Discolouration in drinking water systems: a particular approach

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 van Promoties,
in het openbaar te verdedigen

op maandag 25 juni 2007, om 12.30 uur

door

Joannes Henricus Gerardus Vreeburg

Civiel ingenieur
Geboren te ’s-Gravenhage

Dit proefschrift is goedgekeurd door de promotor:

Prof. ir. J.C. van Dijk

Samenstelling promotiecommissie:

Rector Magnificus, voorzitter
Prof. ir. J.C. van Dijk, Technische Universiteit Delft, promotor
Prof. dr. P.M. Huck, University of Waterloo
Dr. J.B. Boxall, University of Sheffield
Prof. Dr.-Ing. Wolfgang Uhl, Technische Universität Dresden
Prof. dr. ir. W.G.J. van der Meer, Universiteit Twente
Prof. dr. K. Vairavamoorthy, UNESCO-IHE
Prof. dr. ir. F.H.L.R Clemens, Technische Universiteit Delft

ISBN: 978-90-74741-91-0
Pint: Gildeprint BV
Enschede

Voor mijn ouders
To my parents

Content

1 Particles in the drinking water distribution systems

1.1 Introduction

The quality of drinking water in the Netherlands meets high standards as is annually reported by the Ministry of Housing, Spatial Planning and the Environment (VROM)(Versteegh and Dik, 2006). Also the water companies themselves report in the voluntary Benchmark that water quality is one of the least discriminating factors as all the companies 'comply generously'(VEWIN, 2004).
Despite this reported high quality, water companies still report between 3000 and 6000 customer complaints about discolouration annually. The report on the Benchmark mentions that of all the parameters turbidity is causing most of the water quality failures. These figures are based on the data in the report system used to communicate the outcomes of the legal testing programme to the inspectorates (REWAB, 2004).
The most common process associated with the phenomenon of discolouration is historically the corrosion of cast iron pipes as is suggested on the site of the Drinking Water Inspectorate in the UK (DWI, 2007). On many websites of Dutch water companies, however, particles originating during treatment are also identified as the source of deposits in the network. Many authors who studied the corrosion of cast iron in potable water systems conclude that this is a major cause of discolouration. (Smith et al., 1997; McNeill and Edwards, 2001). Recent studies (Prince et al., 2003) have suggested that more sources for particles besides from corrosion play a role in the discolouration problem.

A large proportion of the customer contacts that drinking water supply companies across the world receive, stem from complaints on the occurrence of discoloured water in the drinking water distribution system (DWDS). Fig 1-1 shows a typical breakdown of customer contacts for a UK water company (Vreeburg and Boxall, 2007). Fig 1-2 shows some examples of discoloured water supplied to customers, that have led to the complaints.

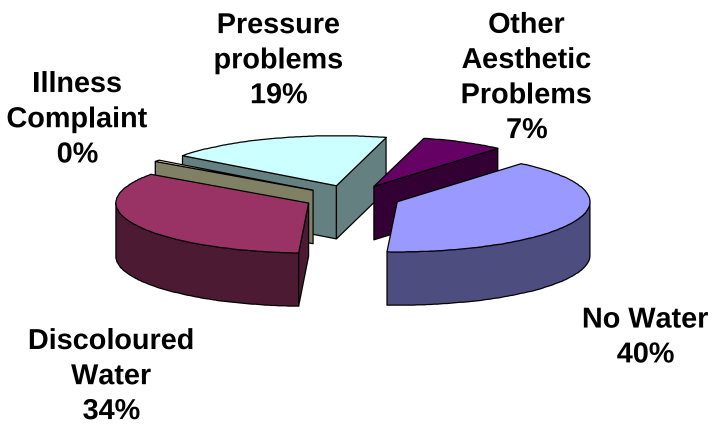


Fig 1-1 Typical breakdown of reasons for customer contacts for a 5-year period for a UK water company.

The pictures in Fig 1-2 show that the black to brownish or red nature of discolouration is not covered by the expression 'brown water'. The different appearances of the discolouration suggest that there is not just one cause of the problems, but probably a mixture of different processes that can lead to discolouration in a broad sense.

Fig 1-2 Examples of discoloured water

Discoloured water incidents as shown in Fig 1-2 greatly affect customers' confidence in tap water quality and the quality of service provided by water companies. Although good customer perception is a major driver for water companies (van Dijk and van der Kooij, 2005), a thorough understanding of the mechanisms and processes that lead to discolouration are currently lacking or at least not widely applied. Hence water companies can only respond to discolouration complaints in a reactive manner. Within modern customer focussed water companies such reactive maintenance is no longer acceptable, particularly within a strict regulatory framework as in the UK. Water companies urgently need a practicable understanding of the processes and mechanisms leading to discolouration incidents and need to develop management tools and techniques.

1.2 Nature of discoloured water

Although referred to as discolouration, the visual effect observed by customers is rarely colour in a strict water quality sense, defined as dissolved contaminants. Typically, if a 'discoloured' water sample is left to stand for a prolonged period (over night) it will clear and material will deposit (Fig 1-2). Hence, it can be concluded that it is particulate matter that the customer experiences as 'discolouration'. The measurable parameter requiring investigation is therefore turbidity. However, different particles have significantly different effects on perceived turbidity, or discolouration. A combination of factors including obscuration, reflection, refraction, diffraction and scatter contribute, although scattering usually dominates. Peak scattering occurs for particles at around half a micron in diameter with a rapid fall off for suspensions of larger or smaller sizes (Russell, 1993). New developments in measuring equipment have made more sophisticated particle counters available that are not dependent on the scattering of light and therefore can give greater detail for a better understanding of the volume of particles involved.

Particulate accumulations are also known to have a relation with biological activity (Gauthier et al., 1999). Of the organic matter 1 to 12% in the particulate accumulations may consist of bacterial biomass, making the deposits an important factor in the hygienic safety of drinking water.

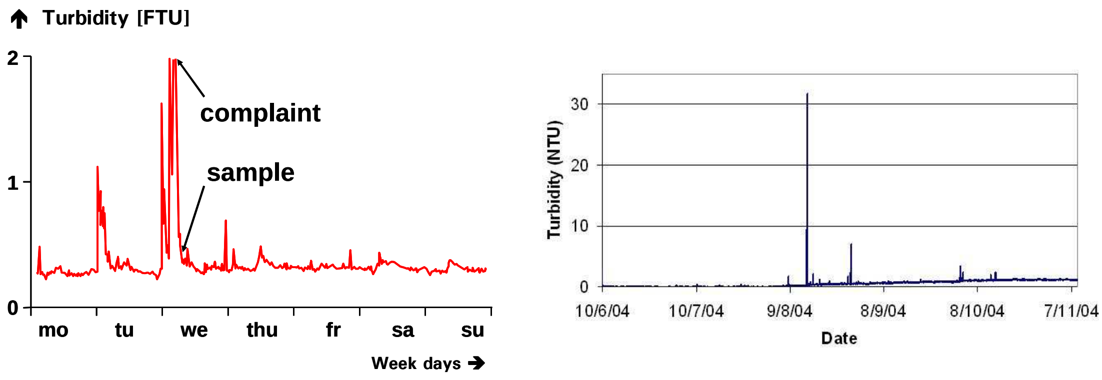


Fig 1-3 Typical discolouration events; both graphs record the turbidity response on a disturbed flow velocity. Left measured in the Netherlands, righ ta similar incident in the UK (Vreeburg, Boxall 2007)

Discoloured water events are difficult to study in real systems because they often occur over short periods seemingly for unpredictable reason. Fig 1-3 shows some typical short duration events captured by turbidity instruments installed in systems in the Netherlands and the UK, respectively. The figures show that discolouration events have the same characteristics: a sharp rise in turbidity that reduces within a few hours, despite considerable differences between the Dutch and UK networks with respect to the materials used. Similar results are found in monitoring turbidity and velocity at different locations in the Melbourne (Australia) drinking water distribution system (Prince et al., 2001).

The largest growth period for network in the Netherlands was from 1945 – 1980, hence, the average age of the network is 42 years and the predominant pipe materials are PVC and AC (see also section 1.5). Conversely in the UK, the networks have not experienced such intensive investment meaning these networks are still dominated by cast iron pipes dating back over the last 100 years or more. The Australian network is more recent than the Dutch network and has concrete or cement mortar lined cast iron and PVC as dominant pipe materials (Prince et al., 2003). The treatment histories of the systems are also different, with systems in the Netherlands having long adopted a very high standard of treatment and a policy of no chlorination, while the UK has seen a variety of different treatment policies, resulting in a variety of levels of service both in quality and quantity. The Australian network is supplied with unfiltered water, and is dosed with chlorine, fluoride and lime (Verberk, 2007).

These historic factors are key to understanding the levels of service and the processes leading to the occurrence of discolouration events as shown in Fig 1-2 and Fig 1-3. This difference is also manifest in the reactive trigger levels that companies use to initiate cleaning in response to discolouration, typically around 4 contacts per 1000 properties in UK compared with 0,5 to 1 contact per 1000 in the Netherlands and 6 contacts per 1000 properties in Australia. This shows that, despite obvious differences in systems, the nature of discolouration problems is the same though the number of problems varies. Intuitively discolouration in the Dutch systems should be almost non-existent considering its history of good treatment, very low leakage and a network with a limited number of cast iron pipes, but turbidity measurement as shown in Fig 1-3 show that discolouration does occur and other processes are involved besides corrosion. This may also highlight the inconsistent nature of customers, with a propensity to complain predominately when the quality of the water changes from what is perceived as ‘normal’. In an area in which usually water is distributed with a low turbidity, a rise to a higher level could cause complaints. The absolute value of this level higher, however, would not cause complaints in an area in which the ‘normal’ level is higher. In other words: the deviation of the level that is perceived as normal rather causes the complaints than the absolute level. This complicates any quantification of discolouration risk in terms of tangible turbidity levels.

1.3 Particle-related processes in the drinking water distribution systems

Discolouration is associated with the mobilisation of accumulated particles from within distribution networks. Such particles have different sizes and densities and, hence, probably different origins, often characterised as either external sources or from processes occurring within the system. Particles can enter the distribution network as background concentrations of organic and inorganic material from the source water (Lin and Coller, 1997; South-East-Water, 1998; Kirmeyer et al., 2000; Slaats et al., 2002; Ellison, 2003), due to incomplete removal of suspended solids at the treatment plant (Gauthier et al., 2001; Vreeburg et al., 2004b) or be added to the water by the treatment plant itself, such as carbon and sand particles, alum or iron flocks and bio particles originating from bio filters (Alere and Hanæus, 1997). The distribution system itself can also produce particles, such as from pipe and fitting corrosion and lining erosion (Stephenson, 1989; Ruta, 1999; Gauthier et al., 2001; Clement et al., 2002; Slaats et al., 2002; Boxall et al., 2003), biological growth (Le Chevallier et al., 1987; Stephenson, 1989; Clark et al., 1993; Meches, 2001) and chemical reactions in which dissolved solids can be transformed to suspended solids (Stephenson, 1989; Sly et al., 1990; Walski, 1991; Lin and Coller, 1997; Kirmeyer et al., 2000); or external contamination that may occur during operations such as pipe repairs (Gauthier et al., 1996; Slaats et al., 2002), intrusion (Gauthier et al., 1999; Kirmeyer et al., 2000; Prince et al., 2001) and backflow. Possibly the most common and significant biological process is biofilm formation which can result from the presence of assimilable organic carbon in the water or the pipe wall (van der Kooij, 2002). The effects of these complex and interacting processes are further complicated by exposure to various different physical and chemical conditions during passage through distribution systems including contact with a range of different pipe materials and ages and different hydraulic conditions. The formation and growth of particles is a very complex process which is currently poorly understood. Factors such as contact times, contact surfaces and hydraulic conditions are likely to play important roles in controlling these processes (Huck and Gagnon, 2004). These sources, external and internal, rarely contribute directly to discolouration events but facilitate the gradual accumulation of material within the distribution system.

Next to the sources and growth of particles, it is important to understand the hydraulic behaviour of the particles to determine the fate of the particles in the network. Research in the UK showed results for the distribution of particle sizes found in discoloured water samples, suggesting a reproducible distribution of particle sizes irrespective of network conditions, source water, etc. (Boxall et al., 2001) It is suggested that the size range of the particles was predominately less than 0.050 mm, with an average size of around 0.010 mm and a significant number of particles in the sub 0.005 mm range. It is unlikely that gravitational settling alone will be a sufficient force for accumulation of such particles since turbulent forces generated by even the lowest flows within a distribution system are likely to be sufficient to overcome gravity settling forces (Boxall et al., 2001). This is particularly valid for the smaller sized particles found within discolouration samples which will dominate discolouration due to their light scattering properties. Fig 1-4 shows material accumulation due to corrosion processes around the complete circumference of pipe samples and a lack of invert deposits, consistent with these concepts. Samples such as these have been installed in a laboratory facility and significant discolouration was generated by exposing them to flushing flow rates, despite the disturbance of weakly adhered material caused by obtaining the samples..

Fig 1-4 Material accumulation around the complete perimeter of cast iron pipe samples

Transport of particles will not only occur through the liquid phase as suspended solids, but can also take place as bed load transport: particles rolling over the pipe wall. Though not mentioned in literature, bed load is a distinct possibility for particle transport. All the aforementioned particle-related processes in a network are visualised in Fig 1-5.

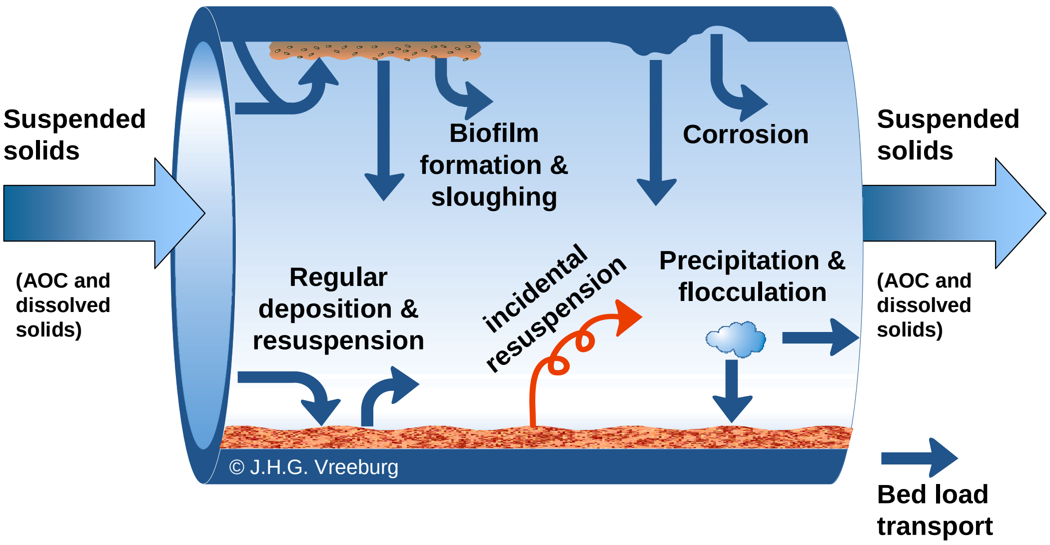


Fig 1-5 Particle-related processes in a network. The direction of the arrows indicate the path particles follow in the pipe. The vertically aimed arrows indicate particles settling on the pipe wall; the horizontally aimed arrows indicate particles moving with the water as suspended solids.

Overall it can be concluded that the mechanism leading to discolouration events are complicated, poorly understood and interactive. However the processes may be understood through the framework presented in Fig 1-5. On this representation the hypothesis of this research is based. The underlying cause of discolouration is presumed to be formed by particles attached by some means to the pipe wall, irrespective of their origin, either imported from outside the network, by the treatment or produced within the network itself. In normal flow the particles regularly deposit and partly resuspend without affecting the aesthetic quality of the water. If flows are increased above normal values in an hydraulic incident, scouring forces and shear stress increase consequently and then the particles may be mobilised. This incidental resuspension may lead to customer complaints.

1.4 Goal of the research

The goal of the thesis is to analyse the processes in the network associated with typical discolouration problems as shown earlier in Fig 1-2, based on the framework of particle related processes presented in Fig 1-5. In the research the behaviour of particles in the network will be explored aiming at an empirical understanding of particle origin and fate in the network. Finally, measures and strategies will be proposed to control the particles in the network to minimise discolouration problems.

1.5 The drinking water network in the Netherlands

The drinking water distribution system is a complicated reactor vessel in which particle-related processes interact with each other and the pipe material. This thesis study has primarily been done within parts of the Dutch network. Though drinking water networks do not differ very much around the world as it comes to diameter and material use, the ratios with which materials are used are different (Fig 1-6).

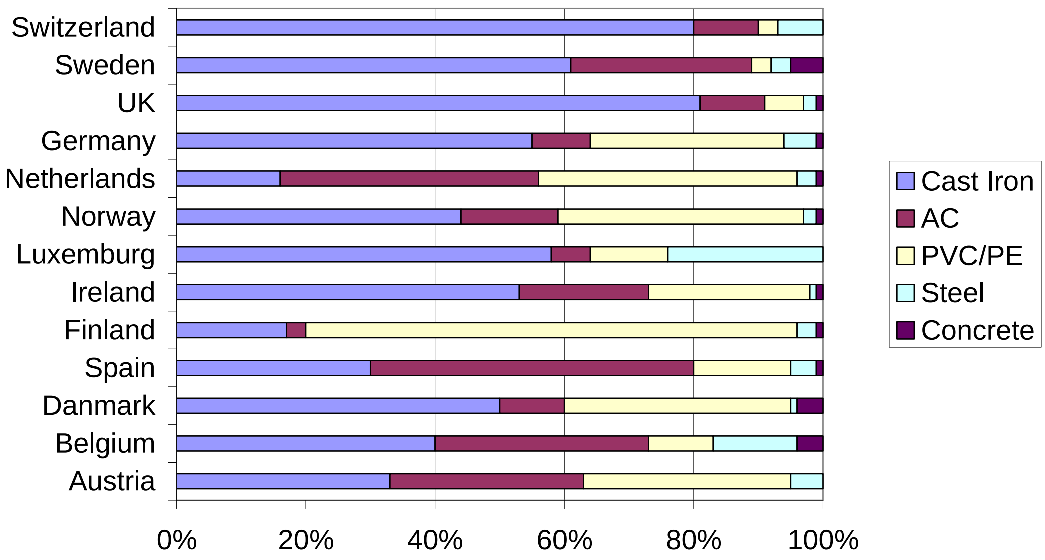


Fig 1-6 Pipe material composition in several European countries in 1990 (Based on several historic data sources in 1990, courtesy Gijs Ekkers)

The main materials used for common distribution networks are Cast Iron (CI), Asbestos Cement (AC) and plastic materials like Poly Vinyl Chloride and Poly Ethylene (PVC and PE). Concrete and Steel are mostly used for the larger transport mains.

The historical development of the Dutch drinking water network is shown in Fig 1-7.

