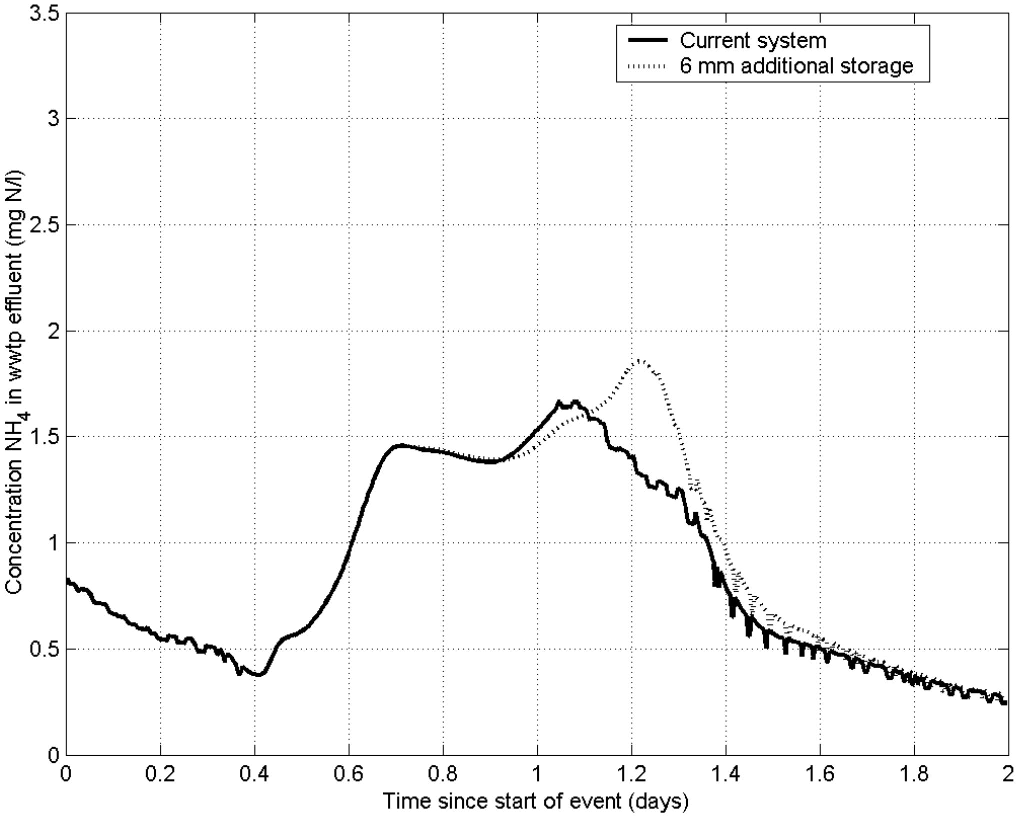


Figure 6.18 Effect of 6 mm of additional external storage on the ammonium concentration in wwtp effluent for storm event A (T = 1 y^{-1}) starting at 08:00 hours. Like for storm event A starting at 22:30 hours, the additional storage causes a deterioration of the effluent quality due to the prolonged loading of the wwtp.

Table 6.7 Impact of additional external storage capacity on loads discharged during storm event A, starting at 08:00 hour.

Point of Discharge	Ammonium load (kg NH_4-N)		N_{total} load (kg N)		$COD_{biodegradable}$ load (kg COD)	
	current system	6 mm additional storage	current system	6 mm additional storage	current system	6 mm additional storage
CSO load	62	40	107	69	735	472
wwtp load	37	50	212	290	76	90
total load	99	90	319	359	812	562

The total nitrogen and $COD_{biodegradable}$ load show the same trend as for storm event A starting at 22:30 hour. The detrimental effect on wwtp performance dominates with respect to the total nitrogen load, whereas the beneficial effect of additional storage on the CSO load dominates for the $COD_{biodegradable}$ load.
A major consequence of the different overall impact of additional storage with respect to the ammonium load is that the optimal configuration of a wastewater system with respect to additional storage varies with time. Therefore, it is concluded that the time varying impact of storm events must be accounted for in wastewater system optimisation studies.

6.3.5 Impact of installed pumping capacity

The pumping capacity or hydraulic capacity of the wwtp is often discussed as one of the key variables determining wwtp performance [e.g. Bruns (1999); Guderian et al. (1998); Erbe et al. (2002)]. The impact of the installed pumping capacity was analysed for the following events:

- storm event A starting at 22:30 hour
- storm event A starting at 08:00 hour
- storm event B starting at 08:00 hour

Storm event A starting at 22:30 hour
Figure 6.19 shows the effect of decreasing the pumping capacity on the ammonium concentration in the wwtp effluent. The nitrification at the wwtp clearly benefits from the reduced hydraulic loading. The CSO volume, however, increases with a lower pumping capacity, from 6.9 mm for a pumping capacity of 0.7 mm/h to 8.6 mm for a pumping capacity of 0.5 mm/h. As a result, the effluent load of ammonium decreases, while the CSO load of ammonium increases with a decreasing pumping capacity, see table 6.8.

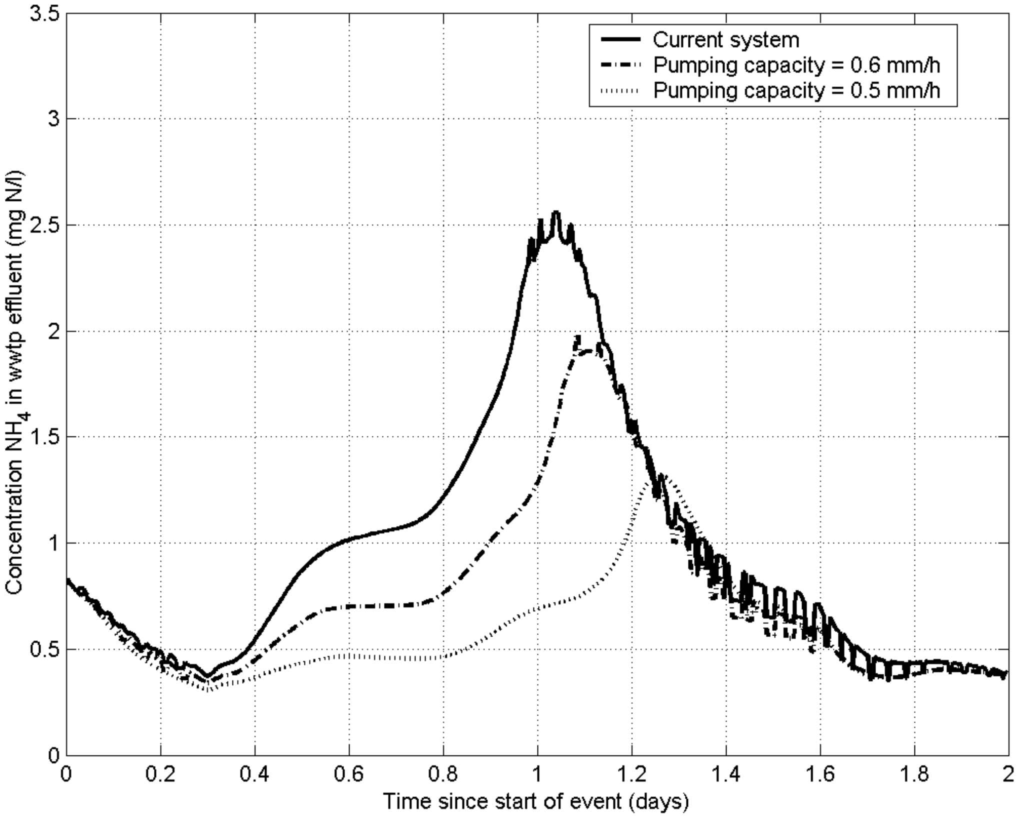


Figure 6.19 Effect of decreasing the pumping capacity on ammonium concentration in wwtp effluent. Storm event A, (T= 1 y^{-1}), starting time of storm event 22:30 hour.

In this case, reducing the pumping capacity is beneficial with respect to the total load of ammonium and total nitrogen, but negative with respect to the total load of $COD_{biodegradable}$. The latter is due to the increase of the already dominating load discharged via the CSO.

Table 6.8 Loads discharged during storm event A, starting at 22:30 hour.

Point of discharge	**Ammonium load (kg NH_4-N)**			**N_{total} load (kg N)**			**$COD_{biodegradable}$ load (kg COD)**		
	current system 0.7 mm/h	0.6 mm/h	0.5 mm/h	current system 0.7 mm/h	0.6 mm/h	0.5 mm/h	current system 0.7 mm/h	0.6 mm/h	0.5 mm/h
CSO load	13	14	16	23	24	29	154	164	196
wwtp load	39	28	19	239	207	176	79	74	69
total load	52	42	35	262	231	204	233	238	264

Storm event A starting at 08:00 hour
For storm event A starting at 08:00 hour, the situation is again, like in the analyses of the impact of additional storage, different from having the same storm event starting at 22:30 hour. A reduction of the pumping capacity slightly decreases the ammonium concentration in the wwtp effluent, as shown in figure 6.20. Table 6.9 shows that reducing the pumping capacity to 0.6 mm/h improves overall wastewater system performance in terms of the total ammonium and total nitrogen load. Further reducing the pumping capacity to 0.5 mm/h, however, causes the total ammonium load to increase relative to the load discharged at a pumping capacity of 0.6 mm/h. In addition, the total $COD_{biodegradable}$ load increases with a decreasing pumping capacity, like for storm A starting at 22:30 hour.
Therefore, it can be concluded again that the proportion of the load discharged via the effluent and the CSO determines the effect of measures in the wastewater system. Moreover, the time at which a storm event occurs determines what pumping rate is to be perceived optimal.

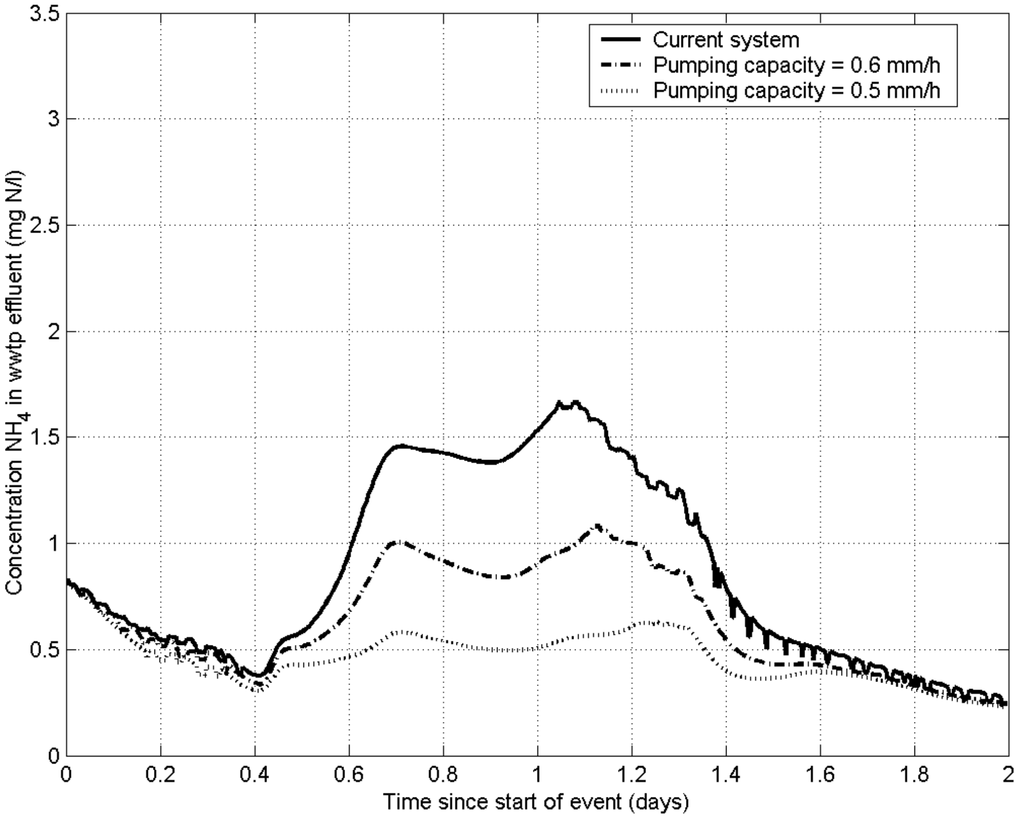


Figure 6.20 Effect of decreasing the pumping capacity on ammonium concentration in wwtp effluent. Storm event A, (T= 1 y^{-1}), starting time of storm event 08:00 hour.

Table 6.9 Loads discharged during storm event A, starting at 08:00 hour.

Point of discharge	**Ammonium load (kg NH_4-N)**			**N_{total} load (kg N)**			**$COD_{biodegradable}$ load (kg COD)**		
	current system 0.7 mm/h	0.6 mm/h	0.5 mm/h	current system 0.7 mm/h	0.6 mm/h	0.5 mm/h	current system 0.7 mm/h	0.6 mm/h	0.5 mm/h
CSO load	62	69	83	107	121	145	735	825	991
wwtp load	37	24	15	212	183	157	76	69	62
total load	99	94	98	320	304	302	812	895	1053

Storm event B starting at 08:00 hour

Storm B did not cause the CSO to spill in the reference situation. Reducing the pumping capacity to 0.6 and 0.5 mm/h, however, results in a CSO volume of 0.7 and 1.6 mm respectively. Figure 6.21 illustrated that, even more significant than for storm event A, a reduction of the pumping capacity results in an improvement of the wwtp effluent quality.

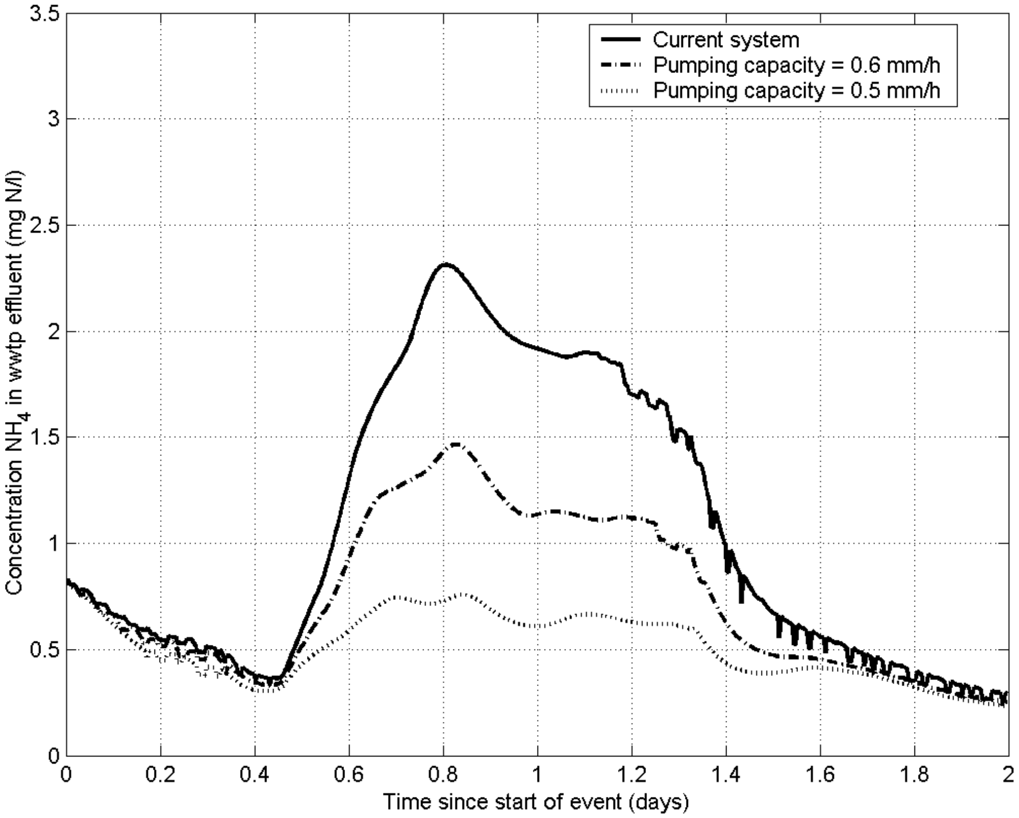


Figure 6.21 Effect of decreasing the pumping capacity on ammonium concentration in wwtp effluent. Storm event B, (T= 0.1 y^{-1}).

The relatively limited CSO volumes due to the reduced pumping capacity cause the CSO load to increase. Nevertheless, with respect to the total load of ammonium and total nitrogen discharged to the receiving waters, the effect of a reduction of the pumping capacity is positive. The $COD_{biodegradable}$ load, however, increases with a reduced pumping capacity. This result is comparable with the results obtained for storm event A, starting at 22:30 hour.

Table 6.10 Loads discharged during storm event B.

Point of discharge	**Ammonium load (kg NH_4-N)**			**N_{total} load (kg N)**			**$COD_{biodegradable}$ load (kg COD)**		
	current system 0.7 mm/h	0.6 mm/h	0.5 mm/h	current system 0.7 mm/h	0.6 mm/h	0.5 mm/h	current system 0.7 mm/h	0.6 mm/h	0.5 mm/h
CSO load	0	3	12	0	5	21	0	35	143
wwtp load	52	33	18	197	169	141	91	81	71
total load	52	36	30	197	174	162	91	116	214

Although reducing the pumping capacity to 0.6 mm/h reduces the total ammonium and total nitrogen load discharged, the, even though limited, CSO discharge causes detrimental effects on especially the total $COD_{biodegradable}$ load. Nevertheless, there is a potential for wastewater system optimisation, as the pumping capacity could be lowered to a level where

CSO does just not spill. In this case, the wwtp effluent benefits from the reduced pumping capacity, while still no CSO event occurs. As most of the storm events occurring do not cause a CSO event (the CSO frequency of Dutch sewer systems complying with the standards is less than 10 y^{-1}, whereas the number of storm events with over 5 mm of precipitation amounts on average to 54 y^{-1} for De Bilt, see table 4.4) a large optimisation potential is available. This would require RTC using weather forecasts to predict to what extent the pumping capacity could be lowered without increasing the number of CSO events.

The results for the two versions of storm event A show that for large storms causing a CSO event the overall impact of reducing the pumping capacity is event dependent, thus complicating wastewater system optimisation.
The results for storm event B show that for storms not causing a CSO event the total load discharged to the receiving waters could be reduced by reducing the pumping capacity to a level just preventing the CSO from spilling. This is only beneficial as long as the temporarily reduced pumping capacity does not have detrimental effects, like settling of sediment in the sewer system.

6.3.6 Impact of hydraulic retention time of pressure main

Pressure mains are common components of wastewater systems consisting of a number of sewer catchments discharging to a central wwtp, especially in areas with a low available hydraulic gradient. As illustrated in section 6.2, pressure mains can contribute significantly to the fluctuations in the influent load arriving at a wwtp. This section discusses the impact of the size, in terms of hydraulic retention time under dwf conditions, of pressure mains on wwtp performance. The impact of anaerobic transformations in the pressure main is not addressed. Figure 6.22 shows the ammonium concentration in the wwtp effluent for three different situations:

- no pressure main
- a pressure main with a dwf hydraulic retention time of 6 hours
- a pressure main with a dwf hydraulic retention time of 24 hours

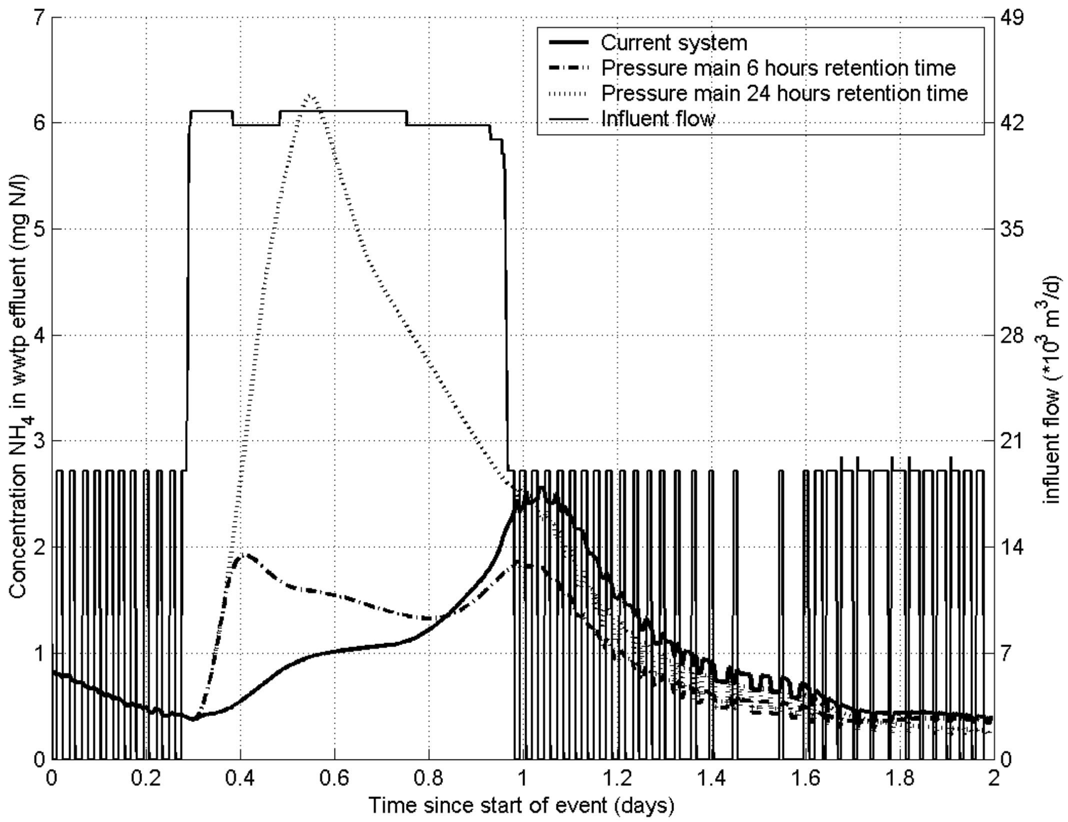


Figure 6.22 Effect of pressure main induced peak loading of the wwtp on the ammonium concentration in wwtp effluent. This effect is due to the fact that wastewater with a dwf concentration arrive at the wwtp with wwf flow rate at the onset of a storm event. Consequently, the longer the hydraulic retention time of the pressure main, the stronger the effect. Storm event A, starting at 22:30 hour.

The pressure main with a retention time of 6 hours seems to be beneficial with respect to wwtp performance, as the peak concentration in ammonium decreases from 2.5 to just below 2 mg N/l. However, the additional peak at the beginning of the storm event coincides with the period of increased hydraulic loading, shown in figure 6.22. Consequently, the total load discharged with a pressure main of 6 hours is 20% higher than without a pressure main, see table 6.11.

The pressure main with a hydraulic dwf retention time of 24 hour has a significant impact on the ammonium concentration in the wwtp effluent. The maximum ammonium concentration in the effluent increases to 6.3 mg N/l and the total additional load discharged increases by 150%.

The size of pressure mains has a significant impact on the total load discharged via the wwtp effluent for all parameters assessed. As pressure mains do not affect the CSO load, the relative contribution of the wwtp to the total load discharged changes. Consequently, also the existence of pressure mains in wastewater systems is important with respect to optimisation of overall wastewater system performance.

Table 6.11 Loads discharged during storm event A, starting at 22:30 hour for different dwf hydraulic retention times of a pressure main.

Point of discharge	Ammonium load (kg NH_4-N)			N_{total} load (kg N)			$COD_{biodegradable}$ load (kg COD)		
	current system	6 h	24 h	current system	6 h	24 h	current system	6 h	24 h
CSO load	13	13	13	23	23	23	154	154	154
wwtp load	39	49	118	239	236	268	79	97	200
total load	52	62	131	262	258	291	233	251	354

6.4 Discussion

The material presented in this chapter illustrates that the dynamic interactions are very important with respect to wastewater system performance and consequently, also for wastewater system optimisation.

The results of the case study confirm statements in literature [Guderian et al. (1998); Rauch and Harremoës (1997); Jack and Ashley (2002); Lau et al. (2002); Ashley et al. (2002b)] that the optimal configuration of a wastewater system in terms of storage and interceptor (or pumping) capacity strongly depends on the assessed parameter, such as ammonium or COD. In general, nitrogen removal at a wwtp is much more sensitive to variations in the influent loading than COD removal.
The different impact on ammonium and BOD/COD is also noticeable in the different proportion of the total load discharged by the CSO and wwtp effluent, therewith directing the focus in optimisation studies to either the sewer system or the wwtp. This focus could change again as soon as the impacts of the receiving waters rather than the total emissions are studied, as in this case also the characteristics of the receiving waters become relevant. This implies e.g. that for parameters causing an acute effect the distribution in time of the total discharged load has to be addressed.

A topic not often discussed in literature on the integrated approach to wastewater systems is the impact of aspects like the existence of pressure mains on the total loads discharged by the wastewater system during storm events. The influent data analysed in section 6.2 indicated that not only pressure mains are important, but that also the volume of wastewater stored in lost storage or stored in the sewer system due to operational aspects (pump failure, selection of switch on/off levels of pumps) can contribute to the fluctuations in wwtp influent loading and CSO discharges. A large scale inventory and statistical analyses, like performed by [Korving (2004)] on pump failure, should reveal to what extent these (operational) aspects are important for wastewater system performance.

The case study also showed that due to normal seasonal and diurnal variations the time at which a storm event occurs can exert a significant influence on sewer system and wwtp performance. Consequently, the optimal configuration of a wastewater system is time-dependent. This implies that the performance of a wastewater system can be improved by either pollution based RTC or by ‘no regret’ measures. The latter are measures that do not have a detrimental effect on the performance of other parts of the wastewater system.

Pollution based RTC [Risholt et al. (2002); Schilling and Kollatsch (1990)] can be entirely based on information from water quality sensors located at strategic locations in the wastewater system or based on sensor data combined with on line simulation. The latter option has been successfully applied in the model based predictive control of wastewater systems as described by [Seggelke (2002); Seggelke and Rosenwinkel (2002)].
Model based predictive control is only possible when simulation times are short enough to be able to optimise the control actions to be taken online. Therefore, the simulation times have to be a small fraction of real time. As discussed in chapter 5, this can be achieved by

applying simplified models [Meirlaen et al. (2002)], provided that these models are capable of reproducing all relevant wastewater system dynamics. In addition, model based predictive control requires sufficient and reliable on line data.
During the last years, a rapid development of water quality sensors has been observed. These sensors are applied at wwtps and/or in sewer systems for a broad range of parameters [e.g. Grüning and Orth (2002); Häck and Lorenz (2002); VanRolleghem and Lee (2003), Lorenz et al. (2002)].
A methodology for the design of a monitoring network for pollution based RTC, however, is still lacking. Further research is necessary to be able to develop such a methodology. This research should probably be based on the general set up for measuring campaigns given by [Vanrolleghem et al. (1999)] or based on the method for designing monitoring networks for hydrodynamic modelling, developed by [Clemens (2001a); Clemens (2002)].

In the case study of section 6.3 only the emissions from the wastewater system were discussed. Within the EU Water Framework Directive [WFD (2000)], assessing emissions is not longer sufficient, as the focus is on receiving water quality. This further complicates wastewater system optimisation, as this implies that the actual (dynamic) response of the receiving waters to the CSO and wwtp effluent discharges has to be accounted for. The large number of possible measures and the strongly non-linear wastewater system behaviour may therefore require the application of advanced optimisation algorithms, such as Genetic Algorithms or Simulated Annealing discussed in chapter 5. Appendix XII gives an example of the application of Simulated Annealing to a wastewater system optimisation problem aiming at reaching the receiving water quality standards with respect to ammonium at minimal costs.
A major problem related to dealing with the receiving water quality, however, is the large uncertainty in the simulated CSO discharges due to limited knowledge of especially sewer sediment transport processes [Ashley et al. (1999)].

In the case study, wastewater system performance was assessed by the total loads, given in tables 6.2 to 6.4 and 6.5 to 6.11. These tables compare the discharged loads for a number of wastewater system configurations under varying boundary conditions. The uncertainties associated with the model results underlying the loads given in the tables, however, were not addressed. Based on the calculated ammonium loads given in table 6.6, it is discussed whether these simulation results are to be considered significant from a statistical point of view. Table 6.12 shows the calculated ammonium loads during storm event A, starting at 22:30 hour for the situation with and the situation without additional storage capacity.

Table 6.12 Impact of additional external storage capacity on the ammonium load discharged during storm event A, starting at 22:30 hour.

Point of discharge	**Ammonium load ($kg\ NH_4$-N)**	
	current system	6 mm additional storage
CSO load	13	2
wwtp load	39	58
total load	52	60

The calculated loads given in table 6.12 can only be considered significant from a statistical point of view if the 95% confidence interval of the calculated CSO load does not exceed 5.5 kg ammonium and the 95% confidence interval of the calculated wwtp effluent load does not exceed 9.5 kg ammonium. The distribution of the uncertainties is unknown. [Korving (2004)] showed after monte carlo simultations that e.g. CSO volumes can be described using a Weibull distribution. As an analysis of the distribution of the uncertainties is considered beyond the scope of this thesis, a Gaussian distribution is assumed in this section.

Assuming a Gaussian distribution of the uncertainties, this gives an acceptable standard deviation in the CSO load of approximately 2.75 kg ammonium and in the wwtp effluent load of approximately 4.75 kg ammonium, see figure 6.23.

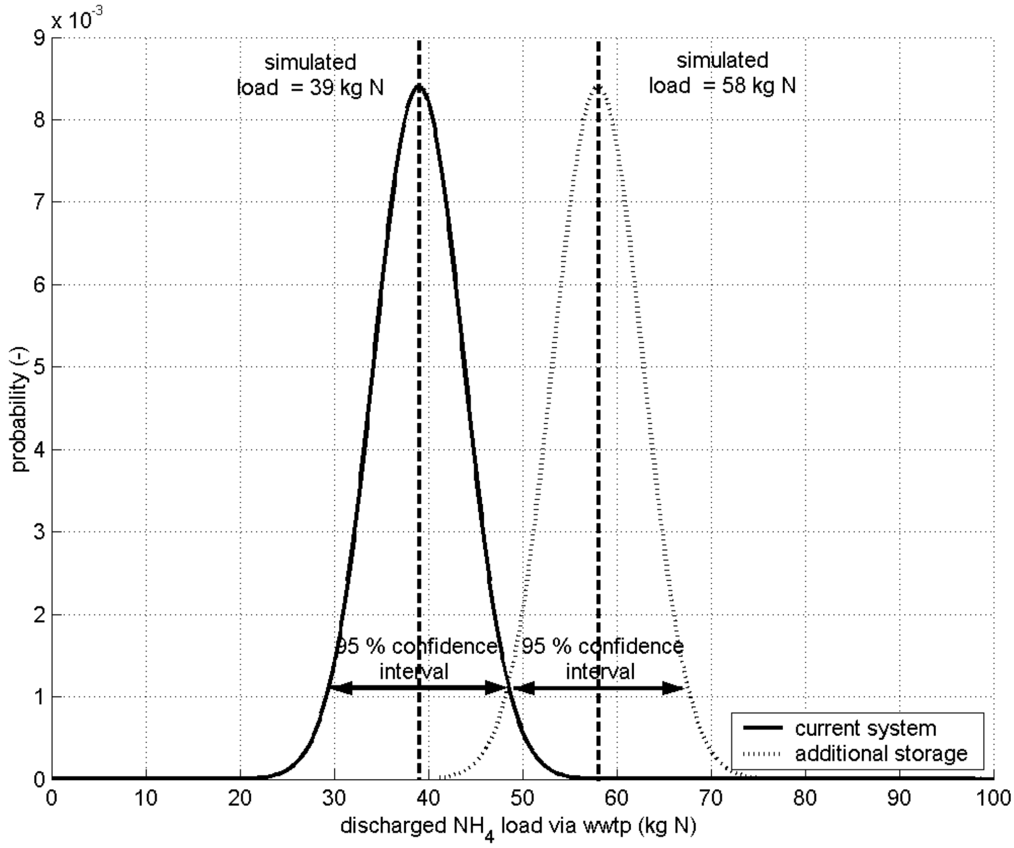


Figure 6.23 95% confidence intervals of calculated ammonium loads discharged via the wwtp. A significant difference in calculated loads is only achieved if the 95 % confidence interval does not exceed ± 9.5 kg ammonium.

As the calculated flows in the simulations are based on the hydrodynamic Loenen model where the pumping capacity is well known, see figure 4.12, it is assumed that the uncertainties in the calculated loads are mainly due to uncertainties related to the simulation of the concentrations. In order to be able to have a standard deviation in the calculated effluent loads not exceeding 4.75 kg ammonium, the acceptable 95% confidence interval of the simulated ammonium concentration in the effluent is 0.1 mg NH_4/l. In section 3.4.8, it is shown that for a period without large measurement errors a mean squared error of 0.23 mg^2 N/l^2 could be achieved with the calibrated Katwoude model. Assuming a Gaussian distribution this MSE is equivalent with a 95% confidence interval of 1.0 mg NH_4/l. It should be noted, however, that this MSE was obtained using measured influent data. In this chapter, the influent data used is produced by the hydrodynamic Loenen model, causing the 95% confidence interval of 1.0 mg NH_4/l of the wwtp model to increase further due to the propagation of errors. Therefore, it is concluded that the accuracy of the model results of the applied wwtp model is not sufficient.
In order to obtain a 95% confidence interval of 2.75 kg ammonium in the CSO load, the 95% confidence interval of the simulated ammonium concentration in the CSO discharge cannot exceed 0.8 mg NH_4/l. In the calibration results given in section 4.3 for the simulation of the ammonium concentration in Ulvenhout, a 95 % confidence interval of 8 mg NH_4/l was obtained. It shoud be noted, however, that this 95% confidence interval of the sewer model is

based on the data of 1 storm only. Therefore, it is advocated to further assess the associated uncertainties for a larger number of storm events.

The required confidence intervals are a factor 10 smaller than the ones obtained in chapter 3 and 4 based on the comparison of measurements and model results. These results show that, even though the reliability of both the applied sewer model and wwtp model is sufficient with respect to the modelling of ammonium, the related uncertainties are still too high. This example illustrates that even a calibrated state of the art model is not capable of producing results with a sufficient accuracy to distinguish between measures to be taken within the wastewater system. It is therefore advocated to extend the probabilistic approach discussed by [Korving (2004)] to both sewer quality and wwtp models in order to be able to identify the main sources of uncertainties and possibly to improve these models.

6.5 Conclusions

Based on the results of the analysis of the effect of wastewater system characteristics for a number of wastewater system performance indicators the following conclusions were drawn:

- the volume of wastewater held within a sewer system (due to the existence of pressure mains, lost storage or due to operational aspects) can exert a significant influence on wastewater system performance in terms of both wwtp effluent quality and CSO discharges. Therefore, these effects must be taken into account when optimising wastewater systems.
- increasing the storage capacity in a sewer system or increasing the pumping capacity reduces the pollution via the CSO. However, these measures can be detrimental with respect to wwtp performance, especially with respect to nitrogen removal. Consequently, it is advised to take into account the dynamic interactions within wastewater systems in wastewater system optimisation studies.
- the wastewater system performance indicator selected has a dominant influence on the solution to be perceived optimal. Therefore, wastewater system optimisation studies should comprise an analysis of the sensitivity of the ‘optimal’ solution to relevant wastewater system performance indicators.
- the optimal solution to a wastewater system optimisation problem is event dependent. Consequently, either pollution based RTC or very robust measures without detrimental effects elsewhere in the wastewater system are to be adhered to.

Chapter 7 Conclusions and recommendations

The material described in this thesis is based on the results of the research project ‘Interactions within the wastewater system I’, started in 1999. This project is to be seen as a first exploration of the field of interactions within wastewater systems. The project identified the importance of the interactions within wastewater systems for wastewater system optimisation. The conclusions given in this chapter reflect the main results and the current state of the art with respect to the interactions. In addition, a number of recommendations for further research is given based on the results described in this thesis. Some of these recommendations are already dealt with in the research project ‘Interactions within the wastewater system II’, started in 2003.

7.1 Conclusions

The objective of this thesis is to identify the possibilities of adopting a water quality based approach for wastewater system optimisation by taking into account the dynamic interactions within wastewater systems.

First of all, it is concluded that the dynamic interactions are important with respect to wastewater system performance, assessed by the emissions to the receiving waters. Wastewater treatment plant (wwtp) effluent quality is affected by quantitative as well as qualitative fluctuations in the influent, which are determined to a large extent by the characteristics of the contributing sewer systems and wastewater transport systems. Consequently, the design and operation of these systems are key elements for wastewater system optimisation as they do not only affect combined sewer overflow (CSO) discharges but also wwtp effluent quality.

It is concluded that current knowledge, incorporated in models for sewer systems and wwtps, allows a realistic assessment of the impact of the dynamic interactions within wastewater systems for ammonium, being the wastewater system performance indicator which is most sensitive to transient conditions.

Furthermore, it is concluded that the selection of parameters used to assess wastewater system performance, such as CSO volume, ammonium, total nitrogen or chemical oxygen demand (COD), determines to a large extent the result of an analysis of wastewater system performance. This is due to:

- the per parameter varying proportion of the pollution discharged via wwtp effluent and CSOs;
- the varying sensitivity to fluctuations in the influent of wwtp performance with respect to nitrogen and COD removal.

Consequently, the knowledge of the interactions within wastewater systems must be used in wastewater system optimisation studies to identify potential detrimental effects of measures taken in either sewer systems or wwtps on the performance of the total wastewater system.

In addition, it is concluded that the potential for optimisation of wastewater systems depends on the characteristics of a storm event. For large storm events causing CSOs to spill, a compromise has to be found between reducing the pollution discharged via the CSO and the wwtp. For smaller storms, comprising the majority of storm events, wastewater system performance can be improved by temporarily reducing the pumping (or interceptor) capacity to a level just not causing the CSOs to spill.

Ideally, the optimal use, given the performance requirements, of available system capacity in terms of pumping and storage capacity is determined for each storm event taking into account the varying (i.e. temperature dependent) biological treatment capacity of a wwtp, the current state of the sewer system and characteristics of the storm event. In practice, this will require predictive, probably model based, control.

Identifying the optimal configuration and operation of a wastewater system can be a very complicated combined problem, given the myriad of possible solutions and the non-linear system behaviour. The two heuristic algorithms discussed and tested in this thesis proved to be capable of dealing with the type of multi-objective, constraint optimisation problem typically encountered within wastewater system optimisation studies. A practical limitation of applying these algorithms, however, is the simulation time, especially if the assessment of wastewater system performance requires impact-duration-frequency relations based on simulations with fully detailed models.

7.2. Recommendations

The results of the research described in this thesis show that incorporating the knowledge of the interactions within wastewater systems in wastewater system optimisation studies can significantly improve the final optimisation results for total emission control. In order to be able to fully benefit from this knowledge, the following recommendations for further research and developments are made.

Pollution based real time control (RTC) of wastewater systems is a very interesting option for dealing with the time varying loading of wastewater systems and the capability to deal with this loading. The knowledge of the interactions can be used to develop new RTC strategies.
The main challenge for further research with respect to pollution based RTC is the development of a methodology for designing monitoring networks providing the data necessary for RTC. This methodology has to comprise at least a tool for selecting measuring locations and determining minimal requirements for sensors to be installed (i.e. in terms of measuring frequency, reliability and accuracy of data). Moreover, it is recommended to develop sensors capable of meeting these requirements and suitable for installation in sewer systems.

The assessment of available monitoring data revealed that the operation and maintenance of wastewater systems is a very important but often overlooked aspect with respect to wastewater system performance. Therefore, it is recommended to further assess the impact of operational aspects on sewer system and wwtp performance in order to be able to fully take advantage of available system capacity. This can of course be combined with the aforementioned pollution based RTC.
In addition, it is recommended to further analyse the causes of the large fluctuations observed in influent loads by scrutinising available influent and operational data for a number of wwtps, possibly combined with additional measuring campaigns. The results of this analysis can be used to decide whether these causes have to be incorporated in models and to develop strategies for dealing with (or preventing) these high influent loads.

Nitrification proved to be the most sensitive process to fluctuations in the influent. Therefore, it is recommended to research which measures can be applied to reduce the detrimental impact of storm events on the nitrification process. These measures can either focus on reducing the fluctuations in the influent load or on measures enhancing the nitrification capacity at a wwtp. An example of the first strategy is the application of urine separation. The second strategy can be applied by e.g. temporarily increasing the number of nitrifiers or novel process design at the wwtp. The number of nitrifiers can temporarily be increased by growing nitrifiers in side stream and adding them to the activated sludge at the onset of a storm event. Another possibility is to optimise the process control at the wwtp during storm events.