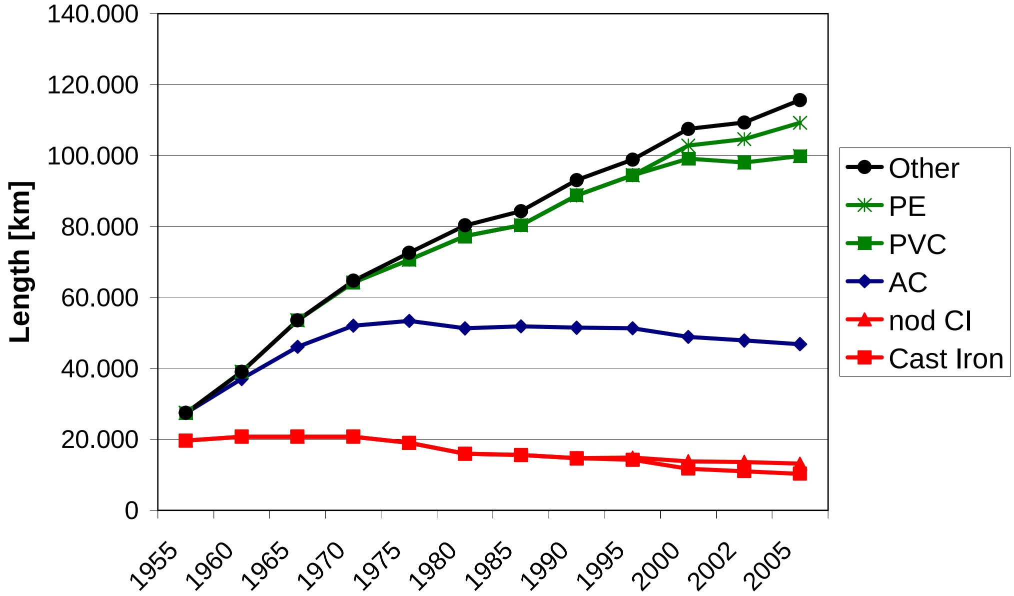


Fig 1-7 Development of the Dutch drinking water system (source: (Geudens, 2006))

Based on these data the network is relatively young (on average 42 years in 2006) and has a minor part of unlined cast iron (less than 10%). The part of unlined cast iron is declining due to ongoing rehabilitation from more than 20.000 km in 1970 to less than 10.000 km in 2005. The "new" cast iron, being the nodular cast iron pipes have a inner wall protection with a cement mortar lining, which eliminates the iron corrosion process.
The use of materials for pipes in the Netherlands can be considered as a 'sign of the time' that also might explain partly the development of material use in the other European countries. The oldest pipes stem from the period pre-1940 in which cast iron was the only material that was available to produce pipes. In 1950 the material Asbestos Cement became available for manufacturing pipes that had many advantages compared to the traditional CI pipes: it was non-corrosive, cheap and easy to handle. The negative effects of the fibre cement finally banned the use of it in new pipes during the 1970-ties. In the early 1960-ties the new plastic materials were developed with obvious advantages in costs and handling and that became the dominant material for new pipes resulting in a network that for almost 50% consists of plastic pipes..

1.6 Setup of the thesis

1.6.1 General

Discolouration is a problem that is as old as public drinking water supply. Until a few years ago this phenomenon had received relatively little attention, however, with other improvements in the supply of drinking water, discolouration is now the single most common reason for customer complaints. Research described in this thesis focuses on the underlying mechanisms of discolouration, going beyond the intuitive and accepted causes like corrosion. New tools and techniques can be developed to support the implementation of planned operation and maintenance strategies to control the discolouration risk. Most of the research, however, concentrates on the composition of the removed sediment and the possible impacts on (microbiological) stability of the water (Gauthier et al., 1996; Gauthier et al., 1999;

Gauthier et al., 2001; Zacheus et al., 2001; Torvinen et al., 2004; Barbeau et al., 2005; Carriere et al., 2005). The actual discolouration risk as such is not assessed or evaluated. For this thesis the role of particles in the DWDS is the central theme. A summary of the particle-related processes as shown in Fig 1-5 will be the leading principle, which leads to the hypothetic development of the discolouration risk in a network, as is sketched in Fig 1-8.

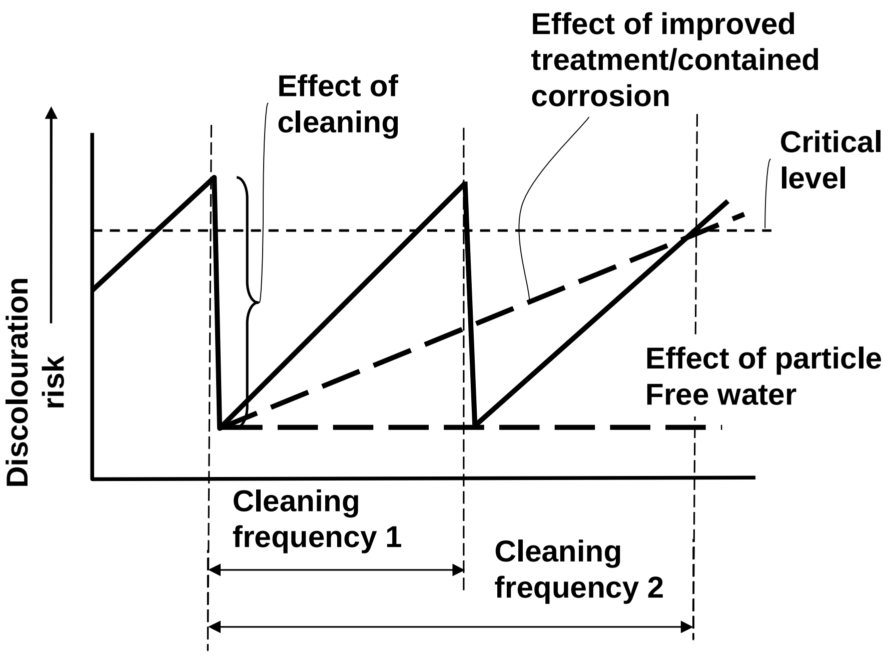


Fig 1-8 The development of the discolouration risk in the DWDS based on the particle related processes.

On the vertical axis the discolouration risk is quantified to a tangible value that comprises elements as the amount and mobility of loose deposits that may cause discolouration events when resuspended. The horizontal axis is the time. The start of the solid line shows the increase in the discolouration risk as a result of the particle-related processes in the network. The discolouration risk is determined by the amount and mobility of loose deposits in the network that may originate from all the processes mentioned in Fig 1-5.

- The net result of the number and volume of particles that are loaded into the system with the incoming water minus the amount and volume of the particles that leave the system with the water that is actually supplied to the customers.
- The net result of the biological activity in the network with the formation of discrete and loose particles attributes also to the discolouration risk. This excludes the Fe and Mn that is incorporated in the biofilm (van der Kooij, 2002; van der Kooij et al., 2003), because these particles are not available for discolouring the water.
- The result of active corrosion processes are loose particles that can either directly discolour the water or add to the reservoir of loose deposits.
- Precipitation and flocculation lead to the growth of small particles to larger ones that can settle in the network. It also may result in the formation of particles from dissolved solids (DS) into suspended solids (SS), for instance in the oxidation of Fe^{2+} to Fe^{3+} and the consequent formation of Insoluble iron hydroxides.
- The net result of the hydraulic behaviour of the particles as suspended and resuspended solids but also some of the particles can leave the network again through the bed load transport.

Chapter 2 of this thesis describes the measuring methods that are developed and applied in this research to substantiate the net result of the particle related processes and their effect on the amount of loose deposits in the network.

The research is next to the empirical understanding of particle origin and fate also aimed at operational measures and strategies to control and/or minimise discolouration problems. Following the hypothesis as illustrated in Fig 1-8 on the development of the discolouration risk the operational measures will be geared to three phenomena:

- The introduction and production of particles in the DWDS
- The hydraulic movement and the accumulation of particles in the DWDS
- The control of the accumulated particles in the DWDS: cleaning the network

Within the framework of the research the three items have been introduced to the drinking water companies in the Netherlands and have become known as the ‘three-stage-rocket’. The thesis describes the three stages in different chapters that are shortly described in the following sections.

1.6.2 Introduction and production of particles

The ambition of water companies is to prevent problems at the customers tap (van Dijk and van der Kooij, 2005). The introduction of particles into the network through the water leaving the treatment plant could be a significant source for particles in the network. To test this hypothesis an experiment is done, described in Chapter 1, in which the sediment build up in two similar networks is compared. The difference between the networks is that one is supplied with typical Dutch drinking water (a groundwater based multi stage treatment) and the other with the same water with an additional treatment by an ultra filtration membrane plant, resulting in a theoretical particle free water. The corrosion process is excluded because the there is no unprotected cast iron within the two networks.
The biological processes should not to be underestimated (van der Kooij, 2002; van der Kooij et al., 2003; Huck and Gagnon, 2004), and will be analysed briefly in the experiment.

1.6.3 Hydraulic movement and accumulation of particles

Some of the particles in the drinking water accumulate in the network probably as a result of the hydraulic circumstances. Water demand varies over the day and especially in the DWDS ‘in the street’ where house connections are made and the water is mostly stagnant or almost stagnant (Blokker et al., 2006). Also in the larger transport lines the velocity of the water varies with low flow and high flows alternating. The hypothesis on which the second stage is built stipulates that if the velocity in the network is sufficiently high during relatively short time periods this will prevent the sediment from accumulating to unacceptable levels. Actually the process ‘regular deposition and resuspension’ shown in Fig 1-5 must be managed in such a way that the net result is zero.
Chapter 1 describes the development of the high velocity self cleaning distribution networks and research into the effects of those networks on the accumulation of particles.

1.6.4 Control of accumulated particles: cleaning of networks

If resuspension of accumulated particles is the main cause for discolouration events, than is containing the amount of loosed sediments in the network a practical measure to reduce the discolouration risk In fact the most common reactive measure used by network operators to deal with discolouration is to clean the network.

The implicit goal of cleaning the network is to reduce the discolouration risk which is visualised with the immediate decrease of the discolouration risk following the solid line in the graph shown in Fig 1-8: the amount of resuspendable sediment is removed.
Chapter 1 inventorises the commonly applied cleaning techniques and their efficacy to manage the discolouration risk leading to a practical approach for cleaning of networks.

1.6.5 Particle composition and behaviour in sediment and transportation systems

Over the years a number of practical studies have been performed that have enhanced the knowledge about the processes in the network and the behaviour of particles. In Chapter 1 three recent case studies are described concerning long distance transport of high quality water with different dominant particle processes and a research into the composition and behaviour of several drinking water distribution systems sediments. In these case studies the new technique particle counting is applied on the transport networks and the potential of the new pre-concentration methods TILVS and Hemoflow is explored.

1.6.6 Discussion and conclusion

At the end of this thesis the results of the chapters will be brought together and the effects of the several processes will be evaluated along with their impact on the practical operation of the network.

2 Measuring discolouration phenomena in drinking water distribution systems

2.1 Introduction

The actual discolouration of water is difficult to measure with traditional sampling because the increased colour or turbidity of the water is temporary and dependent on non-regular hydraulic circumstances that cause increased velocities in the pipes. Nevertheless every water company or distribution network manager receives samples like those shown in Fig 2-1 that are taken by a customer who experiences a discolouration event. These samples show the nature of the discolouration: the resuspension of settled material.

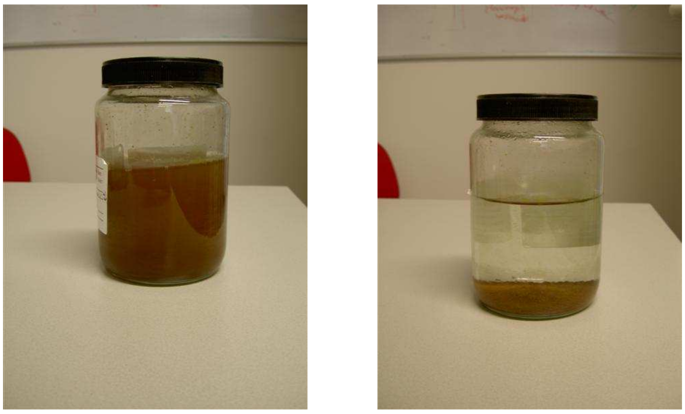


Fig 2-1 Samples handed in by customers. Left the discolouration incident and right the sample after settling

Discoloured water is defined as water with a turbidity that can be seen by a customer with the naked eye. Based on the framework of the particles-related processes (Fig 1-5) this makes turbidity and particles key parameters in the analysis of discolouration. The measuring of turbidity with a turbidimeter and particles with particle counters both use light to quantify the amount of particles. In brief, turbidimeters measure the amount of 90^{o} light scatter from particles in a sample cell. The sample cell can contain a discrete sample or can be flown through with a continuous flow. This reflects the turbidity of the water, relative to a known standard. In this thesis the standard Formazine Turbidity Unit (FTU) is used which is equal to the Nephelometric Turbidity Unit (NTU). Conversely, most on-line particle counters measure a change in light intensity as particles pass through a laser beam. The shadow (light obscuration) cast by each particle is proportional to its size within a defined size range. Particles can be counted and sized within different preset and discrete bands (Hamilton et al., 2003). In this study mostly MetOne PCX particle counters were used, capable of measuring 32 different bandwidths of 1 µm, starting at either 1 or 2 µm.

The change in water quality between the time it leaves the treatment plant and the time it arrives at the customers tap is determined by the processes in the network. The pictures in Fig 2-1 show that accumulation and resuspension of particles is the basic problem. The change in water quality is dependent upon the type of network (transport or distribution), the material used, the hydraulic circumstances and the quality of the treated water that enters the network. Water quality in DWDS is regularly checked through sampling according to the Drinking Water Act (Water-Act, 1958). In the Netherlands the sampling frequency and locations are according this Act determined by the Inspector of Public Health, a part of the Ministry of

Housing, Spatial Planning and the Environment (VROM) and the results are reported annually (Versteegh and Dik, 2006). These samples however give little information about the change in water quality in the DWDS because the frequency of sampling is limited and literally an end-of-pipe quality control.

Particle-related processes in the network in normal circumstances deal with very low concentrations and seldom lead to direct problems. The indirect discolouration problems caused by the resuspension of accumulated particles should be measured to study the effects. The measuring methods that can be used to analyse the particle related processes can be categorised as follows:

- Direct methods
  With these methods the incoming and outgoing water are directly and on-line analysed for different parameters. An example are the continuous monitoring of turbidity or particle counts.
- Effect methods
  These methods measure and quantify the effect of long-term processes in the network. A classic example is the Biofilm monitor (van der Kooij et al., 2003) to measure the effect of AOC on biofilm growth. In this category the Resuspension Potential Measurement (RPM) is developed.
- Concentration methods
  These methods concentrate the water to obtain the low levels of particles in concentrated form to better analyse the amount and composition of particles in the water.

2.2 Direct methods

2.2.1 Introduction

Considering the particle-related processes of Fig 1-5, the quality of the water will change during transport through the network as is found by many authors (van den Hoven and Vreeburg, 1992; Gauthier et al., 1999). The change in water quality can directly be measured with parameters such as turbidity and particle counting. During the transport through the networks particles are lost due to settling or produced by corrosion or biofilm formation resulting in a difference between the incoming and the outgoing water and, consequently, a difference in the turbidity reading or particle counts. The direct way to assess these differences is to monitor these parameters of turbidity or particle counts.
Water quality changes during the water's residence time in the network. Preferably the water that leaves the treatment plant is followed through the network. Grab samples should then be taken with a time lag that equals the residence time. In this way the package of water leaving the treatment plant can be traced in the network and the actual changes can be assessed. This actual tracing is necessary because the water quality at treatment plants will vary with seasons, but also within days or hours. For instance, the turbidity of the water can vary between 0,1 to 0,3 FTU on a daily base. The water in a sample in the network with a value of 0,15 FTU can either have entered the network at a value of 0,1 FTU, but also at a value of 0,3 FTU. This start value is of importance to be able to analyse what happens in the network. From the REWAB system (the Dutch national database in which the results of the samples reported to the Inspector of Public Health are published) it can be observed that the variation in the samples at the treatment plant often are in the same range as the variation in the distribution network. The frequency of sampling and the fact that the samples are taken during office hours make it highly unlikely that the samples reflect exactly the same package of

water. Comparing samples that have been taken on one day at several locations will not lead to valid conclusions.
The benefits of continuous monitoring in the network that will be applied in this research and further illustrated in the next paragraph are already known Parameters that can be analysed continuously are turbidity and particle counts. Other chemical and physical parameters such as temperature, conductivity, pH and oxygen content have been monitored in earlier research but are not considered to give much added information for the purpose of tracing particles (van den Hoven and Vreeburg, 1992).

2.2.2 Continuous monitoring of water quality

Turbidity and particle counts are parameters that characterise the potential discolouration aspects, because the discolouration is defined as a turbidity that can be seen by a customer. The turbidity levels and particle counts of treated water are almost permanently under the thresholds that are prescribed by the Inspector of Public Health (REWAB, 2004; Versteegh and Dik, 2006). The changes in the levels are subtle, which makes it necessary to monitor the water quality continuously. Fig 2-2 shows a scheme of continuous measuring equipment in a network and a picture of the equipment used in this research. Flexibility to install the equipment at various locations is of utmost importance.

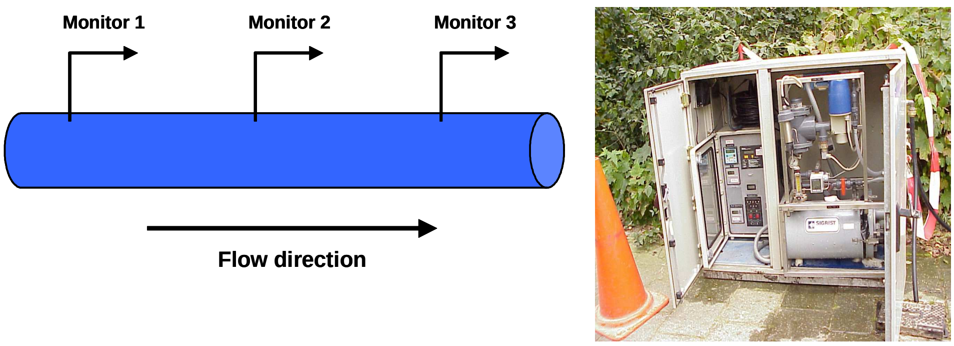


Fig 2-2 Schematic of monitoring with continuous monitoring equipment and a picture of a mobile system.

The blue pipe represents a network and at three locations the water quality is monitored. Location 1 monitors the initial quality, mostly at the treatment plant. The other locations are down stream and monitor the water quality to obtain the, mostly subtle, changes in parameters.

2.2.3 Interpretation of the continuous monitoring of turbidity

Continuous monitoring of turbidity at several locations in a network, starting with the treatment plant can reveal the processes mentioned in section 1.3 (Fig 1-5). Fig 2-3 gives some stylistic patterns of turbidity measurements at three typical locations, being the treatment plant, the transport network and the distribution network. These stylistic patterns each represent a specific process in the network, but rarely can they be measured as such in real life. Most of the processes take place simultaneously, but with one being dominant and explicitly recognisable.

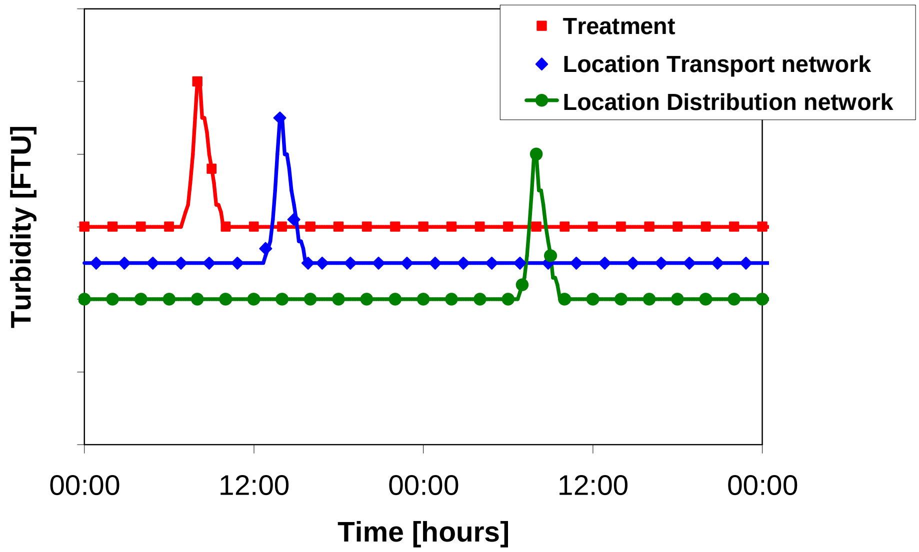


Fig 2-3 Stylistic patterns of continuous turbidity monitoring at three locations. Effect of settling is demonstrated in the decreasing turbidity. The residence time can be determined when the peak value at the treatment plant is followed in the transport an distribution network.

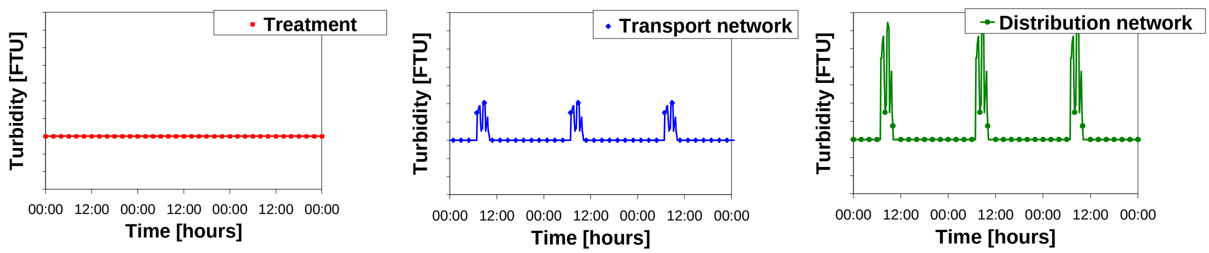


Fig 2-4 Stylistic patterns of turbidity monitoring. Resuspension as result of increased velocity during peak hours. In this case a typical morning peak causes a relatively sharp increase of turbidity. The increase is highest in the distribution network.

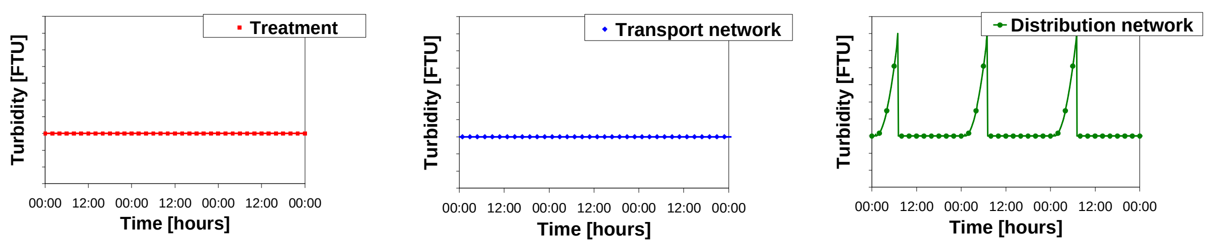


Fig 2-5 Stylistic patterns of turbidity monitoring. The transport network shows the same turbidity as the treatment plant. The pattern in the distribution network is typical for active corrosion: increase during night hours with almost stagnant water that is 'flushed' during the morning peak resulting in a lower turbidity.

In Fig 2-3 trough Fig 2-5 the turbidity traces at the treatment plant, in the transport network and the distribution network are drawn in a graph with turbidity on the vertical axis and time on the horizontal axis. The treatment location is at the beginning of the network, typically at the pressure side of the pump or 'at the fence' of the plant. The transport network is typically a larger size pipe that connects the treatment plant with the demand points. The location in the distribution network is on the smaller sized pipes, 150 mm and smaller, at which the house connections are made. In fact this is the water that actually enters the properties and is experienced by the customers. The colour of the lines (red, blue and green) in this thesis are often used to represent the flow direction with red the upstream location, green the downstream location and blue somewhere in between.

The first process presented in Fig 2-3 is settling of particles, represented by the decreasing turbidity in the flow direction. With settling, particles are lost and that leads to a decrease in turbidity in the flow direction. This phenomenon can often be observed when in the turbidity trace at the treatment plant irregularities occur. These can be the result of a backwash of filters or resuspension as result of a pump switch.
The spikes in the turbidity can be used as natural tracers to monitor the residence time in a network. The peak values at each locations are used to identify the origin of the water and that 'package' of water can be followed through the network. This has proven to be a very powerful tool to actually measure the residence time without adding tracers to the water.

In Fig 2-4 the turbidity trace in the three locations are drawn in separate graphs. The three stylistic traces represent resuspension of particles manifested by local peaks in turbidity, independent of turbidity elsewhere in the network. Mostly the peaks in turbidity can be related to increased velocity in the pipes, for instance as result of a hydraulic incident. In this case the resuspension is caused by the increased flow during the morning peak demand. The increase starts at 7:00 AM or even earlier and ends after roughly three hours. Resuspension can be observed both in the transport network and in the distribution network, though in the transport network there are fewer and mostly lower spikes in turbidity. First, the variation in velocity in the transport network is less than in the distribution network (Blokker et al., 2006), allowing for a more regular settling and resuspension. Second, the sampling point is at the top of the pipe and usually some stratification of the resuspended sediment can be observed. This means that part of the resuspended sediment will not reach the measuring point at the top of the pipe. The peaks in turbidity in the distribution network follow the household demand, with high peaks in the morning and varying peaks in the late afternoon and early evening. The day peaks are also related to the weather conditions and the types of connections: in hot, dry weather in areas with gardens the peak in the evening can be much larger compared to the morning peaks. That also explains why discolouration incidents are often associated with extreme weather conditions.

The graphs in Fig 2-5 are again separated for the three locations and present the turbidity trace that can be measured when active corrosion occurs. Active corrosion changes the water quality (McNeill and Edwards, 2001). The typical pattern is that the turbidity in distribution pipes rises during the night hours when the water stagnates because of low night use. In the morning hours the water consumption increases, flushing out water with an increased turbidity. The rise in turbidity is caused by the particles that are released through the corrosion process. The phenomenon has also been observed when a stable corrosion products scale is damaged by aggressive cleaning action (Vreeburg, 1996). The typical difference between resuspension and corrosion is that with the resuspension phenomenon, the turbidity rises when demand increases while with corrosion the turbidity drops with increasing demands. It can, however, be difficult to distinguish the two processes in the case of a combination of settling and resuspension in a distribution pipe. The pattern of falling and rising turbidity can look the same, but the difference is the timing of the peaks in turbidity. In the case of settling and resuspension the turbidity decreases in the night hours and increases in the day hours, while in the corrosion case this is just the other way around, with increasing turbidity in the night and a decrease during the day. In both cases however customer complaints occur in the morning hours, which makes it impossible to use customer complaints as an indication for what process is dominant

2.2.4 An example of continuous monitoring of turbidity

To illustrate the stylistic graphs of Fig 2-3 through Fig 2-5 and demonstrate that the phenomena occur simultaneously a case study is described. Fig 2-6 gives an example of the concurrent monitoring of turbidity at a pumping station (red line) and two locations in the network (shown with blue and green lines). The monitors used are Sigrist KT65 white light turbidimeters. Measuring accuracy is 0,02 FTU after calibration and the measuring interval is 2 minutes and 30 seconds.

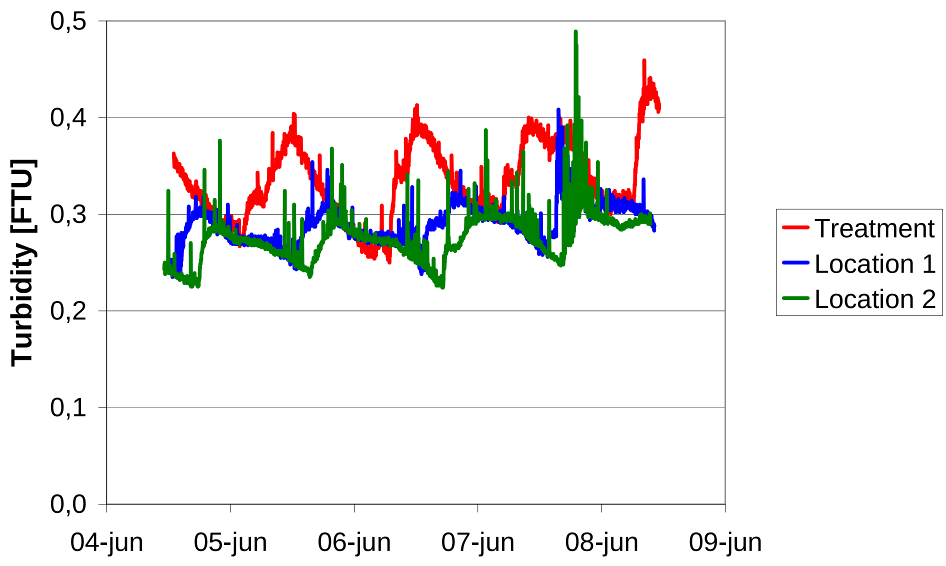


Fig 2-6 Example of continuous monitoring of turbidity at the treatment and two locations in the network. The variation in turbidity at the treatment plant is mirrored in the network, showing a general decrease of the values of turbidity.

The treatment plant is a conventional groundwater treatment with nozzle aeration, rapid sand filtration followed with a second nozzle aeration step and rapid sand filtration (treatment plant Carlifornië of Water Supply Company Limburg). Location 1 and 2 are located in the network at a distance of 12 km and 13 km from the treatment plant.
The average turbidity ex treatment works is 0,33 FTU and the values at the network locations are slightly lower. The averages, maxima and minima are summarised in Table 2-1.

Table 2-1 Turbidity values. Results of four days measuring with a measuring interval of 2 minutes and 30 seconds (n=2300 per location)

	Average [FTU]	Max [FTU]	Min [FTU]
Treatment	0,33	0,46	0,25
Location 1	0,29	0,41	0,24
Location 2	0,27	0,49	0,22

There is however a distinct pattern in the turbidity readings over the day at the treatment plant that is mirrored at the other two locations. From that pattern it is clear that the turbidity in the

flow direction is decreasing. At the end of the measuring period at location 2 there is an increase in turbidity that is not related to the turbidity at the treatment plant and is probably caused by a resuspension of settled material. That is in accordance with the observation that for the majority of time the turbidity drops indicating that material is lost from the fluid and probably settling at the pipe wall. An increase in velocity will increase the sheer stress and resuspend the settled material.
If these locations were monitored with samples, then it would be impossible to analyse the process of regular settling and resuspension of material. Samples are mostly taken during office hours, so the 24-hour information on the pattern will be lost. Moreover at the network locations the levels of turbidity also vary on a shorter time scale with short increases. Those are represented by the short spikes in readings. Given the scale of the time axis, these spikes last for at least half an hour. The chance that samples are taken during those short spikes is unrealistic and that would leave the phenomena unclear.

Following the patterns in the network also allows for determination of the residence time between the treatment works and the network locations following the stylistic pattern in Fig 2-3. The maximum value at the tretament on June 5 was followed in the network; a closer detail of the graph is shown in Fig 2-7 and the indicated values are summarised in Table 2-2.

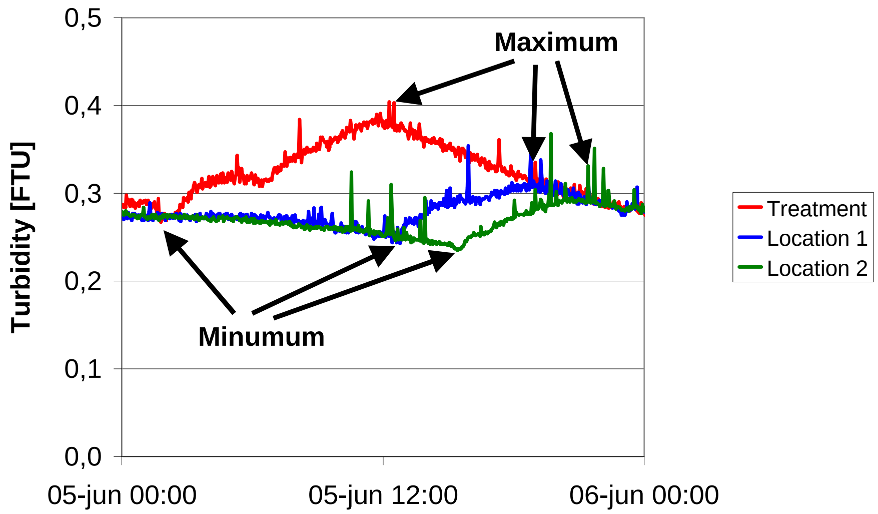


Fig 2-7 Detail of turbidity graph with the maxima and minima indicated

Table 2-2 Summary of maxima and minima continuous monitoring

	Turbidity [FTU]	Date time [day hh:mm]	Res time TM – loc [h:mm]	Average velocity [m/s]	Res time loc - loc [h:mm]
Maximum					
Treatment	0,404	5-jun 12:17			
Location 1	0,346	5-jun 18:47	6:30	0,51	
Location 2	0,331	5-jun 21:25	9:07	0,41	2:37
Minimum					
Treatment	0,269	5-jun 02:02			
Location 1	0,244	5-jun 12:25	10:22	0,32	
Location 2	0,235	5-jun 15:25	13:22	0,27	3:00

The summary in Table 2-2 shows that in the flow direction the turbidity of the water decreases. Because the sampling times could be identified from the patterns, it is possible to actually follow a plug of water through the system. The residence time from the treatment plant to location 1 is 6,5 hours during the day but at night it takes more than 10 hours for the water to travel the same distance. The residence time between location 1 and 2 is two-and-a-half to three hours during daytime, and three hours during night time. Knowing the distance between the locations allows for calculation of the average velocity of the water during the residence time.
If samples were taken, then they probably would have been taken during daytime. If the samples would have been taken in the afternoon, then the differences in turbidity would not have been very distinct but probably in the right order: the highest at the treatment declining in the downstream direction. Earlier in the morning the turbidity at the network location would have been similar, but the treatment plant sample would have been much higher. If the samples, however, were taken in the afternoon of June 8 (see Fig 2-6) than probably the turbidity at the three locations would have been very close to each other or even in reverse order with a higher turbidity at the downstream locations. This means that dependant on the time of sampling different conclusion could have been drawn on the dominant process in the network. The information on residence time and the clear demonstration of resuspension would not have been noticed anyway.

The cumulative frequency distribution of the turbidity for all three locations is presented in Fig 2-8.

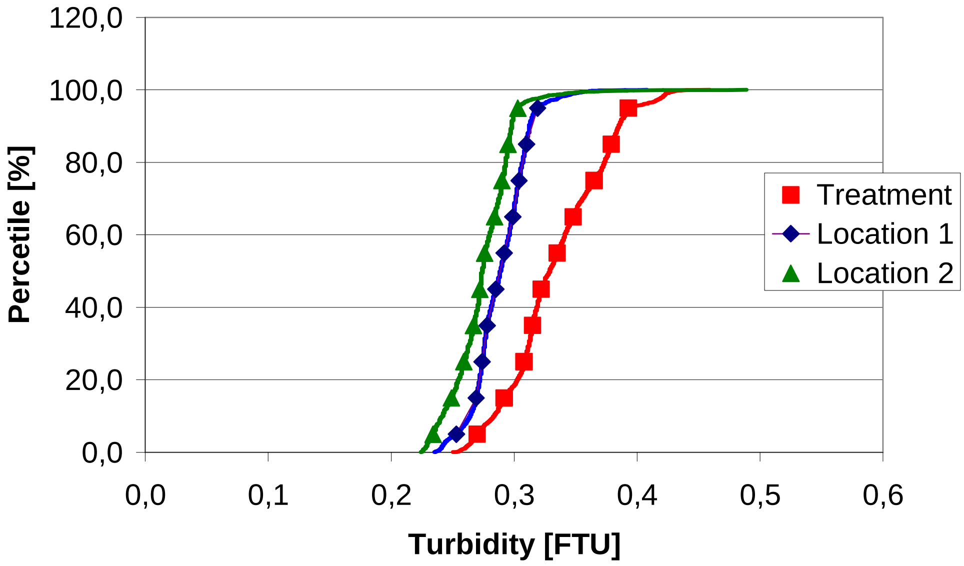


Fig 2-8 Cumulative frequency distribution of turbidity measurements at three locations

The cumulative frequency distribution of the treatment plant has the shallowest inclination showing that that is where the most variation and the highest values lie. In the flow direction the slopes get steeper and shift to the left, indicating lower values and less variation. Only in the 99,0 and beyond percentiles are there higher values, indicating the high peaks when resuspending.
From the frequency distribution the conclusion about whether particles settle or resuspend or not can also be drawn.

2.2.5 An example of continuous monitoring of particle counts

The stylistic phenomena as shown in Fig 2-3 through Fig 2-5 are related to the turbidity of the water. Particle count measurements give more detailed information on the size and number of particles than the general parameter of turbidity. In this study particle counters are used as they are expected to give more information on the particle related processes through the measurement of the size and number of particles. Though turbidity and particle counts are not universally related, they are related for specific water types.

Fig 2-9 shows the results of particle counts at two locations measured with a MetOne PCX monitor in 7 diameter ranges. The measuring frequency is in the same order as the turbidity (1 to 5 minutes) because the time scale of the changes in particle counts is the same as the scale for the turbidity. The particle counts are calibrated according to specifications. The treatment plant operates a conventional groundwater treatment process with rapid sand filtration as the finishing step. The backwash program of the plant is set to start at midnight and the start-up of the filters has an effect on the number of particles in the water.

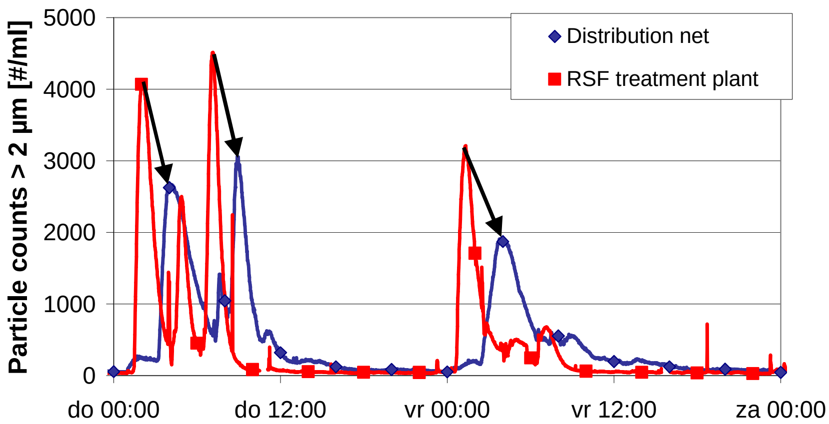


Fig 2-9 Total Particle counts at treatment based on rapid sand filtration (RSF) and distribution network

The arrows in Fig 2-9 indicate the peaks in the particle counts and the mirrored pattern in the distribution network. The same phenomena can be observed as in the case study with the measuring of turbidity at a treatment plant and some downstream locations in the distribution network (section 2.2.4). The downside of using particle counters is the huge amount of data that is generated. Each sampling gives the number of particles in a band width and with the measuring frequency of a few minutes that is required to monitor the short time scale processes this generates large amounts of data. One of the simplest compression techniques that can be used is to present the total amount of particles irrespective of the size as is presented in Fig 2-9. As small particles (< 5 µm) dominate the total number of particles this primarily gives information on these sizes of particles.