The load discharged via the CSO is a very important parameter for wastewater system optimisation. In wastewater system optimisation studies the pollution load discharged by wwtps and CSOs is compared either directly (by assessing the total loads) or indirectly (by weighting the CSO and wwtp discharges or by assessing the impact on the receiving waters).
In the case study of this thesis it has been assumed that the quality of the simulation results, in terms of confidence level, for wwtp effluent and CSO discharges is equal. The uncertainties associated with the simulation of the performance of sewer systems and wwtps, however, differ strongly. Therefore, it is recommended to compare the quality of these simulation results in terms of confidence levels in order to be able to develop requirements for the quality of sewer models with respect to CSO discharges. This may guide the further development of sewer process models and on the other hand enhance the introduction of the concept of uncertainty analysis in wastewater system optimisation studies.

The introduction of knowledge of the interactions within optimisation studies using fully detailed models can be too computationally demanding when impact-duration-frequency relations are required. Therefore, it is recommended to try to simplify the models to an extent that only the relevant dynamics related to the parameters of interest are included.

Finally, it is recommended to extend the research on the interactions to all parameters related to the receiving water quality aspects to be taken into account in the future due the implementation of the EU Water Framework Directive. This also requires an analysis of the impact of discharges from wastewater systems on receiving water quality.

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Abbreviations

AS	activated sludge system
ASM	activated sludge model
AT	aeration tank
bio-P	biological phosphate removal
BOD	biochemical oxygen demand
COD	chemical oxygen demand
CSO	combined sewer overflow
CSTR	Complete mixed Stirred Tank Reactor
DO	dissolved oxygen
dwf	dry weather flow
EMC	event mean concentration
F/M ratio	food to micro-organism ratio or food-to-mass ratio
GA	genetic algorithm
IDF	intensity-duration-frequency
MLSS	mixed liquor suspended solids
MSE	mean squared error
NH_3	ammonia
NH_4	ammonium
$N_{Kjeldahl}$	total ammonium nitrogen and organic bound nitrogen
NTU	normalised turbidity units
NWRW	Nationale Werkgroep Riolering en Waterkwaliteit (national working group on sewerage and water quality)
OUR	oxygen uptake rate
p.e.	population equivalent: the oxygen equivalent to the amount of oxygen demand related to one inhabitant. A p.e. corresponds with 136 g TOD per day or with 54 g BOD per day
PAO	phosphate accumulating organism
PC	primary clarifier
P_{total}	total of phosphorous in wastewater
Q	flow
RIONED	Dutch foundation for applied research on sewerage
RTC	real time control
SA	simulated annealing
SC	secondary clarifier
SCADA	supervisory control and data acquisition system
SS	suspended solids
SSO	sanitary sewer overflow
STD	standard deviation
STOWA	Dutch foundation for applied research on water management
STW	sewage treatment works
SVI	sludge volume index
TEAP	total emissions analysis period
TSS	total suspended solids
UPM	Urban Pollution Management
VAR	variation
VFA	volatile fatty acid
VSS	volatile suspended solids
WFD	Water Framework Directive
WVO	Wet Verontreiniging Oppervlaktewater (Pollution of Surface Waters Act)
wwf	wet weather flow
wwtp	wastewater treatment plant

Summary 'Interactions within wastewater sytems'

The main incentive for the research project 'Interactions within the wastewater system' was the too narrow scope of current approaches for wastewater system optimisation in the Netherlands. Most wastewater system optimisation studies performed in the Netherlands aim at reducing the total provision and operation costs while just meeting the requirements with respect to the discharged CSO volumes. Wwtp performance is often only introduced in the optimisation study as a boundary condition with respect to the hydraulic capacity of the wwtp. Consequently, most wastewater system optimisation studies are strictly quantitative.
This hydraulically based approach, however, has a major drawback. As the dynamic interactions within wastewater systems are not accounted for, the (qualitative) impact of adjustments in sewer system infrastructure or operation on wwtp effluent quality is not taken into account. Moreover, measures like water quality based RTC are not taken into consideration, as current standards do not properly value their reduction to the total pollution load discharged by wastewater system.
The objective of this thesis is to identify the possibilities to extend today's Dutch volume based approach for wastewater system optimisation to a water quality based approach by taking the dynamic interactions within wastewater systems into account.

Chapter 1 Introduction and scope
The history of urban wastewater systems illustrates that today's wastewater systems are the result of over a century of technological, administrative and legislative developments. Consequently, today's infrastructure comprises components of various ages, built with different design philosophies.
The lifetime of infrastructure is a multiple of the lifetime of the wastewater system performance requirements, which are adjusted at an increasing pace. As a result, wastewater system infrastructure has to be adjusted to fully comply with the standards each time the requirements are adjusted, thus prompting a new round of wastewater system improvements. The large investments associated with improving wastewater system infrastructure give rise to wastewater system optimisation studies.
The latest round of wastewater system optimisation studies was entirely volume based. From a more holistic point of view, focussing on volumes only will inevitably lead to suboptimal results when assessing the total pollution load discharged by the wastewater system or the impact on receiving waters.
In this thesis, the introduction of water quality based wastewater system optimisation is anticipated. It is analysed to what extent the knowledge on processes in sewer systems and wwtps is sufficient to be able to take the dynamic interactions within wastewater systems into account.

Chapter 2 State of the art
The pollution load discharged by sewer systems and wwtps clearly affects receiving water quality. Therefore, reducing this pollution load is one of the main drivers for wastewater system optimisation. A complicating factor for wastewater system optimisation is the interaction between sewer systems and wwtps. Sewer system characteristics and operation affect the influent pollutograph, which affects the wwtp effluent quality. The sensitivity of wwtp performance to fluctuations in the influent flow and composition depends on the effluent quality parameter assessed. In addition, this sensitivity depends on the time varying conditions at the wwtp affecting the actual treatment capacity. Consequently, assessing the interactions within wastewater systems requires fully addressing the time varying response of both the sewer system and the wwtp to transient loading. Integrated models, widely described in literature, are often advocated as being capable of addressing the interactions within wastewater systems. Most integrated models, however, are built using available models for sewer systems and wwtps. Consequently, the level of detail and the processes incorporated in these models do not necessarily compare with the level of detail necessary to

properly take the interactions within wastewater systems into account. Moreover, a quantification of requirements for sewer and wastewater treatment models to be applied in an integrated assessment was not found in literature. Therefore, it was concluded that knowledge on the importance of the interactions within wastewater systems and a method to quantify this importance was lacking.
Based on literature, a problem-oriented approach was proposed to quantify the requirements for models to be able to analyse the interactions within wastewater systems. The approach applied in the following chapters of this thesis comprises:

- quantifying to what extent influent fluctuations, in terms of flow and wastewater quality parameters, have a significant effect on wwtp performance;
- deriving minimum requirements for sewer process models;
- analysing the extent in which current sewer process models are capable of meeting these requirements.

Chapter 3 Wastewater treatment and influent fluctuations
Chapter 3 analyses the impact of influent fluctuations on the quality of wwtp effluent. This analysis was based on data obtained via the 'routine' measuring program at wwtps, data obtained in a measuring period at wwtp 'Katwoude' and results of modelling the response of wwtp 'Katwoude' to influent fluctuations using an ASM1 model.
The data analysis learned that the influent and effluent data from 'routine' measurements at Dutch wwtps do not hold much information on the sensitivity of wwtp effluent quality to influent fluctuations. The only noticeable effect is the reduction of nitrogen removal efficiency due to increased wastewater flows. The COD concentration in the effluent does not seem to be affected by fluctuations in the influent.
Data measured at a high frequency, i.e. at a 15 minutes interval, clearly shows the impact of fluctuations in the influent on the effluent quality. A correlation analysis, however, resulted only in moderate correlation coefficients between the influent flow or influent ammonium load and the ammonium concentration in the effluent.
With respect to the modelling of the effluent quality with the ASM1 model, it is concluded that the ASM1 model is capable of reproducing the wwtp effluent quality with respect to nitrogen fractions under transient conditions. The ASM1 'Katwoude' model was fully calibrated with one single set of model parameter values for the complete measuring period from 19 September 2002 to 5 December 2002. Validation of the model with data from 23 February 2001 to 2 March 2001 showed that the model was well validated, especially with respect to nitrification.
The calibrated ASM1 model was used to analyse the requirements for the quality of influent data, provided by either measurements or sewer models. A sensitivity analysis with the 'Katwoude' wwtp model resulted in minimum requirements for sewer process models to be used within an integrated approach. The flow and the ammonium concentration in the influent are almost equally sensitive parameters with respect to wwtp effluent quality. The COD concentration is less important and therefore larger errors with respect to the quality of COD influent data can be accepted.

Chapter 4 Sewer process modelling
The capability of today's commercial software to meet the requirements, set in chapter 3, is discussed in chapter 4. Based on a comparison of model results with the results of measurements and experiments in sewer systems, the quality of model results was analysed for four main in-sewer processes: hydrodynamics, transport of solutes, transport of suspended material and aerobic transformations.
The quality of hydrodynamic modelling was analysed using the set of measurement data from the sewer system of Loenen, comprising 6 months of data on precipitation, water levels and pumping rates. Hydrodynamic models proved to be able to model the flow with a sufficient reliability, given the requirements for influent data developed in chapter 3.
The quality of the modelling of the transport of solutes was analysed using the data from tracer experiments in the sewer systems of Loenen and Beekbergen and a measuring

campaign in Ulvenhout. Today's sewer models showed to have a numerical dispersion higher than the observed physical dispersion. As a result, steep gradients in the concentration of solutes in the sewage will not be properly simulated. In reality, however, gradients in the concentration of solutes in sewer systems are at a level where the high numerical dispersion does not matter. The simulations of the ammonium concentration during a storm event measured in Ulvenhout show that the transport of solutes can, like the hydrodynamics, be described well enough to meet the requirements for influent data.
The modelling of the transport of suspended solids is still at its infancy. Currently, there is still much debate on the characteristics of the suspended solids and the main drivers describing the transport of these solids. An analysis using the well-calibrated hydrodynamic model of Loenen and the measuring results of the turbidity meters located in the sewer system of Loenen, showed that the mean shear stress is a very important driver for the transport and resuspension of suspended solids. Further research on this topic is highly recommended.
The aerobic conversions in sewer systems were analysed in the sewer system of Beekbergen during a 1-day measuring campaign. The aerobic conversions in a typical Dutch sewer had the same order of magnitude as reported in literature. In addition, an adjusted ASM1 model proved to be capable to properly reproduce the observed conversion rates. The variability of the concentrations of total COD and COD fractions in the sewage was, however, an order of magnitude higher than the observed conversions under summer conditions. This indicates that the aerobic conversions have only a relatively limited impact on the composition and fractionation of wwtp influent.

Chapter 5 Wastewater system optimisation: optimisation techniques
The incorporation of the knowledge on the interactions between sewer systems and wastewater treatment plants within the wastewater system optimisation studies complicates these studies as more parameters and potential solutions are involved. In order to enhance the incorporation of the knowledge on the interactions the applicability of two heuristic methods to wastewater system optimisation was discussed in chapter 5.
Both a genetic algorithm and simulated annealing proved to be capable of dealing with a typical wastewater system optimisation problem.
Using a genetic algorithm or simulated annealing inevitably results in a large number of simulations. In practice, this can be a significant drawback due to the potentially long simulation times. However, the results of all these simulations contain a lot of information on the sensitivity of the perceived optimal solution to e.g. the imposed standards. As such, performing a wastewater system optimisation study results in a better understanding of the performance of the wastewater system, which in itself may help improving wastewater system performance in daily engineering practice. Furthermore, this gives an idea of the robustness of the solution perceived optimal.

Chapter 6 Wastewater system performance and interactions
Wastewater treatment plants are subject to large variations in the pollutant load in the influent. An analysis of influent data and wastewater system characteristics showed that the volume of wastewater stored in the sewer system, e.g. in lost storage or pressure mains, at the onset of a storm event strongly affects the total loads arriving at the wwtp. Also operational aspects, such as pumping setpoints or pump failure may significantly affect the pollutant load in the influent.
In a case study, the impact of measures such as additional storage or pumping capacity, on total wastewater performance was analysed for the parameters CSO volume, ammonium, total nitrogen and biodegradable COD. The results confirm literature in showing that the optimal configuration of the wastewater system in terms of storage or pumping capacity depends on the parameter selected to assess wastewater system performance. This is due to the fact that increasing the storage and pumping capacity can have a detrimental impact on wwtp performance, especially with respect to nitrogen removal.
In addition, it was shown that the characteristics of a storm event determine the available optimisation potential of wastewater systems. For large storm events causing CSOs to spill,

a trade-off has to be found between reducing the pollution discharged via the CSO and the wwtp. For smaller storms, comprising the majority of storm events, wastewater system performance could easily be improved by temporarily reducing the pumping (or interceptor) capacity to a level just not causing the CSOs to spill. It has to be noted, however, that this is of course only one of the options available for wastewater system optimisation.
Moreover, it is illustrated that, due to the typically diurnal dwf profile, the time of the onset of a storm event is very important with respect to the total pollutant load discharged via both the wwtp effluent and the CSO.

Ideally, the optimal use, given the performance requirements, of available system capacity in terms of pumping and storage capacity can be determined for each storm event taking into account the varying (i.e. temperature dependent) biological treatment capacity of a wwtp, the current state of the sewer system and characteristics of the storm event into account. In practice, this will require predictive, probably model based, real time control.

Chapter 7 Conclusions and recommendations

The main conclusion is that the interactions are indeed important with respect to wastewater system performance and that current knowledge allows assessing these interactions. Wastewater treatment plant (wwtp) effluent quality is affected by quantitative as well as qualitative fluctuations in the influent, which are determined to a large extent by the characteristics of the sewer and wastewater transport systems. Consequently, the design and operation of sewer and wastewater transport systems are key elements for wastewater system optimisation as they do not only affect combined sewer overflow (CSO) discharges but also wwtp effluent quality.
Furthermore, it is concluded that the parameters used to assess wastewater system performance, such as CSO volume, ammonium, total nitrogen or chemical oxygen demand (COD), determine to a large extent the result of a wastewater system optimisation study.
Another interesting conclusion is that the characteristics of a storm event determine the available optimisation potential of wastewater systems.
Ideally, the optimal use, given the performance requirements, of available system capacity in terms of pumping and storage capacity is determined for each storm event taking into account the varying (i.e. temperature dependent) biological treatment capacity of a wwtp, the current state of the sewer system and characteristics of the storm event.
Therefore, it is recommended to start research into further developing strategies for pollution based real time control (RTC) of wastewater systems. This involves the development of a methodology for designing monitoring networks providing the data necessary for RTC and of sensors capable of meeting these requirements and suitable for installation in sewer systems.
Finally, it is recommended to further assess the impact of operational aspects on sewer system and wwtp performance in order to be able to fully take advantage of available system capacity.

Jeroen Langeveld, September 2004

Samenvatting 'Interacties binnen het afvalwatersysteem'

De belangrijkste aanleiding voor het onderzoeksproject 'Interacties binnen het afvalwatersysteem' waren de tekortkomingen van de gangbare volume gebaseerde aanpak van afvalwatersysteem optimalisatiestudies in Nederland. Deze aanpak neemt namelijk de dynamische (kwalitatieve) interacties binnen het afvalwatersysteem niet mee, waardoor de doorwerking van (operationele) aanpassingen aan de riolering op de werking van de afvalwaterzuivering buiten beschouwing blijft. Dit kan tot gevolg hebben dat maatregelen die nu in het kader van afvalwatersysteem optimalisaties worden genomen bij nader inzien toch niet zo aantrekkelijk zijn doordat zij onverwachte neveneffecten op het functioneren van een afvalwaterzuivering hebben. Daarnaast blijven interessante opties als sturing op waterkwaliteit binnen de huidige volume-gebaseerde aanpak buiten beschouwing.

Het doel van dit proefschrift is het analyseren van de mogelijkheden om de huidige volume gebaseerde aanpak voor afvalwatersysteem optimalisatie uit te breiden tot een aanpak gebaseerd op waterkwaliteit door de interacties binnen het afvalwatersysteem binnen beschouwing te nemen.

Hoofdstuk 1 Introductie en kader

Hoofdstuk 1 biedt een overzicht van de historische ontwikkeling van afvalwatersystemen in Nederland. Verder wordt ingegaan op de huidige aanpak van afvalwatersysteem optimalisatie en op de mogelijke rol van de interacties hierbinnen.

De geschiedenis van onze stedelijke afvalwatersystemen laat zien dat zij het resultaat zijn van ontwikkelingen op het gebied van technologie, organisatie en wetgeving gedurende meer dan een eeuw. Dientengevolge bestaat de huidige infrastructuur uit verschillende componenten die zijn aangelegd gedurende periodes met een soms totaal andere ontwerpfilosofie.

De technische levensduur van de gezondheidstechnische infrastructuur is een veelvoud van de levensduur van de eisen die worden gesteld aan het functioneren van afvalwatersystemen. Telkens wanneer de eisen worden verscherpt is het nodig om de afvalwatersystemen zodanig aan te passen dat ze weer naar behoren functioneren. Het aanpassen van een bestaand afvalwatersysteem vereist over het algemeen grote investeringen, waardoor het lonend wordt om met behulp van afvalwatersysteem optimalisatiestudies na te gaan waar kosten te besparen zijn.

De laatste ronde van afvalwatersysteem optimalisatie was volledig volume georiënteerd. Een meer holistische kijk leert echter dat een dergelijke aanpak onherroepelijk leidt tot suboptimale resultaten met betrekking tot bijvoorbeeld de totaal via het afvalwatersysteem geloosde vuilvracht of de impact op het ontvangende oppervlaktewater.

In dit proefschrift wordt vooruitgelopen op de introductie van een op waterkwaliteit gebaseerde afvalwatersysteem optimalisatie door de interacties binnen het afvalwatersysteem te onderzoeken.

Hoofdstuk 2 State of the art

Hoofdstuk 2 biedt een overzicht van de state of the art over de gehele breedte van de vakgebieden riolering en afvalwaterzuivering.

De vuilvracht die op het oppervlaktewater wordt geloosd door de riolering en afvalwaterzuivering heeft een duidelijk invloed op de oppervlaktewaterkwaliteit. Reductie van deze vuilvracht is daarom een belangrijke drijfveer voor afvalwatersysteem optimalisatie. Een complicerende factor hierbij vormt de interactie tussen riolering en afvalwaterzuivering. De eigenschappen en het operationeel beheer van de riolering beïnvloeden het verloop van het influentdebiet en de influentsamenstelling, die op hun beurt invloed hebben op de effluentkwaliteit. De gevoeligheid van het functioneren van een afvalwaterzuivering voor fluctuaties in het influent varieert per parameter. Bovendien is deze gevoeligheid afhankelijk van de in de loop van de tijd variërende werkelijk beschikbare zuiveringscapaciteit.

Dientengevolge vereist het analyseren van de interacties binnen het afvalwatersysteem het volledig in rekening brengen van de variabele respons van zowel de riolering als de afvalwaterzuivering op buien. In de literatuur wordt vaak verondersteld dat integrale modellen in staat zijn om de interacties binnen het afvalwatersysteem te analyseren. De meeste integrale modellen zijn echter slechts softwarematige koppelingen tussen bestaande modellen voor riolering en afvalwaterzuivering. Dit heeft tot gevolg dat de beschikbare integrale modellen vaak meer beloven dan ze waar kunnen maken doordat bijvoorbeeld belangrijke processen ontbreken. Een goede analyse van de eisen waaraan deelmodellen moeten voldoen voordat ze zodanig kunnen worden gekoppeld dat de interacties binnen het afvalwatersysteem op een juiste wijze in rekening kan worden gebracht ontbreekt in de literatuur.

De literatuur biedt daarentegen met de probleem georiënteerde aanpak wel een opstap om te komen tot dergelijke eisen. In dit onderzoek bestond de aanpak uit:

- kwantificeren in hoeverre influent fluctuaties van belang zijn voor het functioneren van een afvalwaterzuivering (hoofdstuk 3);
- afleiden van minimale eisen voor rioleringsmodellen om gebruikt te kunnen worden voor het analyseren van de interacties binnen het afvalwatersysteem (hoofdstuk 3);
- analyseren van de mate waarin huidige rioleringsmodellen in staat zijn aan deze eisen te voldoen (hoofdstuk 4).

Hoofdstuk 3 Afvalwaterzuivering en influentfluctuaties

Hoofdstuk 3 analyseert de doorwerking van influentfluctuaties op de effluentkwaliteit. Deze analyse is gebaseerd op meetgegevens afkomstig van het ‘routinematige’ meetprogramma op een aantal afvalwaterzuiveringen, op meetgegevens afkomstig van de meetperiode op AWZI ‘Katwoude’ en op de modelresultaten van het ASM1 model van AWZI ‘Katwoude’.

De routinematige meetgegevens bevatten niet veel informatie over de gevoeligheid van het functioneren van afvalwaterzuiveringen voor influentfluctuaties. Het enige duidelijke effect is een afname van het stikstof verwijderingsrendement met een toenemende influentdebiet. De CZV concentratie in the effluent leek niet te worden beïnvloed door fluctuaties in het influent.

De meetgegevens, die voor de meetperiode in Katwoude beschikbaar waren met een frequentie van 15 minuten, laten een duidelijke relatie zien tussen de influentfluctuaties en de effluentkwaliteit.

Met betrekking tot de kwaliteit van de modellering van de dynamiek op AWZI ‘Katwoude’ met behulp van het ASM1 model is geconcludeerd dat het ASM1 model prima in staat is om de effluentsamenstelling voor de verschillende stikstoffracties tijdens buien te reproduceren. Het ASM1 model voor AWZI ‘Katwoude’ kon voor de volledige meetperiode van 19 september 2002 tot 5 december 2002 worden gekalibreerd met slechts een parameterset. Een confrontatie van het gekalibreerde model met een tweede set meetgegevens van 23 februari 2001 tot 2 maart 2001 liet zien dat het model, zeker voor wat nitrificatie betreft, ook goed gevalideerd is.

Het gekalibreerde ASM1 model is gebruikt om eisen op te stellen waaraan influentgegevens, of zij nu afkomstig zijn van metingen of een rioleringsmodel, moeten voldoen om te kunnen worden gebruikt voor een analyse van de interacties binnen het afvalwatersysteem. Deze eisen zijn afgeleid van een gevoeligheidsanalyse met het ASM1 model voor ‘Katwoude’. Het influentdebiet en de concentratie ammonium in het influent zijn beiden gevoelige parameters met betrekking tot de effluentkwaliteit. De gevoeligheid voor variaties in CZV is aanzienlijk minder, waardoor voor CZV een grotere fout in de resultaten van een rioleringsmodel kan worden geaccepteerd.

Hoofdstuk 4 Modelleren van processen in de riolering

Hoofdstuk 4 onderzoekt de mogelijkheden van de huidige commercieel verkrijgbare software om aan de eisen, zoals die gesteld zijn in hoofdstuk 3, te voldoen. Op basis van een confrontatie van modelresultaten met metingen en de resultaten van experimenten in de riolering is de kwaliteit van de rioleringsmodellen geanalyseerd voor vier belangrijke processen in de riolering: hydrodynamica, transport van opgeloste stof, transport van gesuspendeerd materiaal en aërobe omzettingen.

De kwaliteit van de modellering van de hydrodynamica is onderzocht met behulp van de beschikbare metingen uit Loenen, waar gedurende zes maanden in 2001 de neerslag, de waterstanden en het verpompte debiet waren gemeten. Het hydrodynamische model bleek zoals verwacht het verloop van het debiet voldoende nauwkeurig te kunnen reproduceren om aan de eisen zoals gesteld in hoofdstuk 3 te voldoen.

De kwaliteit van de modellering van opgeloste stof is onderzocht op basis van de resultaten van tracer experimenten in de rioolstelsels van Loenen en Beekbergen en een meetcampagne in Ulvenhout. De huidige rioleringsmodellen vertonen een numerieke dispersie die groter is dan de werkelijk optredende dispersie. Hierdoor is het niet mogelijk om hoge gradiënten in de concentratie opgeloste stof goed te simuleren. In de praktijk komen dergelijke hoge gradiënten in de concentratie niet voor, waardoor het probleem van de te hoge numerieke dispersie met betrekking tot de interacties niet ernstig is. Het gebruikte Hydroworks model bleek in staat om het tijdens een bui in Ulvenhout gemeten verloop van de ammoniumconcentratie te reproduceren met een afwijking die binnen de eisen valt die in hoofdstuk 3 aan de kwaliteit van de modelresultaten is gesteld.

Het modelleren van het transport van gesuspendeerd materiaal staat nog in de kinderschoenen. In de literatuur bestaat nog veel discussie over de eigenschappen van gesuspendeerd materiaal en de belangrijkste parameters die het transport van dit gesuspendeerd materiaal kunnen beschrijven. Op basis van een analyse met behulp van het goed gekalibreerde hydrodynamische model van Loenen en de eveneens in Loenen gemeten troebelheid is aangetoond dat de gemiddelde schuifspanning een belangrijke parameter is met het oog op het transport en de resuspensie van gesuspendeerd materiaal. Nader onderzoek naar dit onderwerp is zeer gewenst.

De aërobe omzettingen in rioolstelsels zijn geanalyseerd in het rioolsysteem van Beekbergen tijdens een 1 daags meetprogramma. De aërobe omzettingen in dit typisch Nederlandse riool hebben dezelfde orde van grootte als gerapporteerd in de literatuur. Daarnaast bleek het redelijk goed mogelijk om de gemeten omzettingen met behulp van een voor de riolering aangepast ASM1 model te reproduceren. De variabiliteit van de concentraties (van verschillende fracties van) het CZV lag hierbij een orde van grootte hoger dan de onder zomerse condities geobserveerde omzettingssnelheid. Dit houdt in dat de aerobe omzettingen slechts een beperkte invloed hebben op de variaties in samenstelling van het influent.

Hoofdstuk 5 Afvalwatersysteem optimalisatie: optimalisatietechnieken

Het incorporeren van kennis over de interacties binnen het afvalwatersysteem heeft een complicerend effect op afvalwatersysteem optimalisatiestudies doordat meer parameters en type oplossingen aan bod komen. Om uit het enorme aantal mogelijke verbeteringsmaatregelen het optimale maatregelenpakket samen te kunnen stellen kan gebruik gemaakt worden van daartoe geschikte optimalisatiealgoritmes. In hoofdstuk 5 worden de mogelijkheden voor het toepassen van een tweetal heuristische optimalisatietechnieken binnen afvalwatersysteem optimalisatiestudies verkend.

Zowel een genetisch algoritme als simulated annealing blijken in staat om een typisch afvalwatersysteem optimalisatie vraagstuk aan te pakken.

Het gebruik van beide algoritmes leidt tot een groot aantal simulaties, hetgeen in de praktijk voor problemen kan zorgen door de, afhankelijk van de gebruikte modellen, lange rekentijden. De resultaten van al deze simulaties bevatten echter wel een grote hoeveelheid informatie over de robuustheid van de geselecteerde oplossing voor bijvoorbeeld de gebruikte randvoorwaarden, zoals de normen waarop het functioneren van het systeem wordt getoetst. Het uitvoeren van een optimalisatie kan zodoende leiden tot een beter inzicht in de samenhang binnen het afvalwatersysteem en de gevoeligheid voor externe randvoorwaarden.

Hoofdstuk 6 Functioneren van afvalwatersystemen en interacties

De belasting van afvalwaterzuiveringen kan sterk variëren. Een analyse van influentdata en afvalwatersysteem karakteristieken toonde aan dat de hoeveelheid afvalwater die aan het begin van een bui in de riolering aanwezig is in bijvoorbeeld dode berging of persleidingen van grote invloed is op de variaties in de influentvracht. Daarnaast spelen ook operationele aspecten, zoals in- en uitslagpeilen van pompen en gemaalstoringen een grote rol.

In een casestudy is het effect van maatregelen zoals extra berging of pompcapaciteit op het functioneren van het gehele afvalwatersysteem geanalyseerd voor de parameters overstortingsvolume, ammonium, totaal stikstof en afbreekbaar CZV. De resultaten laten zien dat de optimale configuratie van een afvalwatersysteem met betrekking tot berging of pompcapaciteit afhankelijk is van de parameter op basis waarvan het functioneren van het afvalwatersysteem wordt beoordeeld. Dit is toe te schrijven aan het feit een toename van de bergings- of pompcapaciteit een negatieve invloed kan hebben op het functioneren van een afvalwaterzuiveringsinstallatie, in het bijzonder op de stikstofverwijdering.

Daarnaast is aangetoond dat de eigenschappen van een bui het beschikbare potentieel voor optimalisatie bepalen. Voor grote buien die een overstorting tot gevolg hebben moet een afweging worden gemaakt tussen het lozen via de overstort en via de afvalwaterzuivering. Voor kleinere buien, die de meerderheid van de buien vormen, kan het functioneren van het afvalwatersysteem relatief eenvoudig worden verbeterd door tijdelijk de pompcapaciteit zodanig te verlagen dat juist geen overstorting optreedt. Hierbij moet worden aangetekend dat dit slechts een van de vele mogelijkheden is om het functioneren van het afvalwatersysteem te verbeteren.

Bovendien laten de resultaten zien dat het tijdstip waarop een bui plaatsvindt van grote invloed is op de totale vuilvracht die via de overstort en de afvalwaterzuivering wordt geloosd. Dit verschijnsel wordt veroorzaakt doordat de droogweerafvoer doorgaans een kenmerkend 24-uurs profiel vertoont.

Hoofdstuk 7 Conclusies en aanbevelingen

De hoofdconclusie is dat de interacties binnen het afvalwatersysteem daadwerkelijk van belang zijn voor het functioneren van het afvalwatersysteem en dat de huidige proceskennis toereikend is om de invloed van deze interacties te analyseren. De effluentkwaliteit van afvalwaterzuiveringen wordt beïnvloed door zowel variaties in het influentdebiet als in de influentsamenstelling. Deze variaties worden op hun beurt in belangrijke mate beïnvloed door de eigenschappen van de riolering en de afvalwatertransportsystemen. Het ontwerp en het operationeel beheer van de riolering en de transportstelsels zijn daardoor zeer belangrijke variabelen binnen een afvalwatersysteem optimalisatie, aangezien zij niet alleen van invloed zijn op de via de overstort geloosde vuilvracht, maar zeker ook op de via het effluent geloosde vuilvracht.

Daarnaast is geconcludeerd dat de parameter op basis waarvan het functioneren van een afvalwatersysteem wordt bepaald, zoals het overstortingsvolume, ammonium, totaal stikstof en CZV, voor een belangrijk deel het resultaat van een afvalwatersysteem optimalisatiestudie bepaalt. Een andere interessante conclusie is dat de eigenschappen van een bui het beschikbare potentieel voor optimalisatie bepalen.

Idealiter wordt per bui bepaald in hoeverre de beschikbare systeemcapaciteit, zoals berging en pompcapaciteit, moet worden benut, waarbij de variërende (bijvoorbeeld met temperatuur) biologische zuiveringscapaciteit, de actuele staat van het rioolstelsel en de eigenschappen van de bui in de overweging worden meegenomen.

Om dit ideaalbeeld te verwezenlijken wordt aanbevolen om nader onderzoek te verrichten naar het verder ontwikkelen van waterkwaliteit gebaseerde sturingsstrategieën voor afvalwatersystemen. Dit vereist onder meer de ontwikkeling van een methodologie voor het ontwerpen van netwerken voor het monitoren van afvalwatersystemen en een verdere ontwikkeling van sensoren die geschikt zijn om de vereiste data in de agressieve rioolomgeving te verzamelen.

Tenslotte wordt aanbevolen om een nadere analyse uit te voeren van het belang van operationele aspecten op het functioneren van de riolering en de afvalwaterzuivering om zo onder alle omstandigheden de beschikbare systeemcapaciteit maximaal te benutten.

Jeroen Langeveld, September 2004

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Curriculum Vitae

Jeroen Gerardus Langeveld was born on the 7th of December 1972 in Delft, the Netherlands. After graduating from the secondary school (VWO-B) at Scholengemeenschap Spieringshoek in Schiedam in 1991, he started his study Civil Engineering at Delft University of Technology. He graduated in 1997 at the section of Sanitary Engineering after defending his MSc thesis on the reuse of grey water and use of storm runoff in households. In 1995 he was chairmen of the students association 'Sanitary engineering and Water management'. From 1995, he was employed as student assistant at the section of Sanitary Engineering, where he was employed from 1997 as researcher to assist in teaching and later on in 1998 to do research on the topic of urban water.

In 1999 he continued his career at Delft University of Technology with the research project underlying this thesis 'Interactions within the wastewater system'. The project was funded by a consortium of Arcadis with its two partners HKV and Vertis, Grontmij and Witteveen+Bos. In addition to this research project he has done a succesfull sanitation project in Djenné, Mali. In 2003 he participated in the steering committee of the Dutch guideline on the optimisation of wastewater systems (module B1000 Leidraad Riolering) and of the STOWA project 'Infiltration and inflow'. In the period from 1997 to 2004 he was involved in the organisation of a number of symposia and post academic courses in the field of wastewater systems.

Since June 2004 he is employed at Royal Haskoning, Nijmegen, as consultant in the field of wastewater systems.

Appendix I Sensitivity of effluent quality to influent fluctuations

This appendix gives the sensitivity of effluent quality to influent fluctuations for 2 Dutch wwtps, wwtp Wervershoof a low loaded wwtp with a design capacity of 219,000 p.e. and wwtp Utrecht, a 2 stage wwtp with a high loaded first stage (1.9 g BOD/g MLSS) and a low loaded second stage (0.08 g BOD/MLSS) and a design capacity of 530,000 p.e.. The data for wwtp Wervershoof are from 1999 and 2000, for wwtp Utrecht from the years 1995-1996.

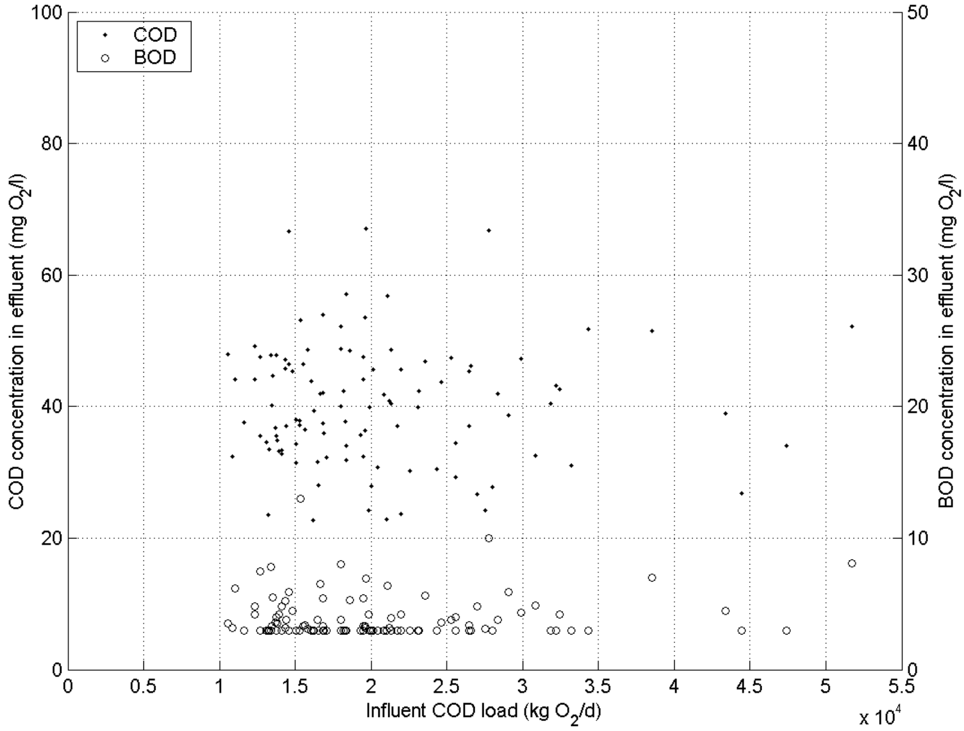


Figure I.1 Wwtp Wervershoof. Influent COD load vs effluent COD and BOD concentration. Average dwf loading is 17,000 kg COD/d.

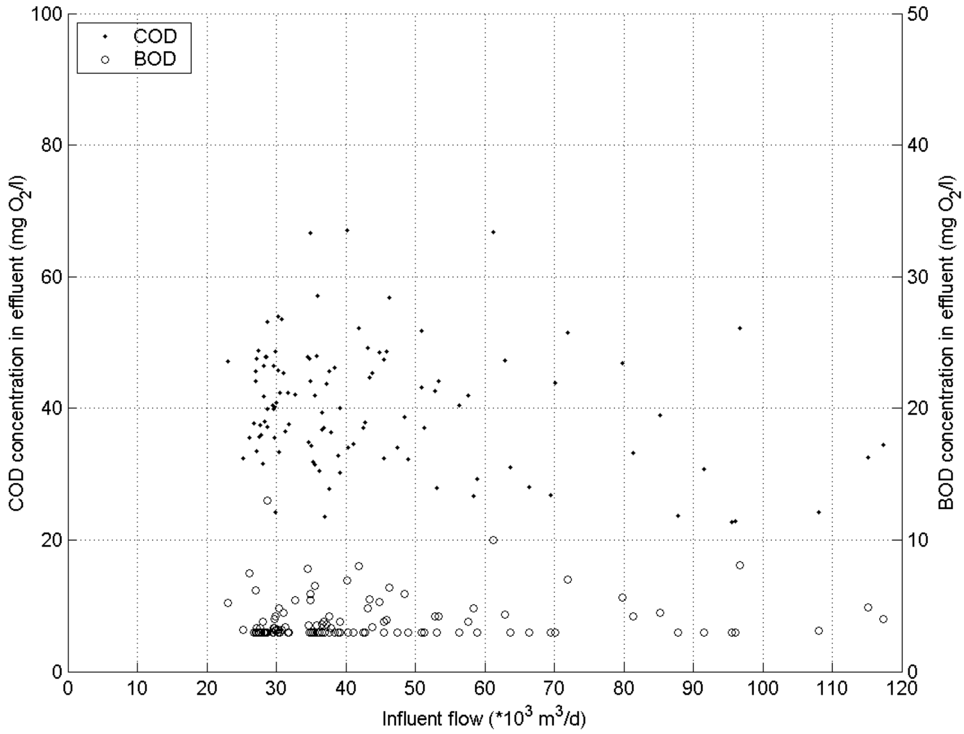


Figure I.2 Wwtp Wervershoof. Influent flow vs effluent COD and BOD concentration. Average dwf is 35,000 m^3/d.

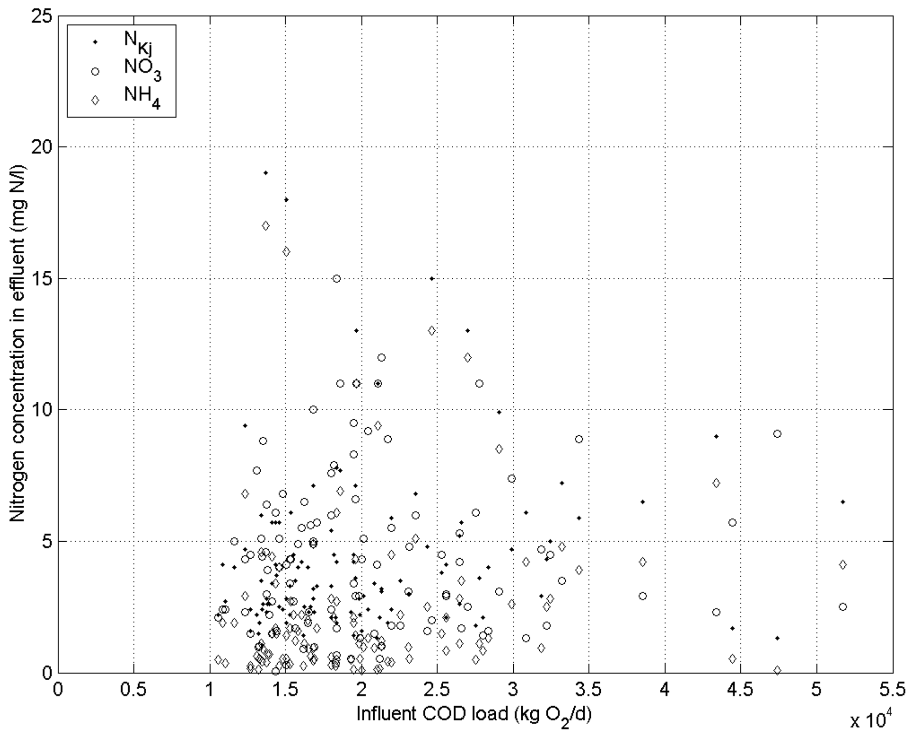


Figure I.3 Wwtp Wervershoof. Influent COD load vs effluent nitrogen concentrations. Average dwf loading is 17,000 kg COD/d.