Other compression techniques are based on describing the relation between the number of particles and the size of the particles. This can be a Pareto-curve, a power law or an improved power law (Ceronio and Haarhoff, 2005). However, these compressions result in the curve characteristics that do not have a physical meaning and thus do not facilitate the interpretation of the measuring data directly.

In this thesis the accumulation of particles is important for the generation of discolouration events in the distribution network. The volume of the particles therefore is of importance rather than the number. Theoretically for instance a flocculation process will decrease the

number of particles, while the volume of the particles will stay the same or increase unmeasurable if sub-micron particles clog to measurable super-micron particles.
The particle volume concentration is also a good indication for those measurements that could contain important information on what happens with the particles in a specific sample. Is, for instance, an increase or decrease in particle volume concentration caused by an in- or decrease of the number or a change in numbers per diameter.

The particle counters that were used is this study were MetOne PCX unless otherwise indicated. These particle counters were able to measure in 32 single µm ranges, allowing for an accurate calculation of the particle volume without any modelling. Each measurement gives the number of particles per ml in 32 ranges. Ranges have a diameter width of 1µm, starting at 1 µm up to the ranges of >31 µm,. The volume is calculated by assuming the particles to be perfectly spherical with a diameter equal to the linear average of the boundaries of a range. This reduces the 32 counts per ranges to one value: the calculated particle volume concentration.

For this study the particle count measurements are presented as calculated particle volume concentration in the unity part per billion volume (10^{-9} m^3/m^3). Per measurement or sample the particle size distribution can be made and presented on a log/log scale (Ceronio and Haarhoff, 2002). Analogue to the interpretation of the turbidity, the particle counts can be followed in the network (Fig 2-7). Also the presentation with the cumulative frequency distribution of the calculated particle volume is analogue to the turbidity graphs. Fig 2-10 gives the calculated volume of the particle counts and Fig 2-11 gives the cumulative frequency distribution of these measurements at a single measuring location at the beginning of a distribution network. The measuring frequency is 2 minutes.

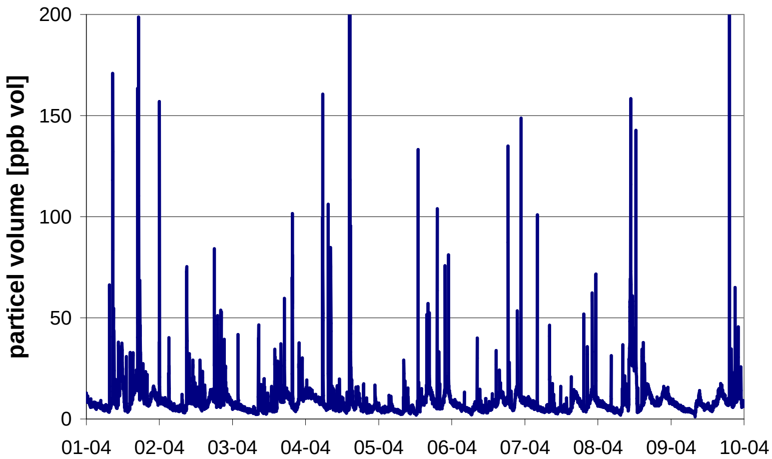


Fig 2-10 Calculated particle volume during a period 9 days of measuring with a 2 minutes sampling frequency

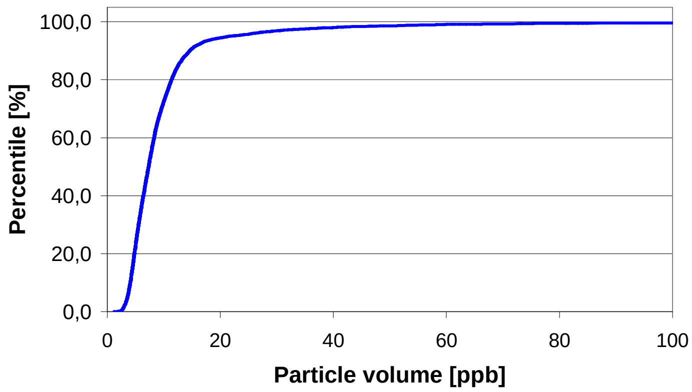


Fig 2-11 Cumulative frequency distribution based on the measurements of Fig 2-10.

This pattern shown in Fig 2-10 has sufficient peaks that can be identified in the network. The cumulative frequency distribution shows how irregular the pattern is. The ratio between the higher percentile values gives an indication of the shape and steepness of the curve. A steep curve indicates a stable pattern, while a more shallow curve shows more variation in the calculated particle volumes. For this research the values above the 80-percentile point gives good information, because this part of the frequency distribution curve shows the 'spikeyness' of the curve. The ratio between the 90% and 99,5% percentile values is used as a indicator for the extremity of the curve (see Table 2-3).
Another characteristic of the calculated particle volume trace is the average value of the calculated particle volume and the distribution of this volume over the relatively short spikes and the more constant lower values. To identify these volumes two parameters are introduced called the Surface-90% and Surface+90%, (Surf-90% and Surf+90%). The Surf-90% gives the ratio between the average value of the CPV's below the 90-percentile measurement and the average value of the CPV over the whole measuring period. The Surf+90% gives the ratio between the average value of the CPV's above the 90-percentile measurement and the total average CPV.

$$Surface-90\% = 0{,}9 * \frac{\overline{CPV_{0-90}}}{\overline{CPV_{0-100}}} * 100\% \qquad Surface+90\% = 0{,}1 * \frac{\overline{CPV_{90-100}}}{\overline{CPV_{0-100}}} * 100\%$$

With

$\overline{CPV_{0-100}}$	:	Average Calculated Particle Volume whole measuring period [ppb vol]
$\overline{CPV_{0-90}}$	:	Average Calculated Particle Volume measurements between percentile values 0 and 90% [ppb vol]
$\overline{CPV_{90-100}}$	:	Average Calculated Particle Volume measurements between percentile values 90 and 100% [ppb vol]

The Surf-90% and the Surf+90% are expressed as percentages of the average value indicating the particle volume concentration that is measured during 90% of the time and 10% of the time respectively. Together the Surf-90% and the Surf+90% give 100% of the total average of all the calculated particle volumes. Graphically the Surface-90% gives the surface between the y-axis and the frequency distribution curve from zero to the 90% value and the Surface+90% gives the surface between the y-axis and the frequency distribution curve from

the 90% value to the 100% value. The higher the Surf+90% value the more spikey the pattern is.
All the characteristics of the example curve of Fig 2-10 are summarised in Table 2-3. The surf-90% and the surf+90% is given as a percentage of the average value of the CPV.

Table 2-3 Characteristics of example curve

Frequency Percentile [%]	[ppb]
90,0	14,57
95,0	21,80
98,0	40,18
99,0	58,39
99,5	84,15
99,9	160,58
ratio 90/99,5	0,17
average [ppb]	9,85
surf -90 [%]	66,4%
surf +90 [%]	33,6%

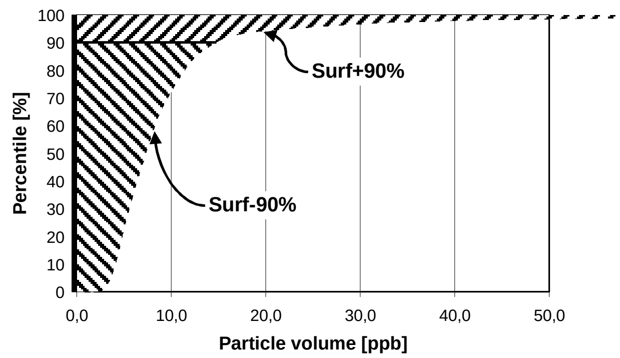


Fig 2-12 Surf+90% and Surf-90%

The information on particle size distribution can be used to see what the differences are between the water with high particle counts and consequent high calculated particle volume and the lower particle counts. For instance the particle size distribution of the calculated particle volume at the 98, 25 and 5 percentile measurements of the example (Fig 2-13) show that the distribution is a proximally identical for the different samples, but that the amount is different, resulting in different calculated particle volumes.

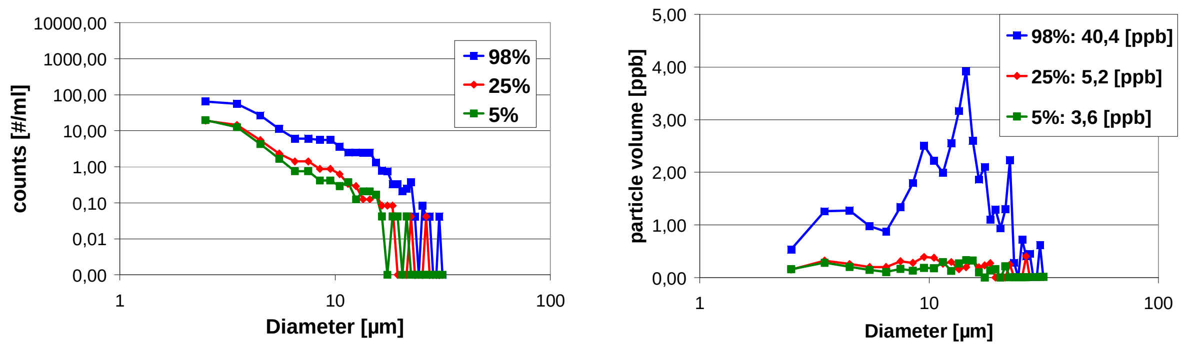


Fig 2-13 Left: Particle size distribution of samples at the 5, 25 and 98 percentile point. Right: Calculated volume distribution over the size ranges for the same samples.

Measuring the particle-related phenomena in the network has improved largely by the introduction of online particle counters that can be used in the network. Up till two years ago, within this research turbidity meters were used to analyse the processes. This is sufficient to get a good understanding of the processes and to get an idea of what the possibilities are to control the processes.
Particle counters add exciting new possibilities for the analysis of the fate of particles in the network. Haarhoff and Ceronio, for example, demonstrated a technique to fingerprint the particle composition of water and to model the particle distribution if insufficient ranges are

available (Ceronio and Haarhoff, 2005). With the multi-range particle counters this is not necessary and the analyses can be performed on the actual data.

2.3 Effect measurements

2.3.1 Introduction

The changes in water quality measured by direct methods give an indication of what happens in the network. The effect of the processes in the DWDS is the accumulation of sediment that increases the risk of complaints. The effect measurements are aimed at determining the discolouration risk itself.

Discolouration events, leading to customer complaints, are mostly connected to 'hydraulic events', disturbance in the normal situation in the network. In a number of cases the relationship to hydraulic events is not obvious, but a closer study of the event typically reveals the underlying hydraulic disturbance (Vreeburg, 1996). Typical 'hydraulic incidents' are pipe burst, hydrant use and valve exercises that result in velocity and flow direction changes in the pipes. The effect measurements that are commonly used and discussed in this chapter are the Resuspension Potential Method (Vreeburg et al., 2004a) and the registration of customer complaints.

2.3.2 The Resuspension Potential Method

The Resuspension Potential Method (RPM) as developed in the Netherlands is based on measuring the mobility of the material in a network (Vreeburg et al., 2004a). The principle of the method is based on the phenomenon of resuspension of particles caused by a hydraulic disturbance. The method is primarily a relative method that is in origin used to compare the presence and mobility of sediment pre and post an intervention in the network. An intervention is for instance a cleaning action. The method is developed to be applied in distribution networks with typical small diameter pipes in the range from 50 to 200 mm. The majority of the pipes tested in practice is in the range of 100 to 125 mm.

The RPM consists of a controlled and reproducible increase in the velocity within a pipe. The reaction of the turbidity on this hydraulic disturbance is measured and translated to a value for the Resuspension Potential. An increase of 0.35m/s, in addition to the actual velocity at the time of measuring, was determined empirically (Vreeburg et al., 2004a). Main reasoning is that this increase in velocity is mild compared to the increase of velocity as result of a pipe failure or the full use of a fire hydrant. The full use of a fire hydrant with a two sided supply on a 100 mm pipe would increase the velocity with at least 0,6 m/s. The hydraulic shear stress as a result of the increase in velocity of 0,35 m/s causes particles to mobilise, affecting the turbidity of the water. The method is mainly applied in 100-150 mm pipes, hence the absolute difference in shear stress caused by the uniform velocity increase is not very large. The turbidity effect is monitored and translated into a ranking for the Resuspension Potential. This Resuspension Potential has an obvious relation with the actual discolouration risk, but not necessarily with actual discolouration events. For a discolouration event, next to the presence of mobile sediment (as measured with the RPM) also a hydraulic disturbance is necessary. In hydraulically quiet networks, meaning networks without large disturbances caused by failures and other unusual high demands, a high RPM can stay without complaints. On the other hand a moderate RPM in a network with a few disturbances can lead to customer complaints.

The Resuspension Potential Method is applied as follows:

- Isolate the pipe for which the discolouration risk is to be assessed, as for uni-directional flushing (Antoun et al., 1999). The isolated length should be at least 315 meters long to be sure that only this single pipe is affected by the 15 minutes disturbance of 0,35 m/s.
- Open a fire hydrant such that the velocity in the pipe is increased by the additional 0.35 m/s above the normal velocity and maintain that rate for fifteen minutes; after this reduce the flow to normal (total length affected is thus 315 m).
- Monitor turbidity in the pipe throughout the fifteen minutes of extra velocity and beyond that until turbidity returns to the initial level.

Schematically this procedure is illustrated in Fig 2-14.

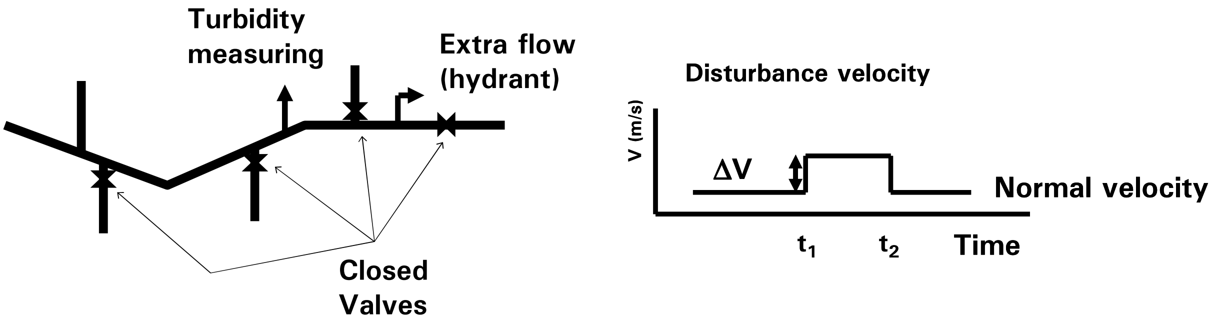


Fig 2-14 Principle Resuspension Potential Method: Increase velocity (Δv) with respect to normal velocity by opening a hydrant in isolated pipe and measure turbidity

The result obtained from an RPM test is the turbidity response of a pipe. A typical example is shown in Fig 2-15 highlighting the four regions of the trace that are utilised to rank discolouration risk:

- Base turbidity level
- Initial increase in turbidity at the start of the hydraulic disturbance
- Development of turbidity during the hydraulic disturbance
- Resettling time and pattern to base (initial) turbidity level

Base turbidity level
The base turbidity level is the level preceding the hydraulic disturbance. Base line turbidity can be linked to the turbidity of the source water and can give some insight into the source of the sediment. Baseline turbidity is also needed to judge the time required for the turbidity to resettle after the increased velocity has stopped.

Initial increase in turbidity
Following the actual disturbance the turbidity will rise immediately to a certain level. This initial increase indicates the instantaneous mobility of the sediment, resulting in peak turbidity. A loose layer that is immediately available causes the initial turbidity.
The initial increase is an indication of the maximum turbidity that can be caused by a hydraulic incident. High initial turbidity increases the chance that discolouration will be noticed.

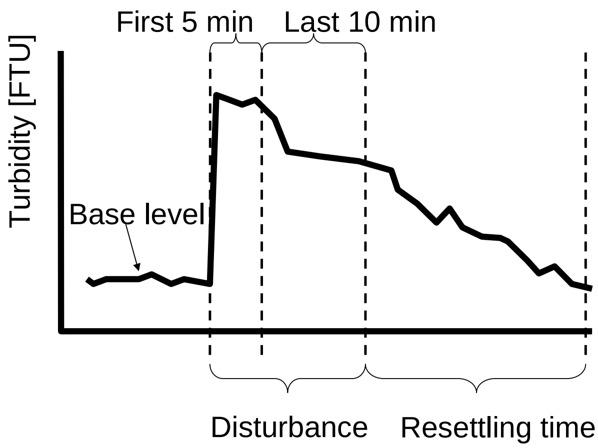


Fig 2-15 Typical RPM turbidity trace resulting from an RPM test, showing the four regions used to rate the discolouration risk.

Development of turbidity during the hydraulic disturbance
The hydraulic disturbance is kept going for 15 minutes, allowing the turbidity to develop to a stable level. If the turbidity stays near the first five-minute level, then the amount of sediment is considerable and the composition of the sediment is homogeneous. A quickly reached level that remains fairly stable during 15 minutes indicates a high discolouration risk.
In many cases the turbidity drops during the 15-minute disturbance time. Three phenomena can explain this:

- A relatively small amount of heavy sediment is present in the pipe. The extra forces in accelerating the flow also promote the initial whirling up of this heavy sediment. The significance of this sediment is limited because it is settling even during the deviating hydraulic circumstances, making the discolouration risk smaller. The chance that the initial discoloured water can actually reach a tapping point in which it can be visually identified is small.
- A too-short length of isolated pipe caused by a wrong isolation, forgotten valves or leaking valves. If the length is less than 315 meters, water is drawn from pipes upstream of the isolated pipe. This water originates from the looped network or from pipes with larger diameters and is less disturbed than the 0,35 m/s.
- A non-homogeneous deposit over the length of the tested pipe, for instance in a hilly area with concentrations of sediment in the depression of a pipe.

In all cases however the level of turbidity following the first peak determines the continued discolouration risk. This level will be present over an extended time, allowing customers more time to see it.

Resettling to base level
After closing the hydrant, it takes a certain amount of time for the turbidity to resettle again to base level. The time needed is important for the discolouration risk or, actually, the complaint risk. If the turbidity stays high during a longer period, the risk of noticing the turbidity in an application, such as filling a white basin (bathtub, washing bin, bathroom sink, etc) is greater.

2.3.3 RPM and discolouration risk

Based on the turbidity trace measured before, during and after the disturbance of the velocity, the Resuspension Potential (RPM) can be determined. The RPM is based on a ranking of five aspects of the turbidity trace:

- Absolute maximum value of turbidity during first five minutes of disturbance;
- Average value of turbidity during first five minutes of disturbance;
- Absolute maximum value of turbidity during last ten minutes of disturbance;
- Average value of turbidity during last ten minutes of disturbance;
- Time needed to resettle again to initial turbidity level.

The ranking system is based on the process that determines a discolouration event through resuspension of sediment. The turbidity during the first five minutes represents the effect of the acceleration of the flow velocity and is rated on the peak and the average. The turbidity during the last ten minutes is the effect of a longer disturbance and can be lower than the initial acceleration effect. This is also judged on the peak and average value. The time to resettle adds to the discolouration risk because it determines the time that an increased turbidity level is present and can be noticed by a customer. A high turbidity during the disturbance that resettles quickly to the base level has a lower discolouration risk than a lower turbidity that takes a long time to resettle.

For each aspect validation on a 0 to 3-point scale is made: 0 is the lowest or best rating and 3 the highest or worst rating. The lowest value equivalent with ‘no resuspension potential’ is thus 0 (zero) and the highest value or ‘maximum discolouration risk’ is 15. This flexible rating makes the RPM a measuring method that is primarily used for the comparison of different situations and less as a absolute measuring tool.
For the rating per aspect a scale must be made that is calibrated to the turbidity equipment being used. Also, site-specific elements can be taken into account. If, for instance. the intuitive feeling of a network is that the discolouration risk is moderate, then the rating scale can be adjusted to this level. Changes in the discolouration risk that may occur when, for example, the treatment is improved or a cleaning program is started can be related to the earlier objectified level. A cleaning program should lead to less sediment in the pipes that should result in lower levels of turbidity during the disturbance of the velocity, leading to a lower value of the RPM. The discolouration risk established in this way is a relative figure that can be company-specific or even area or site specific. The effects of changed operation of the network can be assessed specifically and instantaneously. For every situation and type of measuring equipment a ranking table can be made, depending on the type of turbidity-measuring equipment being used, local circumstances but also the goal of the measurements. Table 2-4 and Table 2-5 give the values for discolouration risk for the Sigrist KT65 turbidimeter at a dedicated measuring point and the Dr Lange Ultraturb equipment at the flushing point, respectively.

Table 2-4 Example of ranking RPM for discolouration risk using the Sigrist KT65 equipment at a dedicated measuring point

	0	1	2	3
Absolute max first 5 min	<0,3 FTU	0,3-1,0 FTU	1,0-2,4 FTU	>2,4 FTU
Average first 5 min	<0,3 FTU	0,3-1,0 FTU	1,0-2,4 FTU	>2,4 FTU
Absolute max last 10 min	<0,3 FTU	0,3-1,0 FTU	1,0-2,4 FTU	>2,4 FTU
Average max last 10 min	<0,3 FTU	0,3-1,0 FTU	1,0-2,4 FTU	>2,4 FTU
Time to clear	< 5 min.	5-15 min	15-60 min	>60 min

Table 2-5 Example of ranking RPM for discolouration using the Dr Lange Ultraturb equipment at the flushing point

	0	1	2	3
Absolute max first 5 min	<3 FTU	3 – 10 FTU	10-40 FTU	>40 FTU
Average first 5 min	<3 FTU	3 – 10 FTU	10-40 FTU	>40 FTU
Absolute max last 10 min	<3 FTU	3 – 10 FTU	10-40 FTU	>40 FTU
Average max last 10 min	<3 FTU	3 – 10 FTU	10-40 FTU	>40 FTU
Time to clear	< 5 min.	5-15 min	15-60 min	>60 min

The ranking tables have a variety in the borders of the ranges that shows the flexibility of the methodology. Firstly, it allows for different turbidimeters and for different locations of measuring being used. The Sigrist KT65 measures with a white-light lamp and the Dr Lange Ultraturb with an infrared lamp resulting in different values for the same event. The Sigrist KT65 is used by Kiwa Water Research because of the robustness of the equipment that was available when the monitoring system was constructed in the early 1980-ties. The Dr Lange Ultraturb is a more recent turbidimeter that is often used by water companies, because of it's size and simplicity in use. The first table uses a dedicated measuring point at the researched pipe itself, as is shown in Fig 2-14. The second table is meant for measuring at the flushing point, which is more practical, but gives higher values for turbidity (Slaats et al., 2002). Both measurement strategies can be tuned to each other in a calibration experiment using both methods simulateously,

Secondly, the ranking can be tailored to the actual application. The first table is used to distinguish subtle differences in resuspension to see what the effects are of different cleaning methods. For that application it is important that the measurement be sensitive to distinguish the different levels of resuspendable sediment that are left after cleaning. In this case it is not that important to know how high the RPM in absolute turbidity values was prior to the cleaning.

The second table is used to prioritise the need for cleaning in a whole area that has a certain level of complaints. In that case the sensitivity should be more in the higher ranges to rank the areas that most urgently need cleaning. In this case sensitivity in the lower ranges is not important as the same cleaning method is used for the whole network.

With the ranking the sensitivity of the measurement can be tuned to the actual needs and equipment. This allows for a change in measuring equipment or measuring locations without older data being lost. The ranking can also be adjusted, for instance, to changing standards or company policies.

In the practical application of the RPM, the measuring of the resettling time is often limited to a maximum, for instance 30 minutes, or is ignored. If it is ignored the maximum ranking is set to 12 instead of 15. As in the research will be shown the RPM can also be adjusted for the time the disturbance is performed. This influences the value of the RPM for that case and hampers the possibilities to use the RPM in an absolute sense, but makes it only suited to compare pre- and post levels in a specific area.

2.3.4 Typical RPM-curve

Fig 2-16 shows an example of an RPM indicating a high discolouration risk. The measurement has been made with Sigrist-equipment, so Table 2-4 is applicable. Scale of the equipment was set at a maximum of 2,4 FTU and the sampling frequency is two-and-a-half minute. The base turbidity is rather constant and low (0,22 FTU). The disturbance is performed from 10:15 to 10:30. The increased turbidity during the first five minutes as well as during the following 10 minutes is high and above 2,4 FTU, which is the maximum turbidity. This leads for the first four aspects to the maximum score. The time to clear is several hours, meaning that on all the items the maximum score is reached, resulting in the maximum score of 15. Despite the fact that the base line turbidity is very constant and low, this location will experience a high turbidity with any hydraulic event. The main problem in this area however is the long time that a high turbidity is visible, even with this mild disturbance.

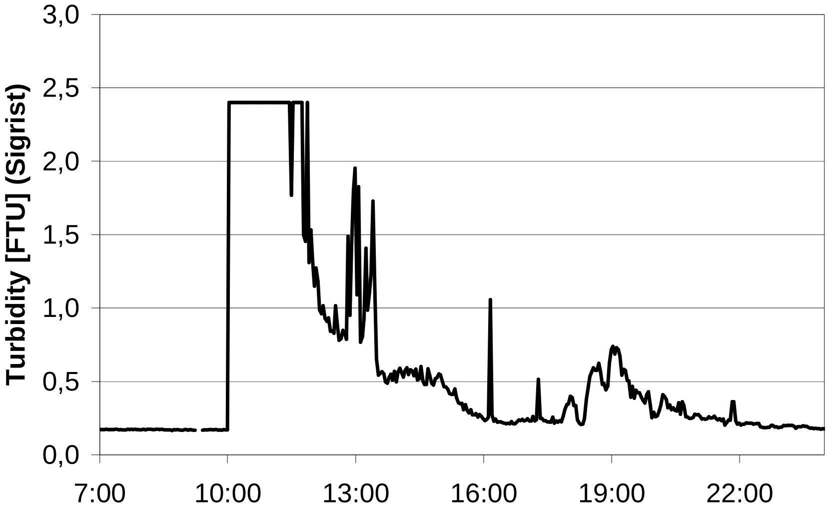
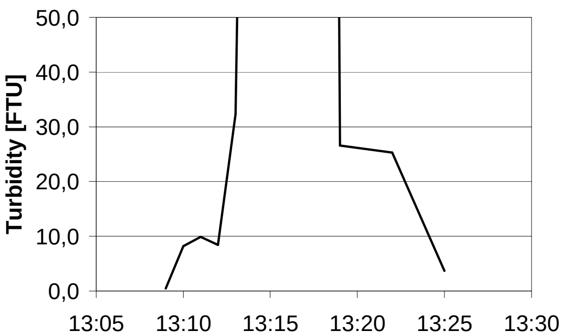


Fig 2-16 Example of RPM, measured with Sigrist-equipment with maximum scale 2,4 FTU, indicating a high discolouration risk with a maximum score of 15. The increased velocity occurred from 10:00 to 10:15 and the effect is long measurable.

An example of the application of the RPM as an evaluating tool for a cleaning action is given in Fig 2-17. The left picture is the turbidity during the disturbance, which was from 13:10 to 13:25. The RPM is ranked using the boundaries of Table 2-5. The maximum during the first five minutes was above 40 FTU, but the average was between 10 and 40 FTU. Both the maximum and the average of the last ten minutes were above 40 FTU. The resettling time is not measured to save time.

After cleaning the same procedure is applied. The disturbance starts now a 9:59 and ends at 10:14. One point is credited because the maximum in the last ten minutes is just above 3 FTU. In this case it can be concluded that the cleaning was effective resulting in a reduction of the RPM which means that the resuspendable sediment has been removed.



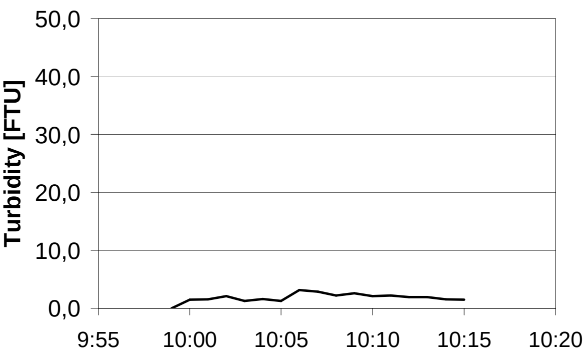


Fig 2-17 Results of RPM method applied pre- and post-cleaning to evaluate the effectiveness of the operations. The pre-cleaning RPM is 11 according to Table 2-5 and post-cleaning is 1; resettling time is ignored

The RPM is suited for the evaluation of the build up of sediment layers that have a certain mobility. Every type of measurement, however, influences the parameter that is actually measured and the RPM is not an exception to that rule. When the RPM is measured a certain amount of sediment is removed during the 15 minutes disturbance. If the RPM is repeated after a short while and the same result is measured, than it should be kept in mind that there was a loading of the system because the removed sediment during the first measurement was replaced with 'fresh' sediment. Especially when a low loading of the system is expected, the repeated RPM influences the measuring location.

2.3.5 Flexibility of the RPM and alternatives

The RPM was originally designed as a comparative measuring tool to asses the effect of changing boundaries in a network. For instance the evaluation of the efficacy of cleaning as is demonstrated in Fig 2-17. As an increasing number of water companies apply the principle of the method, also an increasing variety of practical applications occurs. All the different application have in common that the effect of a disturbance on the turbidity is measured.
A full evaluation of the RPM on all five aspects requires a measuring time of at least a few hours, mostly consumed by determining the initial base level and pattern and the resettling time. It also requires turbidity-measuring equipment that can be expensive and which requires skill to operate. As said, implementation of the principle of the RPM during the last decade has led to some alternatives in the analysis methods. The first adjustment is to limit the measuring time of the base level turbidity to some minutes prior to the disturbance or the time needed to isolate the pipe. Limiting the time used to measure the resettling time to a specific time, for instance 30 minutes is another adjustment. The examples shown in Fig 2-17 use this abbreviated version of the RPM. As seen the resettling time is not clear in the first graph. However the overall impression is that the discolouration risk is high (>12) and action is required.
Limitation of the measuring time allows for more measurements in a working day. It is possible to do four measurements in an 8-hour working day.
Replacing the continuous monitoring by taking 5 to 8 grab samples during and after the disturbance limits the total measuring time even further. All the adjustments, however, cause loss of information, specifically on the base level and the resettling time. For the assessment of the trigger level towards the discolouration risk, this is less important as the examples in Fig 2-17 show.

2.3.6 Customer complaints

Customer complaints are the ultimate effect-measuring 'tool' for actual discolouration events. The measuring tool itself, namely the observation of a customer combined with the motivation to inform the company, is however not a reliable tool. The turbidity level at which customers are inclined to complain cannot be strictly defined, but is dependant on local circumstances. If, for instance, discoloured water is used in an application with a running tap as washing hands, than the turbidity or discolouration is more difficult to notice than in case it is used to fill a white bath. To determine a threshold for the turbidity at which customers would complain a duplo experiment is performed in which delegates at a technical conference were presented a number of white and continuously stirred bowls (Slaats et al., 2002). Each bowl had water with different turbidity that was a dilution range of a large flush sample from a drinking water network. The threshold for complaining is around 10 FTU.
Next to the actual threshold level for turbidity is difficult to define also the recording of the complaints by the company is not always reliable and consequential. A classic example is a large pipe burst with a lot of complaints; after a while the customer service centre releases the

message that the cause of the problems is known and that everything is being done to solve the problem as quickly as possible; at that point the complaints are usually not recorded any more. Relatively small companies, especially, record the complaints around large incidents insufficiently.
With the ongoing merging of the Dutch water companies the scale of the companies is increasing and the customer service centres are becoming more professional.
A good customer complaints registration records every complaint and passes it on to the relevant department in the company. Although it sounds contradictory, the registration of complaints should be done without knowledge of the cause and nature of the complaints.

The former Nutsbedrijf Regio Eindhoven (NRE) had a good customer complaints registration system with a 24/7 manned call centre that fit the description that "every complaint is registered", including the multiple complaints during incidents. The results of this meticulous complaint registration are shown in Fig 2-18

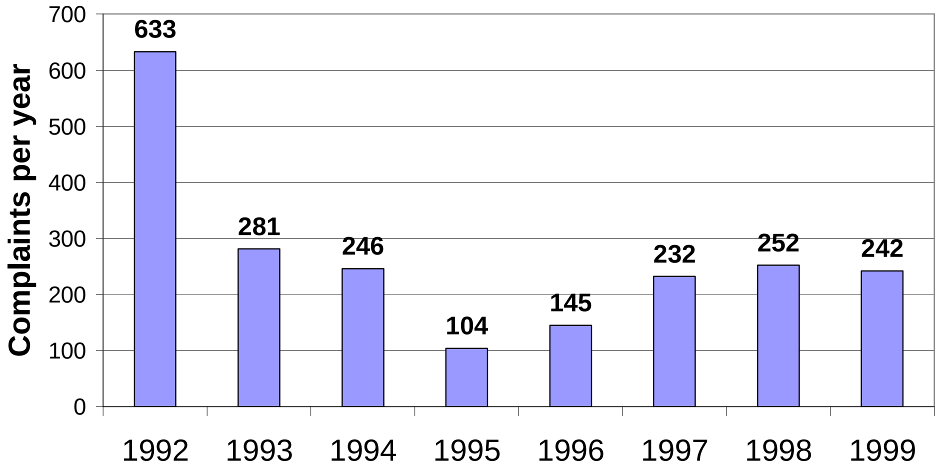


Fig 2-18 Discolouration complaints Nutsbedrijf Regio Eindhoven 1992 – 1999.

In the period 1992-1993 the water company performed a carefully planned unidirectional flushing program that met the operational requirements (see paragraph 5.3). With the customer complaints registration, the effect of the program was clearly demonstrated: the number dropped from 633 to 245 in 1994 and even dropped further, to 104, in 1995. Then, the number of complaints gradually increased again which led in 1999 to a measuring campaign. The RPM was applied in several series to evaluate the discolouration. RPM locations were selected both with and without recorded complaints to see a difference.
The measuring campaign revealed that most of the locations had a high RPM, but the level of complaints varied. There was no clear relationship between the number of recorded complaints and the RPM. Almost all the analysed areas had an elevated RPM, but not all the areas had the same elevated number of complaints. This shows that the RPM is an indication of the discolouration risk, but that for an actual discolouration event also a disturbance of the velocity is necessary that obviously is not always the case. During the RPM-series, however, some extra complaints were recorded, as is shown in Fig 2-19.

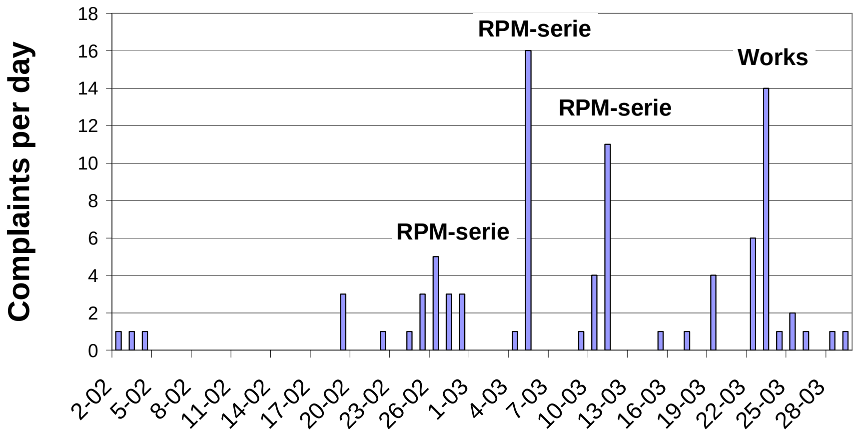


Fig 2-19 Complaints during the RPM measuring campaign at NRE

Also, planned work on 23 March caused some complaints indicating the discolouration risk was increased and that the combination with the disturbance of flow conditions resulted in complaints.

This case shows the capabilities of customer complaint registrations as a monitoring tool for discolouration risk. A complaint occurs when two conditions are met: a relatively high level of resuspendable sediment in the pipe and a hydraulic disturbance. Without the disturbance the discolouration risk may stay unnoticed until something actually happens.

The customer complaints level can be applied as a monitoring tool, when the registration system is well defined and consistently applied. It is difficult to give absolute figures on what levels are acceptable. Complaint levels of 1 per 1000 connections per year can for the one company be a trigger to take action while the other company aims at reducing the level to 4 per 1000 a year like is mentioned in the targets for 2008 of the Melbourne water company Yarra Valley Water.

2.4 Concentration measurements

2.4.1 Introduction

The difference between the level of incoming and outgoing particulate mass is subtle as is shown by the measurements of turbidity and particles counts. To assess the composition of the particles, a concentration is necessary for chemical and volumetric analysis. Based on the calculated particle volume (see for instance Table 2-3) the volume of the particulate mass is 10 to 100 ppb. With a density of 1100 kg/m^3 this means 11 to 110 µg/l. For a valid gravimetric analysis at least 2,5 mg is necessary (APHA, 1998), meaning that a sample of at least 25 litre must be taken. These samples should be taken over an extended time-period to ensure that variations are sufficiently balanced over the sampling time. As is shown in Fig 2-6 the cycle of variation can be captured if sampling times are 24 to 72 hours.
Two experimental techniques of concentration have been tried in this study: The Time-Integrated Large Volume Sampling (TILVS) and the Hemoflow.