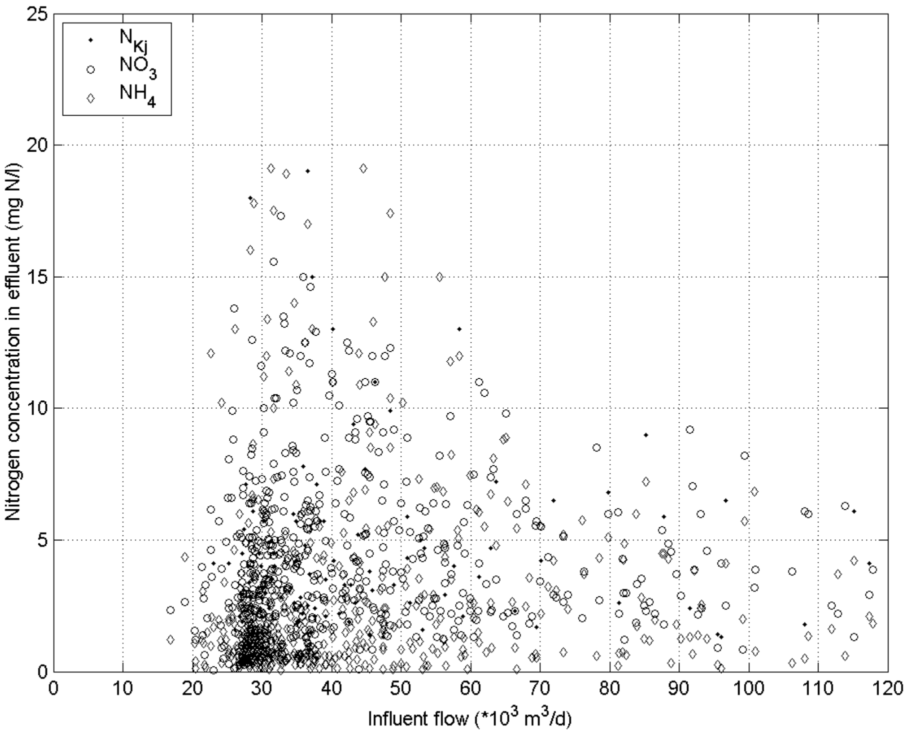


Figure I.4 Wwtp Wervershoof. Influent flow vs effluent nitrogen concentration. Average dwf is 35,000 m^3/d.

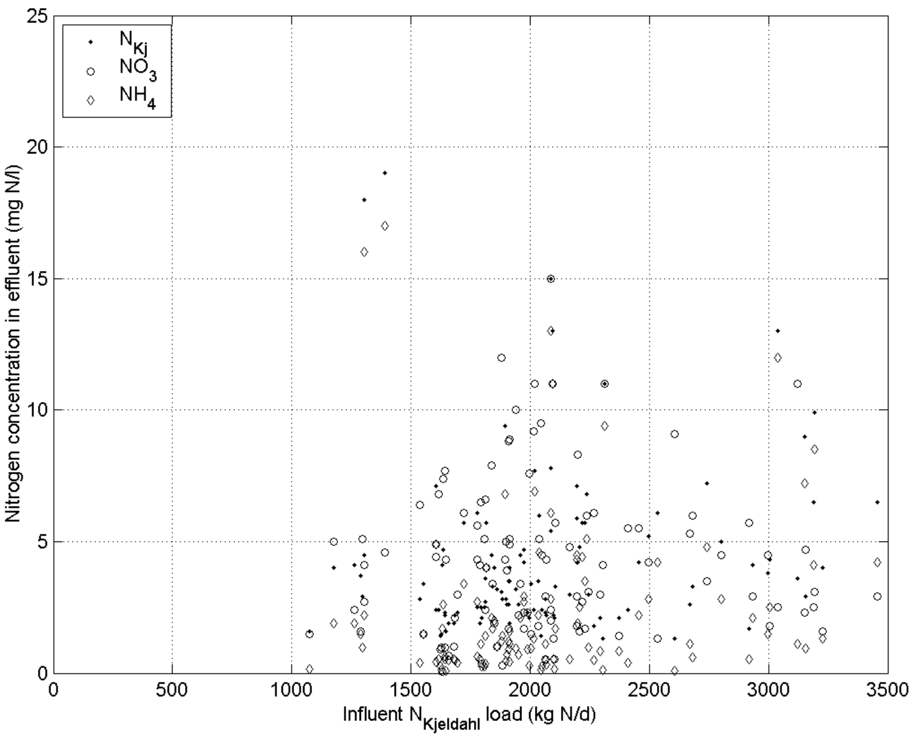


Figure I.5 Wwtp Wervershoof. Influent nitrogen load flow vs effluent nitrogen concentration. Average dwf loading is 1,700 kg N/d.

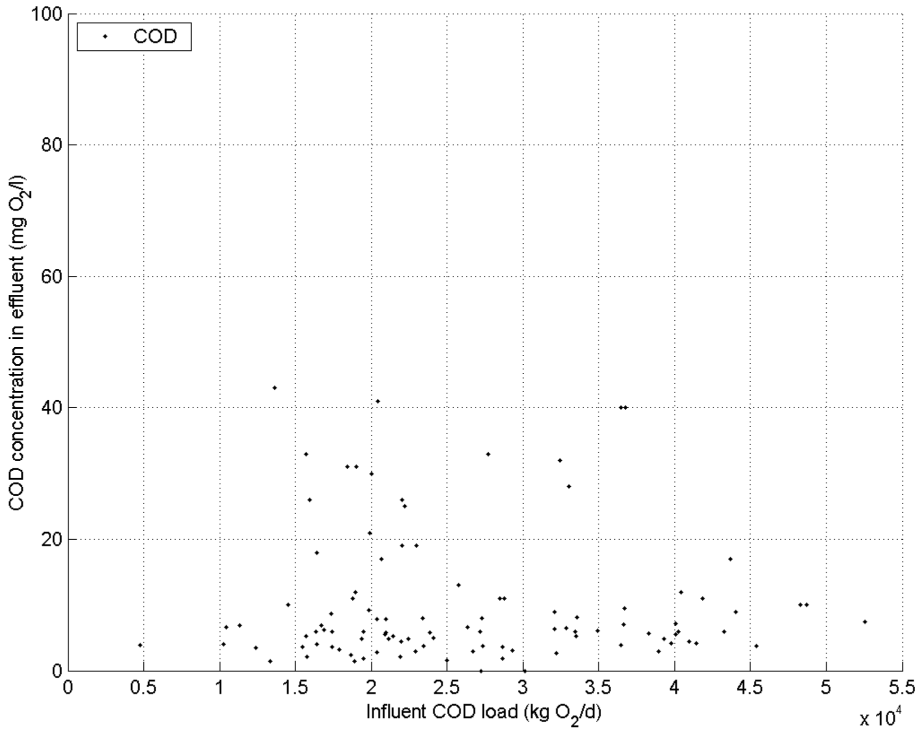


Figure I.6 Wwtp Utrecht. Influent COD load vs effluent COD and BOD concentration. Average dwf loading is 21,000 kg COD/d.

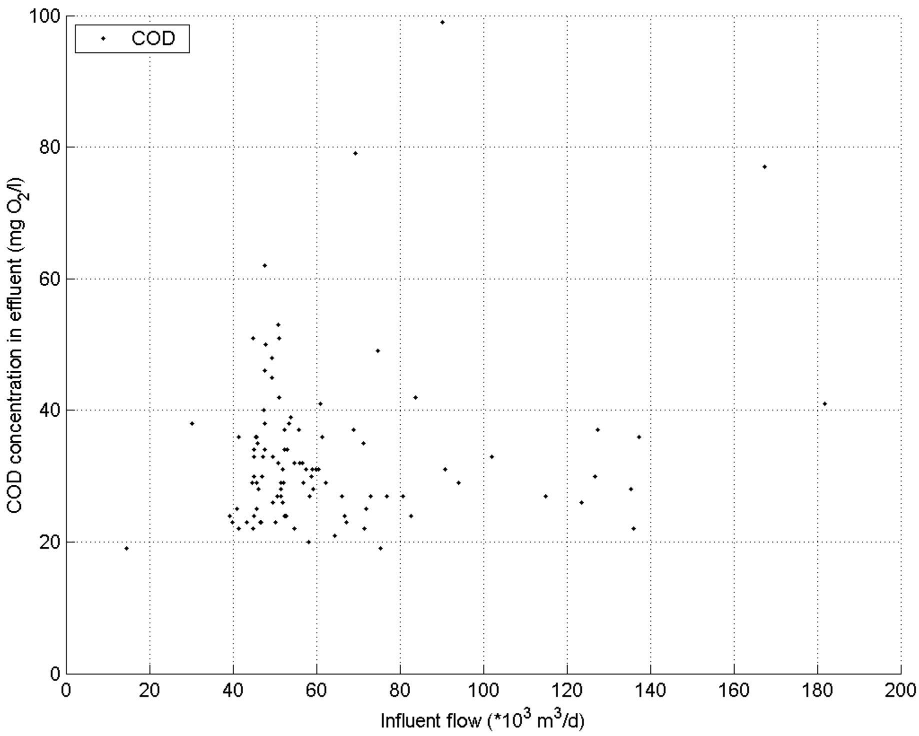


Figure I.7 Wwtp Utrecht. Influent flow vs effluent COD and BOD concentration. Average dwf is 45,000 m^3/d

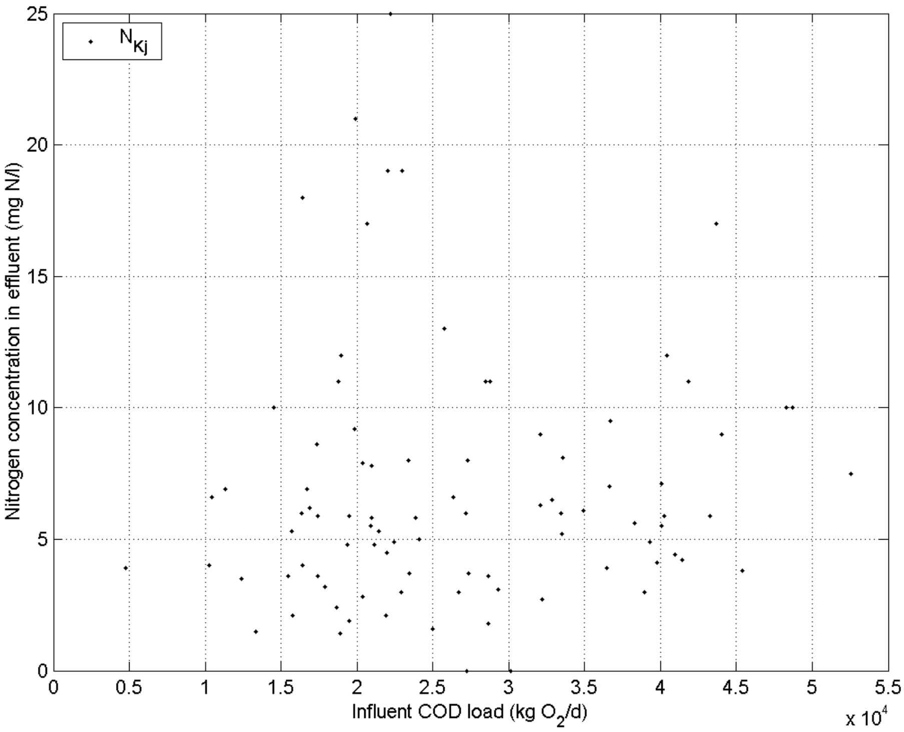


Figure I.8 Wwtp Utrecht. Influent COD load vs effluent nitrogen concentrations. Average dwf loading is 21,000 kg COD/d.

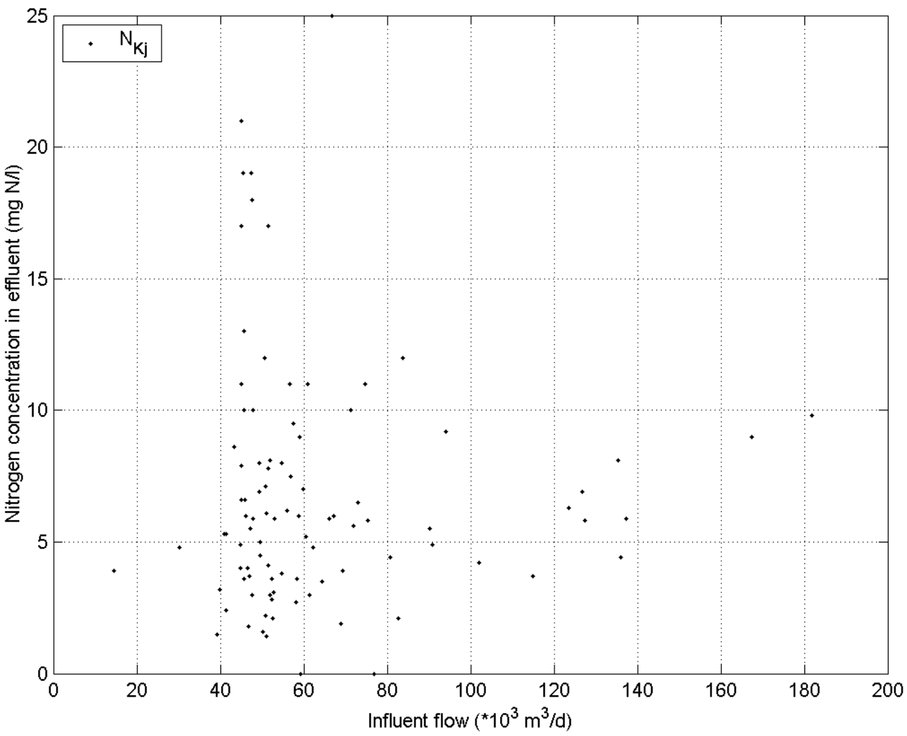


Figure I.9 Wwtp Utrecht. Influent flow vs effluent nitrogen concentration. Average dwf is 45,000 m^3/d.

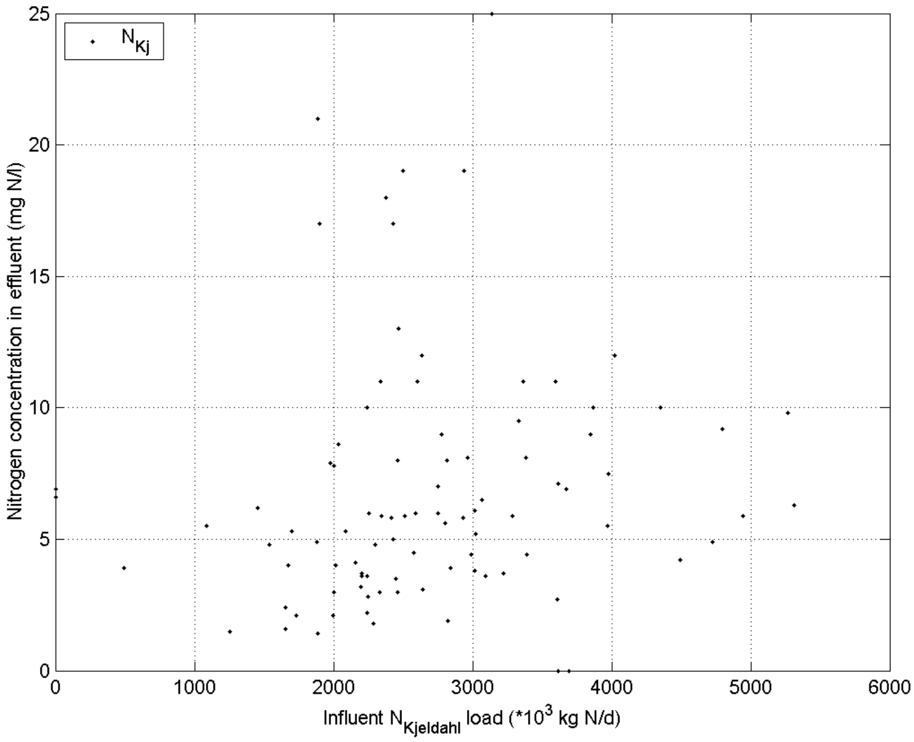


Figure I.10 Wwtp Utrecht. Influent nitrogen load flow vs effluent nitrogen concentration. Average dwf loading is 2,000 kg N/d.

Appendix II ASM 1 model

Table II.1 Process kinetics and stoichiometry [from: Henze et al. (1987)].

j	Component → i / Process ↓	1 S_I	2 S_S	3 X_I	4 X_S	5 $X_{B,H}$	6 $X_{B,A}$	7 X_P	8 S_O	9 S_{NO}	10 S_{NH}	11 S_{ND}	12 X_{ND}	13 S_{ALK}	Process Rate, ρ_j $[ML^{-3}T^{-1}]$
1	Aerobic growth of heterotrophs		$-\frac{1}{Y_H}$			1			$-\frac{1-Y_H}{Y_H}$		$-i_{XB}$			$-\frac{i_{XB}}{14}$	$\hat{\mu}_H\left(\frac{S_S}{K_S+S_S}\right)\left(\frac{S_O}{K_{O,H}+S_O}\right)X_{B,H}$
2	Anoxic growth of heterotrophs		$-\frac{1}{Y_H}$			1				$-\frac{1-Y_H}{2.86Y_H}$	$-i_{XB}$			$\frac{1-Y_H}{14\cdot 2.86Y_H}-i_{XB}/14$	$\hat{\mu}_H\left(\frac{S_S}{K_S+S_S}\right)\left(\frac{K_{O,H}}{K_{O,H}+S_O}\right)\times\left(\frac{S_{NO}}{K_{NO}+S_{NO}}\right)\eta_g X_{B,H}$
3	Aerobic growth of autotrophs						1		$-\frac{4.57-Y_A}{Y_A}$	$\frac{1}{Y_A}$	$-i_{XB}-\frac{1}{Y_A}$			$-\frac{i_{XB}}{14}-\frac{1}{7Y_A}$	$\hat{\mu}_A\left(\frac{S_{NH}}{K_{NH}+S_{NH}}\right)\left(\frac{S_O}{K_{O,A}+S_O}\right)X_{B,A}$
4	'Decay' of heterotrophs				$1-f_P$	-1		f_P					$i_{XB}-f_P i_{XP}$		$b_H X_{B,H}$
5	'Decay' of autotrophs				$1-f_P$		-1	f_P					$i_{XB}-f_P i_{XP}$		$b_A X_{B,A}$
6	Ammonification of soluble organic nitrogen										1	-1		$\frac{1}{14}$	$k_a S_{ND} X_{B,H}$
7	'Hydrolysis' of entrapped organics		1		-1										$k_h \frac{X_S/X_{B,H}}{K_X+(X_S/X_{B,H})}\left[\left(\frac{S_O}{K_{O,H}+S_O}\right)+\eta_h\left(\frac{K_{O,H}}{K_{O,H}+S_O}\right)\left(\frac{S_{NO}}{K_{NO}+S_{NO}}\right)\right]X_{B,H}$
8	'Hydrolysis' of entrapped organic nitrogen											1	-1		$\rho_7(X_{ND}/X_S)$
	Observed Conversion Rates $[ML^{-3}T^{-1}]$	$r_i=\sum_j \nu_{ij}\rho_j$													
	Stoichiometric Parameters: Heterotrophic yield: Y_H; Autotrophic yield: Y_A; Fraction of biomass yielding particulate products: f_P; Mass N/Mass COD in biomass: i_{XB}; Mass N/Mass COD in products from biomass: i_{XP}	Soluble inert organic matter [M(COD)L^{-3}]	Readily biodegradable substrate [M(COD)L^{-3}]	Particulate inert organic matter [M(COD)L^{-3}]	Slowly biodegradable substrate [M(COD)L^{-3}]	Active heterotrophic biomass [M(COD)L^{-3}]	Active autotrophic biomass [M(COD)L^{-3}]	Particulate products arising from biomass decay [M(COD)L^{-3}]	Oxygen (negative COD) [M(−COD)L^{-3}]	Nitrate and nitrite nitrogen [M(N)L^{-3}]	$NH_4^+ + NH_3$ nitrogen [M(N)L^{-3}]	Soluble biodegradable organic nitrogen [M(N)L^{-3}]	Particulate biodegradable organic nitrogen [M(N)L^{-3}]	Alkalinity – Molar units	Kinetic Parameters: Heterotrophic growth and decay: $\hat{\mu}_H$, K_S, $K_{O,H}$, K_{NO}, b_H; Autotrophic growth and decay: $\hat{\mu}_A$, K_{NH}, $K_{O,A}$, b_A; Correction factor for anoxic growth of heterotrophs: η_g; Ammonification: k_a; Hydrolysis: k_h, K_X; Correction factor for anoxic hydrolysis: η_h

Table II.2 Model parameters ASM1 [Henze et al. (1987)].

parameter and definition		unit
S_i	inert soluble COD	mg O_2/l
S_s	readily biodegradable substrate	mg O_2/l
X_i	inert particulate matter	mg O_2/l
X_s	slowly biodegradable substrate	mg O_2/l
X_{bh}	active heterotrophic biomass	mg O_2/l
X_{ba}	active autotrophic biomass	mg O_2/l
X_p	particulate decay products	mg O_2/l
S_O	Oxygen	mg O_2/l
S_{NO}	nitrate and nitrite	mg N/l
S_{NH}	NH_4^+ and NH_3 nitrogen	mg N/l
S_{ND}	soluble biodegradable organic nitrogen	mg N/l
X_{ND}	particulate biodegradable organic nitrogen	mg N/l
S_{ALK}	Alkalinity	mg N/l
Q	Flow	m^3/d

Table II.3 Standard values for kinetic and stoichiometric parameters for ASM1 [STOWA (2000)].

symbol and definition		default	range	Unit
Y_H	heterotrophic yield	0.67	0.46-0.69	gCOD/gCOD
Y_A	autotrophic yield	0.24	0.07 – 0.28	gCOD/gN
i_{XB}	nitrogen mass per mass of COD in biomass	0.086		gN/gCOD
i_{XP}	nitrogen mass per mass of COD in products from biomass	0.06		gN/gCOD
f_p	fraction of biomass leading to particulate products	0.08	0.08 – 0.20	-
μ_H	maximum specific growth rate for heterotrophic biomass	6	3.0-13.2	d^{-1}
K_S	half-saturation coefficient for heterotrophic biomass	20	10-180	gCOD/m^3
K_{OH}	oxygen half-saturation coefficient for heterotrophic biomass	0.2	0.01-0.20	gO_2/m^3
K_{NO}	nitrate half-saturation coefficient for heterotrophic biomass	0.5		gNO_3/m^3
b_H	decay coefficient for heterotrophic biomass	0.62	0.05-1.6	d^{-1}
b_A	decay coefficient for autotrophic biomass	0.15		d^{-1}
η_g	correction factor for μ_H under anoxic conditions	0.8		-
$\eta_{h,}$, η_{NO3}	correction factor for hydrolysis under anoxic conditions	0.4	0.6-1.0	-
k_h	maximum specific hydrolysis rate	3	1.0-3.0	d^{-1}
K_X	half-saturation coefficient for hydrolysis of slowly biodegradable substrate	0.03	0.01-0.03	gCOD/gCOD
μ_A	maximum specific growth rate for autotrophic biomass	0.8	0.34-0.80	d^{-1}
K_{NH4}	ammonia half-saturation coefficient for autotrophic biomass	1.0		gNH_3-N/m^3
K_{OA}	oxygen half-saturation coefficient for autotrophic biomass	0.4		gO_2m^3
K_a	ammonification rate	0.08		m^3 COD/g.d.

Appendix III Danfoss Sensor specifications

Table III.1 Specifications Danfoss INSITU 4100 Transmitter.

Transmitter	**Ammonium NH_4-N**		**Orthophosphate PO_4-P**	
Measuring range (mg/l)	0-20	0-100	0-6	0-15
Standard method	Indophenol blue method		Molybdenum blue method	
Measuring uncertainty	0.6-20 mg/l ±10% of actual concentration <0.6 mg/l: ±0.06 mg/l	1-100 mg/l: ±10% of actual concentration <1 mg/l: ± 1 mg/l	0.3-6 mg/l ±10% of actual concentration <0.3 mg/l: ±0.03 mg/l	2.4-15 mg/l ±10% of actual concentration <2.4 mg/l: ±0.24 mg/l
Response time	10 min.	24 min.	15 min.	15 min.
Measuring interval	Continuous measurement			
Current output (passive)	4-20 mA (scaleable by HART®) galvanic isolated. Max. load 750 ohm @ 30 V d.c.			
Transmitter cable	4 metres shielded cable (8 mm) with IP 67 plug			
Enclosure rating	IP 68 to IEC 529			
Ambient temperature	Storage: 0 to +50°C, Operation: Air: -20 to +40°C, Medium: 0 to +35°C			
Power supply	20-28 V d.c. 6A			
Automatic calibration	Every 72 hours (user selectable) with internal standards			
Change of reagents	Every 10 weeks			
Approvals	CE and C-tick approval, Emmision: EN 50081, Immunity: EN 61000-6-2			
Enclosure material	PUR and stainless steel (AISI 316)			
Weight/size	15 kg/height: 600 mm, diameter 350 mm			

Table III.2 Specifications Danfoss INSITU 4100 Transmitter.

Transmitter	**Nitrate and Nitrite NO_x-N**
Measuring range (mg/l)	0-10 mg/l
Measuring principle	UV absorption
Measuring uncertainty	0.6-10 mg/l: ± 10% of actual concentration <2 mg/l: +/- 0.2 mg/l
Response time	15 min.
Measuring interval	Continuous measurement
Current output (passive)	4-20 mA (scaleable by HART®) galvanic isolated. Max. load 750 ohm @ 30 V d.c.
Transmitter cable	10 metres (2x1.5 mm² + 2x0.36 mm2 shielded cable
Enclosure rating	IP 68 IEC 529
Ambient Temperature	Storage: 0 to +50°C, Operation: Air: -10 to +35°C, Medium: 2 to + 30°C
Power supply	20-28 V d.c. 2A
Interval of membrane exchange	Every 3 months, depending on application
Interval of carrier exchange	Every 3 to 6 months depending on measuring mode
Approvals	CE, Emission: EN 50081; Immunity EN 61000-6-2
Materials	PBT/PC/PPS
Size	Diameter: 300 mm
Weight	6 kg with liquids, 5 kg without liquids

Appendix IV SIMBA protocol

This appendix gives an overview of the SIMBA protocol, applied in chapter 3 for the calibration of the ASM1 wwtp Katwoude model. The figures are reproduced with permission from [Hulsbeek et al. (2002)], who summarised the original Dutch STOWA report [STOWA (2000)].

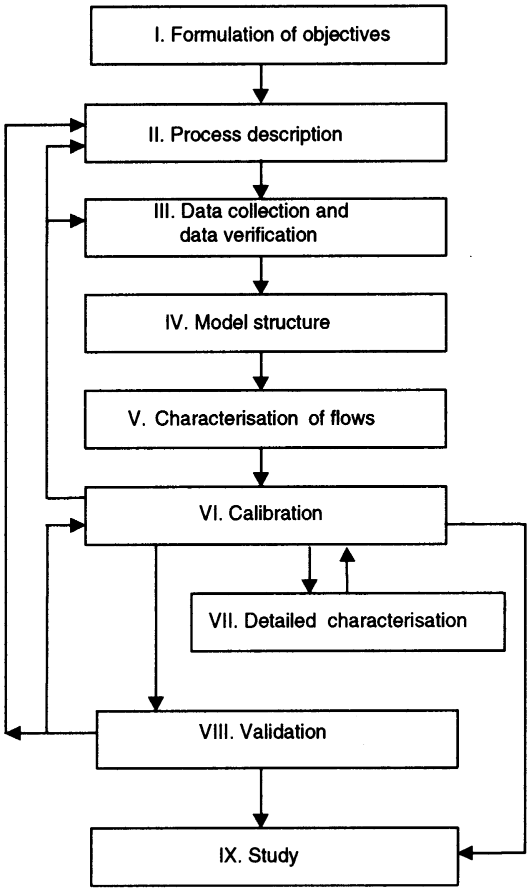


Figure IV.1 Structure of SIMBA protocol.

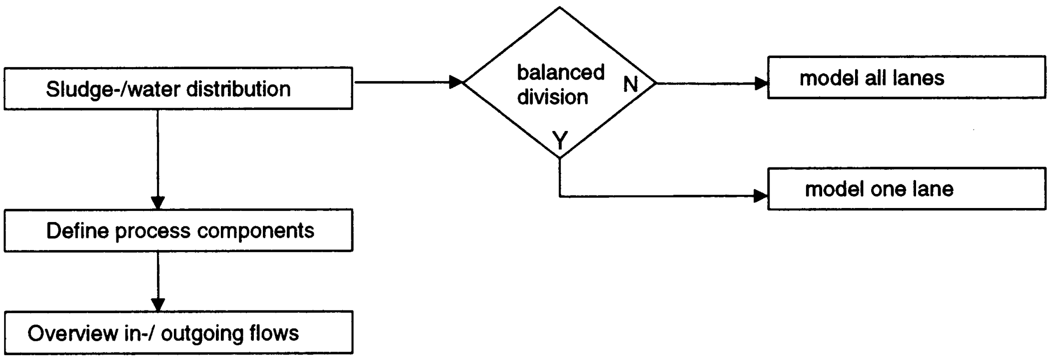


Figure IV.2 Phase II Process description.

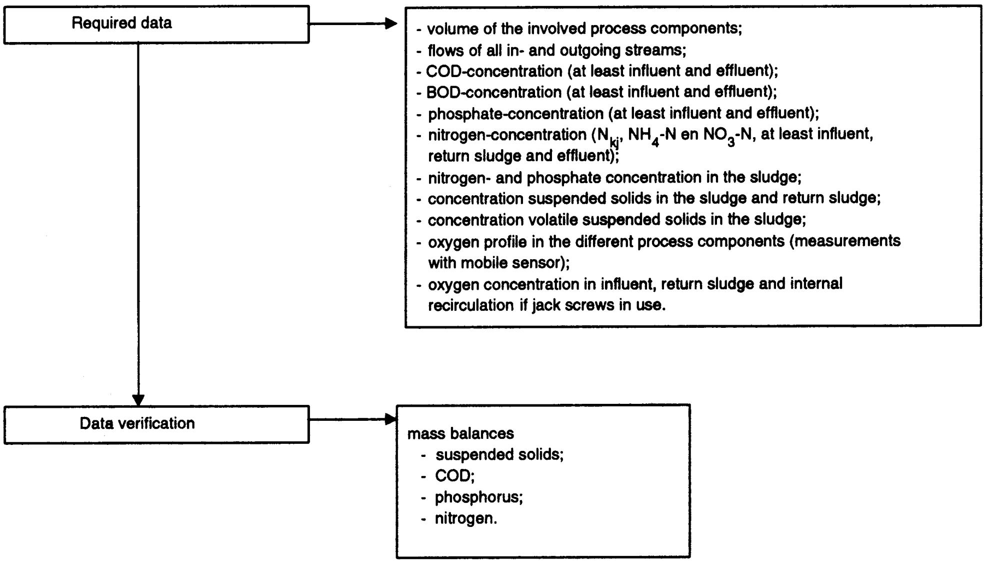


Figure IV.3 Phase III. Data collection and verification.

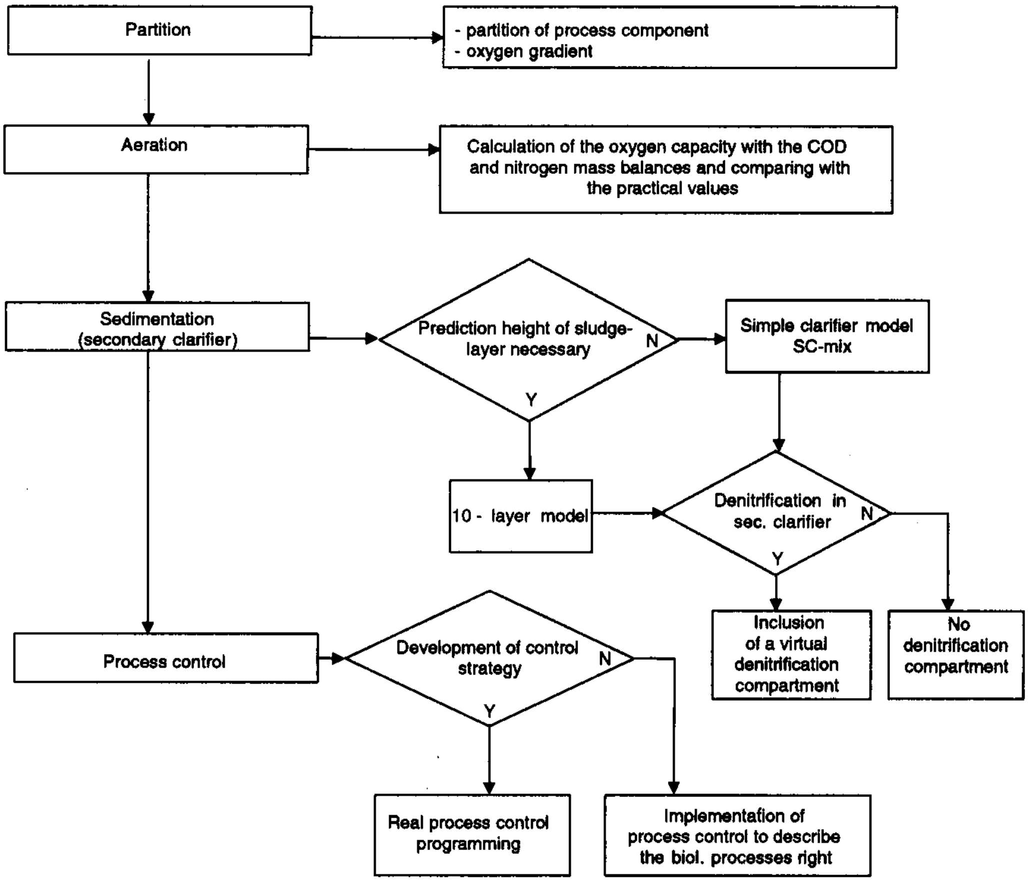


Figure IV.4 Phase IV. Model structure.

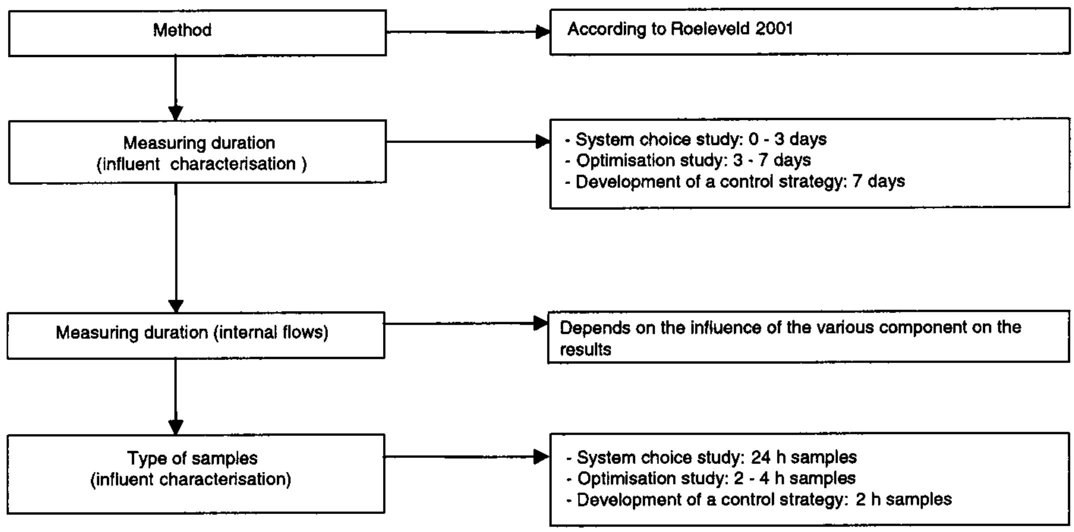


Figure IV.5 Phase V. Characterisation of flows.

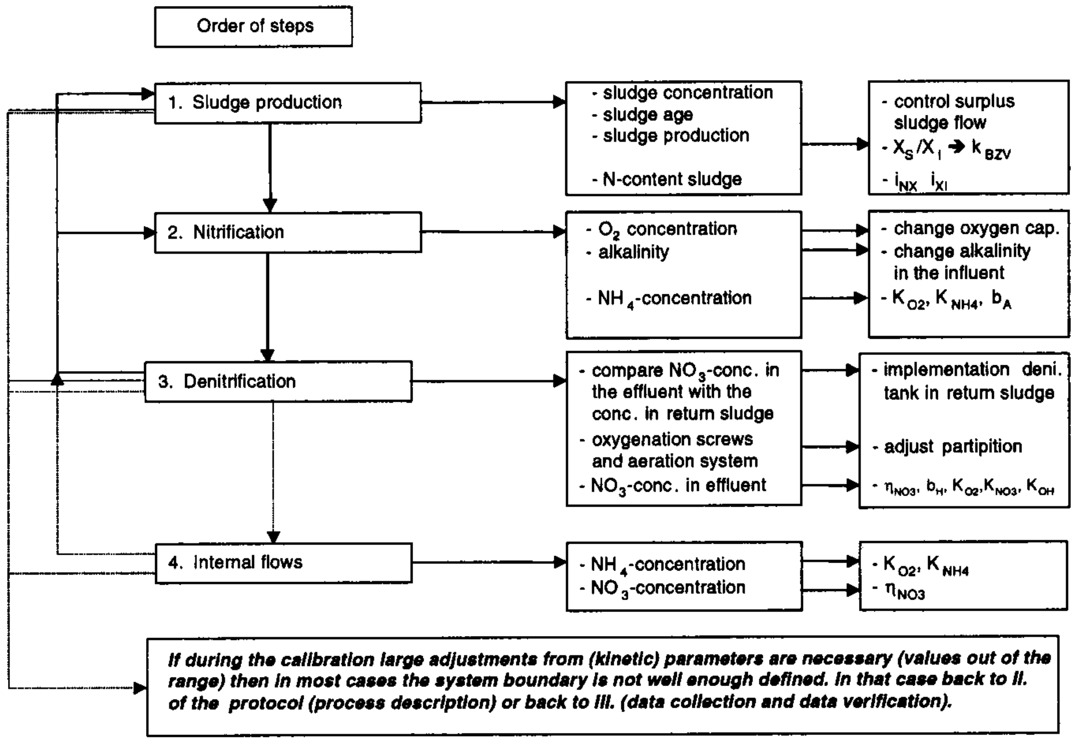


Figure IV.6 Phase VI. Calibration.