2.4.2 Time-Integrated Large Volume Sampling (TILVS)

Within the frame work of this study, an online filtration device is developed using a dosing displacement pump that feeds a constant flow onto a small filter membrane, termed a 'Time-

Integrated Large Volume Sampler' (TILVS) (Fig 2-20). By having a continuous delivery of water through the filter, a large amount of water can be sampled, preconcentrating the low numbers of particles and providing a time-integrated sample. This is done by using a True-dos M (Grundfoss) pump to deliver a constant flow to the filter by applying changes in pressure to keep the flow constant as the filter fouls and resistance increases. The filtration unit is made of stainless steel and can withstand up to 10 bars of pressure. The unit also has a water overflow vessel to allow the pump to passively sample from the sampling tap as the inflow on the True-dos pump is not suitable for sampling directly from a pressurised system. The pump delivers a constant flow of water to the filter, allowing the total volume that is filtered over the duration of the sampling period to be calculated. The updated version of the TILVS, based on the experience in this study and advice of the manufacturer, is equipped with a pressure sustaining valve fitted before the filter unit. This improves the accuracy of the pump.

Flow rates may range from 0.5-4.0 L/h over a period of 19-72 hours depending on the particle content of the sampled water. The flow rates should be chosen so that the sampling can continue for 24 hours without clogging the filter to trans membrane pressure over 10 bar. As filter 0.45 µm cellulose acetate filters are used and 1.2 µm cellulose acetate filters are also tested. Only the results of the 0.45 µm filters will be discussed.

The membrane filters used in this study had a much smaller pore size than filters used in comparable studies. (Gauthier et al., 1997) used 5 µm cellulose acetate filters, while (Nguyen et al., 2002) used a non-destructible 1 µm filter cartridge. There were two reasons why a smaller pore size is chosen for this study. As a direct result of the multi-barrier approach the sediment loading of the Dutch drinking water (in the order of 0.05 to 0.2 ppm) is much lower than elsewhere. Gauthier reported a concentration of 0.35 ppm in his study (Gauthier et al., 1997), while Nguyen reported a value of 0.65 ppm (Nguyen et al., 2002). The advantage of filters with a pore size of 0.45 µm is that this pore size is close to the lower detection size range of the used particle counters.

Filters are prepared by rinsing with deionised water and drying in a 105 ºC oven. All glass and plastic used for the preparation and handling of the filters were acid bathed in a 1:10 dilution of AR grade nitric acid (Merck) before use. Cellulose acetate filter membranes are used as they are suitable for both inorganic and organic (volatile suspended solids) analysis. Filters were analysed gravimetrically for total suspended solids (TSS) after drying in a 105ºC oven and cooling in a desiccator.

Fig 2-20 TILVS-unit. On the left is the pump, in the middle the water overflow vessel and on the right the filter unit.

2.4.3 Hemoflow

Another experimental pre-concentration technique is called the Hemoflow sampling technique, developed in house by Kiwa Water Research. Fig 2-21 shows a picture of the equipment. Originally, this device was used to pre-concentrate large samples for biological analysis. The pumps used have a low impact on the turbulence of the water to restrict the damage of bacteria as much as possible. The device is based on kidney rinsing machines and the membrane filtration unit is developed to be used in kidney dialysis equipment.

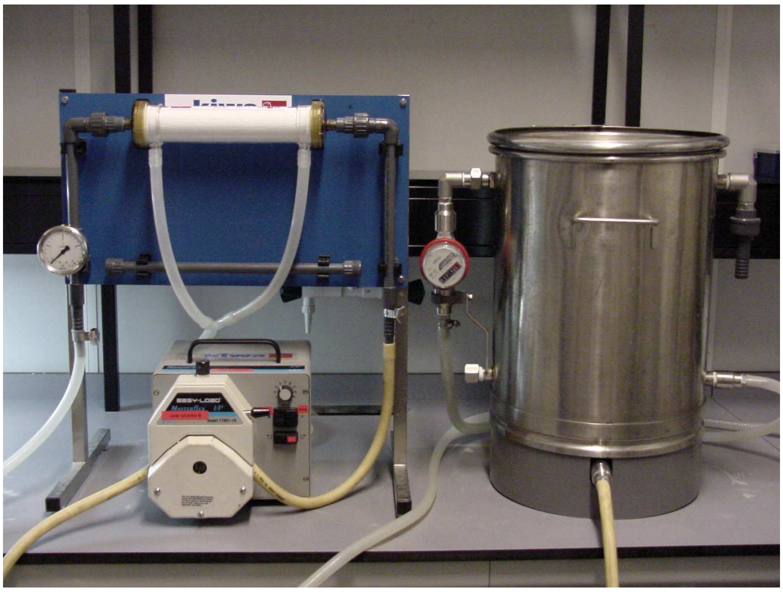


Fig 2-21 Picture of Hemoflow pre-concentration unit

The installation consists of a stainless steel recirculation vessel shown on the right in Fig 2-21. The vessel is controlled by a floater tap and the flow is measured using a standard household flow meter. With a conventional hose displacement pump, often used in medical applications to pump blood, the water from the vessel is fed in cross-flow over the membrane filter, while the concentrate is fed back to the vessel. The permeate is discharged as it no longer contains particulate matter.
The flow over the filter is restricted to 800-900 ml/hour, allowing more than 2000 litres of water to be concentrated over 24 hours. The concentrated sample can be analysed on particulate matter either by filtering or by evaporation at 105 ^{o}C. The residue can be analysed analogous to the analysis of the TILVS filters, i.e. VSS, TSS and elemental analysis through ICPMS.
Concentration techniques as the Hemoflow and the TILVS are still experimental and need to be further developed in future research. One of the main reasons for further development is to get a good database with relevant values that can serve to better manage the treatment facilities and the discolouration problems in the network.

3 Effects of particle-free water in a common drinking water distribution system

3.1 Introduction

One of the sources for loose particles in the Drinking Water Distribution System (DWDS) is the treated water. Particles can enter the distribution network as background concentrations of organic and inorganic material from the source water (Lin and Coller, 1997; South-East-Water, 1998; Kirmeyer et al., 2000; Slaats et al., 2002; Ellison, 2003), due to incomplete removal of suspended solids at the treatment plant (Gauthier et al., 2001; Vreeburg et al., 2004b) or be added to the water by the treatment plant itself, such as carbon, sand or calcium particles, alum or iron flocs and bio particles originating from bio filters (Alere and Hanæus, 1997).

To quantify the contribution of particles to the accumulation of loose deposits in the network two similar areas in a large DWDS were isolated and supplied with different types of water. The first area was supplied with particle-free water and the second with normal drinking water. The particle-free water was produced by *in situ* post-treatment of finished drinking water using an Ultra Filtration membrane type 0,1 µm pore size. This first area is referred to as 'Research Area' (Res) and the second as 'Reference Area' (Ref).
The hypothesis is that in the Reference Area, supplied by normal drinking water, a build-up of a sediment layer will be observed, while in the Research Area this build up will be much less or even non-existent. With reference to the particle-related processes in the network (Chapter 1, Fig 1-5) the particles in the new layer originate from the drinking water itself, from either a biological process or precipitation and flocculation process in which the soluble form of especially iron will change to the particulate form which may add to the sediment layer. The corrosion process is ruled out as there is no unprotected cast iron in the areas considered.

3.2 The experiment

3.2.1 Set up

Two similar areas with respect to pipes lengths, number of connections and pipe material in a large DWDS were isolated from the rest of the network and supplied through a single dedicated line. Every area contains approximately 500 household connections and the total length of the pipes is around 7500 meters. The schematic and real maps of both Research and Reference Areaa are given in Fig 3-2 and Fig 3-3 and further described in the next section.

For the supply of particle-free water to the Research Area two UF installations were installed each with a capacity of 7,5 m^3/h. The Research Area was pressurised with a separate pump group in combination with a 400 m^3 clear water balancing reservoir for the UF water. The separate pump group was monitored within the general operation centre of the water company and can be remotely controlled. A scheme of the connection of the Research Area and the Reference Area to the larger network is given in Fig 3-1. The starting point is the transport network that is directly pressurised by the pumps at the treatment plant Spannenburg (see detailed description in section 3.2.3). From the network, drinking water is drawn into a clear water reservoir that serves as a process buffer. From the reservoir the water is fed to two UF-installations. The filtered water is stored in a 400 m^3 UF-reservoir from which it is pumped to the Research Area. Because the Research Area has actual household connections, the continuity of supply should be guaranteed. Emergency valves 1 and 2, closed during normal operation, are opened if one of the components fails. If the pump for the Research Area fails, valve 1 will be opened and the Research Area can be supplied with the normal drinking water pressurised by the pumps at the treatment plant. Valve 2 will be opened if the level in the UF-reservoir drops below a critical level due to malfunctions of the UF-installation. The UF-reservoir will fill with normal drinking water and supply can then continue. During the entire research period the emergency valves 1 and 2 stayed closed as both the UF-installation and the pump worked continuously.

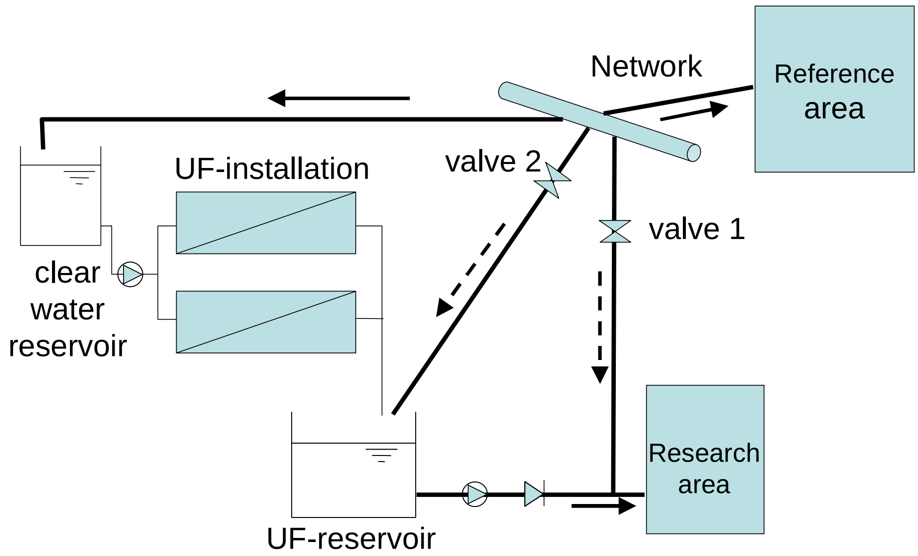


Fig 3-1 Scheme of UF-installation connection. Valve 1 is controlled by the pressure in the Research Area, valve 2 is controlled by the level in the UF-reservoir. The solid arrows indicate the normal flow direction, the dashed arrows the flow in case of failure of the UF-installation or the pump

Prior to commissioning the UF installation both areas were cleaned using a unidirectional flushing program based on a velocity of 1,5 m/s. The effect of the cleaning was checked with an adjusted RPM measuring campaign (Vreeburg et al., 2004a) which also monitored the build-up of a sediment layer.
Initially both areas were cleaned by flushing in the period 22-26 June 2005. The Research Area was again flushed on October 5, 2005, because the UF installation was commissioned on October 24, 2005 after a 4 month delay due to starting-up problems. During the experiment incoming water was analysed with particle counters in two periods of two weeks. oncurrently particle counters were employed in the network to monitor the changes in water quality.
The incoming water was also analysed in one period with the concentration technique Hemoflow (see section 2.4.3).
After a 12-month period both areas were cleaned again in the period 6-14 November 2006. The flushed water was analysed for amount and composition of the removed sediment. A summary of the measuring campaign during and prior to the experiment is given in Table 3-4 at the end of the Materials and Methods section.

3.2.2 The Research and Reference Areas

Fig 3-2 shows the characteristics of the Research Area that was supplied by the particle-free UF-water. The asbestos cement network was mainly constructed in the 1970. In 1974 a new part was constructed of PVC pipes. The network is a conventional network, primarily dimensioned to supply sufficient fire flows and it accommodates 550 connections with on average 2,6 persons per connection and a daily demand of 122 litre (data 2004). Every connection is equipped with a water meter.

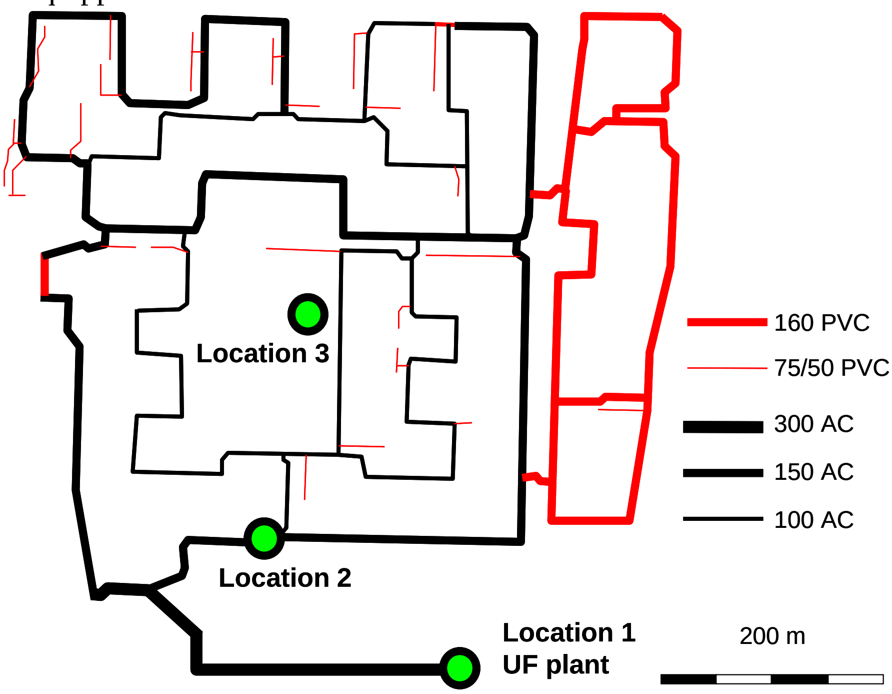


Fig 3-2 Research Area supplied with UF water. The red lines represent PVC pipes, the red lines AC pipes. Diameter indications for PVC pipes are typical outside diameters and AC typical inside diameters. Location 1, 2 and 3 are the particle count monitoring locations

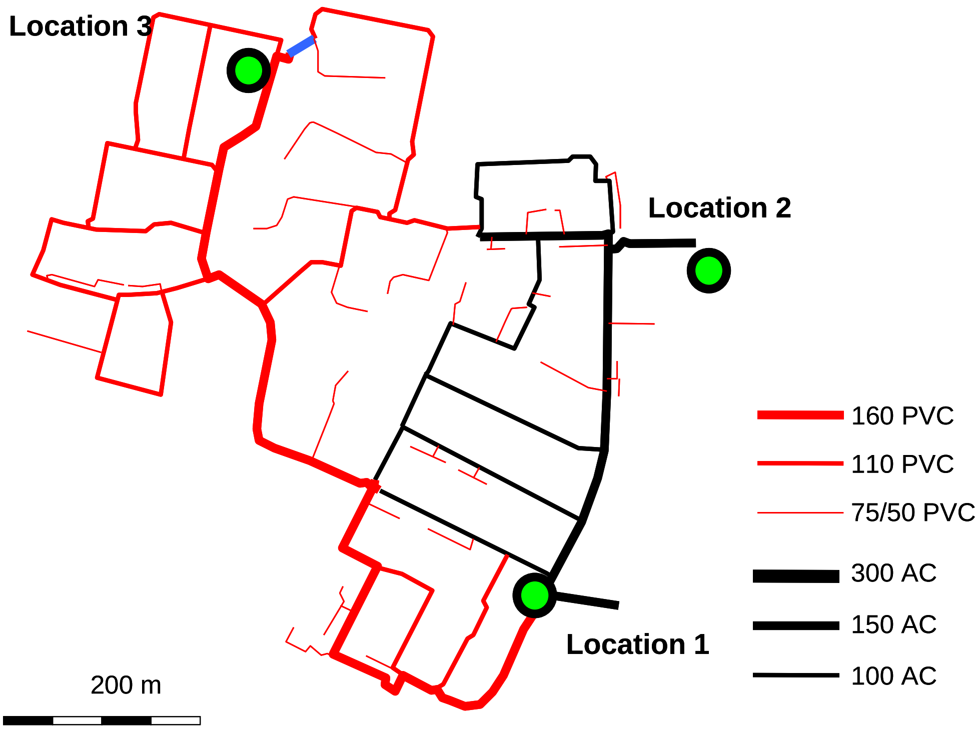


Fig 3-3 Reference Area supplied with drinking water The red lines represent PVC pipes, the red lines AC pipes. Diameter indications for PVC pipes are typical outside diameters and AC typical inside diameters. Location 1, 2 and 3 are the particle count monitoring locations

The dimensions of the network and the material used is a result of the typical company policies in that time. Until the early 1970-ties a lot of Dutch water companies used AC as a standard material for the construction of new networks. For this water company the policy on use of material changed in 1973 and PVC was the new material of choice. The layout of the network is a based primarily on the supply of fire fighting water (see also chapter 4) with consequent low velocities in the network. The Locations 1, 2 and 3 indicate the particle count monitoring locations. Fig 3-3 shows the characteristics of the Reference Area that is supplied with the normal drinking water.

The network in the area was constructed during three periods in the time span from 1968 to 1998. The asbestos cement pipes (black lines) were mainly laid during the period 1968-1969. The PVC extension of the network where measuring location 1 is situated was constructed in 1976 and shows the typical material use and dimensioning of that time. The PVC-part of the network that contains measuring location 3 was constructed in de period 1995 – 1999. This part of network is also a conventional network, primarily dimensioned to supply sufficient fire flows, but has some smaller pipes in the loops in the supply area. The total network accommodates 520 connections with on average 2,7 persons per connection and a daily demand of 118 litres (data 2004). Every connection is equipped with a water meter. The average age of this network and the total building area is younger than that of the Research Area. This is also indicated by the average number of people per connection and the slightly lower per capita per day demand: In a new building area typically young families live, with a larger number of young children that use less water (diaper use versus toilet use) and have more modern water saveing equipment.
Both areas are situated in the city of Franeker in the northern part of the Netherlands and are supplied with water from the treatment plant Spannenburg.

The actual pipe data is summarised in Table 3-1:

Table 3-1 Pipe length and material at Research and Reference Areas

Material	Diameter [mm]	Research Area Length [m]	Reference Area Length [m]
AC	300	84	3
AC	150	2097	611
AC	100	2500	1273
HPE	160		31
PVC	160	31	1174
PVC	110	1408	2291
PVC	75	53	86
PVC	50	1140	1684
Total		7313	7152

3.2.3 The treatment plant

The city of Franeker is supplied by the treatment plant Spannenburg that is located 38 kilometres south of Franeker. The pumping station at the treatment plant is connected to the city of Franeker by a network of mainly AC and PVC mains in which the water has a residence time of roughly 24 hours. The water is pumped to the water tower and further distributed to the local DWDS. The water tower is connected in line which means that the tower functions as a fixed pressure point. The storage capacity is too small to serve as a true balancing reservoir. The average water quality data from Spannenburg are summarised in Table 3-2. The UF installation is also located at the water tower.