Appendix V SIMBA model wwtp Katwoude

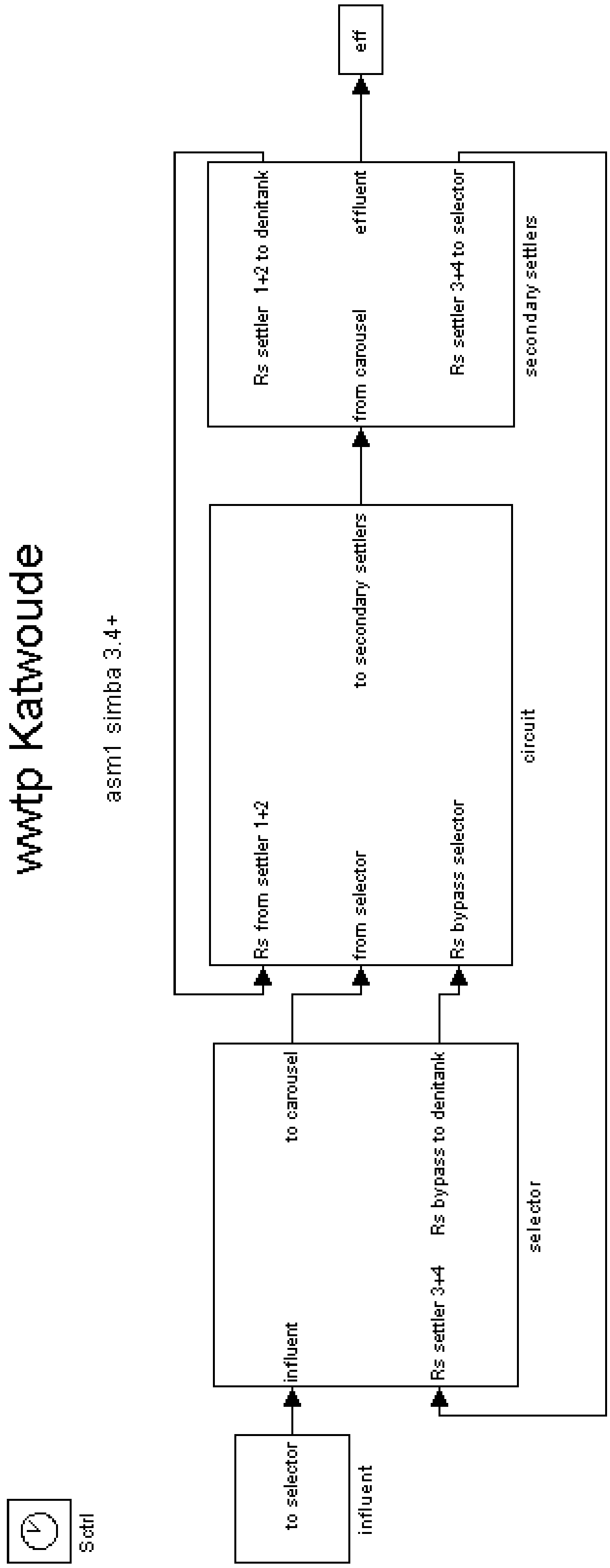
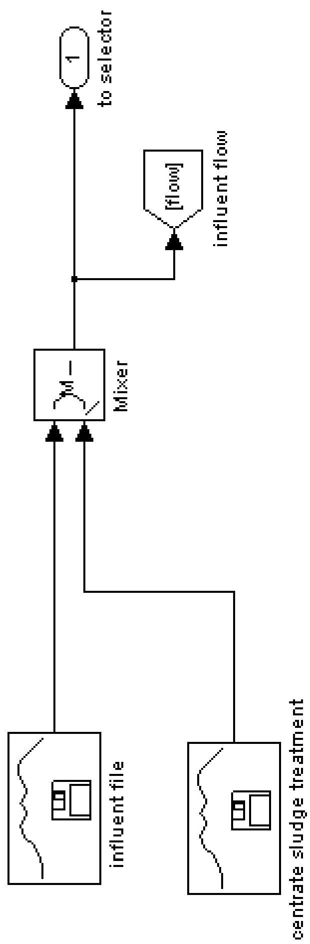


Figure V.1 Model layout. The first set up for the model has been made by Hielke van der Spoel of waterboard Hollands Noorderkwartier. This model, adjusted by Jeroen Stok [Stok (2003)], has been used as a basis for the modelling of wwtp Katwoude. The model comprises four blocks: influent, selector, circuit and secondary settlers.



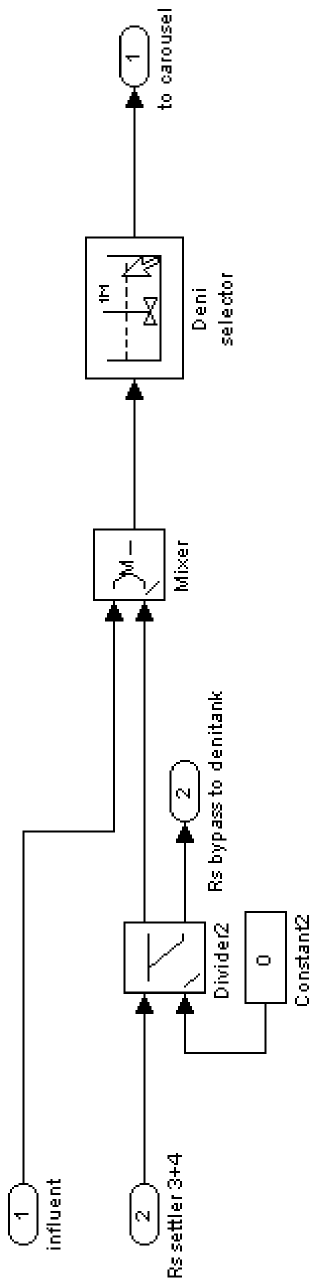


Figure V.2 Influent block.

Figure V.3 Selector block.

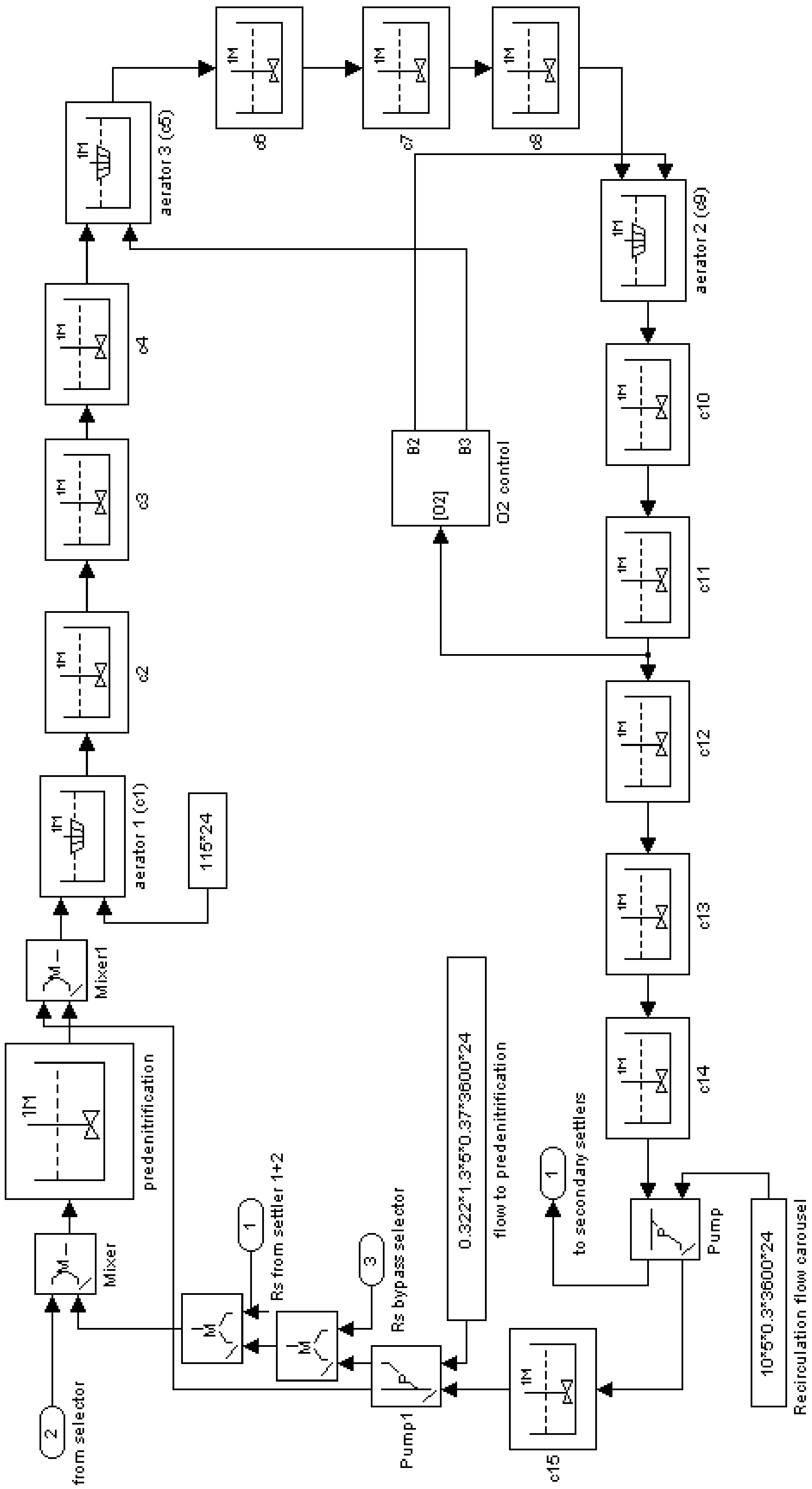


Figure V.4 Circuit block.

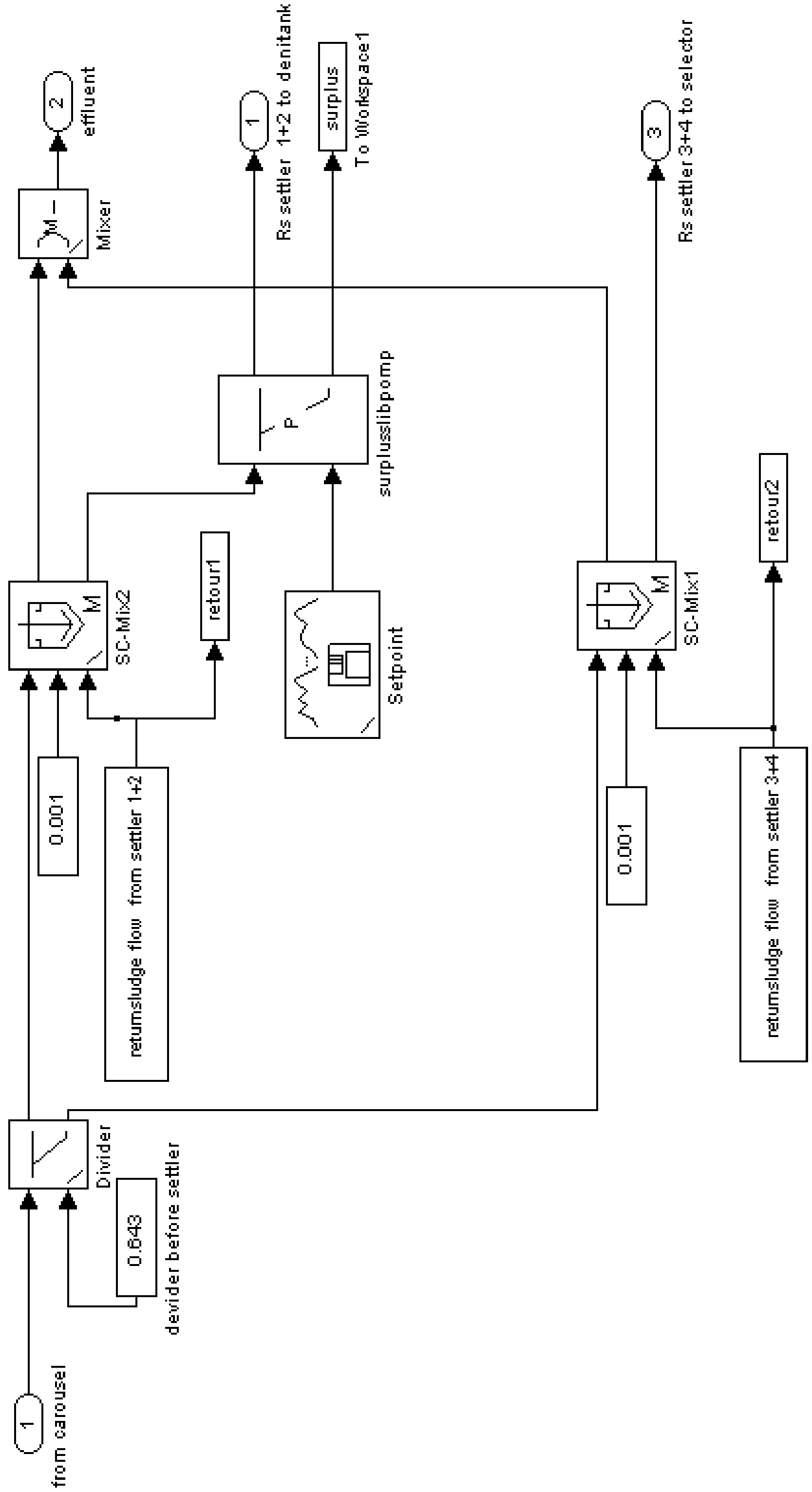


Figure V.5 Secondary settlers block.

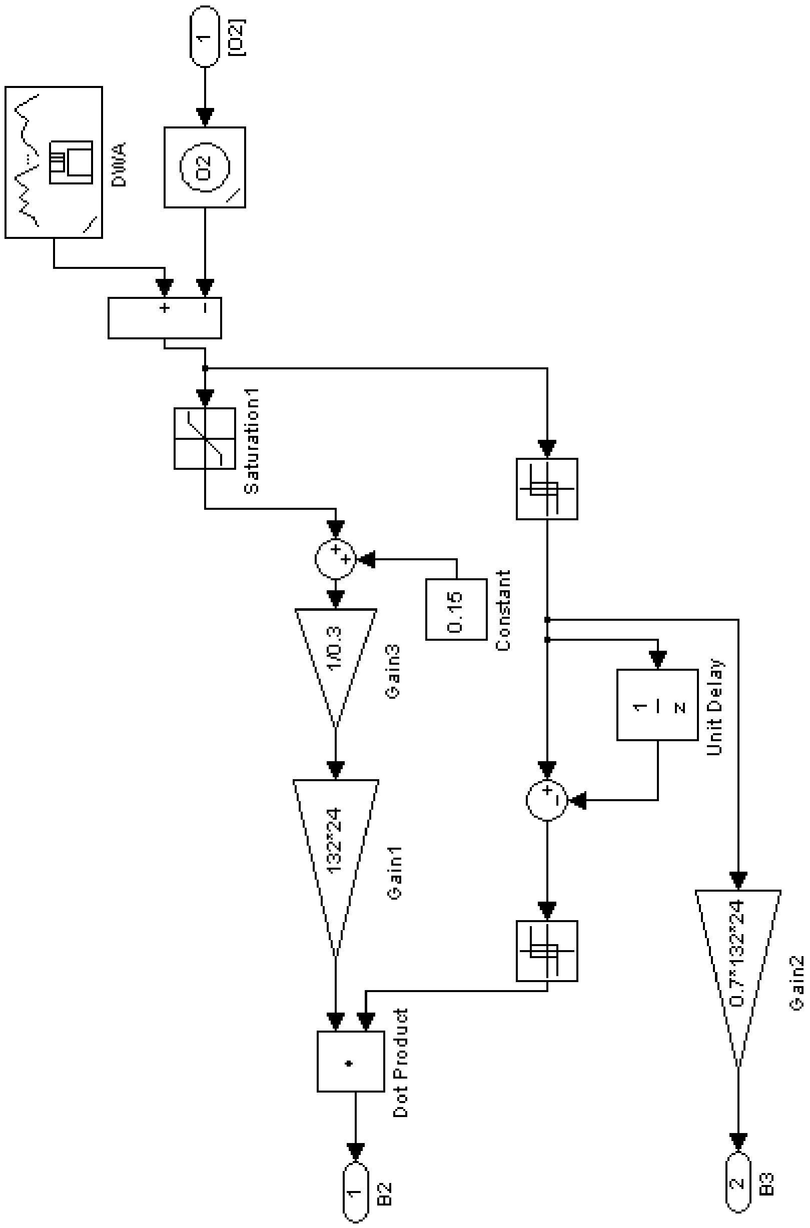


Figure V.6 Aeration controls.

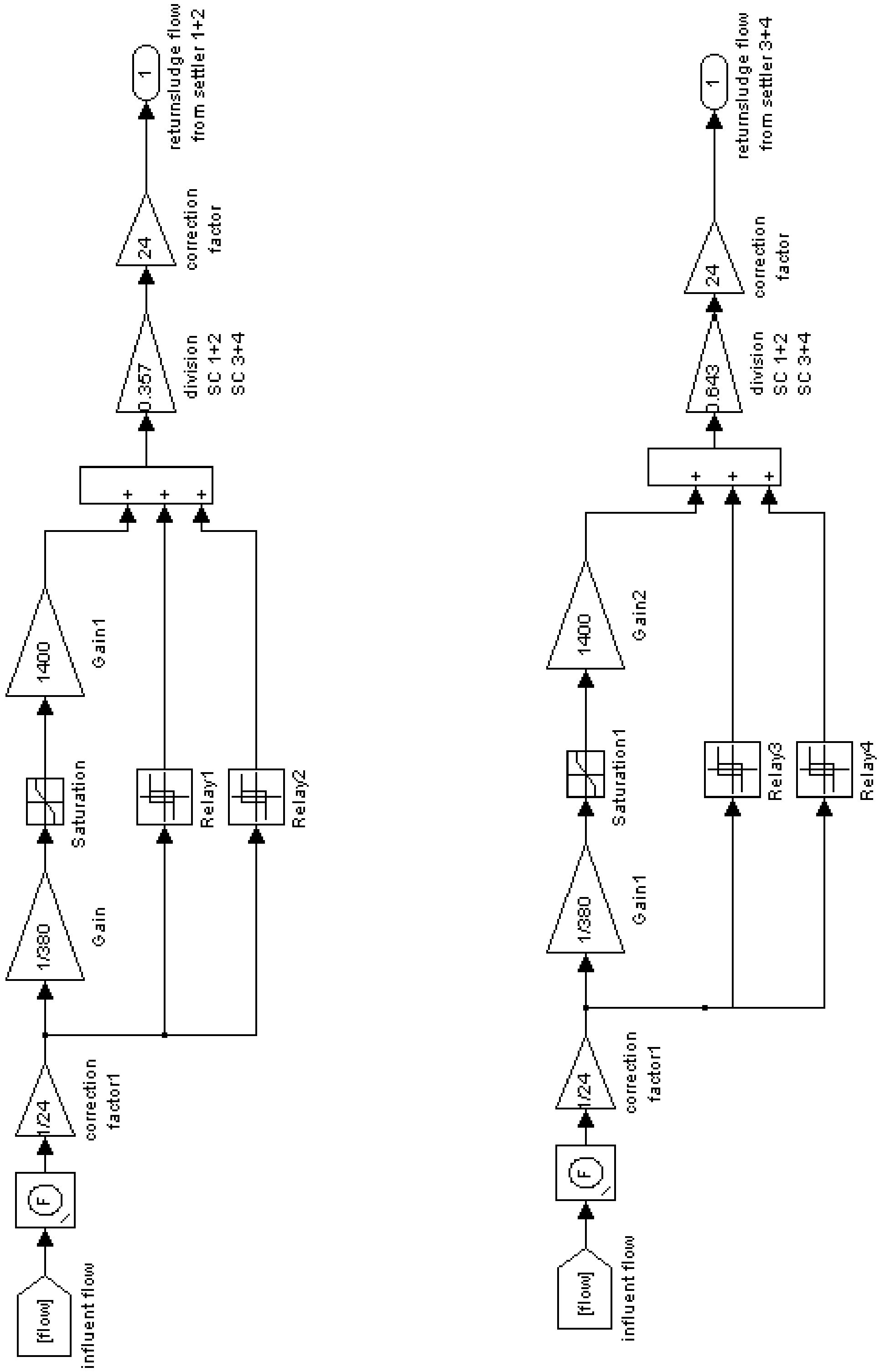


Figure V.7 Secondary settler controls.

Appendix VI Simulation results for nitrification. Wwtp 'Katwoude'

Simulation results for the complete measuring period from 19 September 2002 (= day 1) to 5 December 2002 (= day 77). The upper graphs show measured data and the model results, the lower graphs show the residuals only if the measured data are reliable.

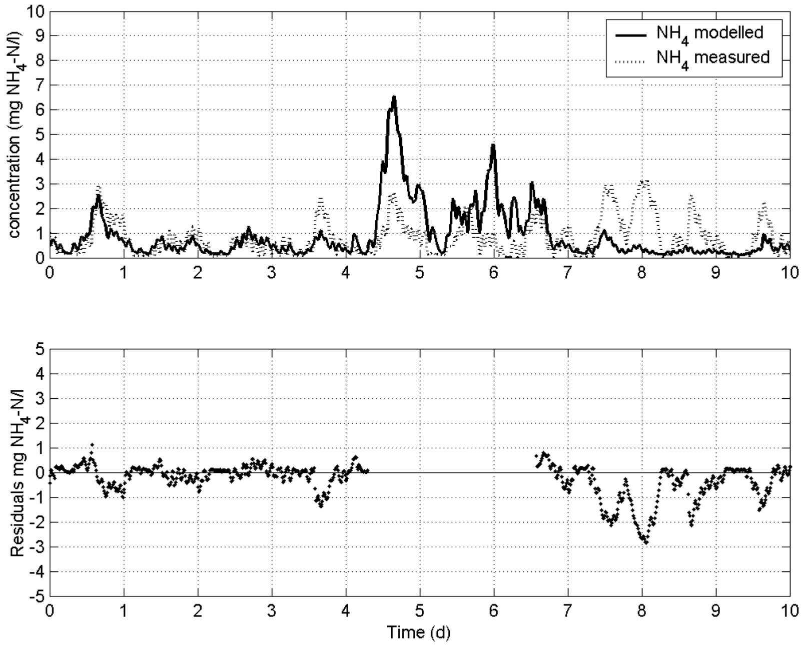


Figure VI.1 Modelled vs measured ammonium concentration in AT effluent. Day 0 to 10 (19 September to 29 September). The ammonium sensor measuring the influent gave unreliable too high readings between day 4 and 7.

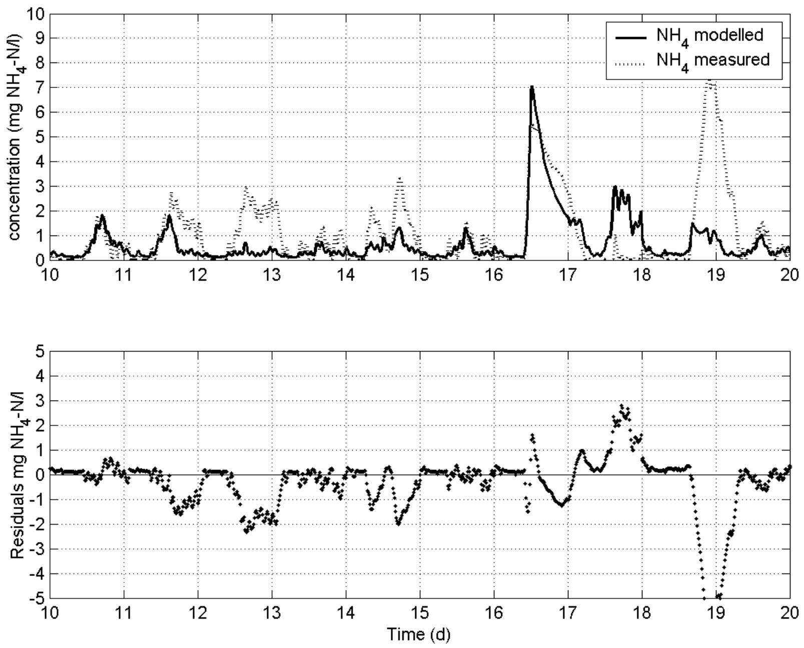


Figure VI.2 Modelled vs measured ammonium concentration in AT effluent. Day 10 to 20 (29 September to 9 October). The large difference between the model and measured ammonium concentration in the AT effluent on day 19 are likely due to low quality sensor data. However, for this date no control measurements were taken.

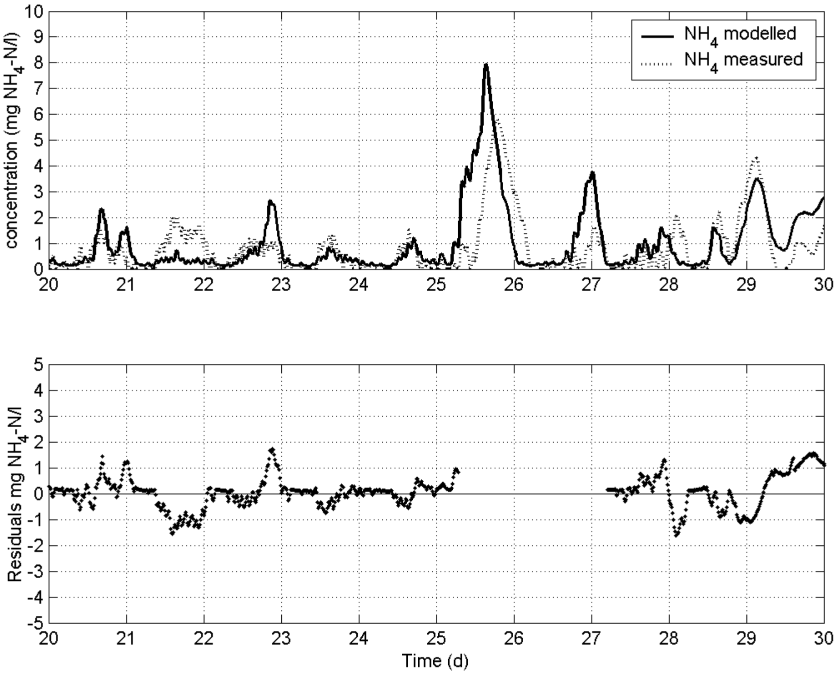


Figure VI.3 Modelled vs measured ammonium concentration in AT effluent. Day 20 to 30 (9 October to 19 October). The ammonium sensor measuring the influent gave unreliable readings on day 25 and 27.

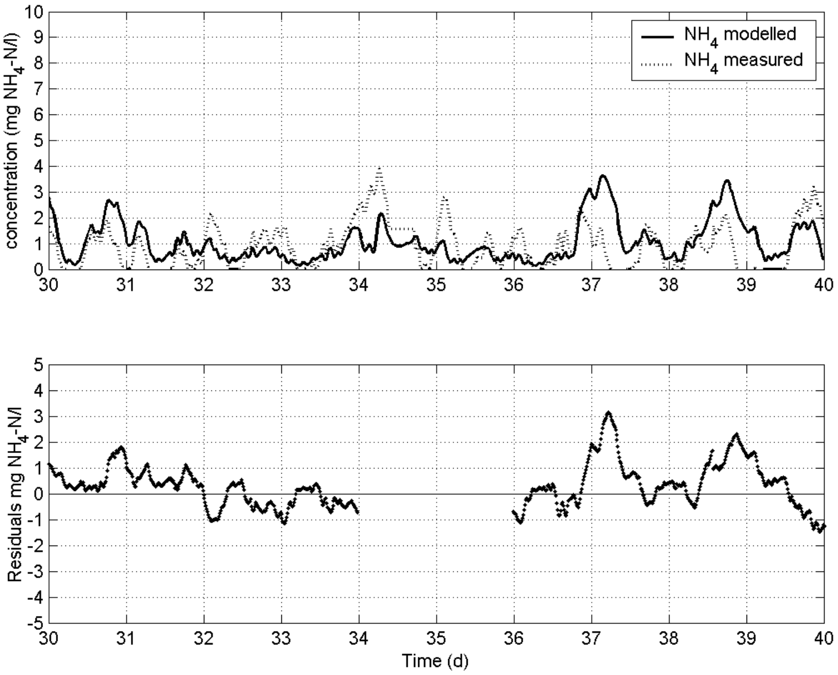


Figure VI.4 Modelled vs measured ammonium concentration in AT effluent. Day 30 to 40 (19 October to 29 October). The ammonium sensor measuring the influent gave unreliable readings from day 34 to day 36.

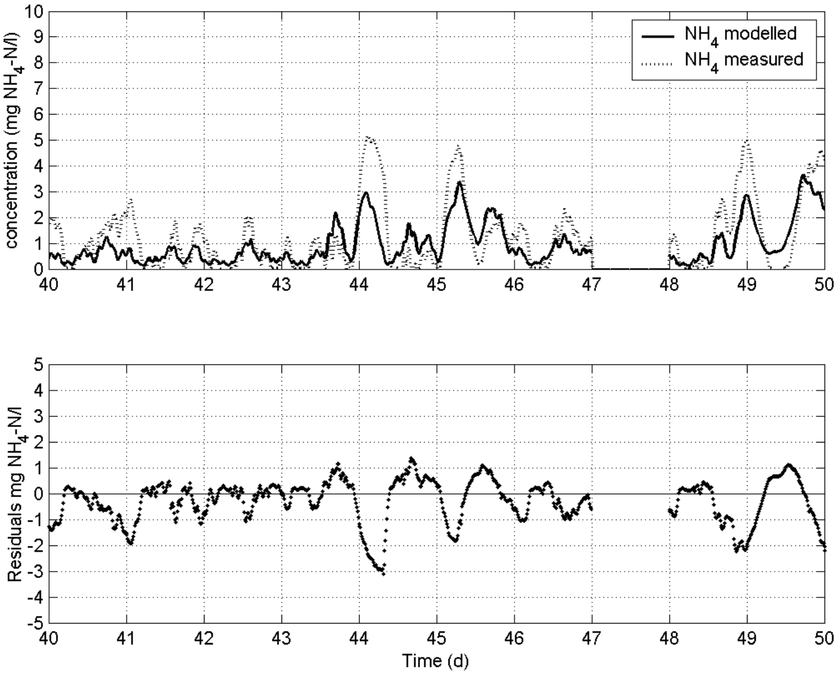


Figure VI.5 Modelled vs measured ammonium concentration in AT effluent. Day 40 to 50 (29 October to 8 November). On day 47 the data acquisition system failed.

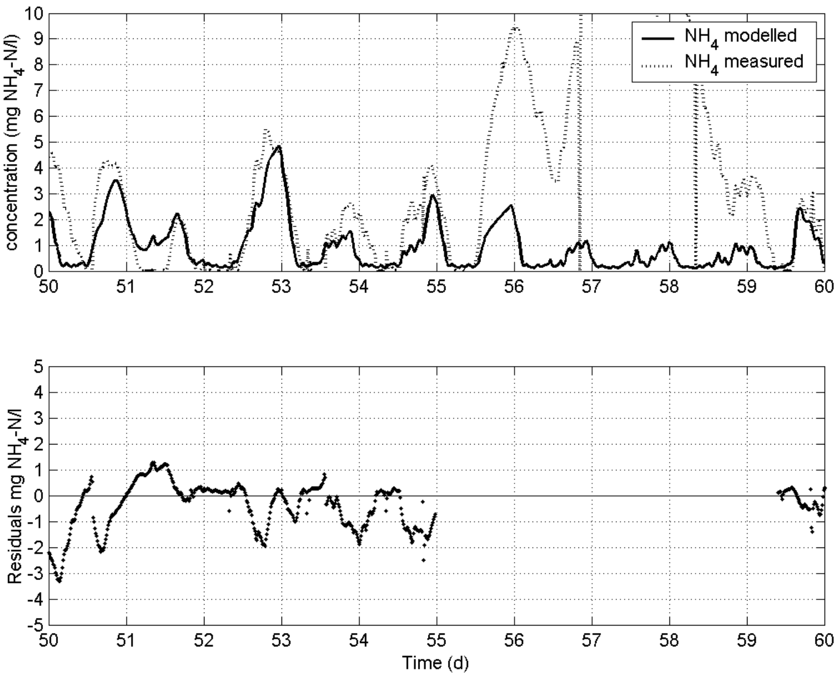


Figure VI.6 Modelled vs measured ammonium concentration in AT effluent. Day 50 to 60 (8 November to 18 November). From day 55 to day 59 the aeration and controls were under repair, resulting in an extremely high (up to 10 mg N/l) ammonium concentration in the AT effluent.

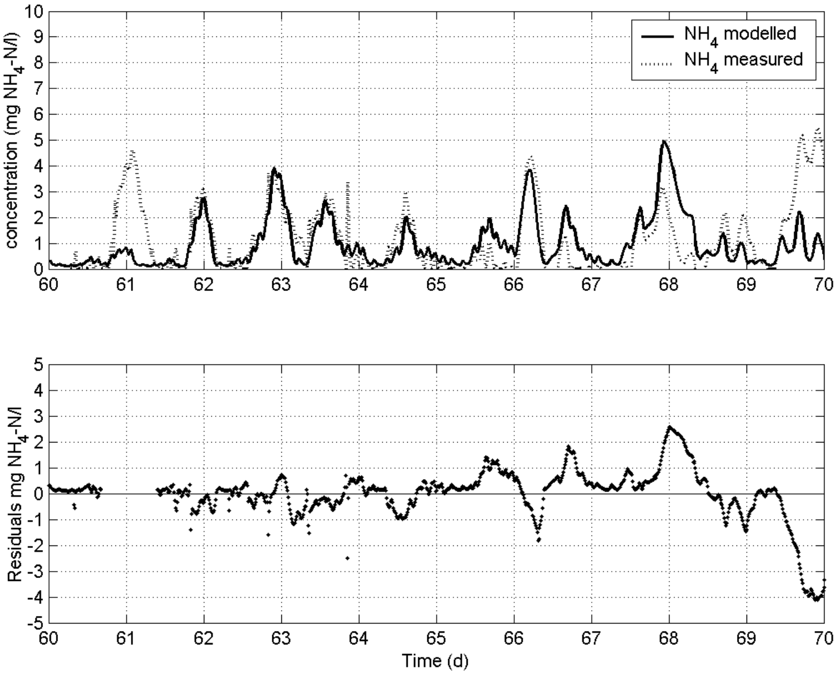


Figure VI.7 Modelled vs measured ammonium concentration in AT effluent. Day 60 to 70 (18 November to 28 November). The ammonium sensor measuring the influent gave false readings round day 61. The large residuals on day 69 are also likely due to unreliable sensor readings.

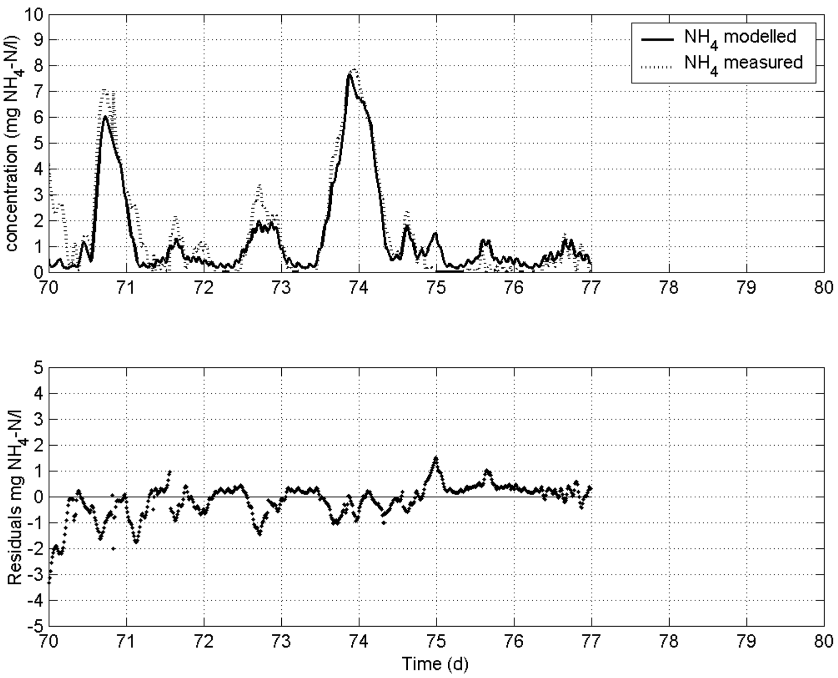


Figure VI.8 Modelled vs measured ammonium concentration in AT effluent. Day 70 to 77 (28 November to 5 December).

Appendix VII Sensitivity of selected boundary for significant errors

VII.1 Sensitivity to value of boundary for determining acceptable systematic error in influent data

In the assessment of the impact of significant errors the deviation of daily averaged ammonium with more than ± 0.5 mg N/l was considered significant. In this appendix the sensitivity of the results to the assumed value of ± 0.5 mg N/l was analysed by performing the same analysis for a deviation of ± 0.4 mg N/l and ± 0.6 mg N/l. The final results for both analysed storm events did not deviate much from the values found for the original criterion, see table VII.1 and table VII.2.

Table VII.1 Systematic errors, rounded to 5% values, causing significant deviations in simulation results (in %). Storm 1 December 2002.

		flow			**NH_4 in influent**			**COD_{susp} in influent**		
criterion	**boundary**	dilution phase	recovery phase	total storm event	dilution phase	recovery phase	total storm event	dilution phase	recovery phase	total storm event
averaged NH_4	- 0.4 mg N/l	-25	-50	-10	-10	-50	-10	-	-	-
	+ 0.4 mg N/l	25	25	10	20	30	10	-	-	-
	- 0.5 mg N/l	-25	-	-25	-25	-	-20	-	-	-
	+ 0.5 mg N/l	25	30	10	25	40	15	-	-	-
	- 0.6 mg N/l	-25	-	-25	-25	-	-20	-	-	-
	+ 0.6 mg N/l	30	30	15	25	50	15	-	-	-

Table VII.2 Systematic errors causing significant deviations in simulation results (in %). Storm 17 October 2002.

		flow			**NH_4 in influent**			**COD_{susp} in influent**		
criterion	**boundary**	dilution phase	recovery phase	total storm event	dilution phase	recovery phase	total storm event	dilution phase	recovery phase	total storm event
averaged NH_4	- 0.4 mg N/l	-20	-	-15	-50	-	-15	-	-	-
	+ 0.4 mg N/l	25	50	15	20	40	15	-	-	-
	- 0.5 mg N/l	-50	-	-10	-50	-	-20	-	-	-
	+ 0.5 mg N/l	25	50	10	25	50	15	-	-	-
	- 0.6 mg N/l	-50	-	-20	-50	-	-25	-	-	-
	+ 0.6 mg N/l	50	50	10	30	50	15	-	-	-

VII.2 Sensitivity to criteria selected for determining acceptable systematic error in influent data

In addition to the criterion for the acceptable deviation of the daily average ammonium concentration in the effluent of 0.5 mg/l, as described in section 3.5, three other criteria were analysed:

- peak deviation in momentarily ammonium concentration in the AT effluent
- deviation in daily averaged N_{total} concentration in the AT effluent
- deviation in removal efficiency for N_{total}

The concentration COD in the wwtp effluent was not taken into account, as this concentration is not sensitive to fluctuations in the influent [Leinweber (2002)], as long as the secondary clarifier is not overloaded and sufficient aeration capacity is available.

For each criterion, a level of significance was defined, as shown in table VII.3, in order to be able to determine an ‘acceptable’ level of systematic errors in sewer models during the different stages of the storm event.

Table VII.3 Boundaries for significant errors.

criterion	boundary
1. maximum deviation in NH_4 concentration in the AT effluent	± 2 mg NH_4-N/l
2. deviation in daily averaged NH_4 concentration in the AT effluent	± 0.5 mg NH_4-N /l
3. deviation in daily averaged N_{total} concentration in the AT effluent	± 1 mg N_{total}-N/l
4. deviation in removal efficiency for N_{total}	± 10 %

The effect of systematic errors in the aforementioned influent parameters was analysed for each criterion given in table VII.3. Table VII.4 shows the results for the storm of 1 December 2002.

Table VII.4 Systematic errors, rounded to 5% values, causing significant deviations in simulation results (in %). Storm 1 December 2002.

		flow			**NH_4 in influent**			**COD_{susp} in influent**		
criterion		dilution phase	recovery phase	total storm event	dilution phase	recovery phase	total storm event	dilution phase	recovery phase	total storm event
1. NH_4	- 2 mg N/l	-30	-50	-20	-20	-50	-15	+ 100	+ 100	+ 50
	+ 2 mg N/l	30	35	15	25	50	15	-	-	-
2. averaged NH_4	- 0.5 mg N/l	-25	-	-25	-25	-	-20	-	-	+ 100
	+ 0.5 mg N/l	25	30	10	25	40	15	-	-	-
3. averaged N_{total}	- 1 mg N/l	-50	-	-30	-50	-	-20	-	-	-
	+ 1 mg N/l	40	50	15	30	50	15	-	-	-
4. N_{total} removal efficiency	- 10%	35	50	15	40	70	20	-	-	-
	+ 10%	-30	-	-25	-	-	-30	-	-	100
level of significance	lower	-25	-50	-20	-20	-50	-15	-	-	-
	upper	25	30	10	25	40	15	100	100	50

The results per criterion have the same order of magnitude for the parameters flow and ammonium. The final rows of the table give the most stringent value of the four criteria.

The results for the storm of 1 December 2002 (as shown in table VII.4) and the results for the storm event of 17 October (see table VII.5) have the same order of magnitude for all criteria, even though the characteristics of the storm events differ significantly.

Table VII.5 Systematic errors causing significant deviations in simulation results (in %). Storm 17 October 2002.

		flow			**NH_4 in influent**			**COD_{susp} in influent**		
criterion		dilution phase	recovery phase	total storm event	dilution phase	recovery phase	total storm event	dilution phase	recovery phase	total storm event
1. NH_4	-2 mg N/l	-50	-50	-20	-50	-	-25	+ 100	+ 100	+ 50
	+ 2 mg N/l	25	50	15	30	50	25	-	-	-
2. averaged NH_4	- 0.5 mg N/l	-50	-	-10	-50	-	-20	-	-	+ 100
	+ 0.5 mg N/l	25	50	10	25	50	15	-	-	-
3. averaged N_{total}	- 1 mg N/l	-50	-50	-25	-50	-50	-50	-	-	-
	+ 1 mg N/l	50	50	25	50	50	20	-	-	-
4. N_{total} removal efficiency	- 10%	25	40	10	35	50	20	-	-	-
	+ 10%	-25	-50	-10	-50	-50	-25	-	-	-
level of significance	lower	-25	-50	-10	-50	-50	-25	-	-	-
	upper	25	40	10	25	50	15	100	100	50

Appendix VIII Calibration curve SCUFAs

The SCUFA sensors were calibrated in the laboratory to known dilutions of Rhodamine WT in clean water in order to be able to relate the sensor readings to concentrations of Rhodamine WT dye.

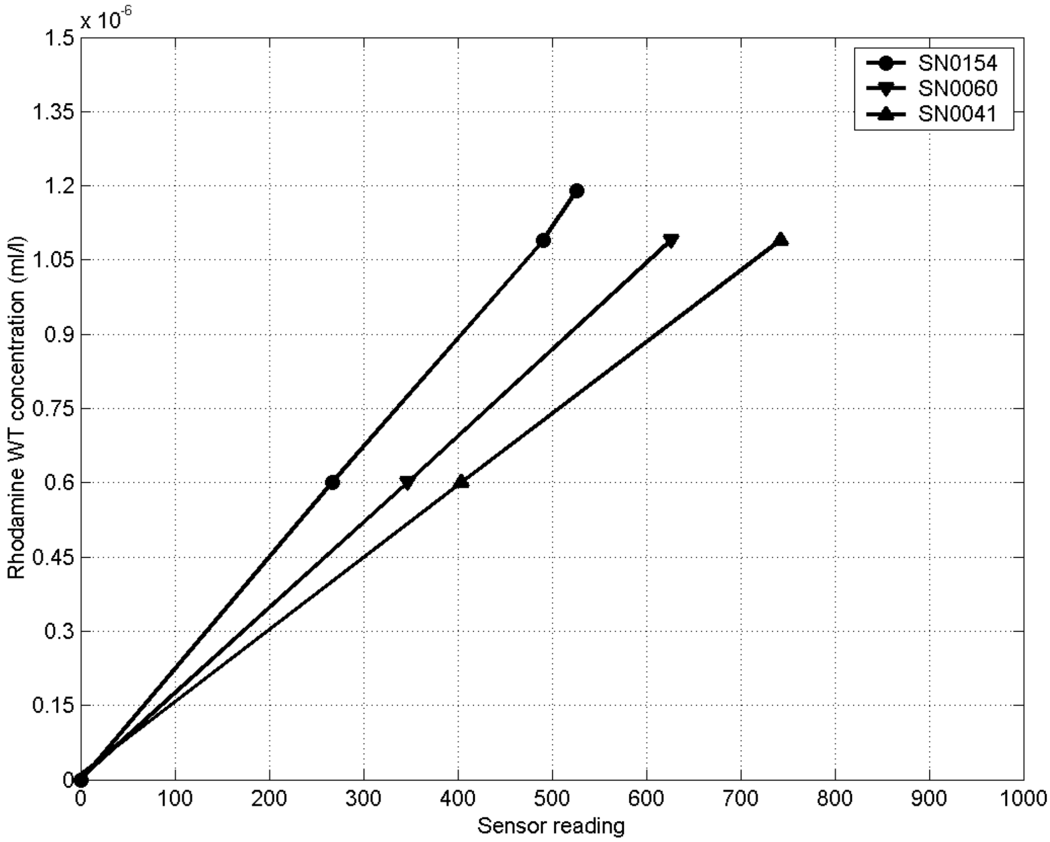


Figure VIII.1 Calibration curve SCUFAs.

Appendix IX NWRW 4.3 Runoff model

The runoff model commonly applied in the Netherlands comprises 4 processes, as shown in figure IX.1 (see [Van Luijtelaar and Rebergen (1997)]):

- initial losses: wetting of dry surface and storage in local surface depressions
- infiltration
- evaporation
- flow routing

The initial losses are introduced as an average constant value depending on the type of contributing surface, as shown in table IX.1.
The infiltration is calculated using the Horton infiltration model [Horton (1940)], given in equation IX.1 and IX.2:

Decrease of infiltration capacity:

$$f_t = f_e + (f_b - f_e)e^{-k_d t} \quad \text{(eq. IX.1)}$$

Recovery of infiltration capacity:

$$f_t = f_b + (f_b - f_e)e^{-k_r t} \quad \text{(eq. IX.2)}$$

where

f_t = infiltration capacity at time t (mm/h)
f_b = maximum infiltration capacity (t=0) (mm/h)
f_e = minimum infiltration capacity (mm/h)
k_d = time factor decrease of infiltration capacity (mm/h)
k_r = time factor recovery of infiltration capacity (mm/h)
t = time (h)

The evaporation is introduced as a monthly constant value to be able to empty the surface storage in between storm events.
The overland flow towards the entry of the sewer system (e.g. gullypots) is introduced by equation IX.3:

$$q = c * h \quad \text{(eq. IX.3)}$$

where

q = flow (mm/min)
c = routing constant (min^{-1})
h = level of rainfall stored on surface (mm)

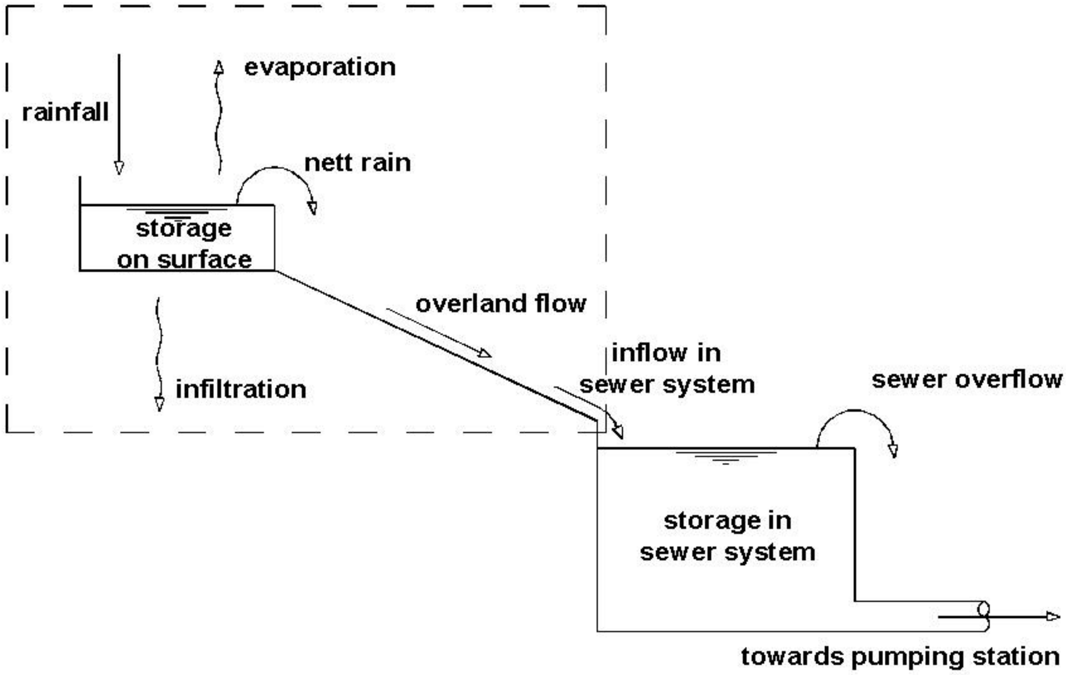


Figure IX.1 Rainfall runoff model.

Table IX.1 Default runoff parameters Leidraad Riolering C2100 [Stichting RIONED (1999)].

	type of surface		routing constant (min^{-1})	storage on surface (mm)	infiltration capacity (mm/h)		time factors (h^{-1})	
					max	min	decrease	recovery
1	impervious	sloping	0.5	0.0				
2		flat	0.2	0.5				
3		flat, large area	0.1	1.0				
4	semi-	sloping	0.5	0.0	2.0	0.5	3.0	0.1
5	pervious	flat	0.2	0.5	2.0	0.5	3.0	0.1
6		flat, large area	0.1	1.0	2.0	0.5	3.0	0.1
7	roofs	sloping	0.5	0.0				
8		flat	0.2	2.0				
9		flat, large area	0.1	4.0				
10	pervious	sloping	0.5	2.0	5.0	1.0	3.0	0.1
11		flat	0.2	4.0	5.0	1.0	3.0	0.1
12		flat, large area	0.1	6.0	5.0	1.0	3.0	0.1

Appendix X Relative bias and residuals hydrodynamic modelling

This appendix gives the modelled and measured water levels, the residuals and the relative bias for all calibrated storm events, except for storm 18/07/01, which was discussed in chapter 4.

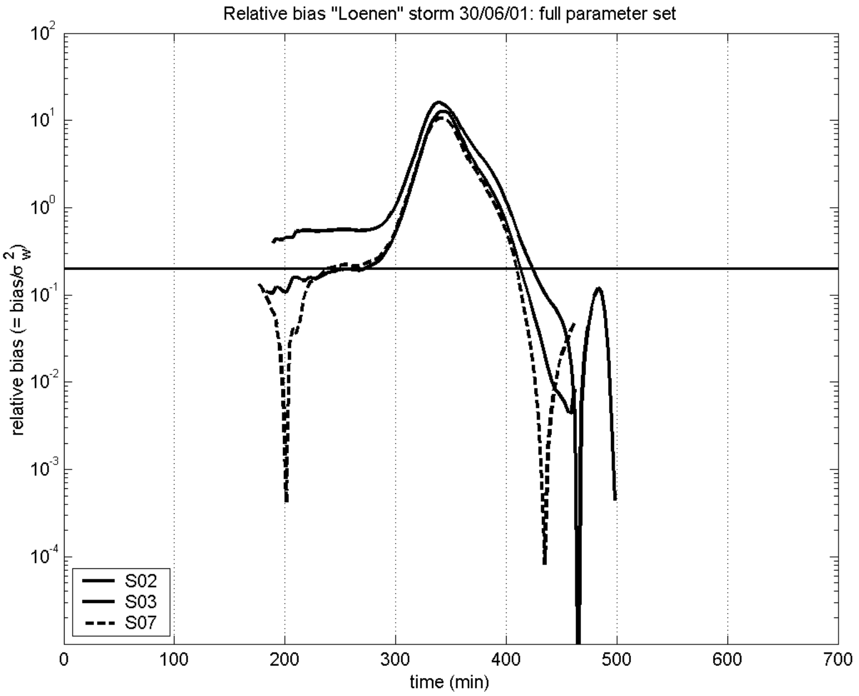


Figure X.1 Relative bias storm 30/06/01.

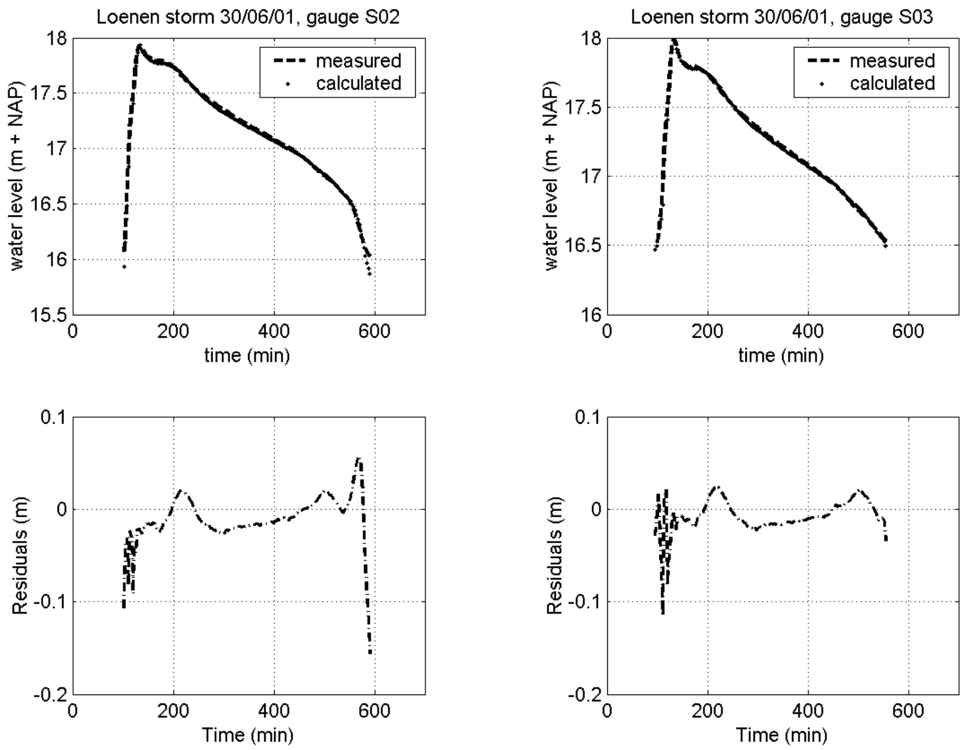


Figure X.2 Residuals storm 30/06/01, gauge S02 (left) and gauge S03 (right).

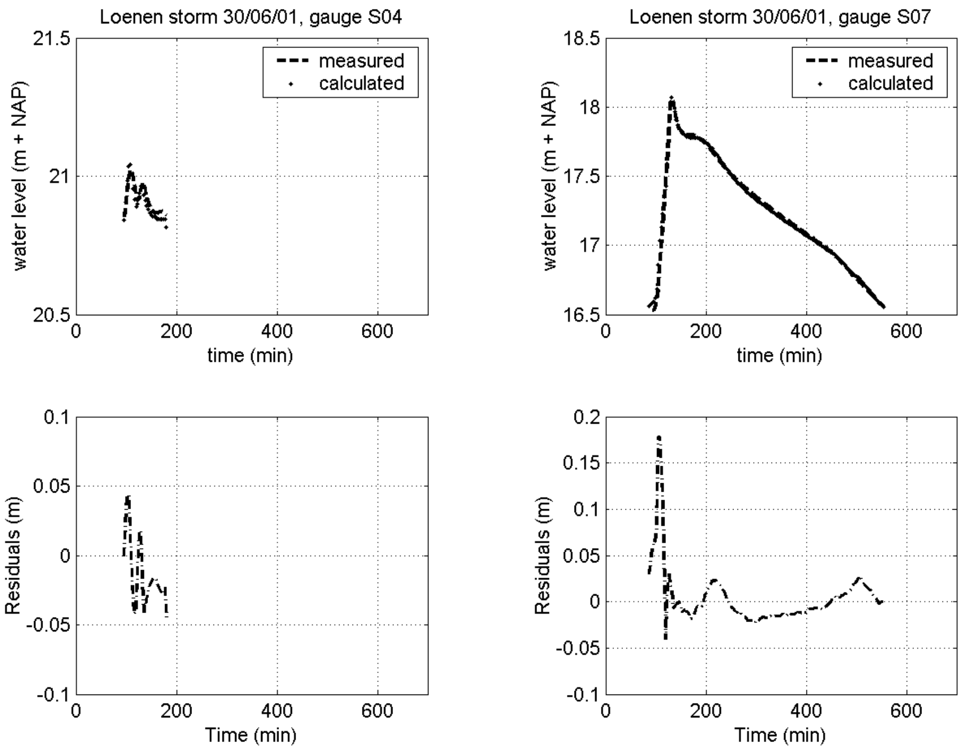


Figure X.3 Residuals storm 30/06/01, gauge S04 (left) and gauge S07 (right).

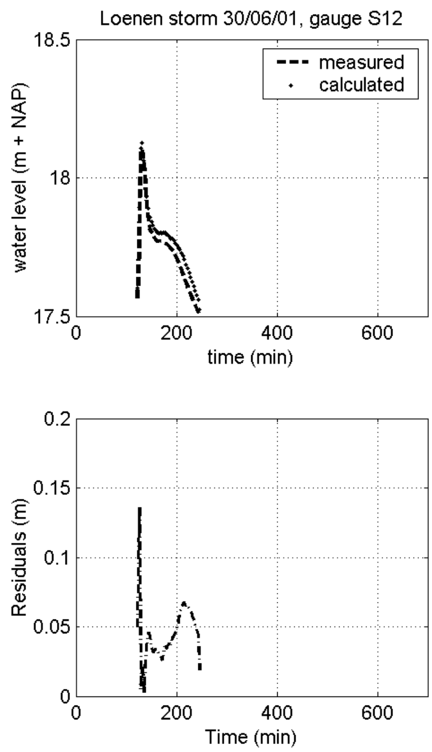


Figure X.4 Residuals storm 30/06/01, gauge S12.

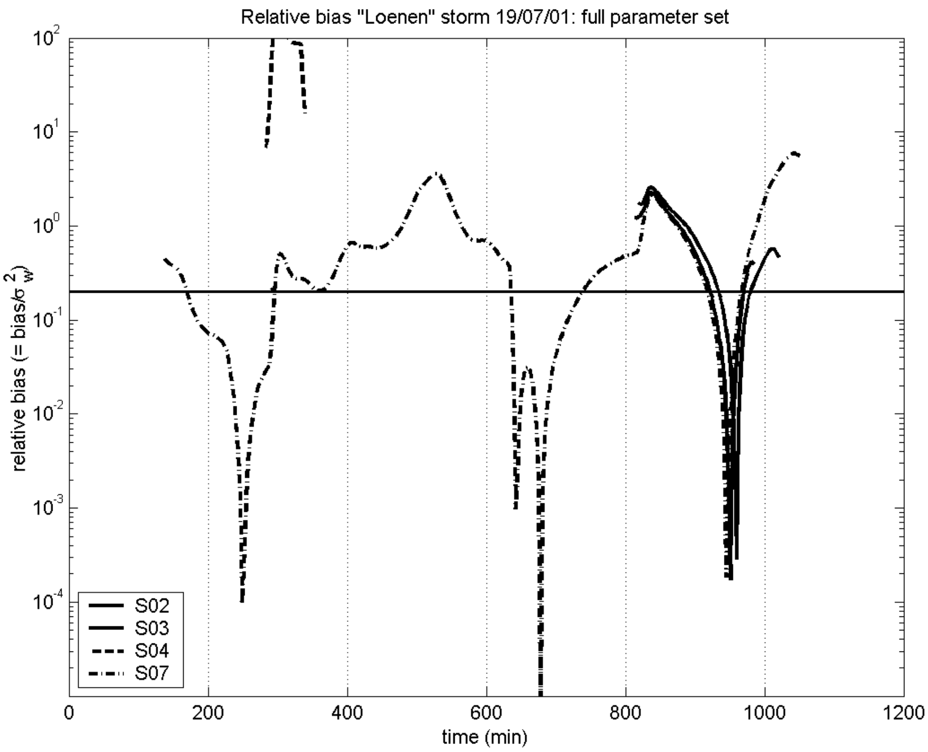


Figure X.5 Relative bias storm 19/07/01.

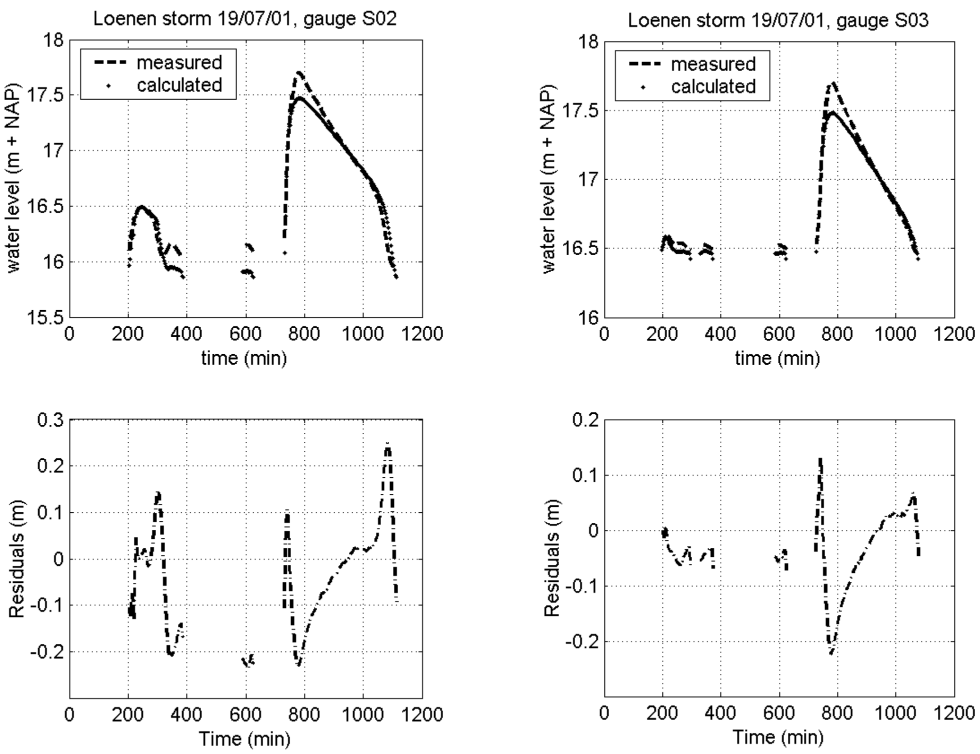


Figure X.6 Residuals storm 19/07/01, gauge S02 (left) and gauge S03 (right).

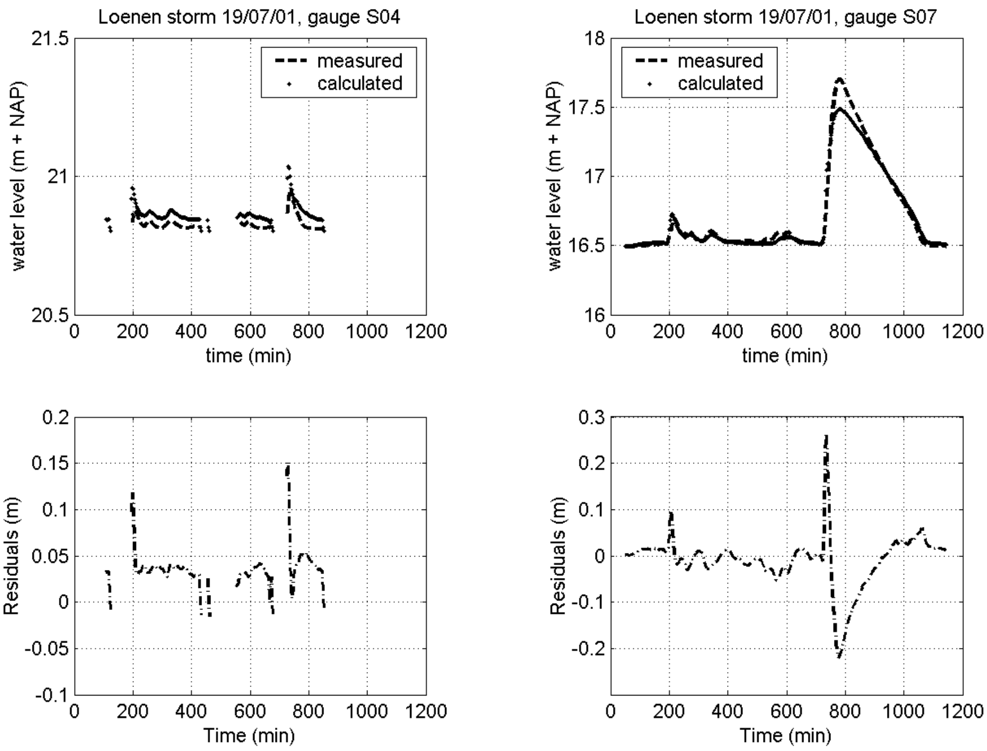


Figure X.7 Residuals storm 19/07/01, gauge S04 (left) and gauge S07 (right).

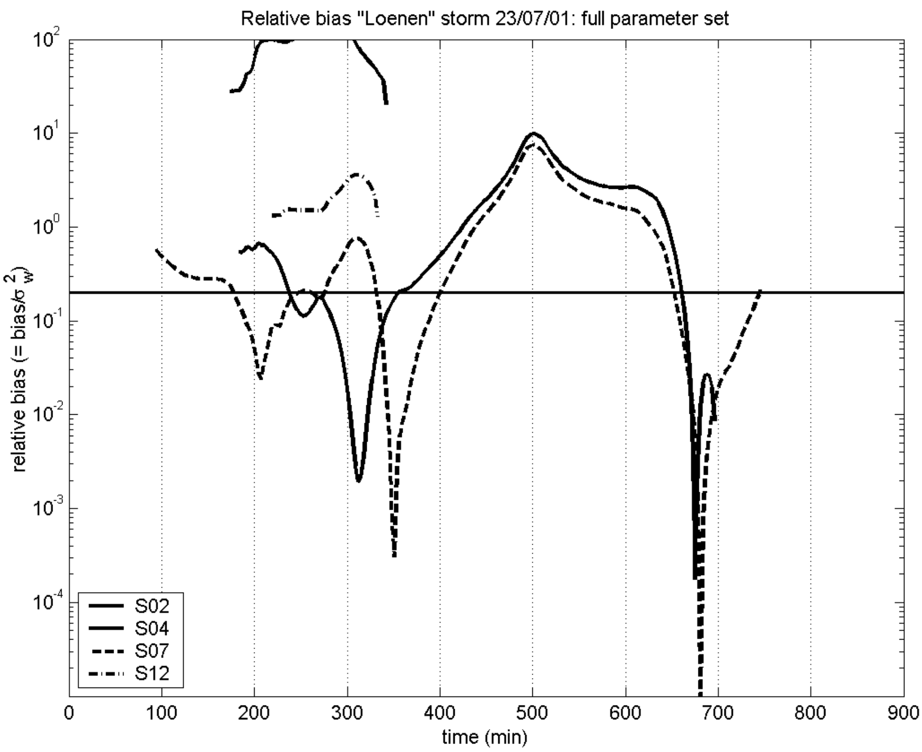


Figure X.8 Relative bias storm 23/07/01.

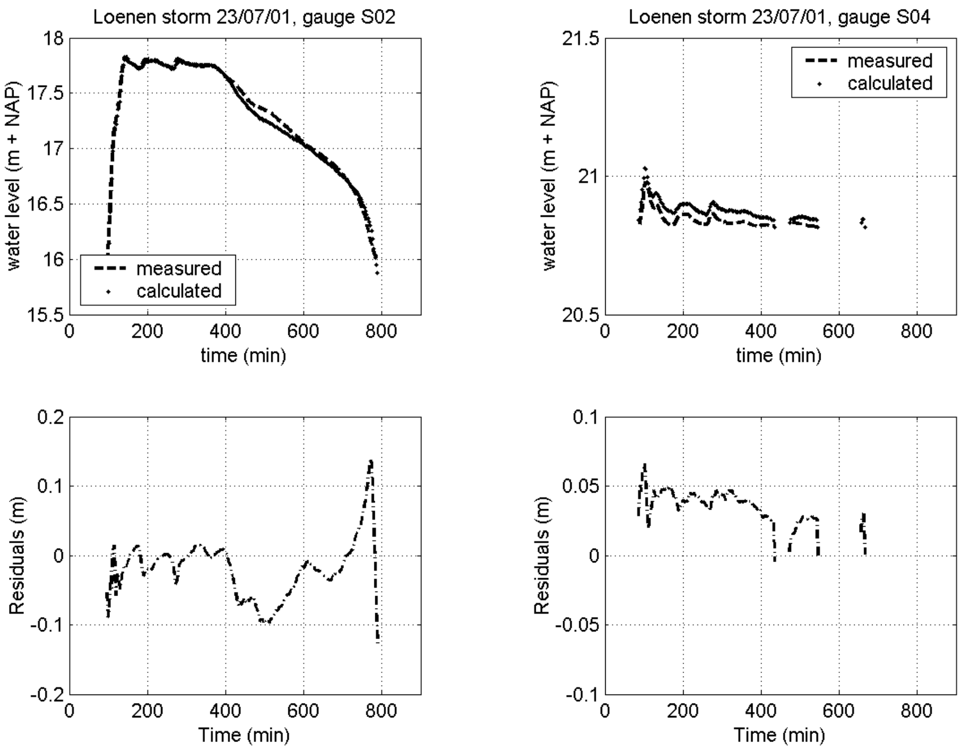


Figure X.9 Residuals storm 23/07/01, gauge S02 (left) and gauge S04 (right).

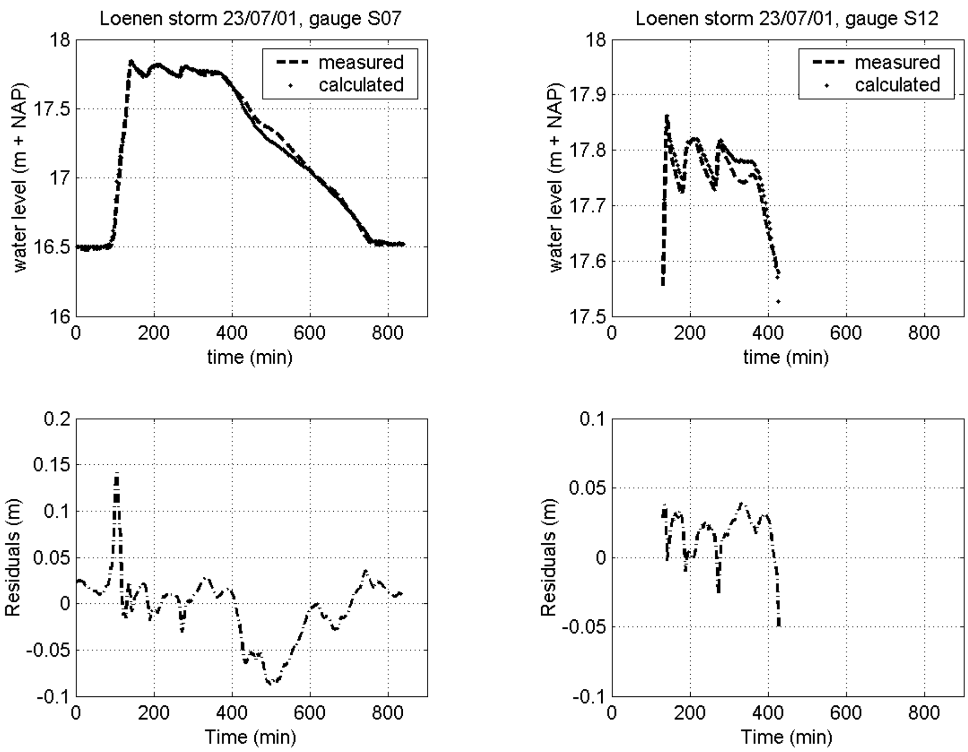


Figure X.10 Residuals storm 23/07/01, gauge S07 (left) and gauge S12 (right).

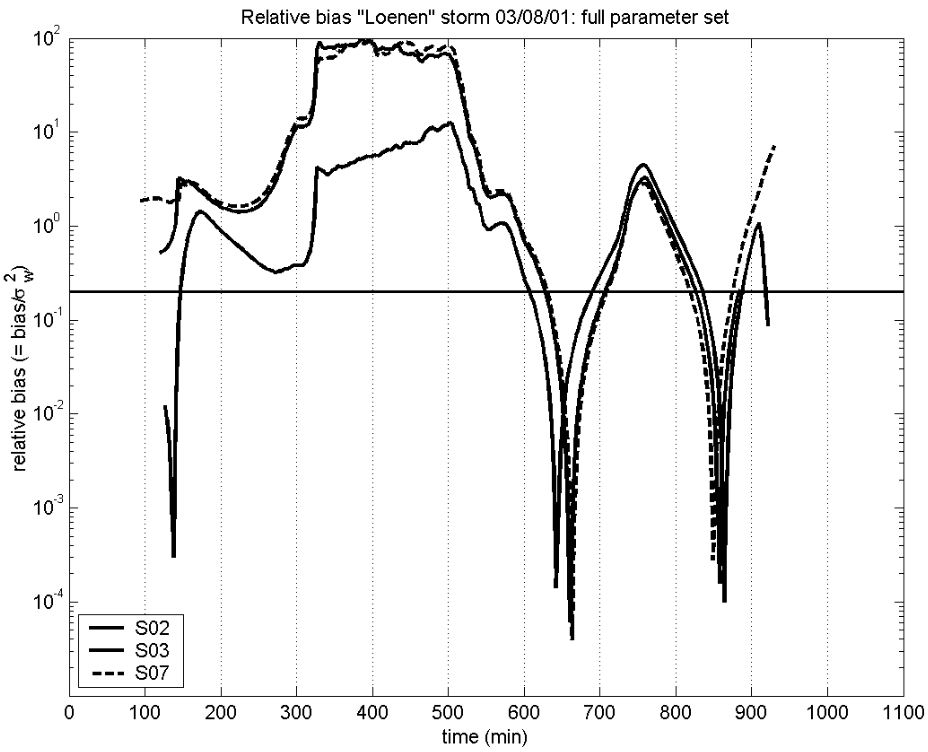


Figure X.11 Relative bias storm 03/08/01.

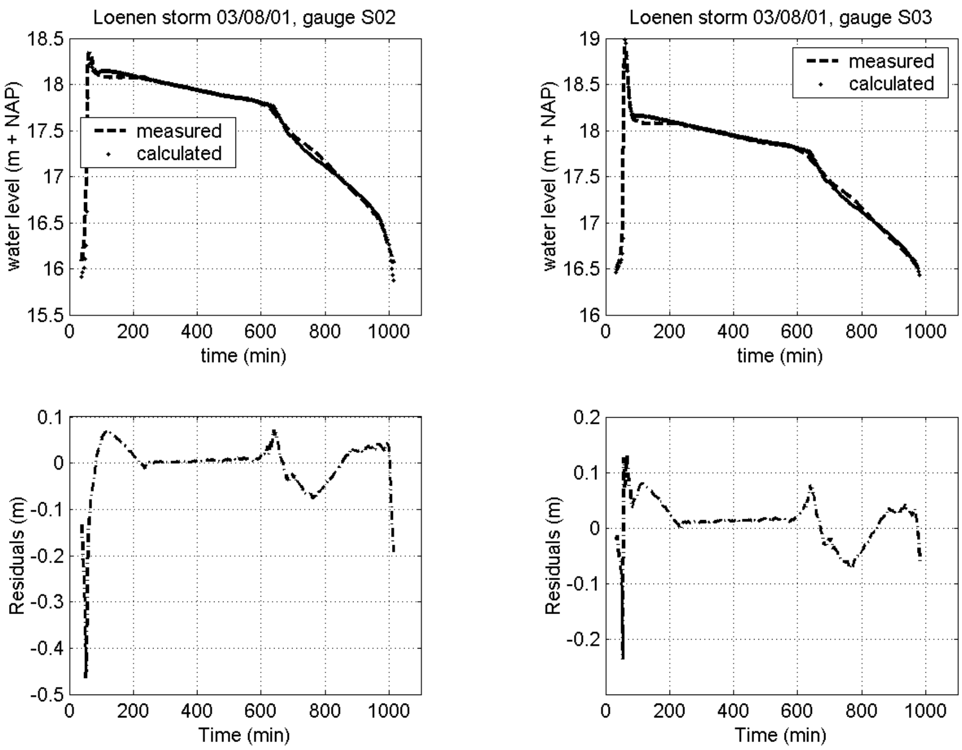


Figure X.12 Residuals storm 03/08/01, gauge S02 (left) and gauge S03 (right).

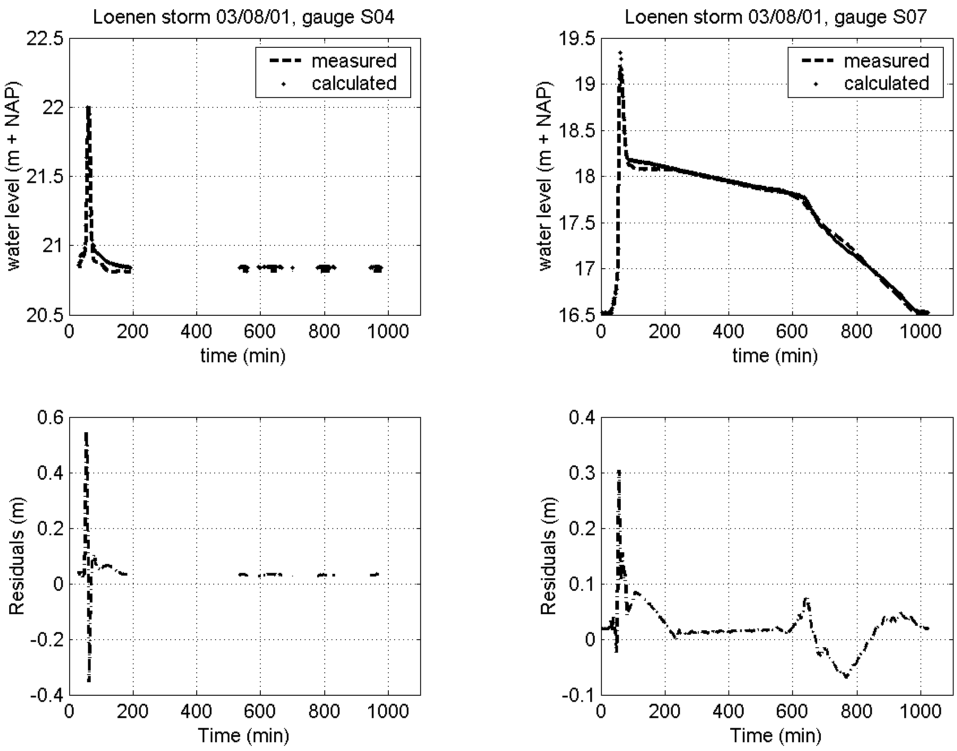


Figure X.13 Residuals storm 03/08/01, gauge S04 (left) and gauge S07 (right).

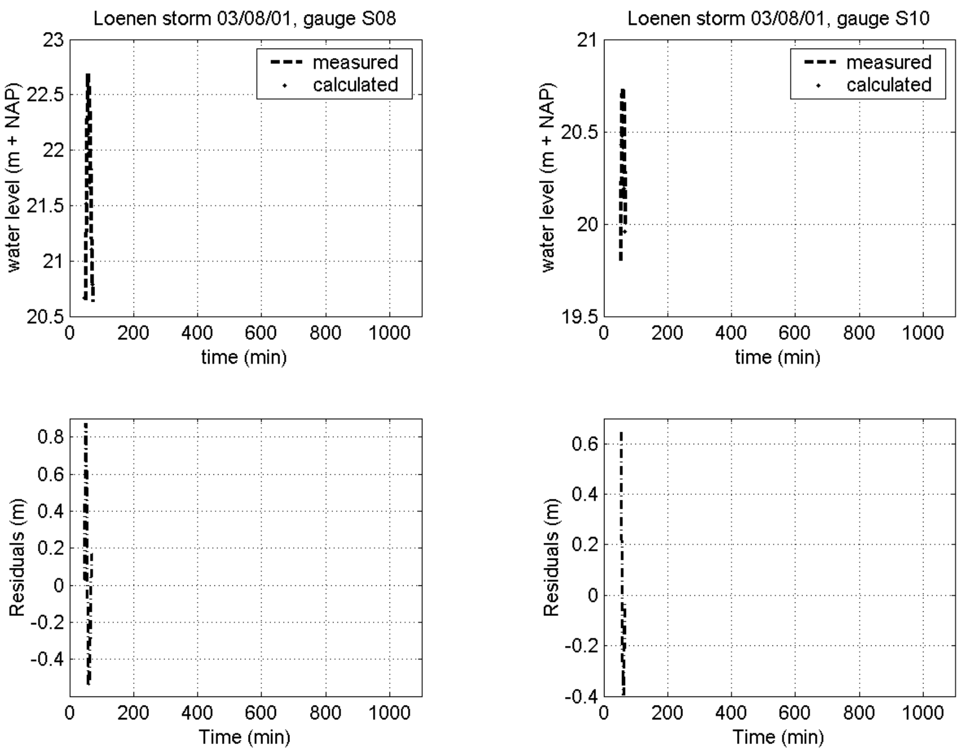


Figure X.14 Residuals storm 03/08/01, gauge S08 (left) and gauge S10 (right).

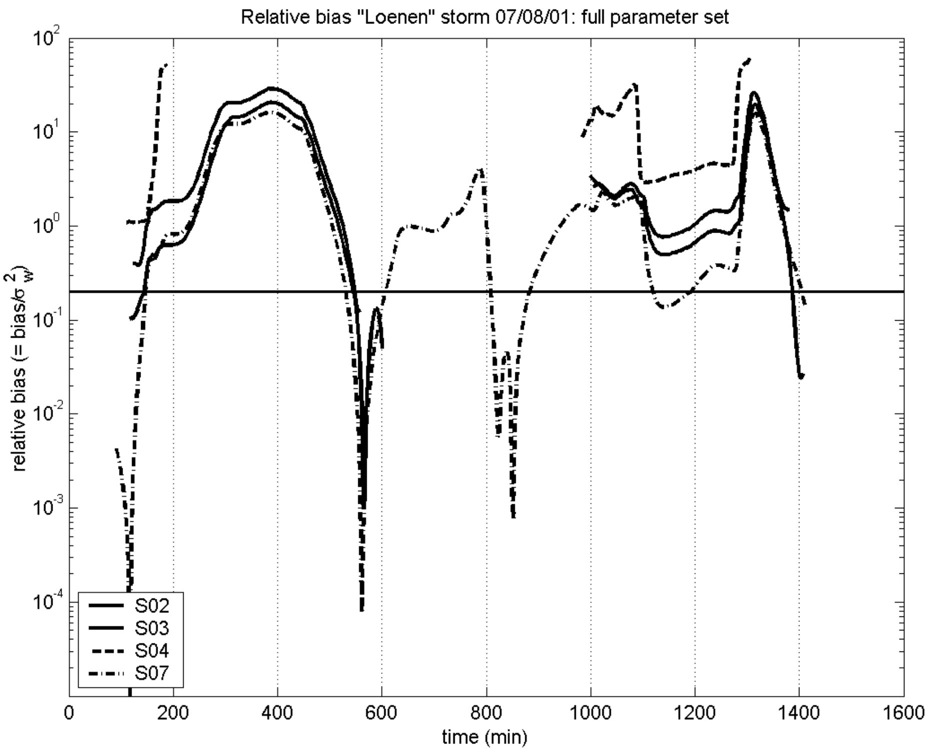


Figure X.15 Relative bias storm 07/08/01.