Table 3-2 Average water quality data from treatment plant Spannenburg based on obligatory sampling scheme

		Spannenburg			
		n	avg	min	max
Turbidity	FTU	52	0,21	<0,05	1,30
Temperature	ºC	54	12,0	10,0	13,0
pH	-	52	7,55	7,40	7,65
SI	-	52	-0,20	-0,33	-0,04
Dissolved Oxygen	mg/l	52	6,1	4,6	7,6
DIOC	mg/l	4	7,5	7,3	8,1
Aeromonas	n/100 ml	16	10	6	29
Chloride	mg/l	4	30	30	31
Iron	mg/l	12	0,03	0,02	0,08
Manganes	µm/l	12	<10	<10	<10
Aluminium	µm/l	4	<5	<5	<5
Magnesium	mg/l	52	9,4	8,2	10,0
Calcium	mg/l	52	31	25	41

The treatment process is a conventional groundwater treatment that has an initial aeration step and two rapid sand filtration steps separated by a second aeration step and pH and softening. The turbidity of the treated water, measured directly after the clear water pumps, has a certain pattern that is related to the backwash program of the filters, as is shown in Fig 3-4. This pattern of peaks in turbidity is typical for treatment plants with rapid sand filtration as their final treatment step. Depending on the size of the clear water storage facility, the patterns can be more or less recognisable.
The underlying hypothesis on treated water as a main source for particles in the network is that, especially in the relatively short times of increased turbidity, the major loading of the network takes place.

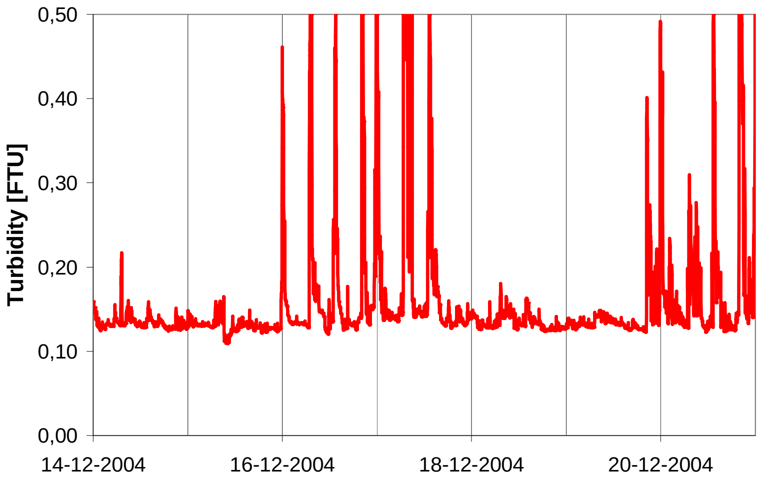


Fig 3-4 Turbidity of treatment plant Spannenburg during several days, measured with Sigrist KT65

3.3 Materials and Methods

3.3.1 General

Several measuring methods were used during the experiment to monitor the build-up of the sediment layer and the loading of both the areas with particles. The particle load to the areas was directly measured by analysing the incoming water on particle content and the effect of the particle load was measured with the Resuspension Potential Method (Vreeburg et al., 2004a), paragraph 2.3). The changes in particle content were measured by concurrent particle measurements at different locations in the area. Finally, the overall effect was analysed by harvesting all the sediments in both areas at the end of the experiment and analysing them. Experimentally a few Hemoflow measurements were performed to get an idea of the composition of the particles. A summary of the application and timing of the different materials and methods is given in Table 3-4.

3.3.2 Particle counters and turbidity

During two periods the incoming water was monitored with three MetOne PCX particle counters. Two particle counters measured in 32 ranges starting at the range 1-2 µm particles with an increment of 1 µm up to a range >31µm. One particle counter was calibrated to start with the range 2-3 µm with increments of 1 µm up to a range >31µm. The monitors were calibrated according to factory specifications and used a measuring frequency of 2 minutes. Monitoring locations for the Research Area are indicated in the map of Fig 3-2. Location 1 is at the pressure side of the pumps right behind the 400 m^3 UF-reservoir (see Fig 3-1).Location 1 and 2 were directly connected to the distribution pipe with dedicated taps. In location 3 the monitor was connected in an inner installation of a primary school.
The monitoring locations for the Reference Area are indicated in Fig 3-3. The monitors at location 1 and 3 were installed in the inner installation of a one-family house. Location 2 was also situated in a primary school. The school locations obviously were closed during weekends and nights and did not have any demand at those times, apart from the flow needed for the equipment, that is estimated at 200 l/h.
During the first measuring period concurrently with the particle counts, the turbidity of the drinking water at the inlet of the UF treatment (at location 1 in Fig 3-2) was measured with a Sigrist KT65 white light turbidimeter.

3.3.3 Resuspension Potential Measurements (RPM)

The Resuspension Potential Method measures the effect of a mild disturbance of the flow velocity in a pipe on the turbidity of the water and is described in section 2.3. The mild disturbance (0,35 m/s) resuspends loose sediments, increasing the turbidity of the water. The standard RPM (Vreeburg et al., 2004a) is, for this application, adjusted to a shorter time of disturbance: 5 minutes instead of 15 minutes, but with the same velocity. The main reason is that the original procedure requires 315 meters of isolated pipe (15 minutes disturbance at 0,35 m/s), which would restrict the number of viable measuring locations to one or two for each area. The adjusted procedure requires 105 meters of isolated pipe, enabling more locations in each area. For each area, six locations are identified that are indicated in Fig 3-5 based on the actual maps of the system. The schematics of the networks are presented in Fig 3-2 and Fig 3-3. The locations are selected considering some practical point. First point is that the use of valves should be limited. Each location can be disturbed by closing one valve and using the hydrant in the neighbourhood. Second criterion is that the locations are spatially apart from each other avoiding that the measuring of one location disturbs the other locations . The RPM influences the measuring location, because with the disturbance a certain amount of sediment is also flushed out. For this reason the RPM was limited analysed to minimise the effect on the locations during the experiment. Three locations were kept undisturbed, thus unmeasured, until the end of the measuring period.

The ranking table for the adjusted RPM is given in Table 3-3. The boundaries for the ranges were chosen low, because we did not expect a high load of sediment to either area. With the visibility threshold for discolouration of 10 FTU (Slaats et al., 2002) at the maximum value it indicates a realistic discolouration risk when the RPM is high.

Table 3-3 Ranking table adjusted RPM: 5 minutes disturbance and measuring on the flushing point with a Dr Lange Ultraturb

Points	0	1	2	3	4
Max during disturbance [FTU]	<1	1 -3	3 – 5	5 – 10	>10
Average during disturbance [FTU]	<1	1 -3	3 – 5	5 – 10	>10
Resettling time [min]	<1	1-5	5-10	10-15	>15

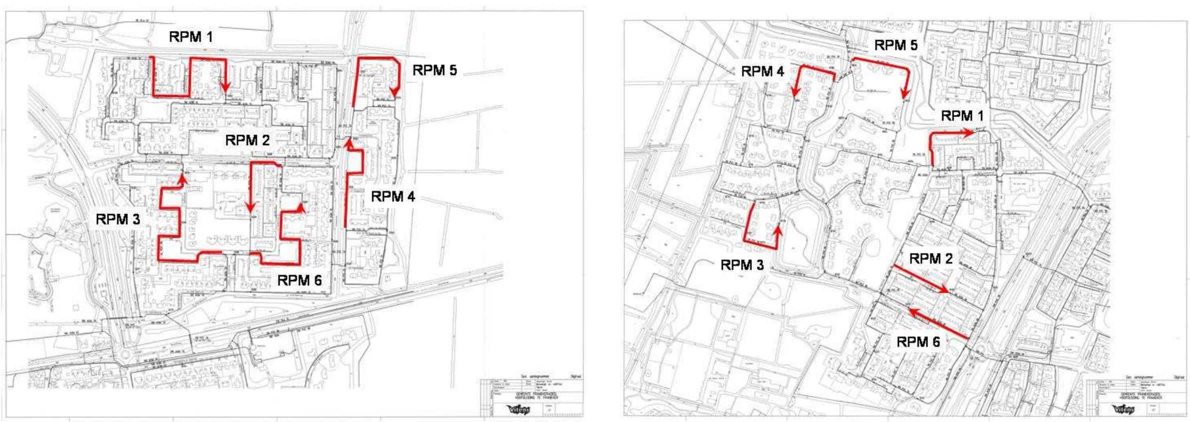


Fig 3-5 RPM locations for the Research Area (left) and the Reference Area (right)

The RPM is measured in three periods:
- The -1 measurement to evaluate the RPM before cleaning the area
- The 0-measurement to evaluate the RPM just after cleaning
- The 1- measurement to evaluate the development of the RPM half way the experiment. This has only been done at three locations (1, 2 and 4) to protect the other locations from disturbance by the RPM itself, as previously mentioned.
- The 2-measurement at the end of the period to evaluate the end result after one year of normal operation of the Reference Area and one-year feeding of UF-water to the Research Area

3.3.4 Total sediment analysis

The sediment that accumulated during the research period was in both areas flushed out using a dedicated unidirectional flushing program (see paragraph 5.3). The layouts for the flushing programs are designed and documented following the principles for water flushing (1,5 m/s flushing velocity, three pipe turnovers and a clear water front). The flow and turbidity of the flushed water is monitored continuously. From each first turnover a sample was taken and analysed for TSS, VSS, Fe and Mn concentrations. The samples were drawn from the first turnover because then almost all of the sediment is removed. A calibration curve of the relationship between turbidity and each of the analysed parameters was made and used to calculate the total volume of deposits and its composition. These calibration curves are specific for this type of water and are different for the different areas.

3.3.5 Hemoflow measurements

The Hemoflow method (section 2.4.3) was experimentally applied in the last stage of the experiment to assess the composition and weight of the particles entering the Reference Area. The heart of the methodology is that a large sample volume is concentrated using a cross-flow membrane filter. The concentrated samples were analysed for TSS, VSS, ATP, Fe and Mn levels.
In the period 2 – 13 November 2006 the Hemoflow measurement was operated and four samples were collected. The sampling time varied between 24 and 72 hours, resulting in sample volumes of 2000 to 6000 litres. To verify the performance of the UF-installation also a sample was taken at the outflow of the installation.
The method is still in development and thus experience is limited. In this particular case the four preconcentrated samples of the drinking water were analysed on total suspended solids by evaporating the complete sample at 105 ^oC. Obviously, this is not the correct way to analyse the total suspended solids, because this should have been done through filtering the sample and weigh the residue according to (APHA, 1998).
The residue that was collected through evaporation also contained the mineral salts that were dissolved in the normal drinking water. The TSS that is calculated from the residue should be corrected for these dissolved minerals. This is done through an ICPMS-scan of the supernatant water of the preconcentrated sample after destruction with nitric acid. The amounts of minerals found are compared with the average mineral level at the treatment plant. If they are in the same order, then the contribution of these minerals is subtracted from the residue resulting in the correct TSS.
The complete residue was analysed for the organic fraction by combusting the sample at 450^oC according to (APHA, 1998).

The last sample from the UF installation was analysed correctly for TSS by filtering and weighing the residue. As could be expected the residue was too little to analyse for VSS.

3.3.6 Summary of measuring activities

The measuring dates and types of measurement during the experiment are summarised in Table 3-4

Table 3-4 Summary of measuring activities

	Research Area						Reference Area					
	Loc 1		Loc 2		Loc 3		Loc 1		Loc 2		Loc 3	
Particle count												
First period	14-3-06 / 30-3-06		21-3-06 / 30-3-06		14-3-06 / 30-3-06		30-3-06 / 10-4-06		30-3-06 / 10-4-06		30-3-06 / 10-4-06	
Second period	12-10-06 / 18-10-06		12-10-06 / 20-10-06		12-10-06 / 20-10-06		30-10-06 / 10-11-06		30-10-06 / 4-11-06		30-10-06 / 10-11-06	
Turbidity							Input UF installation 21-03-06 / 10-4-2006					
Hemoflow	21-11-06 / 27-11-06						2-11-06/3-11-06; 3-11-06/6-11-06; 6-11-06/10-11-06; 10-11-06/13-11-06					
Resuspension Potential Method (RPM)	1	2	3	4	5	6	1	2	3	4	5	6
-1 (2005)	24-6	27-6	24-6	24-6	24-6	24-6	22-6	22-6	22-6	22-6	23-6	23-6
0 (2005)	13-7	13-7	13-7	14-7	14-7	14-7	7-7	7-7	7-7	7-7	8-7	8-7
1 (2006)	14-3	14-3	-	14-3	-	-	14-3	14-3	-	14-3	-	-
2 (2006)	6-11	6-11	6-11	6-11	7-11	7-11	6-11	7-11	7-11	7-11	7-11	7-11
Cleaning period												
Initial	5 October 2005						24 June 2005					
End	9 and 10 November 2006						13 and 14 November 2006					

3.4 *Results*

3.4.1 Particle counters: general

The particle measurements show the particle concentration for various ranges. For this experiment the particle concentrations were interpreted in two ways: the calculated particle volume and the particle-size distribution. The calculated particle volume gives an indication of the amount of sediment or suspended particles in the incoming water. Simultaneous measurements in the DWDS showed how the calculated particle volume changed and what, presumably, was left in the pipes. The particle-size distribution indicates how the properties of the particles can change during the transport and distribution. (see also section 2.2)
The measuring data is presented ‘as is’ and no smoothening in the form of a running average has been applied. In the cumulative distribution curves the data is processed to smoothen the data more and draw conclusions from that.

3.4.2 Particle counters: particle volumes

The concentration of particles in the different ranges was used to calculate the volume of the particles, assuming that the particles have a spherical shape with a diameter equal to the linear average value of the borders of the range. Obviously, the total particle volume concentration of the water feeding the Reference Area is higher than the that of the UF-water that feeds the Research Area. To illustrate this, for two days during the first measuring period the particle volumes of the drinking water at location 1 in the Reference Area and the UF-water at location 1 in the Research Area were calculated and then graphically presented as shown in Fig 3-6. The unit is parts per billion volume (ppb: 10^{-9} m^3/ m^3). The measurements were not simultaneous but gave a good impression of the difference in particle load on the system.

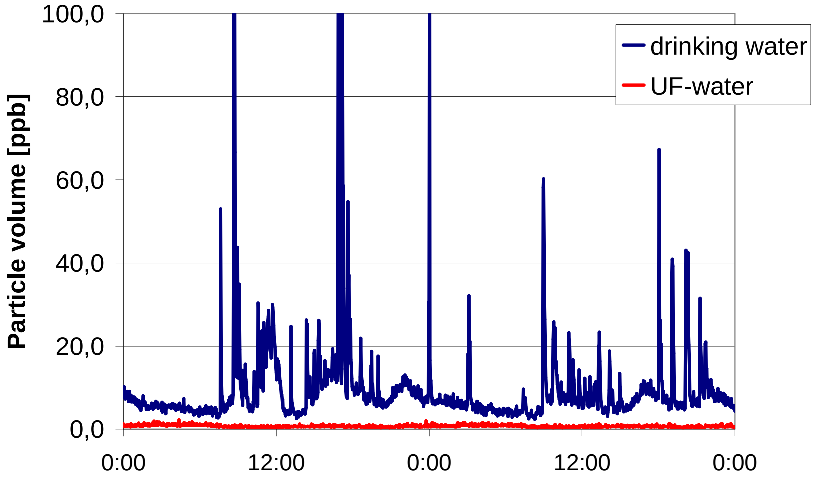


Fig 3-6 Particle volumes of drinking water and UF-water during two representative days in the first measuring period (March-April 2006)

The particle volume trace does not match the turbidity trace from the treatment plant as is presented in Fig 3-4. The distance between the treatment location and the entrance of the Reference Area is about 38 kilometres. During the transport the characteristics have changed in such a way that the regular pattern has disappeared. According to the stylistics of section 2.2.2 there is a resuspension type of trace at the Reference Area with peaks during the high demand hours and low values during the low demand hours.
For each measuring period a typical period with valid data was selected from the total measuring period as mentioned in Table 3-4 and presented with two graphs and one table. The first graph represents the total calculated particle volume concentration in ppb. The second graph is part of the frequency distribution of the calculated particle volume concentration and the table is a summary of various frequency percentiles, the average particle volume concentration and the parameters Surf-90% and Surf+90% (see paragraph 2.2.5).
The graphs for the Research Area and the Reference Area are on a different scale, because the particle load on the Research Area is less than that on the Reference Area (Fig 3-6). The scales differ by a factor of 10, both on the first graph for the calculated particle volume and on the frequency distribution graph. To characterise the variation of the calculated particle volumes the ratio between the 90 percentile and the 99,5 percentile is calculated. When this ratio approaches one the variation in the calculated particle volume is very low, meaning that the shape of the frequency distribution is steep. When the ratio approaches zero, the variation is large and the slope of the frequency distribution is shallow.
Another parameter to characterise the irregularities or spikes in the traces the Surf-90% and the Surf+90% are used as are explained in section 2.2.3. When the Surf+90% exceeds 20% the pattern has some spikes in it, indicating that 20% of the overall average is contributed in 10% of the time.
For calculation of the particle volume, the particle-diameter range 1-2 µm is not used, as only two of the three particle counters used were capable of measuring this specific range.

The Research Area

The results of the first measuring period in the Research Area are given in Fig 3-7 and Table 3-5 for Location 1 and 2. the monitor at the third location malfunctioned during the complete period and the results were disregarded. The calculated particle volume at both locations are very similar as can be seen from Fig 3-7: the traces are overlapping and the cumulative distribution curves are almost identical indicating there is little difference between the two locations.

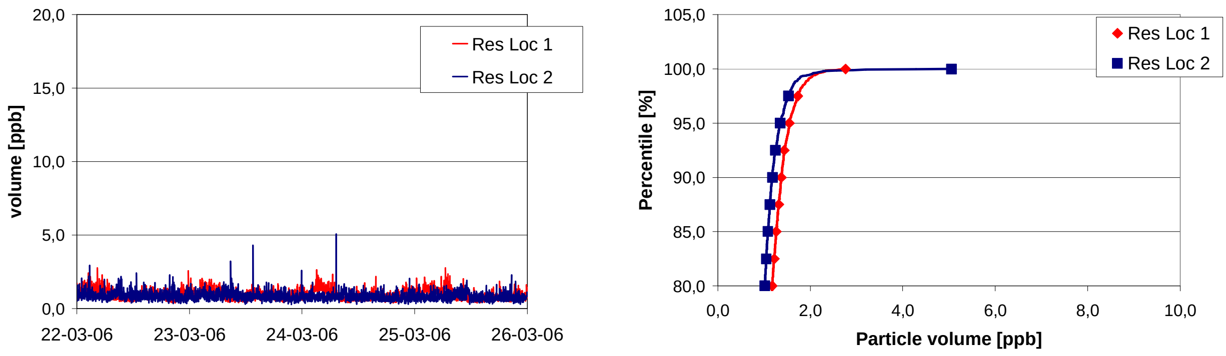


Fig 3-7 Calculated particle volumes in Research Area and cumulative frequency distribution during first measuring period. Curves are almost identical, indicating a stable situation

Table 3-5 Frequency percentiles Research Area 22-03-2006 to 26-03-2006

Frequency percentile [%]	22-3-2006 to 26-3-2006 Res Loc 1 [ppb]	Res Loc 2 [ppb]
90,0	1,37	1,18
95,0	1,55	1,35
98,0	1,78	1,57
99,0	1,95	1,73
99,5	2,12	1,99
99,9	2,55	2,92
Ratio 90/99,5	0,65	0,59
Average [ppb]	0,92	0,83
Surf -90 [%]	82,3%	82,5%
Surf +90 [%]	17,7%	17,4%

The second measuring period (12 to 20 October 2006) in the Research Area is summarised in Fig 3-8 and Table 3-6. Three locations could be monitored of which the first two are similar to the first period. In the third location an deviating pattern can be recognised.