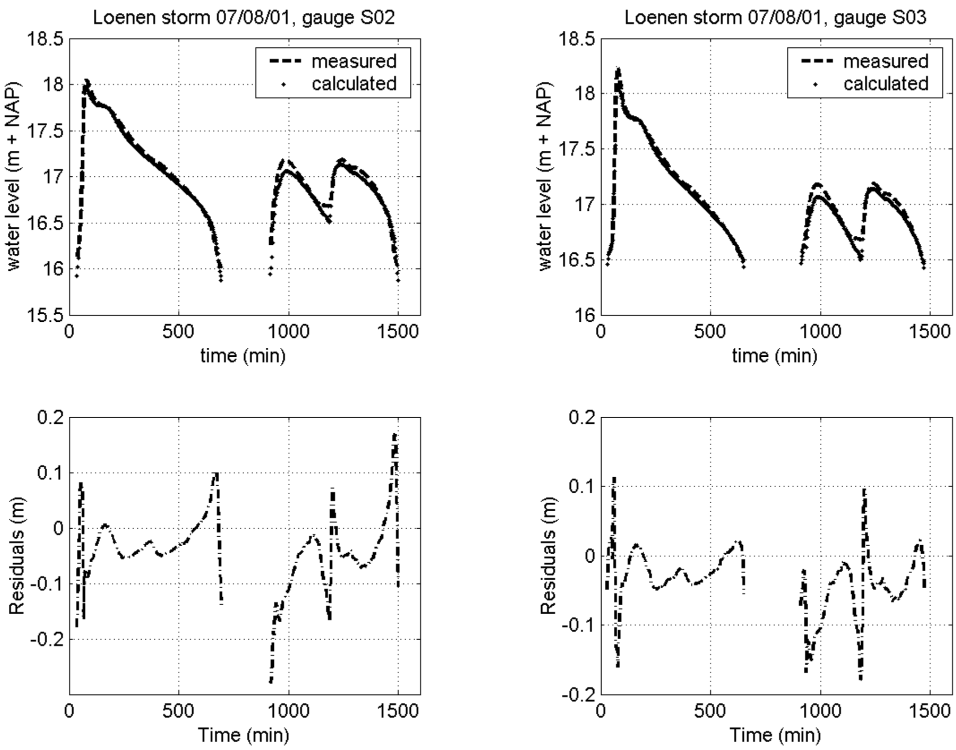


Figure X.16 Residuals storm 07/08/01, gauge S02 (left) and gauge S03 (right).

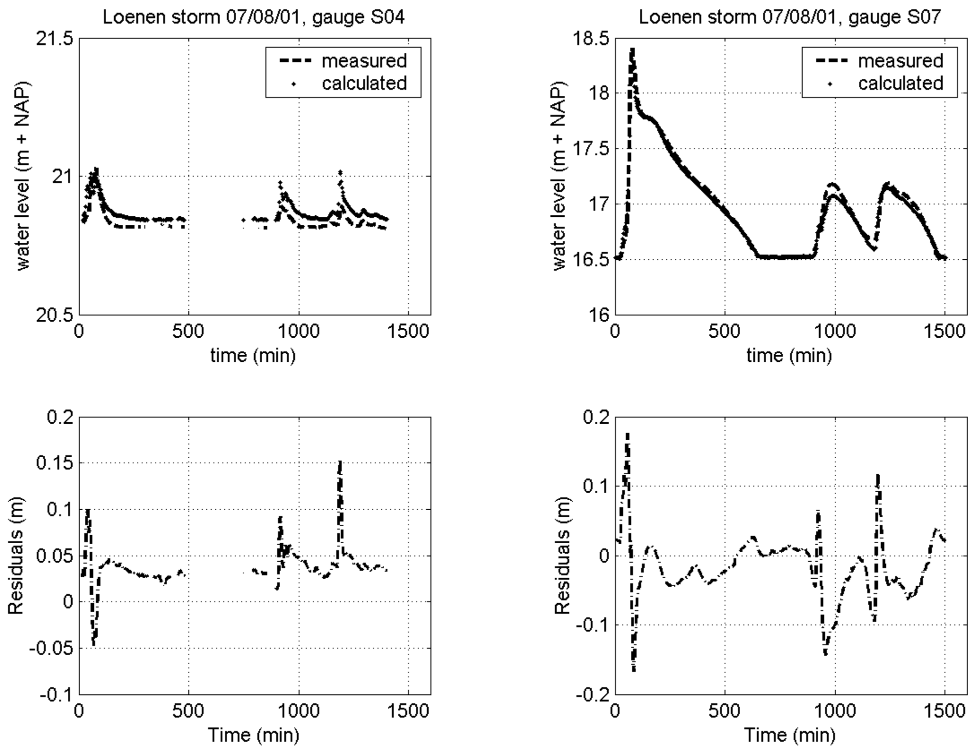


Figure X.17 Residuals storm 07/08/01, gauge S04 (left) and gauge S07 (right).

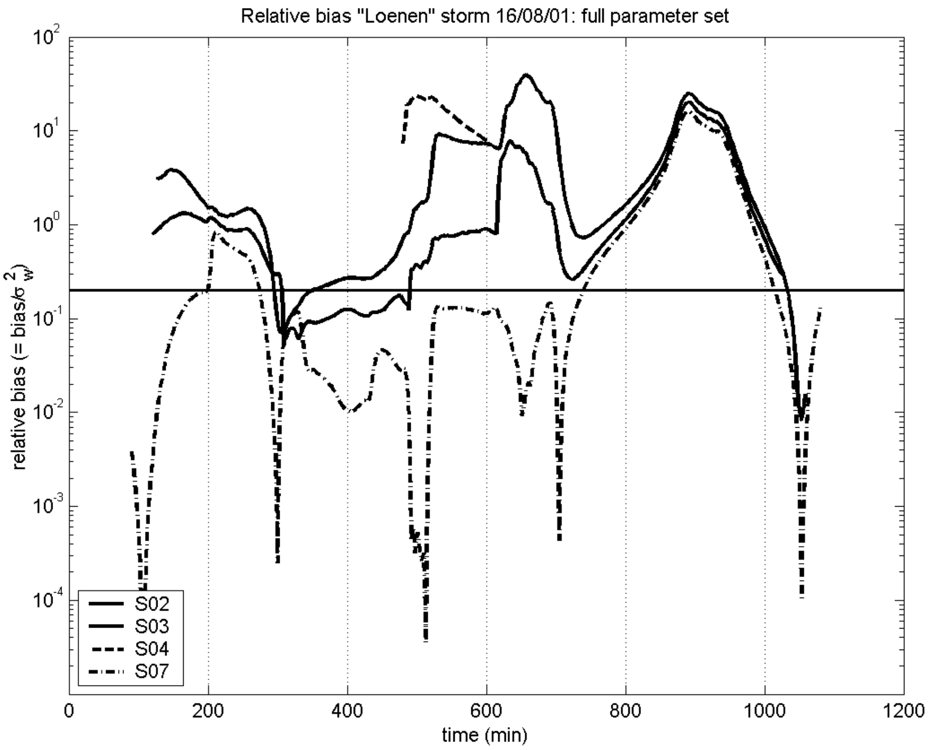


Figure X.18 Relative bias storm 16/08/01.

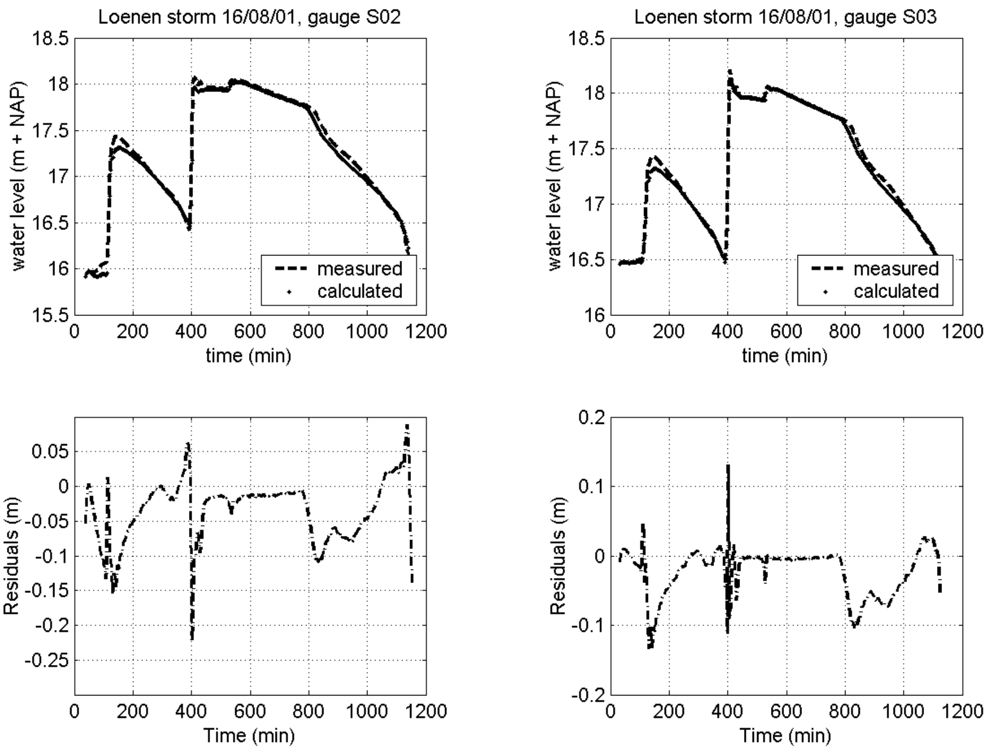


Figure X.19 Residuals storm 16/08/01, gauge S02 (left) and gauge S03 (right).

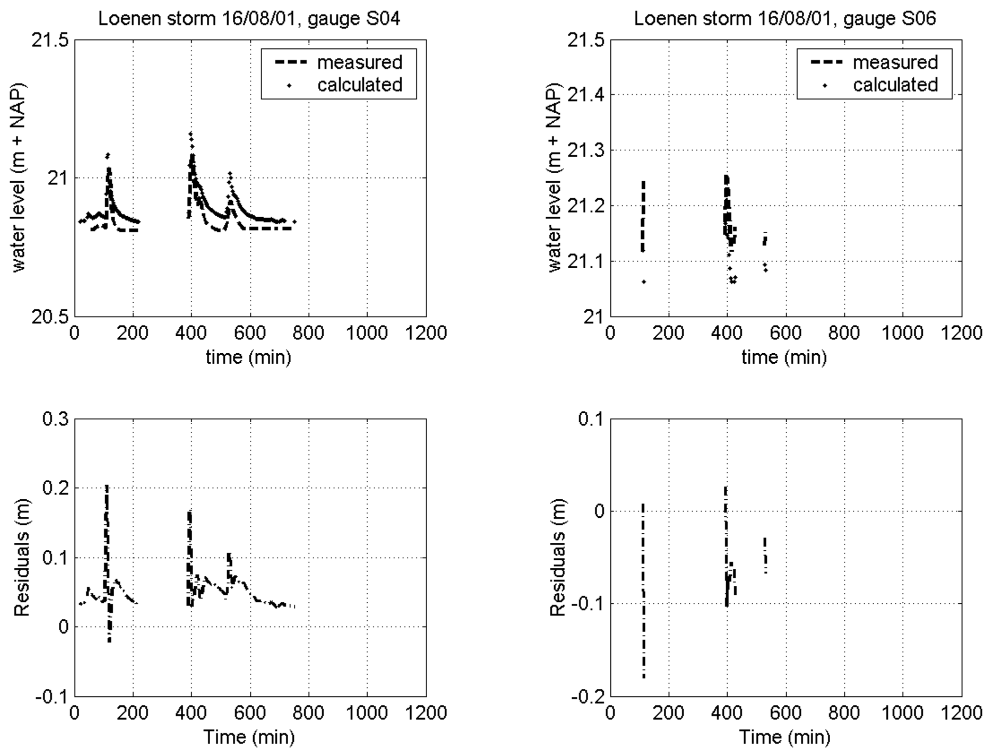


Figure X.20 Residuals storm 16/08/01, gauge S04 (left) and gauge S06 (right).

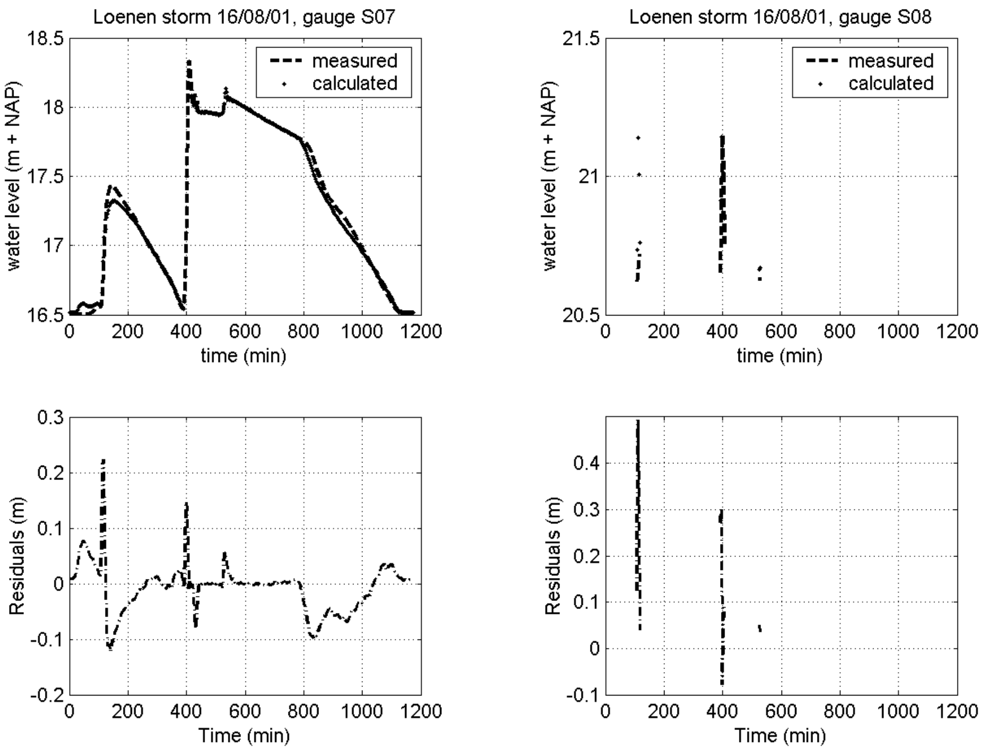


Figure X.21 Residuals storm 16/08/01, gauge S07 (left) and gauge S08 (right).

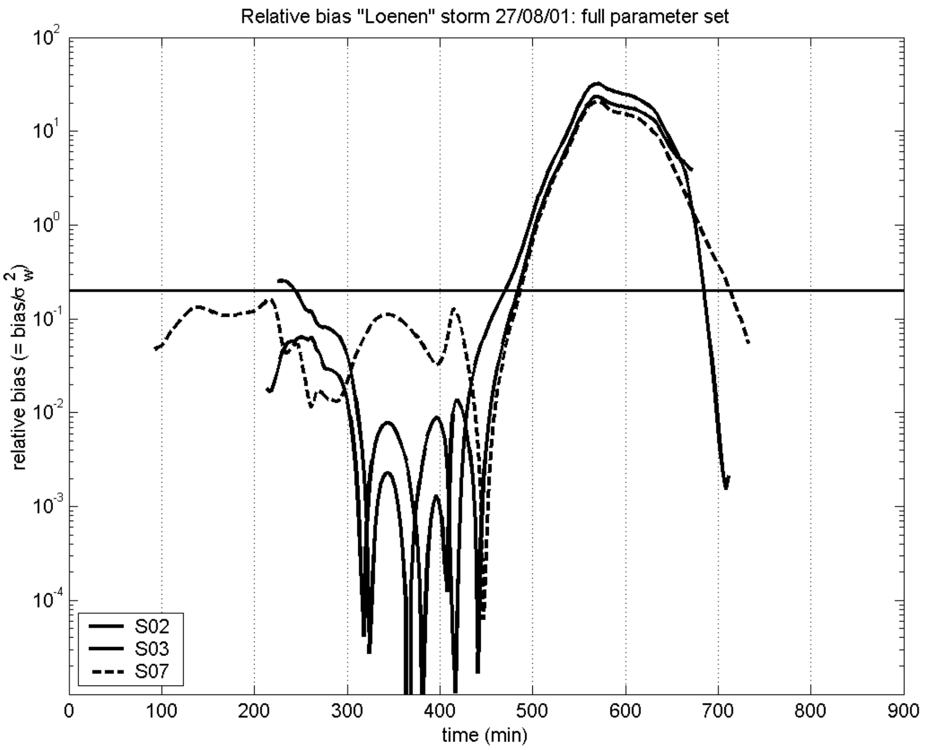


Figure X.22 Relative bias storm 27/08/01.

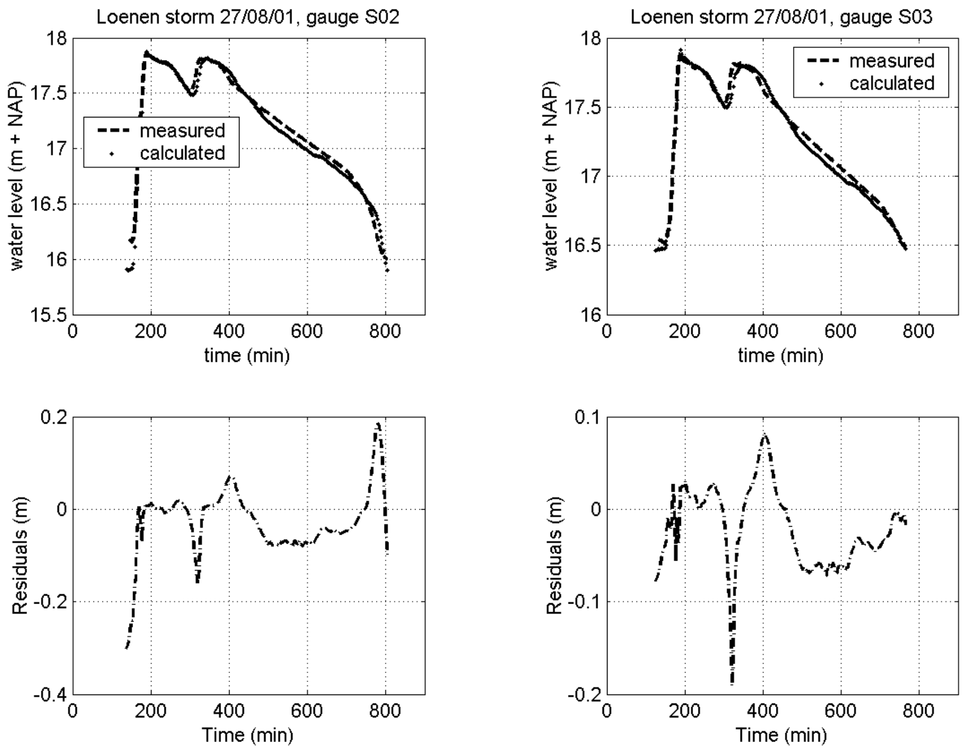


Figure X.23 Residuals storm 27/08/01, gauge S02 (left) and gauge S03 (right).

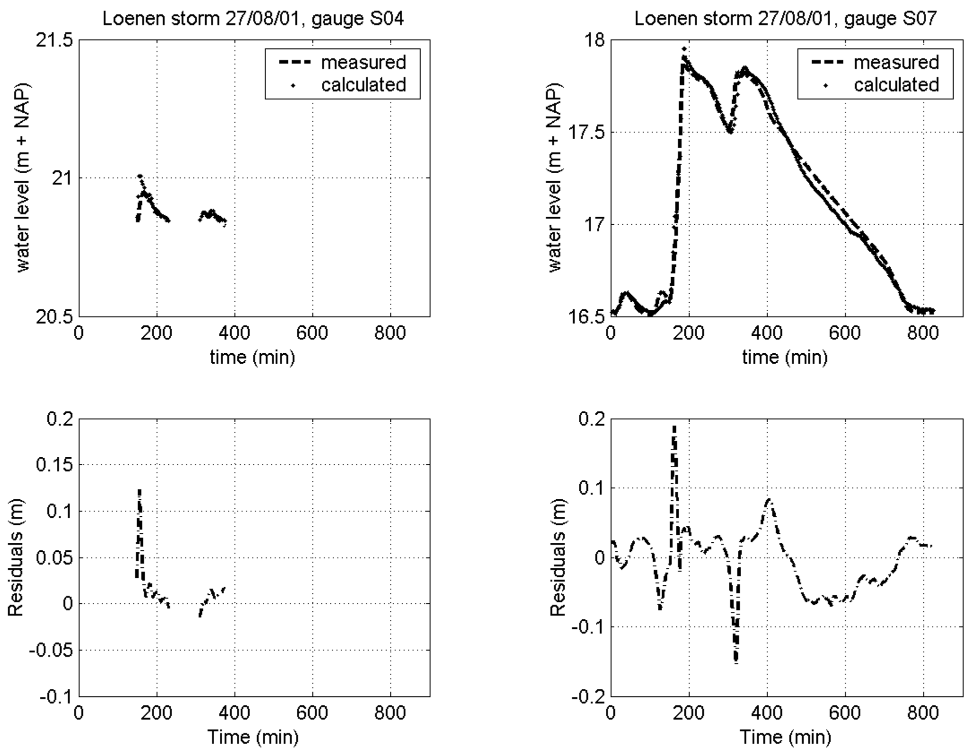


Figure X.24 Residuals storm 27/08/01, gauge S04 (left) and gauge S07 (right).

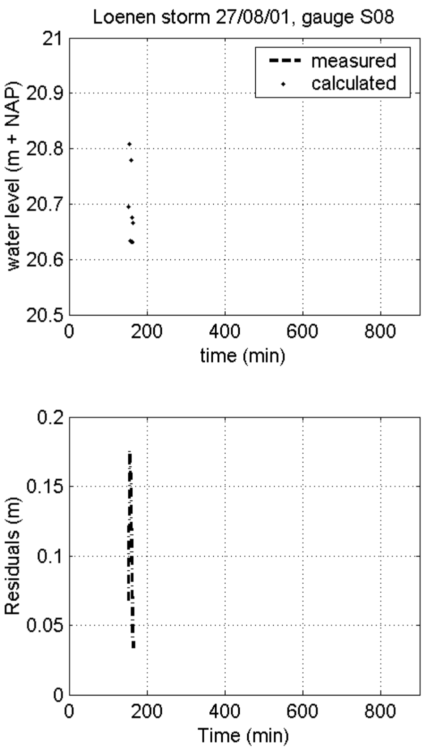


Figure X.25 Residuals storm 27/08/01, gauge S08.

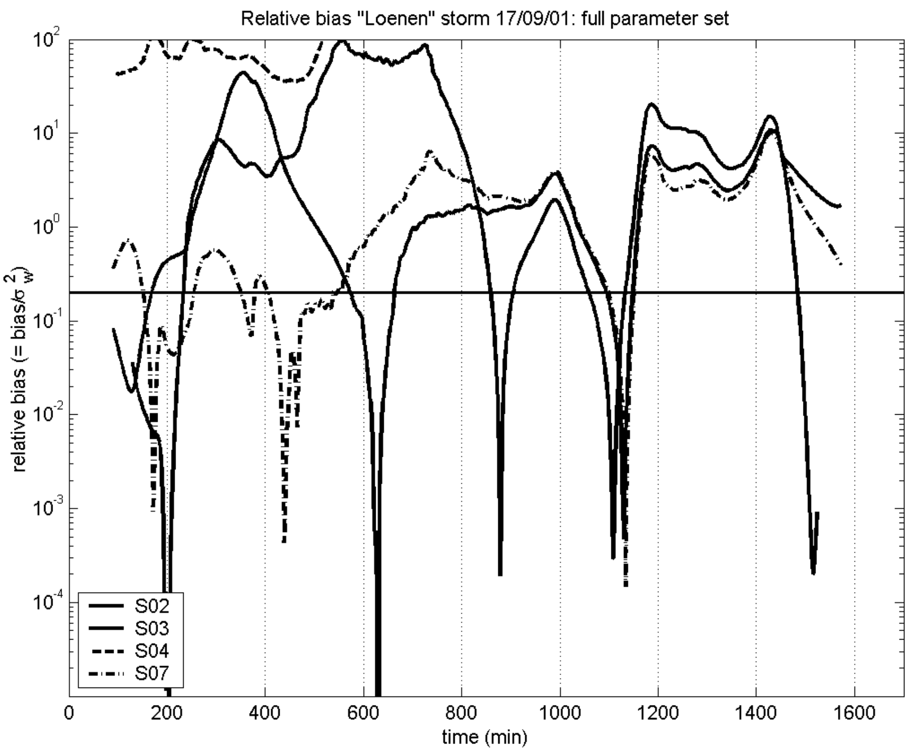


Figure X.26 Relative bias storm 17/09/01.

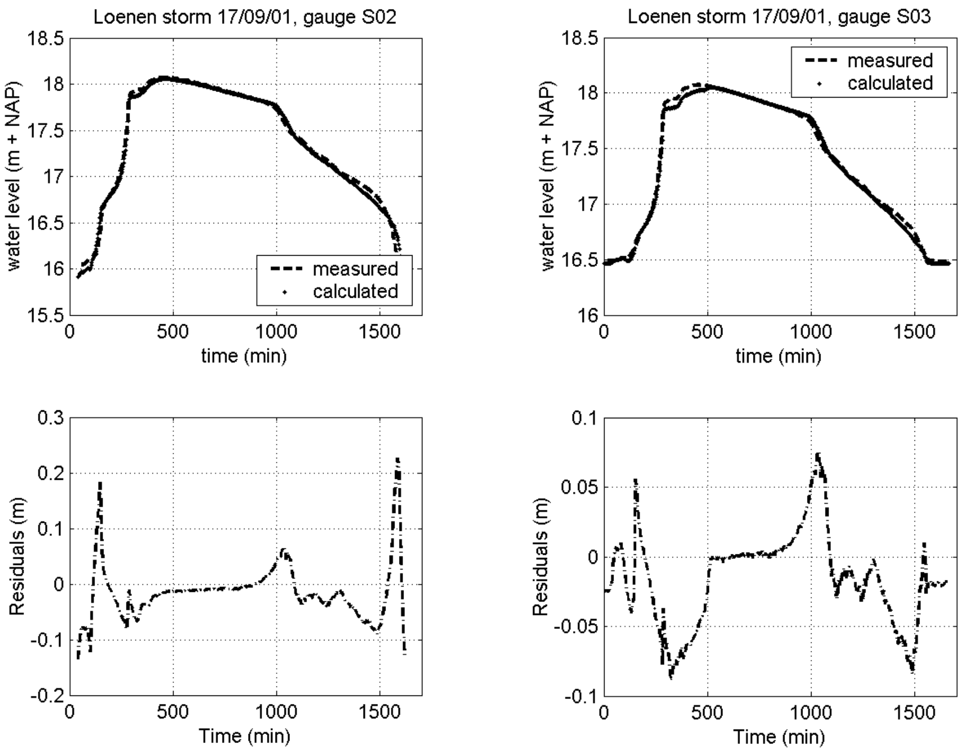


Figure X.27 Residuals storm 17/09/01, gauge S02 (left) and gauge S03 (right).

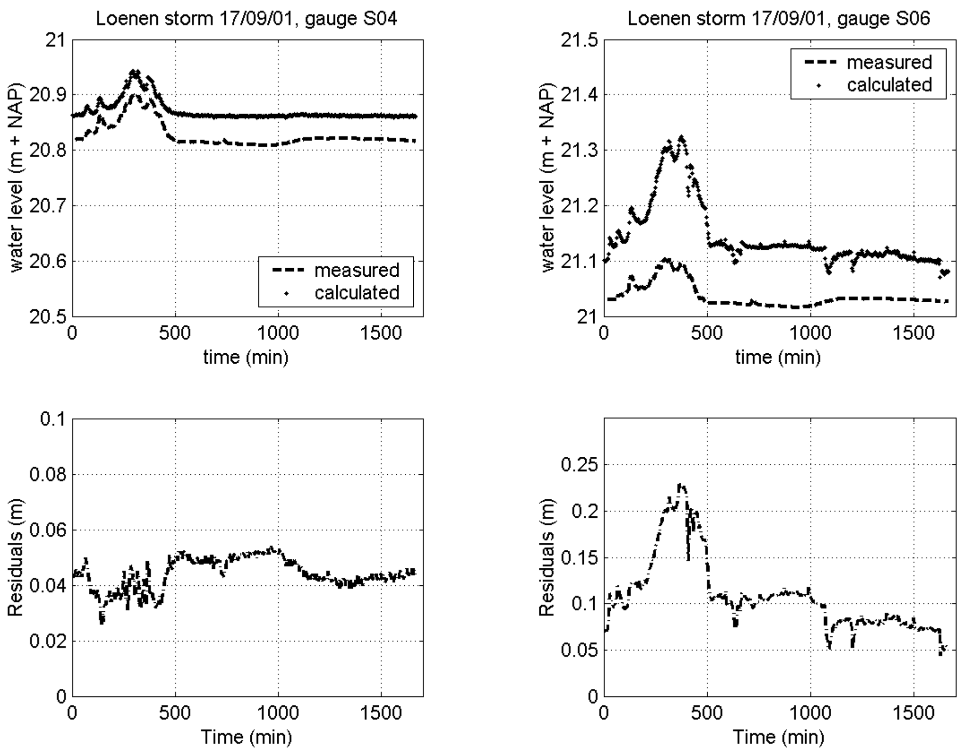


Figure X.28 Residuals storm 17/09/01, gauge S04 (left) and gauge S06 (right).

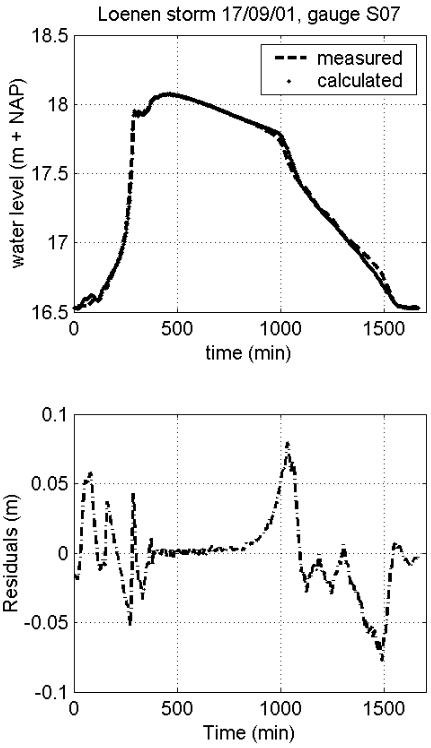


Figure X.29 Residuals storm 17/09/01, gauge S07.

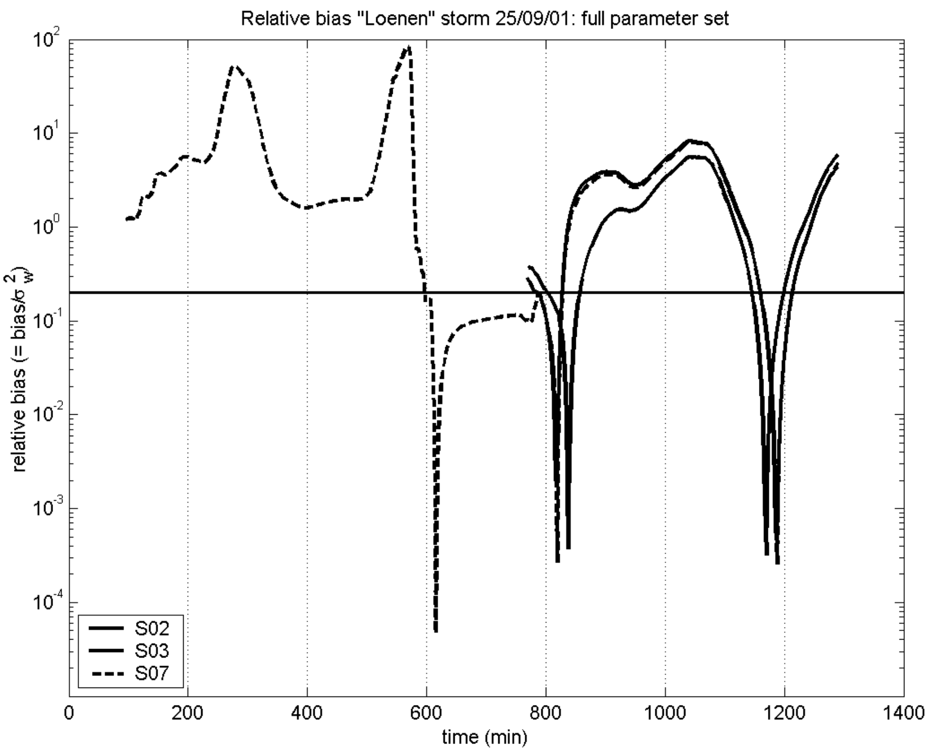


Figure X.30 Relative bias storm 25/09/01.

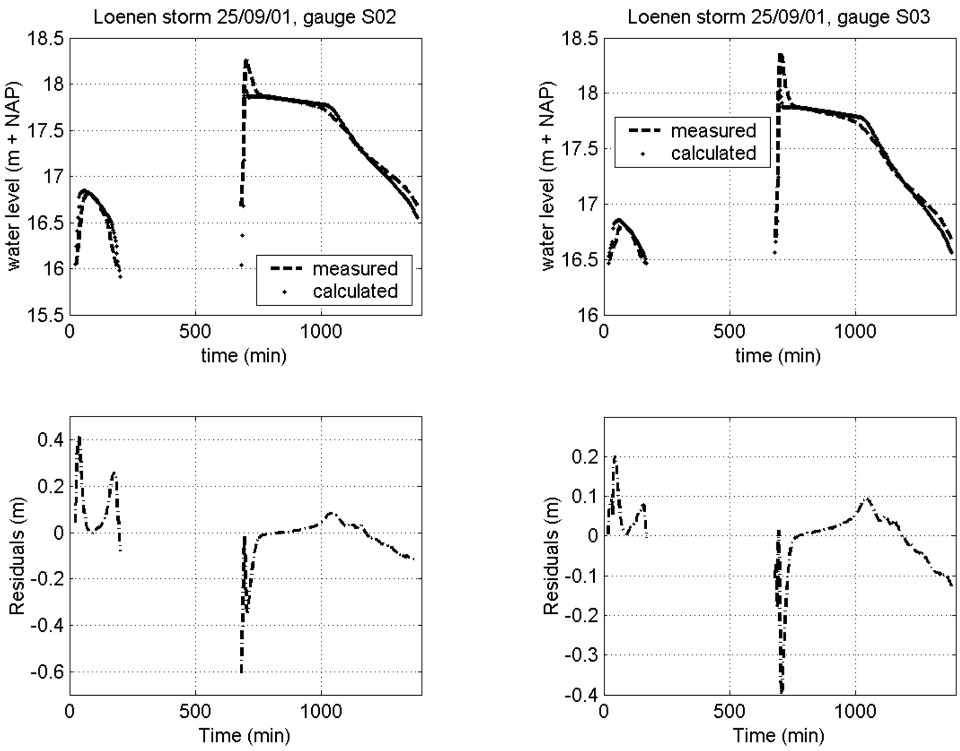


Figure X.31 Residuals storm 25/09/01, gauge S02 (left) and gauge S03 (right).

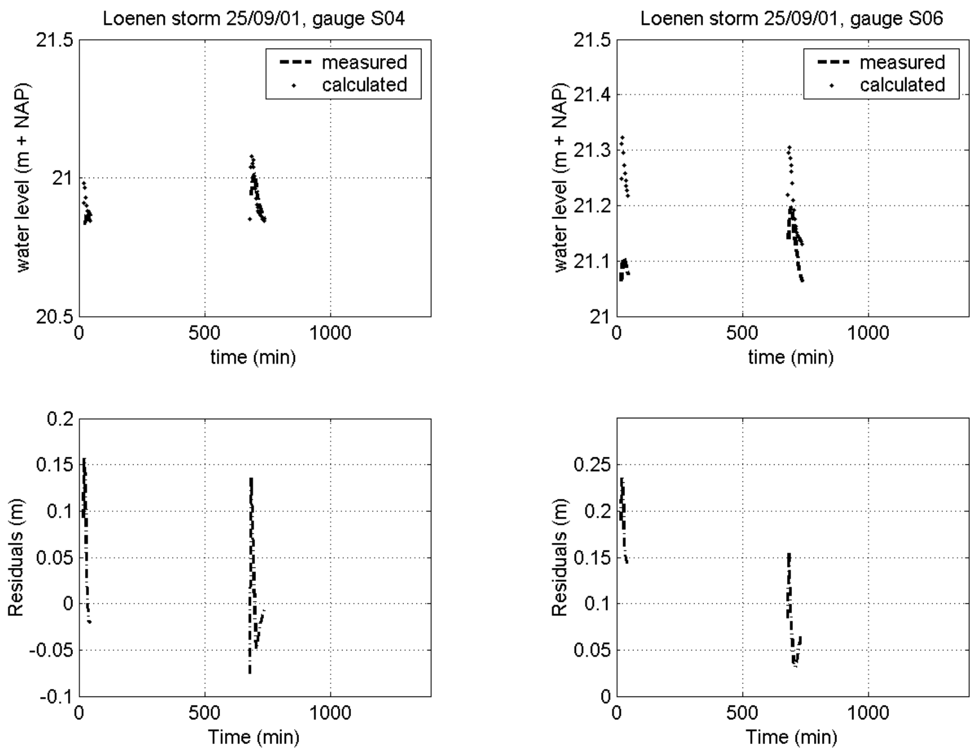


Figure X.32 Residuals storm 25/09/01, gauge S04 (left) and gauge S06 (right).

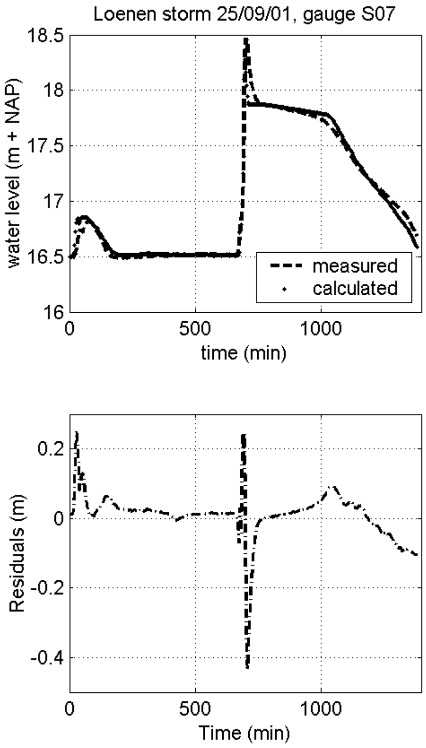


Figure X.33 Residuals storm 25/09/01, gauge S07.

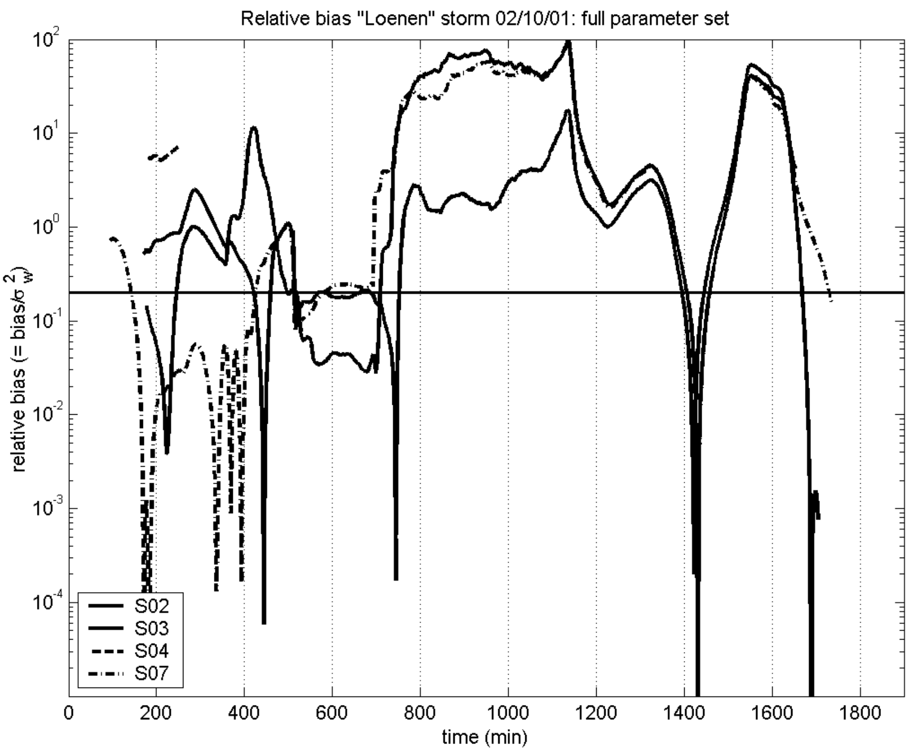


Figure X.34 Relative bias storm 02/10/01.

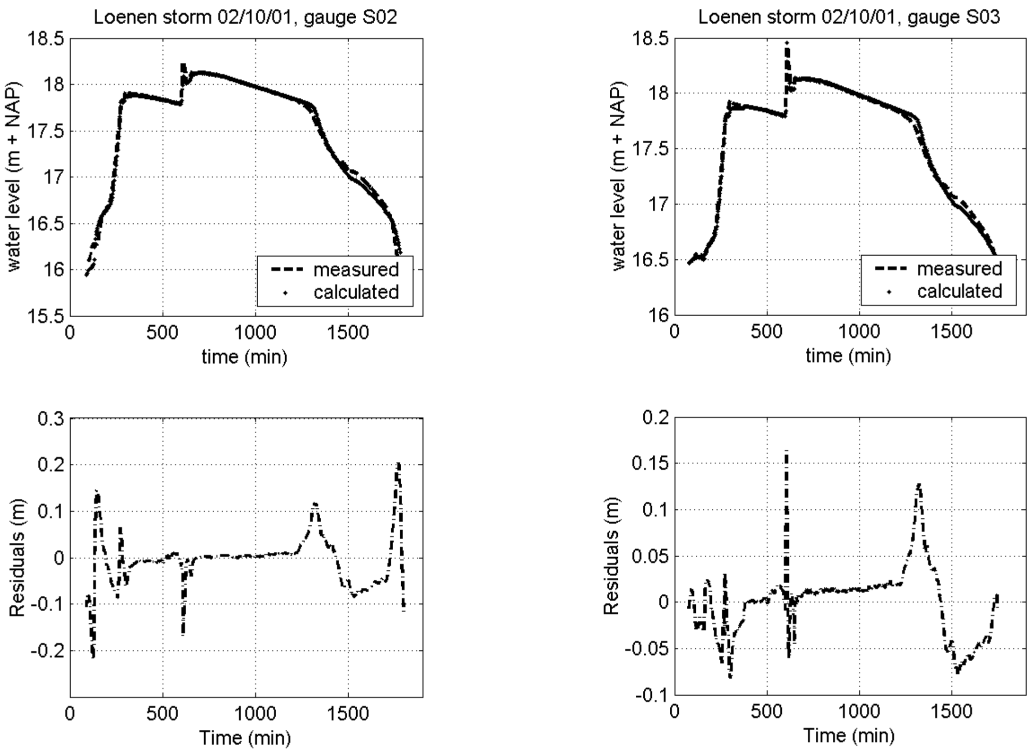


Figure X.35 Residuals storm 02/10/01, gauge S02 (left) and gauge S03 (right)

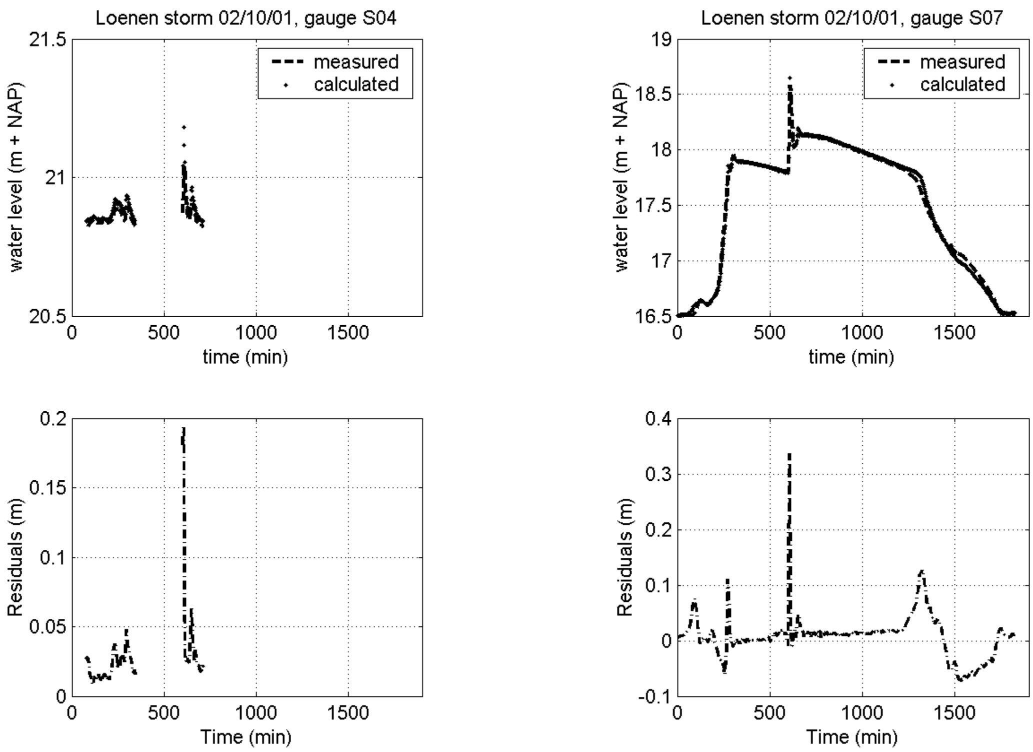


Figure X.36 Residuals storm 02/10/01, gauge S04 (left) and gauge S07 (right)

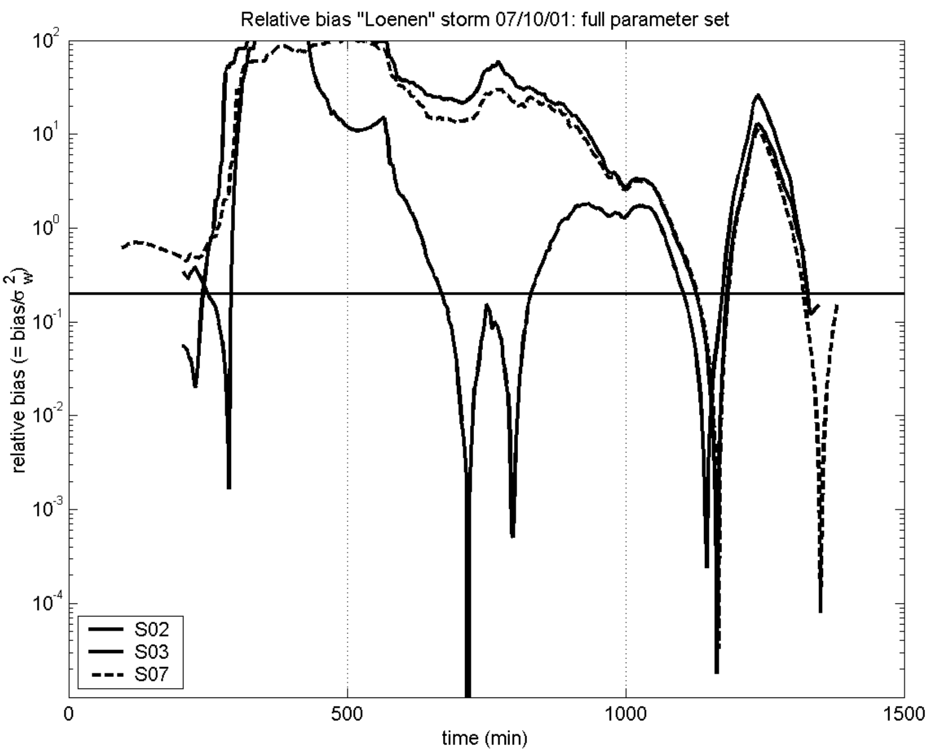


Figure X.37 Relative bias storm 07/10/01.

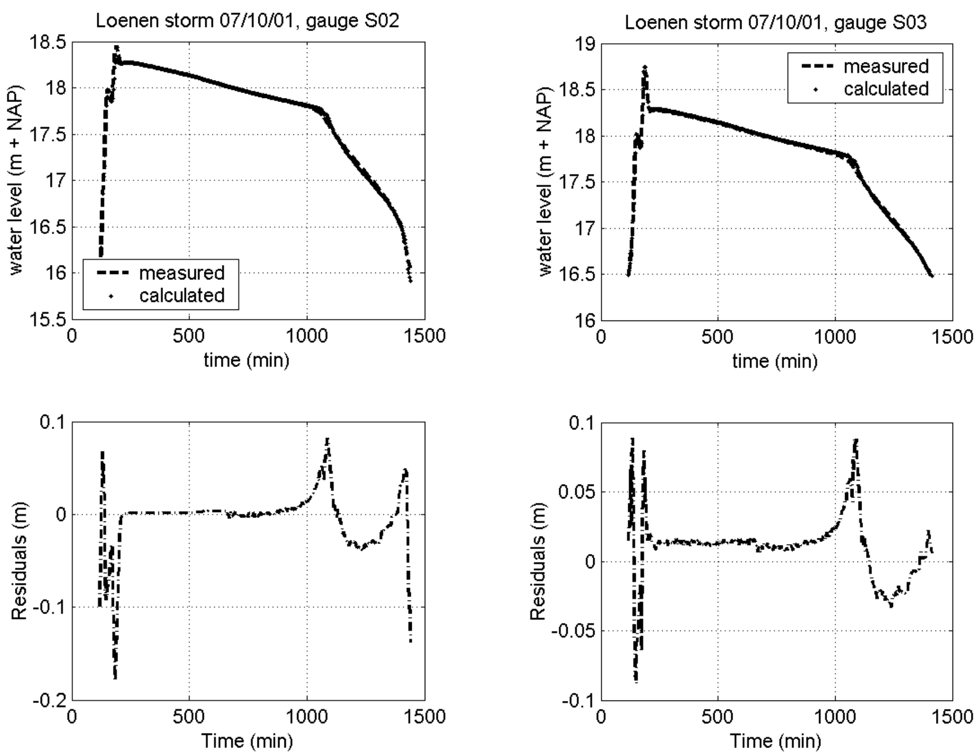


Figure X.38 Residuals storm 07/10/01, gauge S02 (left) and gauge S03 (right)

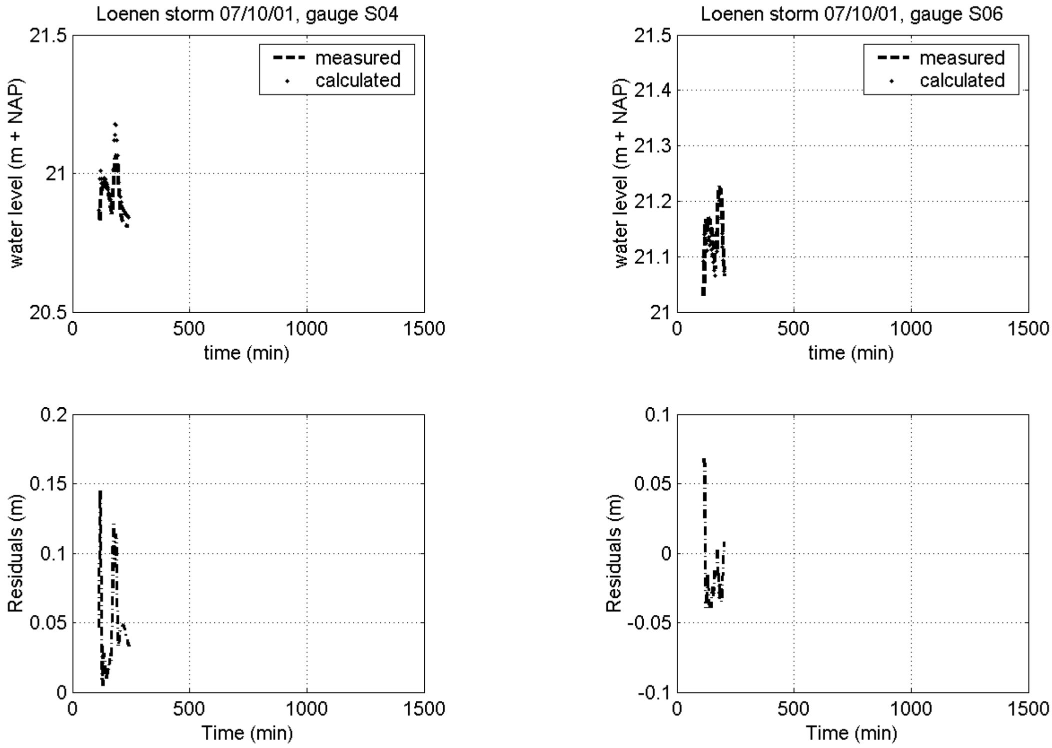


Figure X.39 Residuals storm 07/10/01, gauge S04 (left) and gauge S06 (right)

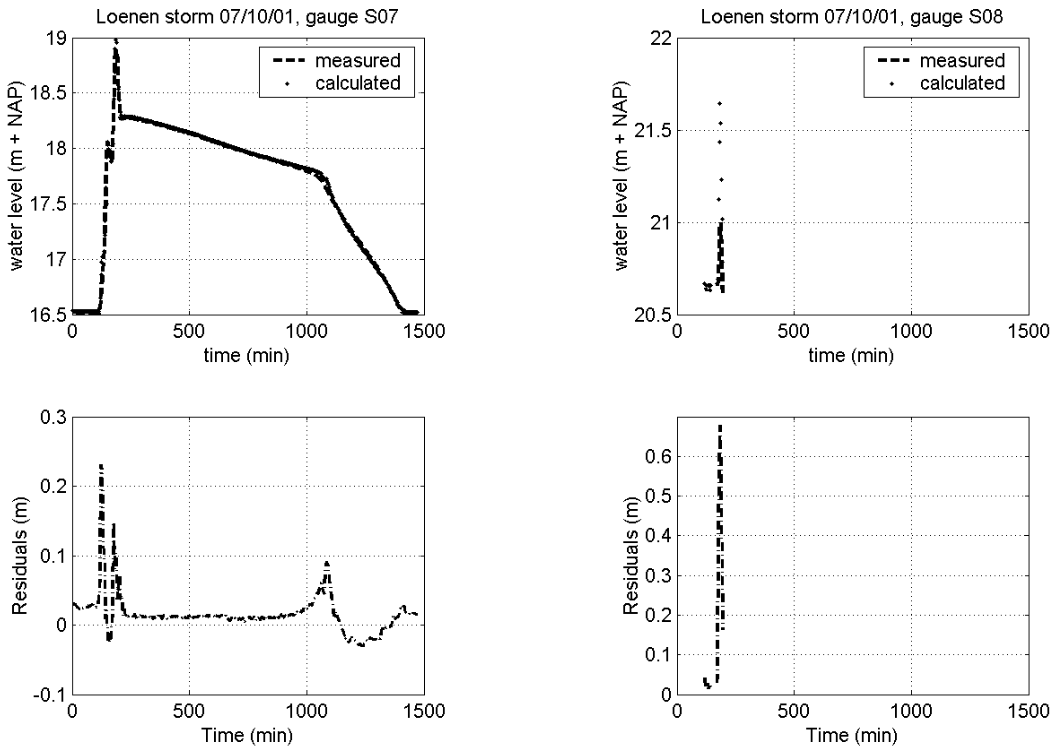


Figure X.40 Residuals storm 07/10/01, gauge S07 (left) and gauge S08 (right)

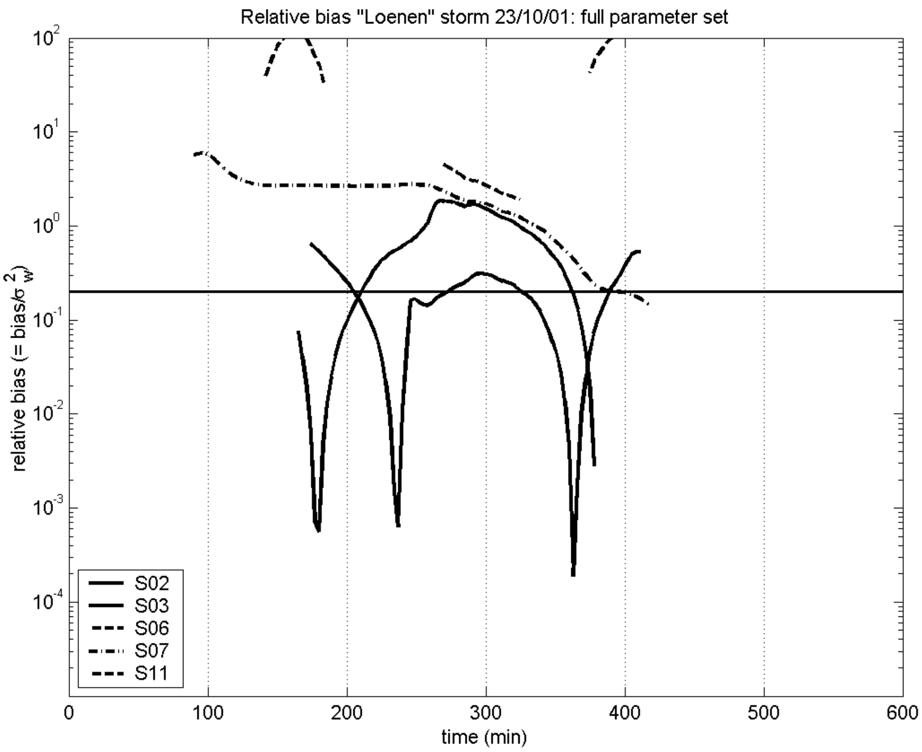


Figure X.41 Relative bias storm 23/10/01.

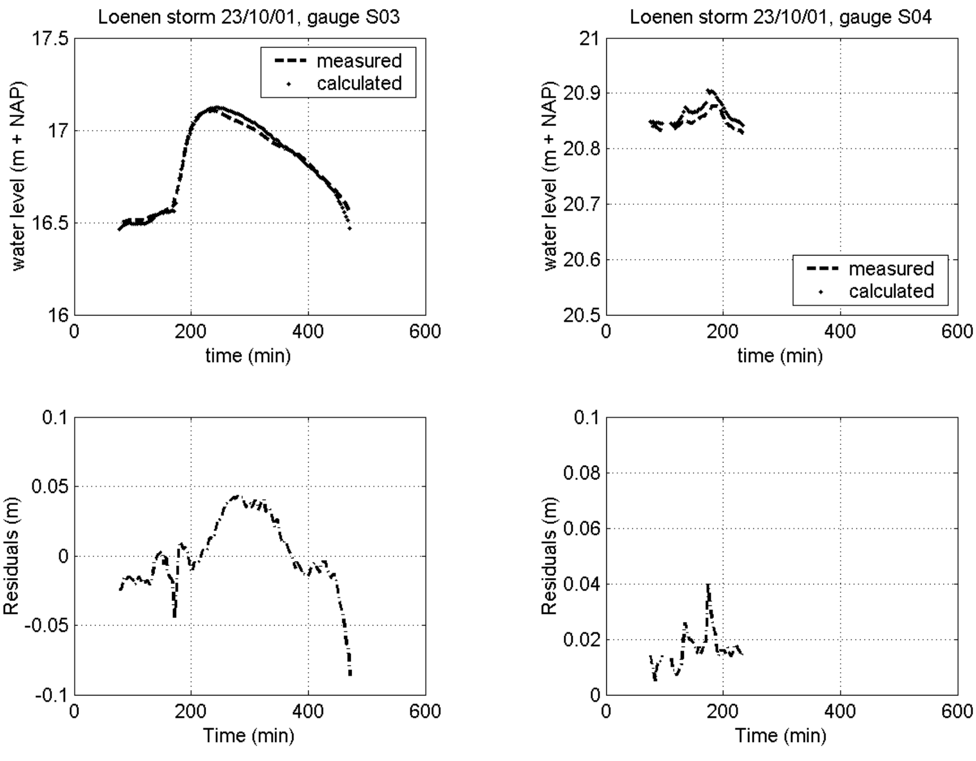


Figure X.42 Residuals storm 23/10/01, gauge S03 (left) and gauge S04 (right).

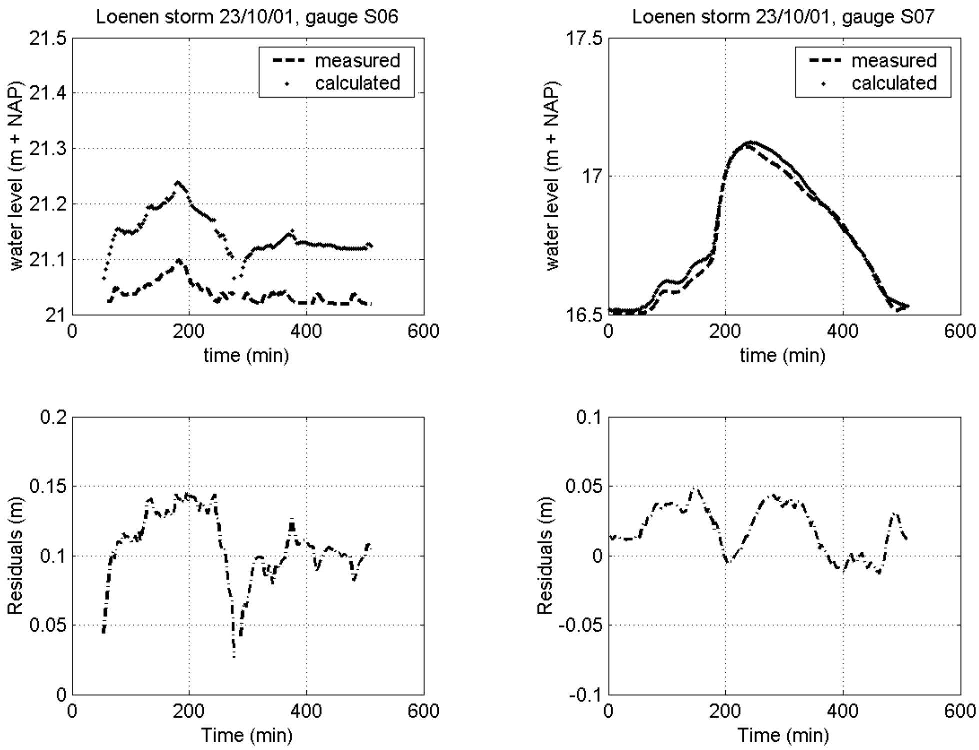


Figure X.43 Residuals storm 23/10/01, gauge S06 (left) and gauge S07 (right).

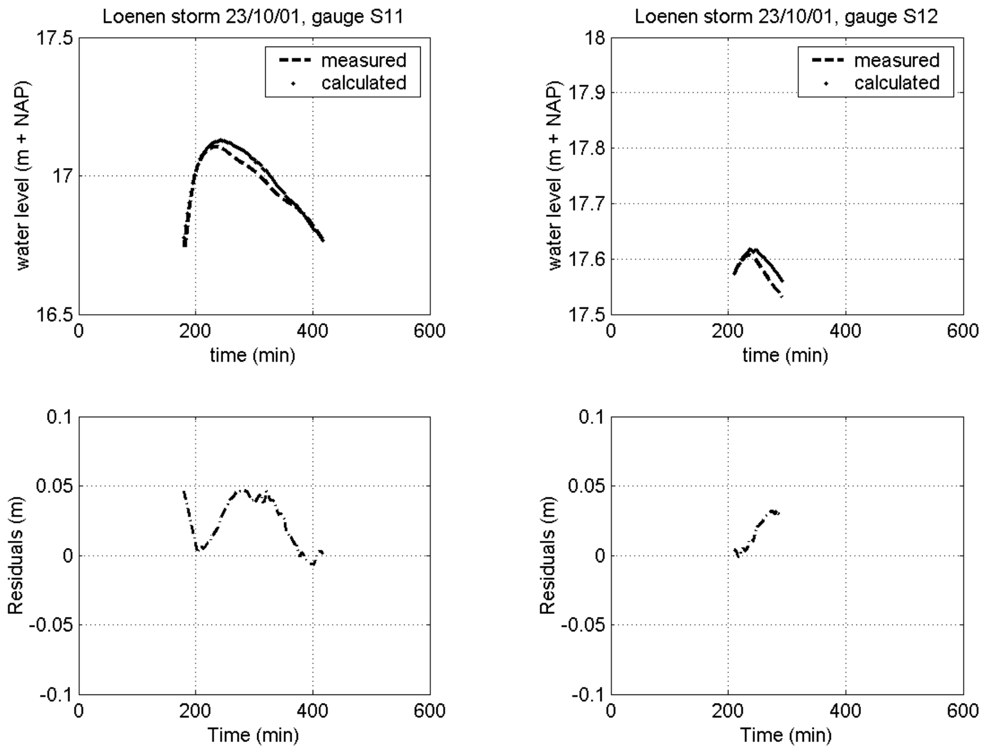


Figure X.44 Residuals storm 23/10/01, gauge S11 (left) and gauge S12 (right).

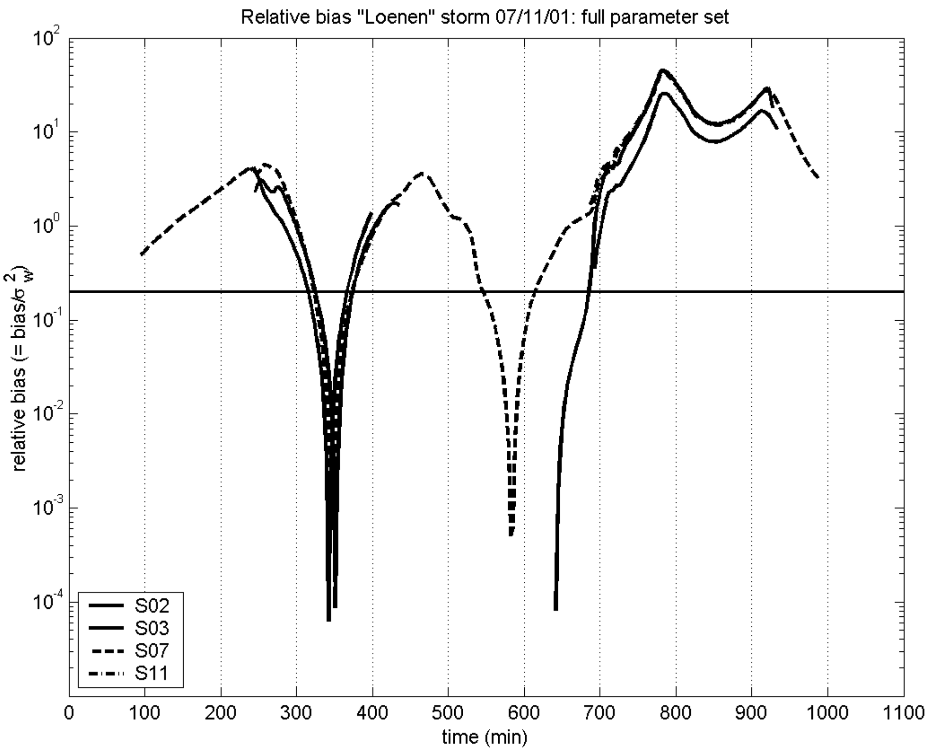


Figure X.45 Relative bias storm 07/11/01.

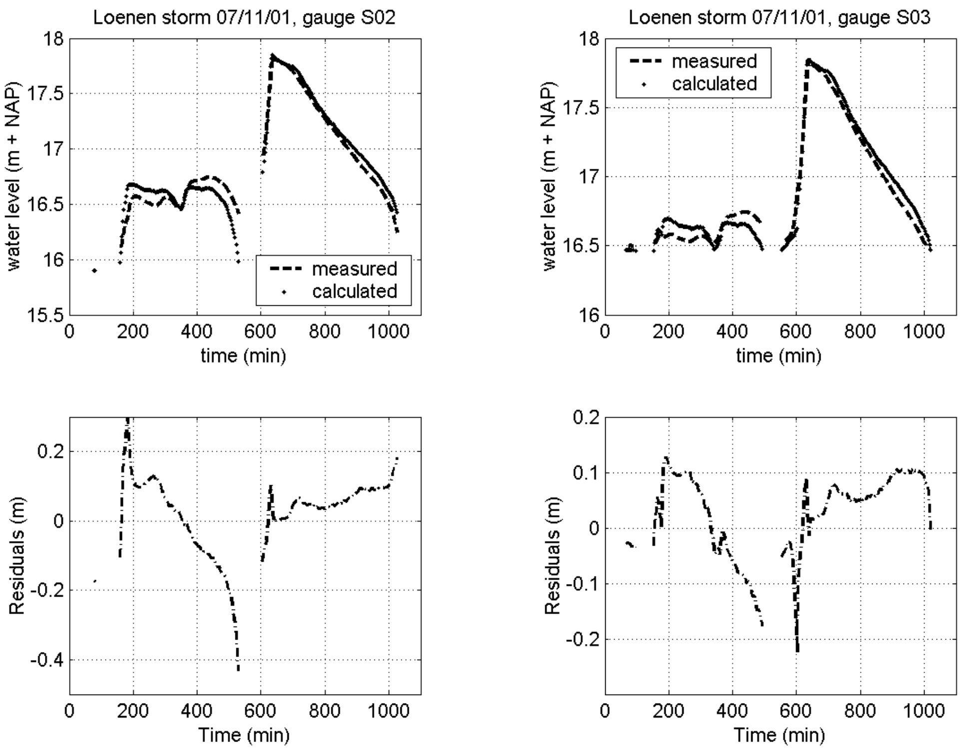


Figure X.46 Residuals storm 07/11/01, gauge S02 (left) and gauge S03 (right).

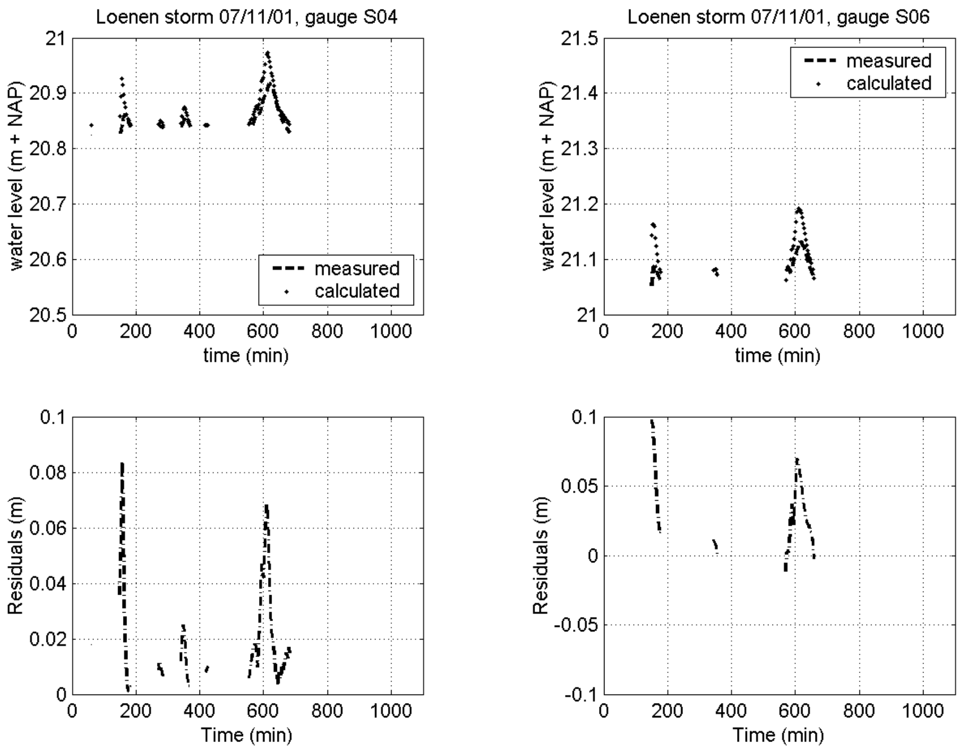


Figure X.47 Residuals storm 07/11/01, gauge S04 (left) and gauge S06 (right).

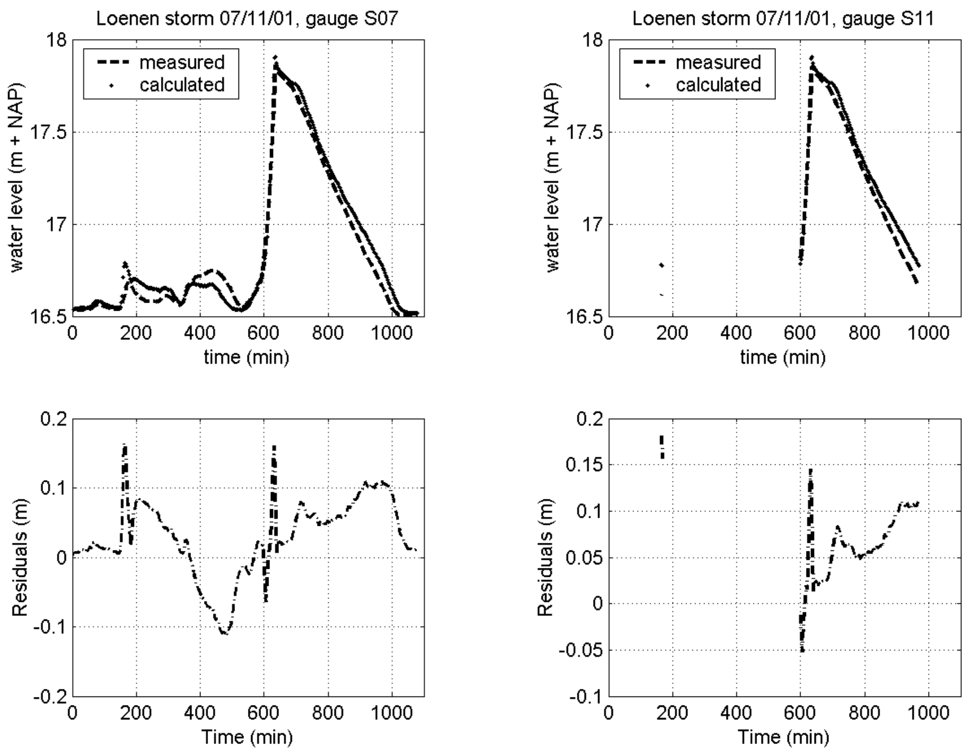


Figure X.48 Residuals storm 07/11/01, gauge S07 (left) and gauge S11 (right).

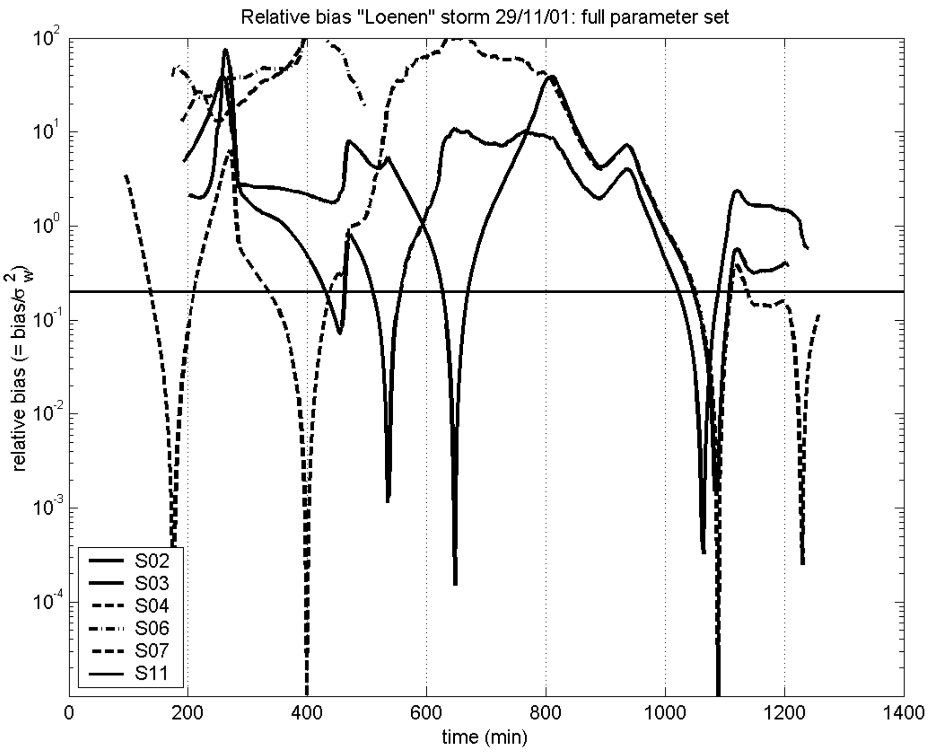


Figure X.49 Relative bias storm 29/11/01.

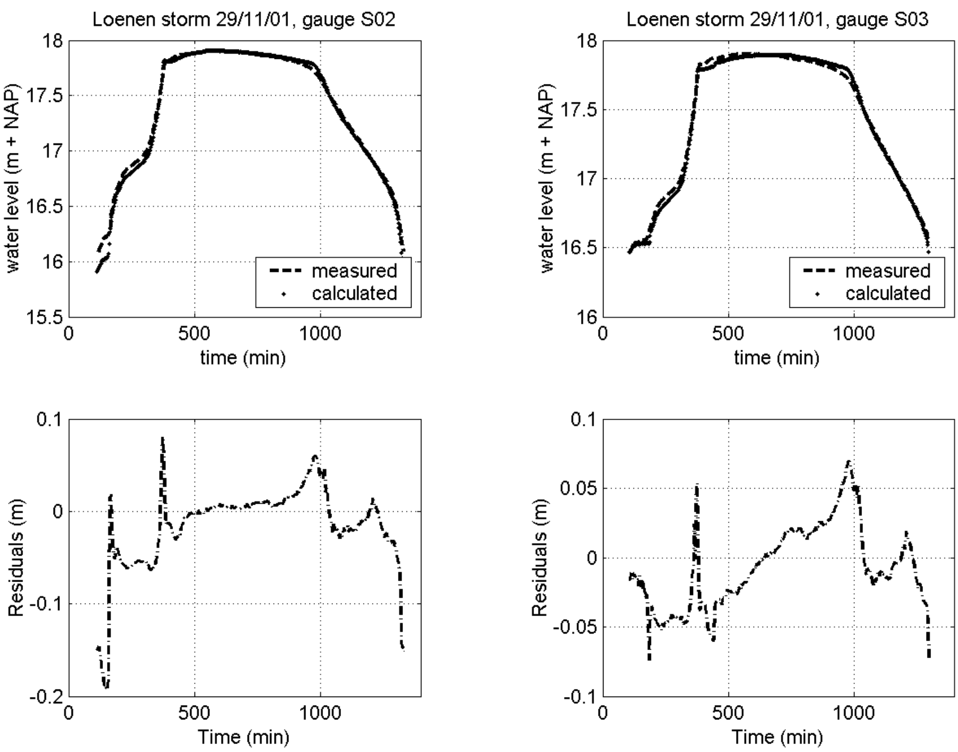


Figure X.50 Residuals storm 29/11/01, gauge S02 (left) and gauge S03 (right).

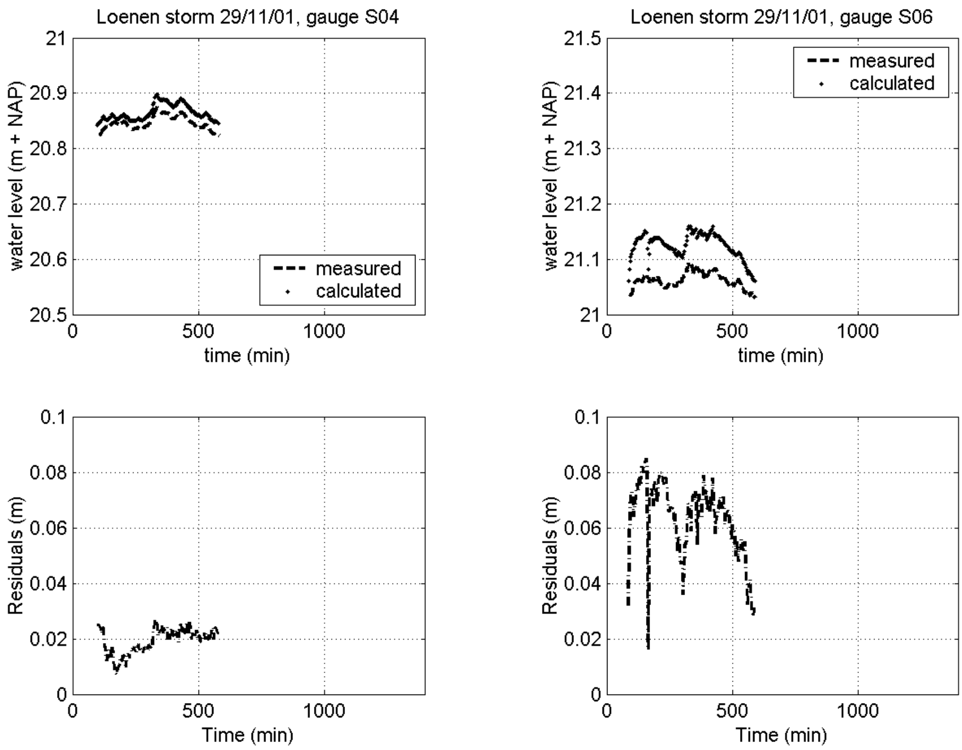


Figure X.51 Residuals storm 29/11/01, gauge S04 (left) and gauge S06 (right).

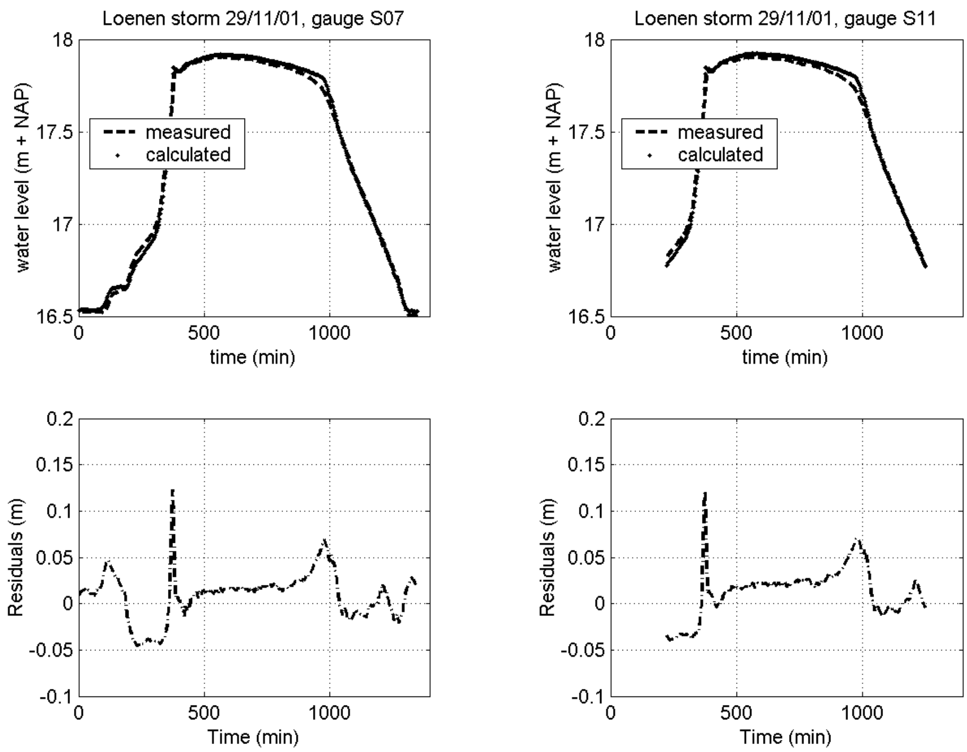


Figure X.52 Residuals storm 29/11/01, gauge S07 (left) and gauge S11 (right).

Appendix XI General process for wastewater system optimisation studies

Appendix XI.1 gives a brief overview of the general process for wastewater system optimisation studies, described in the guideline 'Optimisation of wastewater systems' as part of the Dutch 'Leidraad Riolering' [Stichting RIONED (2003b)]. This process comprises all aspects related to wastewater system optimisation studies, from the first initiative to the operation and management of measures implemented.
Apart from this process, appendix XI.2 proposes a procedure for the technical phase of the optimisation process, facilitating the introduction of knowledge on the interactions within the wastewater system. This procedure is a slightly adapted version of the procedure described by [Boomgaard et al. (2002)].

XI.1 Dutch guideline for optimisation of wastewater systems

The guideline 'Optimisation of wastewater systems' as part of the Dutch 'Leidraad Riolering' has been issued recently [Stichting RIONED (2003b)]. T guideline aims at increasing the success rate of wastewater system optimisation studies by providing a general structure for wastewater system optimisation studies. The practical guideline, see figure XI.1, for wastewater system optimisation describes the process of optimisation, irrespective of the optimisation objective, including administrative, political and technical issues.

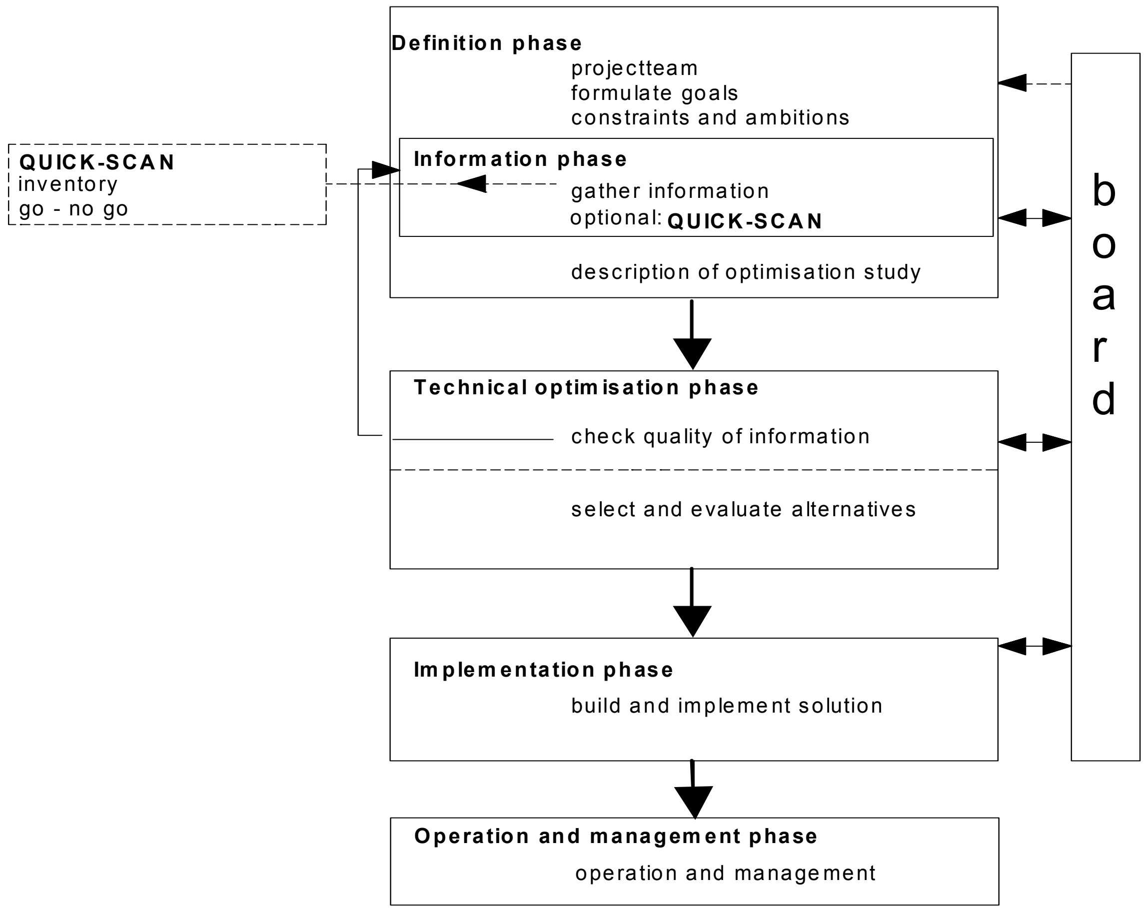


Figure XI.1. Process chart for wastewater system optimisation [Stichting RIONED (2003b)]. In this thesis the focus is on the technical optimisation phase. The board represents the local authorities, i.e. the water boards and municipalities.

The first phase, the definition phase, is often initiated by the responsible authorities. As a first step, a project team is formed, consisting of representatives of the responsible authorities for the sewer systems, the wastewater treatment and the receiving waters. For large wastewater systems, the number of team members may easily reach double figures. Once formed, the team will have to formulate objectives, constraints and ambitions. This is considered to be a very important step within the whole optimisation process, since the problem definition will determine the necessary level of detail. If, for instance, the performance of a sewer system is assessed by the average annual number of CSO events considerably less detailed data and models are necessary compared to the situation in which performance is assessed by the CSO load per event.
Within the definition phase special attention has to be paid to the collection of information on the wastewater system. The availability and quality of the data, such as sewer system dimensions or lay out and wastewater treatment plant performance, determine to a large extent the quality of the optimisation study result. Since the availability and quality of the information is of utmost importance the collection of the necessary information is often performed within an intermediate information phase.
A ‘quick-scan’, a brief inventory of available data and system characteristics, is often advocated to determine the potential for optimisation and to identify noticeable wastewater system characteristics [Mameren, van (2001)]. A quick-scan may reveal parts of the wastewater system having a striking over- or under capacity, such as unused hydraulic or biologic treatment capacities or, in contrast, overloaded secondary clarifiers. In general, no simulations are performed at this stage of the process, but merely a comparison of system performance to well-known reference figures.
The results of the quick-scan are often used to be enable the responsible authorities to take a go/no go decision for the rest of the optimisation study.

Once the optimisation problem has been properly defined and all the necessary information is available the technical optimisation phase starts. This phase is often considered to be the ’real’ optimisation phase and comes down to *evaluating* a number of potential measures in order to be able to come up with the ‘*optimal*’ set of measures given the constraints. A more detailed description of how the various measures can be evaluated and the available methodologies for identifying the optimal set of measures are given in chapter 5.

As soon as the authorities have selected a set of measures the implementation of the measures may take place. Within this implementation phase, it is of utmost importance to regularly check whether the assumptions made during the optimisation study still hold. In daily engineering practice, the actual provision costs are known to differ strongly from the assumed costs, as often not all aspects involved in costing can be taken into account in advance [FWR (1998)]. At the end of the implementation phase, the operation and management phase of the implemented measures commences.

XI.2 Procedure for wastewater system optimisation

A wastewater system optimisation problem can be defined by its objectives and constraints. The definition of these objectives and constraints is the responsibility of all actors involved (water board(s), municipalities, WWTP operator) in the optimisation process. Special attention, however, should be given to the quantifiability of the objectives and constraints. Once the optimisation problem has been defined, the technical optimisation phase begins. For this phase, the procedure shown in figure XI.2 is proposed.

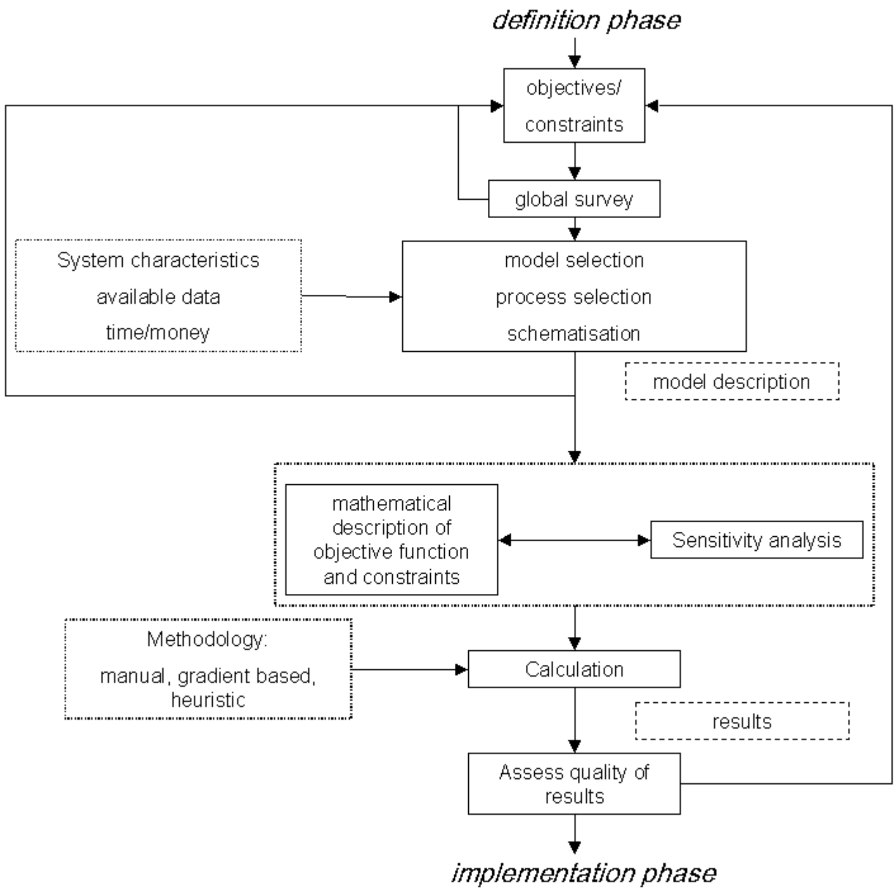


Figure XI.2 Optimisation procedure.

Global system survey

The waste water system is globally analysed to identify uncommon characteristics or possible causes of unusually weak system performance. These are characteristics that can be identified by expert judgement in a simple survey, giving a global indication of system performance and main limitations. Examples of uncommon system characteristics are:

- incompatibility of pumping capacities with discharge capacities of connected rising mains or of booster stations with contributing pumping stations [Witteveen+Bos (2001)];
- capacity of pumping station exceeding by far actual loading [Bixio et al. (2001)]. As a result, the pump chamber acts as a settling tank (although without sludge removal) during DWF;
- performance of WWTP not conforming expected performance. In this case, [Kappeler and Gujer (1993)] state that as long as the WWTP does not perform well, further optimisation of the wastewater system is useless.

When the global survey reveals uncommon system characteristics either the objectives and constraints should be reconsidered or measures should be taken to solve the encountered problems. It should be noted, however, that global system surveys might already have taken place as part of a quick-scan during the definition phase of the optimisation process.
Furthermore, during the global survey expert judgement will have to provide an overview of possible types of measures. This inventory of measures will be one of the inputs for the model composition phase, as the models to be applied need to be detailed enough to calculate the effect on wastewater system performance of each measure.

Model composition

Model composition is the phase of the optimisation study incorporating knowledge on the interactions within the wastewater system.

The models to be used in an optimisation study should be as simple as possible (thus limiting data acquisition and calculation efforts) and at the same time detailed enough to be able to judge whether wastewater system performance complies with all standards [FWR (1998); Rauch et al. (1998)]. Consequently, the standards, normally embedded in the objectives and constraints of the optimisation study, have a strong relation with the necessary level of detail of the sewer system and wwtp models to be applied.

The impact of the standards is especially noticeable with respect to:

- parameters involved: the level of faecal coliforms in wwtp effluent is fairly constant and does not necessitate detailed modelling [Rauch et al. (1998)], whereas the level of ammonium in wwtp effluent can vary significantly, necessitating a more detailed model. The same holds for sewer system modelling: assessing the theoretical annual overflow frequency [Ribius (1951); Vaes (1999)] is less demanding in terms of detail in the sewer model than the annual COD load discharged through a CSO.
- time scale: wwtp effluent e.g. can be judged by 2 hour peak values [Urbaniak (1998)] or by the daily average.
- incorporation of statistics into the standards. Two extremes can be noted: assessment of wastewater system performance based on mean values or by return periods. Examples of the mean value approach are annual overflow volumes for sewer systems [CUWVO (1992)] or annual nitrogen removal efficiency [Velde, van de (2002)]. The assessment of return periods, possibly expressed by amplitude/duration/frequency relationships, requires long-term statistical information. This information can be obtained by the following approaches [Schütze et al. (2002)]:
- continuous simulation. a long-time series of rainfall is used as input to the model and a statistical analysis is performed on the model results a posteriori;
- event based simulation approach: a number of events is used as input to the model and the amplitude/duration/frequency relationship is based on the a priori known statistics of the rainfall. This approach is only valid for linear systems.

  Since wastewater systems often do not show linear system behaviour, normally continuous simulation is necessary, resulting in unacceptable simulation times. A reduction of the rainfall series (eliminating inter-event dry weather periods) is an often applied technique [Vaes (1999)], which works well for quantitative modelling, but has only a rather limited effect as soon as water quality aspects are taken into account [Jack (1999); Schütze et al. (2002)]. Another approach is the application of simplified models [Meirlaen et al. (2002); Willems (2000); FWR (1998)]. Although simulation times are significantly reduced, the effects of this simplification has to be taken into account when analysing and interpreting models results.

The standards also determine to what extent the interactions between the sewer system and the wastewater system are important in an wastewater system optimisation study. If wwtp performance, assessed according to the standards, is affected by fluctuations in influent flow and/or quality, knowledge of the fluctuations in the influent will be required. As this knowledge is to be provided by a sewer model, the requirements for sewer models depends on both the CSO discharge standards and indirectly, the wwtp effluent standards.

Apart from the standards, figure XI.2 illustrates that model composition also depends on:

- system characteristics. The characteristics of a sewer system and a wwtp affect the dynamic response of a wastewater system under transient conditions. E.g. the layout of a sewer system determines the peakedness of the hydrographs and pollutographs. If a sewer system discharges to the wwtp through a pressure main, the plug flow properties of the pressure main cause the pollution load arriving at the wwtp to increase proportional with the flow until all dwf has been replaced by wwf [Graaf, van der (1992)]. Consequently, the larger the volume of the pressure mains, the higher the peaks of the wwtp influent pollutograph will be. The same phenomenon has been observed for large interceptor sewers, containing a large volume of wastewater during dwf [Krebs et al. (1999)]. For wastewater systems with large (in terms of hydraulic retention time) interceptor sewers the plug flow behaviour may dominate the shape of the pollutograph and consequently, detailed modelling of the contributing catchments may not be worthwhile with respect to the interaction with the wwtp.

- available data: the available data can be classified by:
- geometric and structural data of the wastewater system
- inflow to the wastewater system: contributing areas and connected households
- operational data on system performance: pumping rates, influent and effluent monitoring data, additional measured data

  The quality of the available data, in terms of reliability and accuracy, has a strong influence on the quality of the results of the optimisation procedure. Ideally, there is a balance between the quality of the data used and the level of detail applied within the models used for wastewater system optimisation.

  [Korving (2004)] describes a methodology, based on Bayesian statistics, to check the probability of taking the wrong measures within sewer system management, given the uncertainty of the data. This approach can also be applied to wastewater system optimisation.
- available time and money: normally, each wastewater system optimisation study has a certain budget, which, in engineering practice, strongly influences the maximum achievable level of detail. Moreover, the time and money budget determine to what extent additional (e.g. measuring) data can be acquired [Stichting RIONED (2003a)].

The final result of the phase of model composition is a model of (parts of) the wastewater system, schematising the system in sufficient detail while accounting for the processes considered to be important. In case e.g. necessary data on system characteristics is missing and model composition is not possible, additional data acquisition will be necessary or otherwise it is possible to start the optimisation procedure over again and redefine the objectives and constraints.

Mathematical description of objectives and constraints

Usually, the objective(s) and constraints for a wastewater system optimisation study are described in general terms. In order to be able to take these into account within an optimisation study, quantifiable objectives and constraints need to be formulated [Rauch and Harremoes (1999a), (1999b)]. This will result in an objective function, combined with constraints either embedded within the objective function or addressed separately. In general, wastewater system optimisation is a trade off between costs and performance.

The inclusion of the costs within wastewater system optimisation studies is in the Netherlands normally limited to provision costs [Boomgaard et al. (2001a)], although sometimes also operational costs are included [e.g. Willemsen (2000)].
In the UK, a whole life costing approach is being developed for sewer asset management. Within this approach, the emphasis is on including all relevant costs, including provision, replacement, maintenance and operation [Cashman et al. (2002)]. The level of detail of the assessment of costs should be defined in an early stage of the optimisation process by the stakeholders.

The performance of a wastewater system is to be assessed by performance indicators. In wastewater optimisation studies performed in the Netherlands, the performance indicators were limited to wwtp effluent parameters and CSO discharge parameters [Mameren, van (2001)]. Theoretically, all aspects involved in wastewater system performance should be taken into account by the use of measurable performance indicators [Matos et al. (2002) (2003); Ashley and Hopkinson (2002)]. However, a sewer system benchmark study in the Netherlands revealed that the data, necessary to assess the performance of sewer systems to other topics than directly related to the standards and regulations, is very hard to get [Stichting Rioned (2003b)].

The costs are normally incorporated in an objective function, shown in equation XI.1, as they are relatively easy to quantify. In addition, a penalty function can be included to embed the performance of the wastewater system in the objective function. The latter requires capitalising wastewater system performance, potentially introducing subjectivity in the objective function, as e.g. no universal value for the discharge of 1 m^3 wastewater through a CSO exists. This topic is addressed in more detail by [Korving (2004)]. However, wastewater performance can also be addressed separately [Boomgaard et al. (2001a)].

$$S_{(costs)} = \sum_{i=1}^{n} \alpha_i (M_i) + \alpha_p (P_{required} - P) \qquad \text{(eq. XI.1)}$$

where:

S	objective function
α_i	specific cost function
M_i	specific measure
α_p	penalty function
P	performance level

Sensitivity analysis

Even a small wastewater system consists of a large number of conduits, manholes pumping stations, CSOs and a wwtp. Therefore, theoretically, a myriad of combinations of improvement measures, such as increasing pumping capacities, building additional storage or applying RTC, exists. An optimisation procedure processing all possible variables makes no sense, as in general part of the parameters is of minor influence. A sensitivity analysis is used to eliminate these elements from the final set of optimisation parameters, thereby reducing the magnitude of the optimisation problem. The methodology applied within the sensitivity analysis can range from a practical engineering approach to a singular value decomposition performed on model results [Clemens (2001)]. Mostly, the number of measures to be evaluated within the following part of the optimisation study are significantly reduced after performing a sensitivity analysis.

Search for optimum solution of objective function

The actual optimisation comes down to searching for the combination of measures best complying with the combination of objectives and constraints. This search can be performed:

- manually; a limited number of combinations of measures will be selected manually and evaluated. The best option is supposed to represent the 'optimal solution'.
- automatically; an algorithm searches automatically for the 'best' solution. In general, two families of search algorithms can be distinguished:
- gradient based methods
- heuristic methods

The search methodology to be considered to be most appropriate strongly depends on the optimisation problem. Manual optimisation is limited to clear optimisation problems with a rather limited number of variables, while automatic optimisation is more suited for larger and more complicated optimisation problems. However, objective functions for wastewater system optimisation typically show multiple (local) optima and discontinuities, limiting the applicability of many search algorithms such as classical gradient based techniques [Rauch and Harremoes (1999b)]. Heuristic methods have been reported to be capable of dealing with the type of optimisation problem encountered in wastewater system optimisation [Gill et al. (2001)]. Section 5.3 discusses the applicability of two heuristic search algorithms to be applied within wastewater system optimisation studies. Once a satisfying algorithm has been selected, the optimum combination of measures can be searched for.

Assess quality of results

The search for the optimum combination of measures given the objectives and constraints may result in a number of different solutions. Since not every aspect can be included in an optimisation routine, the limited number of ‘best’ options will have to be cross-examined for robustness and applicability much closer by a sensitivity analysis and subsequent expert judgement. The quality of the results strongly depends on the quality of the applied data and models. Especially insignificant results, i.e. the optimal solution performs 5% better than the actual situation, while the uncertainty interval of the model results is ± 50%, should be considered with care.

Appendix XII Optimisation of ammonium concentration in a small river using Simulated Annealing

This appendix builds on the case study of section 6.3. The wastewater system is shown in figure XII.1. In addition to the models described in section 6.3, the river was modelled as a plugflow reactor. Furthermore, it was assumed that both the CSO and the wwtp discharge at the same location and that the discharged effluent and CSO volume mix instanteneously over the cross section of the river. The river has a base flow of 1 m^3/s and a background ammonium concentration of 0.1 mg N/l. The water quality standard for ammonium in the river is 1 mg/l.

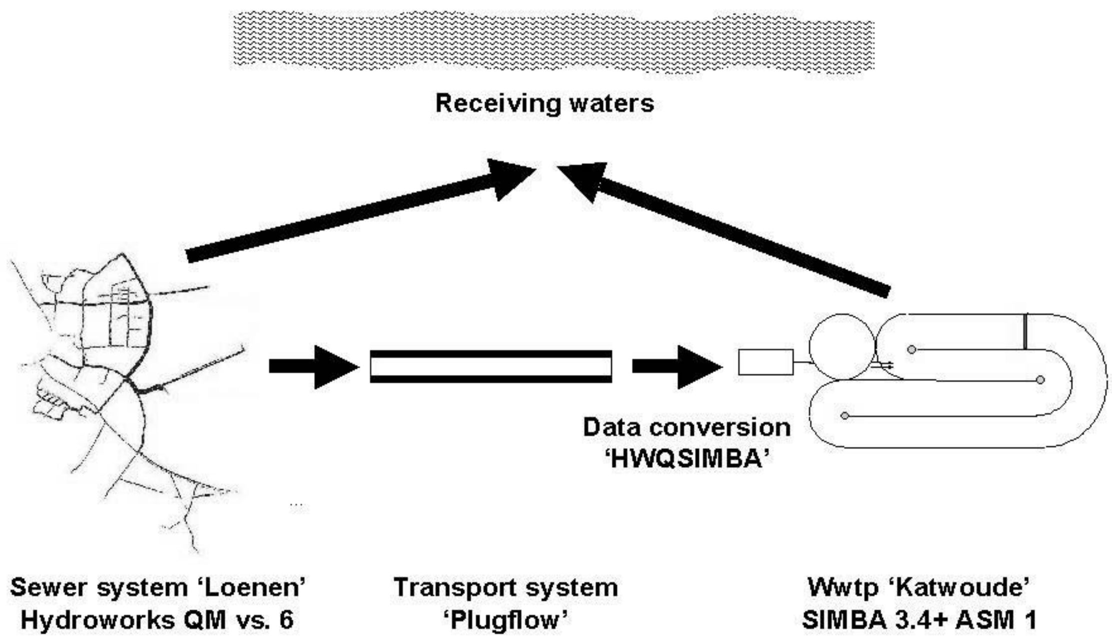


Figure XII.1 Semi-hypothetical urban water system.

In section 6.3 wastewater system performance was among other things assessed by the total ammonium load discharged. The total discharged ammonium load, however, has no direct relation with receiving water quality, as it does not incorporate the distribution over time of the discharged load. Figure XII.2 shows the impact of the discharged load during storm event B on the ammonium concentration in the river for a varying pumping capacity. The system with a pumping capacity of 0.7 mm/h exceeds the river standards with approximately 0.2 mg N/l during 6 hours. Decreasing the pumping capacity to 0.6 mm/h decreases the total duration of non compliance with the standards, but during the overflow event occurring just before T = 0.8 days, the ammonium concentration in the river peaks to 1.8 mg N/l. Further reducing the pumping capacity to 0.5 mm/h results in a peak concentration of ammonium of even 5.5 mg N/l.

As discussed in section 6.3 reducing the pumping capacity enhances wwtp performance, but is counterproductive with respect to CSO discharges. This results in an optimisation problem to find a trade off between CSO and wwtp induced water pollution. In this example the optimisation problem was defined as trying to minimise the exceedance of the river water quality standard at minimal provision costs.

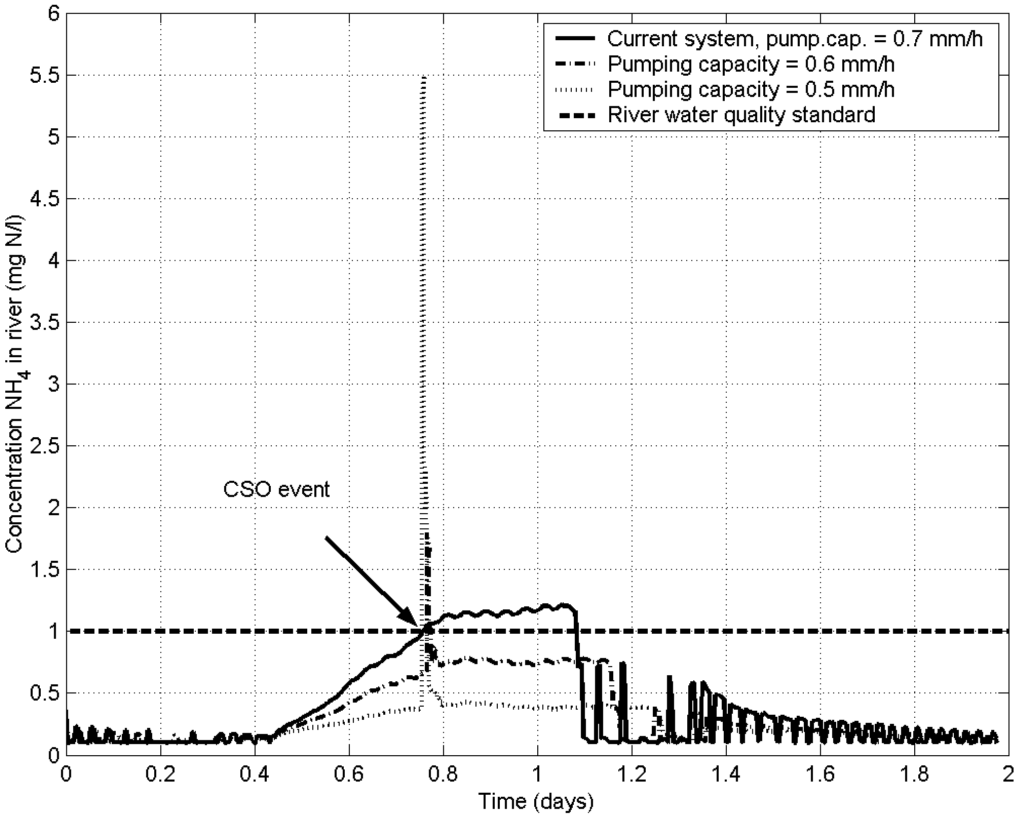


Figure XII.2 Effect of pumping capacity on ammonium concentration in the river. Storm event B, T = 10ºC.

The parameters to be adjusted during the optimisation were the pumping capacity, within the 0.5 – 0.7 mm/h range, and the additional storage capacity, within the 0 – 6 mm range, as given in table 6.1. Adjusting the pumping capacity did not affect the provision costs, whereas for the installation of additional storage a fixed price of Euro 1500/m^3 was assumed.

Simulated annealing was used to solve this optimisation problem, using the integrated model for the semi-hypothetical wastewater system shown in figure XII.1

Figure XII.3 gives the final result of the optimisation problem. In this case, it was not possible to fully meet the requirements by adjusting the pumping capacity and therefore a small volume of additional storage was necessary to fully comply with the standards.

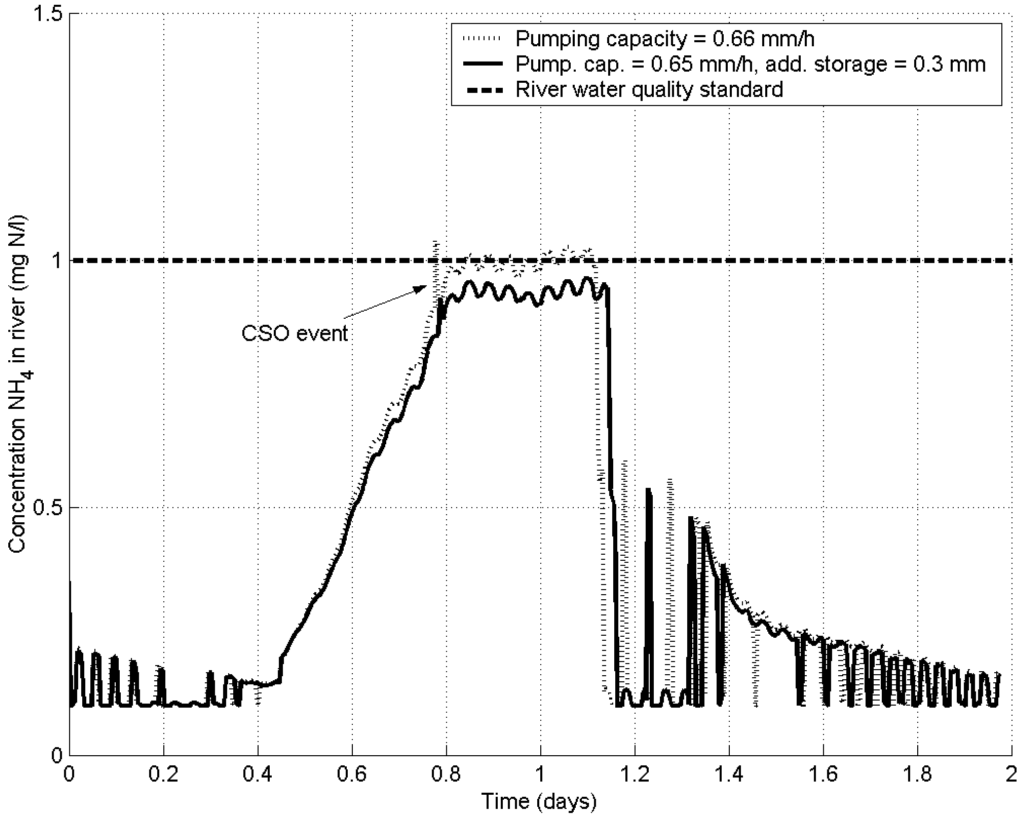


Figure XII.3 Result of optimisation using Simulated Annealing. Reducing only the pumping capacity only results in the situation with a pumping capacity of 0.66 mm/h (the dotted line). Further reducing the pumping capacity causes the standards to be exceeded due to the CSO, increasing the pumping capacity causes the standards to be exceeded by the wwtp effluent. Consequently, building additional storage capacity will be required to be able to meet the standards. The thick line represents a situation with a pumping capacity of 0.65 mm/h and an additional storage capacity of 0.3 mm. As this line is well below the required river water quality, still some potential for optimisation remains, i.e. reducing the additional storage capacity.

Appendix XIII Van Leer Limiter

The Van Leer Limiter applied in chapter 4 was developed by Van Leer [Van Leer (1974)] and is in fact Ven Leer's Second scheme. This scheme can be derived from the Lax-Wendroff scheme, written in flux form. The Lax-Wendroff flux is written as the sum of the upwind flux plus a second order correction. This correction is then multiplied by a limiter, which will temper the second order correction to prevent wiggles. The Van Leer limiter is very simple, while still being second order accurate (except near extrema) and conservative.This appendix is based on a publication by Pourquié [Vreugdenhil and Koren (1993)].
Figure XIII.1 gives the staggered grid used.

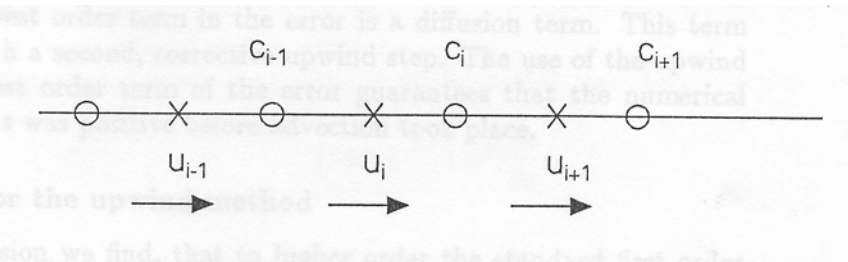


Figure XIII.1 Staggered grid used. Arrows indicate fluxes at cell boundaries.

Van Leer's second scheme can be derived from the Lax-Wendroff scheme, written in flux form:

$$f_i = f_i^{upwind} + \frac{1}{2}u(1-\sigma)(c_{i+1} - c_i) \qquad \text{(eq. XIII.1)}$$

where

f	flux	(kg/m^2/s)/
u	flow velocity	(m/s)
σ	courant number	
c	concentration	(kg/m^3)

The higher order part of the flux is now limited using the Van Leer Limiter. The limiter is a function of the regularity of the solution, which is measured by:

$$\theta = \frac{(c_i - c_{i-1})}{(c_{i+1} - c_i)} \qquad \text{(eq. XIII.2)}$$

called the monotononicity monitor. With the Van Leer Limiter as a function of θ equation XIII.1 can be adjusted using the limiter:

$$f_i = f_i^{upwind} + \frac{1}{2}u(1-\sigma)(c_{i+1} - c_i)L(\theta) \qquad \text{(eq. XIII.3)}$$

The function L(θ) selected by Van Leer is:

$$L(\theta) = \frac{|\theta| + \theta}{1+\theta} \qquad \text{(eq. XIII.4)}$$

when using this limiter, the final scheme, for a positive u is given by:

$$\begin{aligned} C &= c_{i-1} + (1-\sigma)\frac{(c_i - c_{i-1})(c_{i+1} - c_i)}{(c_{i+1} - c_{i-1})} && if \theta \geq 0 \\ C &= c_{i-1} && if \theta < 0 \end{aligned} \qquad \text{(eq. XIII.5)}$$

Figure XIII.2 gives an illustration of the performance of the Van Leer Limiter, coded in Matlab.

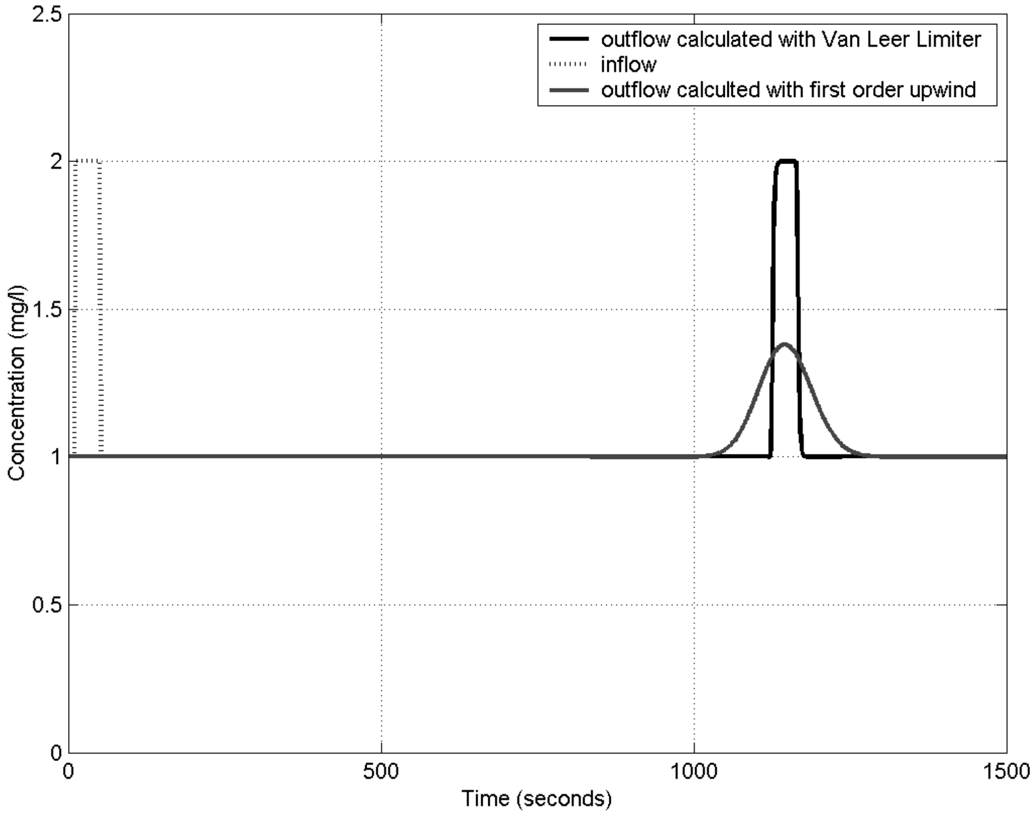


Figure XIII.2 Example of calculation results using the Van Leer Limiter coded in Matlab, compared with a first order upwind scheme. A block shaped influent was simulated for a sewer reach of 454 m long with a flow velocity of 0.4 m/s.