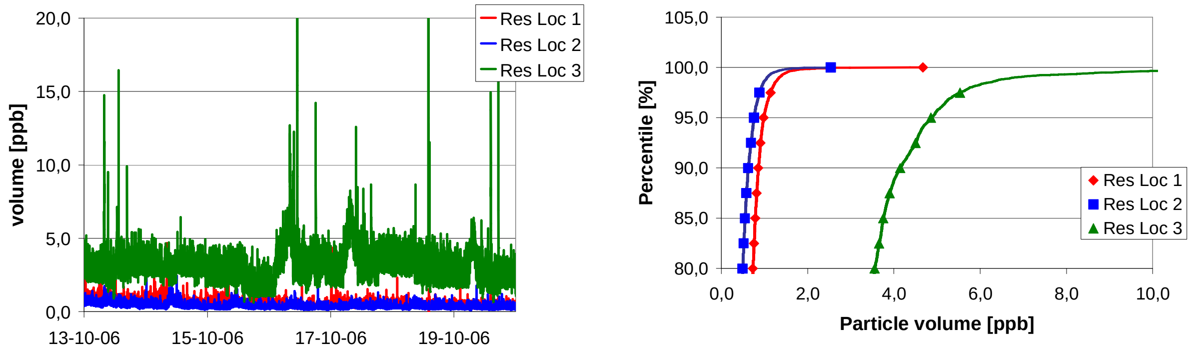


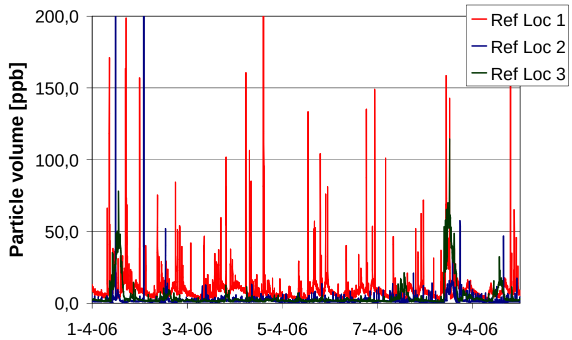
Fig 3-8 Calculated particle volumes in Research Area and cumulative frequency distribution during second measuring period

Table 3-6 Frequency percentiles, Research Area 13-10-2006 to 20-10-2006

	13-10-2006 to 20-10-2006		
Frequency percentile [%]	Res Loc 1 [ppb]	Res Loc 2 [ppb]	Res Loc 3 [ppb]
90,0	0,85	0,62	4,15
95,0	0,98	0,75	4,86
98,0	1,18	0,92	5,76
99,0	1,33	1,05	6,78
99,5	1,51	1,21	8,90
99,9	2,17	1,62	16,39
Ratio 90/99,5	0,56	0,51	0,47
Average [ppb]	0,55	0,38	2,93
Surf -90 [%]	80,7%	78,7%	80,9%
Surf +90 [%]	19,3%	21,3%	19,1%

The calculated particle volume loading on the Research Area was not zero, which theoretically could be expected. Though the UF-reservoir was cleaned before it was used for storage of the UF-water, it could have some sediment residuals that were resuspended during the research period. The frequency distribution of both periods was almost identical for the first two locations, which indicates that the UF-installation functioned well. During the course of the experiment the particle load picked up from the reservoir decreased as can be seen from the percentile values that are lower at location 1 in the second period. The ratio between the 90% percentile and the 99,5% percentile were for the first period higher than for the second period, indicating that there was less variation. The absolute values in the second period were lower, indicating a lower calculated particle load overall. For the third location the ratio is lower, indicating that more variation occurs, which is confirmed by the pattern on the left of Fig 3-8. In this graph more peaks occur during the daytime. During the weekend of 14 and 15 October 2006 there were no peaks and all the other peaks occurred during daytime. This indicates that the peaks were caused by an increased velocity in the inner installation which indicates that resuspendable sediment is present either in the installation itself or in the distribution pipe that the installation is connected to.

The Reference Area
The results of the first measuring period in the Reference Area are presented in Fig 3-9 and Table 3-7. All monitors were functional so for all three locations relevant data is available.



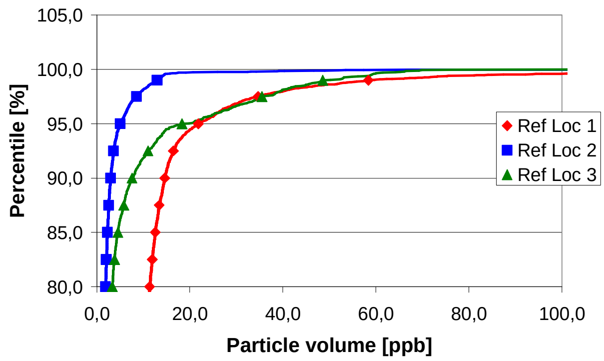
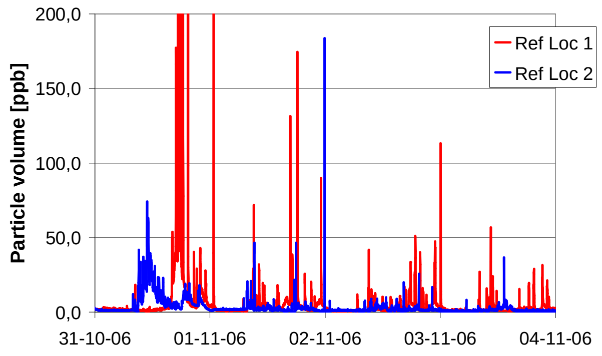


Fig 3-9 Calculated particle volumes in Reference Area and frequency distribution during first measuring period

Table 3-7 Frequency percentiles Reference Area 1-4-2006 to 10-4-2006

	1-4-2006 to 10-4-2006		
Frequency percentile [%]	Ref Loc 1 [ppb]	Ref Loc 2 [ppb]	Ref Loc 3 [ppb]
90,0	14,57	2,95	7,55
95,0	21,80	5,00	18,32
98,0	40,18	9,56	39,45
99,0	58,39	12,94	48,60
99,5	84,15	14,42	59,09
99,9	160,58	49,01	69,25
Ratio 90/99,5	0,17	0,20	0,13
Average [ppb]	9,85	1,98	4,49
Surf -90 [%]	66,4%	56,6%	43,8%
Surf +90 [%]	33,6%	43,4%	54,9%

The results of the second measuring period in the Reference Area are presented in Fig 3-10 and Table 3-8 and are limited to two locations, again because of malfunctioning of the particle counter at location 3



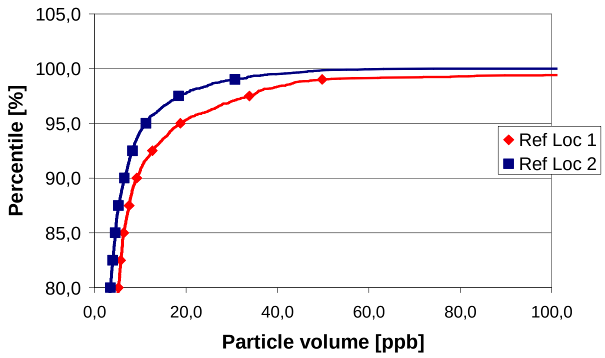


Fig 3-10 Calculated particle volumes in Reference Area and frequency distribution during the second measuring period

Table 3-8 Frequency percentiles Reference Area 31-10-2006 to 4-11-2006

Frequency percentile [%]	31-10-2006 to 4-11-2006 Ref Loc 1 [ppb]	Ref Loc 2 [ppb]
90,0	9,25	6,53
95,0	18,77	11,25
98,0	36,26	20,94
99,0	49,77	30,71
99,5	117,97	39,57
99,9	307,70	55,12
ratio 90/99,5	0,08	0,16
average [ppb]	5,77	3,11
surf -90 [%]	38,2%	49,2%
Surf +90 [%]	61,8%	50,4%

In both measuring periods in the Reference Area, the frequency distribution of the particle volume for location 2 was steeper than for location 1. Also, the percentile values were lower (Table 3-7 and Table 3-8). This indicates that the particle volume between the two locations decreased and also that the peaks were lower. The third location measured during the first period, shows a steeper frequency distribution than the first location, but also has higher values. The 90/99,5 ratio is low indicating more variation in values. Overall, the calculated particle volume at the entrance of the Reference Area was highest and had the most peaks. Towards the second location a settling of particles occurs that is seen in the lower values of the calculated particle volume and the 90/99,5 percentile ratio. Towards the third location more influence of resuspension is observed in the lower 90/99,5 percentile ratio.

The average load to the Reference Area was a factor of 10 higher in particle volume during both the periods. The distribution of the load over time was smoother in the Research Area than in the Reference Area. In the Reference Area during the first period, 33,6% of the load was delivered in 10% of the time. The Surf+90% in the Reference Area has values in the range 34-62% and was much higher than in the Research Area with values in the range 17-22%. This indicates that a large part of the particle load to the Reference Area was in the peaks: in 10% of the time, 34 to 62% of the load was supplied. This is, however, not a quantitative determination as the volume flow data are lacking. If high peaks coincide with high volumes, the actual mass load would even be higher.

3.4.3 Particle counters: Particle size distribution

Particle-size distribution for momentary samples shows how the number of particles is distributed over the different particle sizes. This gives added information on how the calculated particle volume is built up. A certain volume can for instance consist of many small particles or a few larger particles.

Fig 3-11 gives the particle distribution for samples in the 98-percentile value at the three measuring locations(see Fig 3-2) for the measuring locations in the Research Area. Note that the axes have a logarithmic scale.

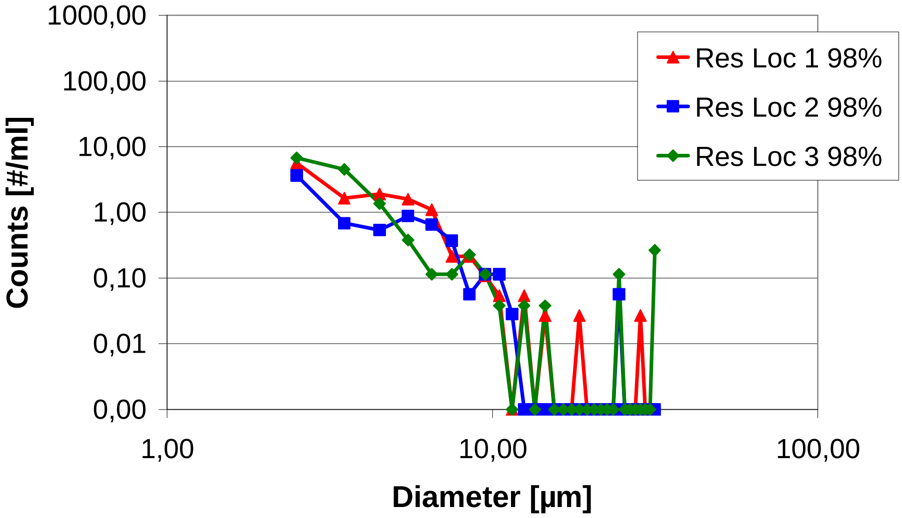


Fig 3-11 Particle distribution in Research Area 98% values period 13-10-2006 / 20-10 2006. According to the low variability in particle volume the lower percentile values are not so different.

The same graph is made for the 98-percentile and the 25-percentile samples at the measuring locations in the Reference Area (Fig 3-12).

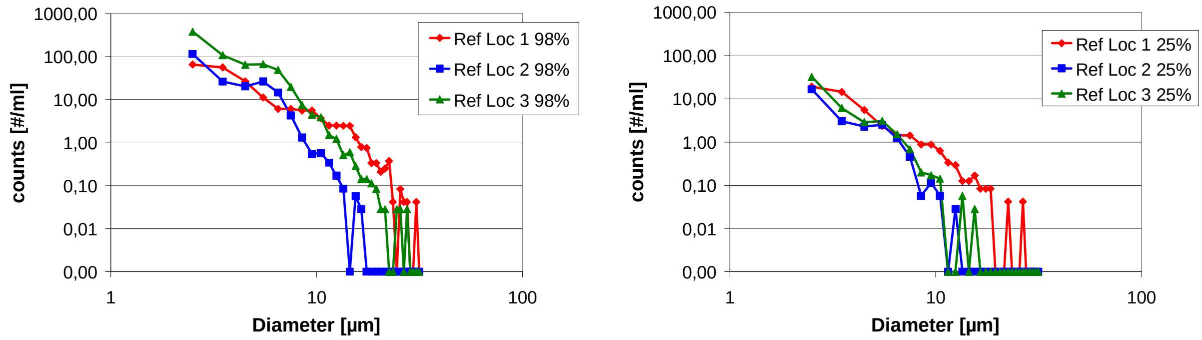
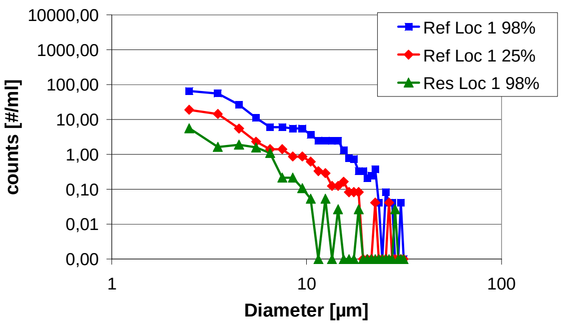


Fig 3-12 Particle distribution in Reference Area: left the high counts at the 98% percentile value and right the low counts at the 25% percentile value.

There is an order of magnitude difference between the 98 percentile and 25 percentile counts in the Reference Area that, in its turn, differs again an order of magnitude with the counts in the Research Area.

The graph with the 25-percentile samples in the Reference Area (right graph in Fig 3-12) shows that, at location 1, more large particles were present that disappear in the network towards locations 2 and 3. This is consistent with the calculations over longer periods, as is shown in Table 3-7 and Table 3-8, which show a decline in calculated particle volume from location 1 to location 2 and a more moderate increase towards location 3. The decrease in the amount of larger particles from location 1 to the other locations occurred also in the 98-percentile samples.

The effect of the UF treatment on the particle distribution is illustrated in Fig 3-13, in which the distribution of the 98-percentile sample in location 1 from the Research Area is compared with the 98- and 25-percentile samples at location 1 in the Reference Area.



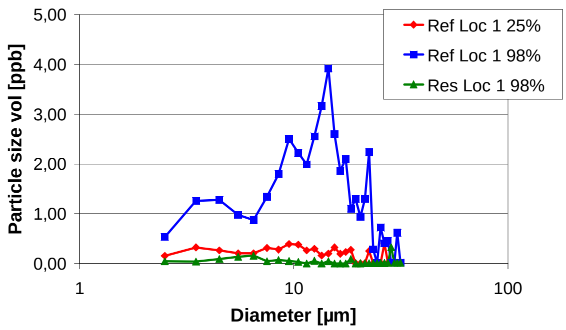


Fig 3-13 Particle distribution of drinking water (location 1 Reference Area) and UF-water after UF reservoir (location 1 Research Area) and volume calculation per diameter size.

The contribution of the diameter-size ranges to the total volume shows that the difference between the 25-percentile Reference Area sample and the 98-percentile Research Area sample is not very large. The influence of the very small number of larger particles is not very large. The volume of the particles in the 98-percentile Reference Area sample is dominantly determined by particles in the mid-size range from 7 to 23 µm.
The particle volume in the 98-percentile sample of the UF installation is probably picked up in the UF-reservoir. The total volume of the particles in the samples is given in Table 3-9.

Table 3-9 Volume particles typical samples drinking water and UF water

	volume particles [ppb]
Ref Loc 1 98%	40,41
Ref Loc 1 25%	5,23
Res Loc 1 98%	1,30

The UF-treatment cuts down the total volume of the particles considerably, which could be expected, and also shows in the values presented in earlier tables. The particle volume from 'peak-events' that are typically in the high percentile samples may differ considerably up to a factor of more than 30, but also in the normal range the volume is cut by a factor of 4. This sustains the hypothesis that most of the particle load originates from incidents, either leading to high particle volume at the treatment location or by resuspended particles that have already accumulated in the network.

3.4.4 Resuspension Potential Measurements

The results of the ranking of the RPM for both the areas are given in Table 3-10.

Table 3-10 RPM measuring results. In every column per measurement the rankings for the three categories are given and the total. Each measurement is characterised with the average of all the measurements

Research area	-1 measure				0-measure				1-measure				2-measure			
Date	24-6-2005				13-7-2005				14-3-2006				6-11-2006			
location				tot				tot				tot				tot
Res loc 1	4	4	3	11	1	1	1	3	2	1	0	3	3	3	2	8
Res loc 2	4	4	4	12	1	1	1	3	4	3	0	7	2	1	0	3
Res loc 3	4	4	3	11	1	0	1	2					2	1	0	3
Res loc 4	4	4	2	10	0	0	0	0	1	1	0	2	1	1	0	2
Res loc 5	4	4	3	11	2	1	2	5					1	1	0	2
Res loc 6	4	4	2	10	3	1	1	5					4	2	0	6
Average				10,83				3,00				4,00				4,00
Standard deviation				0,75				1,9				2,65				2,449

Reference area	-1 measure				0-measure				1-measure				2-measure			
Date	22-6-2005				7-7-2005				14-3-2006				6-11-2006			
location				tot				tot				tot				tot
Ref loc 1	3	2	2	7	1	1	1	3	0	0	0	0	0	0	0	0
Ref loc 2	4	4	3	11	2	1	1	4	4	4	2	10	4	4	3	11
Ref loc 3	4	4	4	12	2	1	1	4					4	4	2	10
Ref loc 4	3	3	1	7	4	3	1	8	3	1	0	4	4	4	3	11
Ref loc 5	4	3	3	10	0	0	0	0					4	4	2	10
Ref loc 6	4	3	1	8	0	0	0	0					4	4	3	11
Average				9,17				3,17				4,67				8,83
Standard deviation				2,14				2,99				5,03				4,355

The results of the -1 measurement show that both areas had a relatively high RPM that was evenly spread over the whole area, considering the standard deviation. Considering that the visibility threshold for turbidity is 10 FTU (Slaats 2002), almost all the locations had a realistic discolouration risk. After the initial cleaning, not all the locations had a clear RPM. The average, however, dropped for both areas to the same level, so the starting point for both areas was equal. The standard deviation for the Reference Area is somewhat higher that that of the Research Area.

The ranking table for this RPM experiment is adjusted in this way that the original situation is almost the maximum score, which is in this case the original -1 level.

The RPM's after a year give the clearest picture of the development of the discolouration risk. In the Research Area the increase in the RPM from right after the initial cleaning was only 1 point with a standard deviation that is constant for both measuring periods. It can be concluded that the accumulation of mobile sediment was of minor importance and the discolouration risk stayed almost the same during this period. For the Reference Area one location (location 1) had a very low RPM, which has no obvious explanation but caused a relatively high standard deviation in the results. All the other locations are more or less back on the same RPM as the -1 measurement. This means that within approximately one-and-a-half years, the network was recharged with particles to the same realistic discolouration level. The results of the RPM measurements are shown graphically in Fig 3-14 with the averages of the RPM.

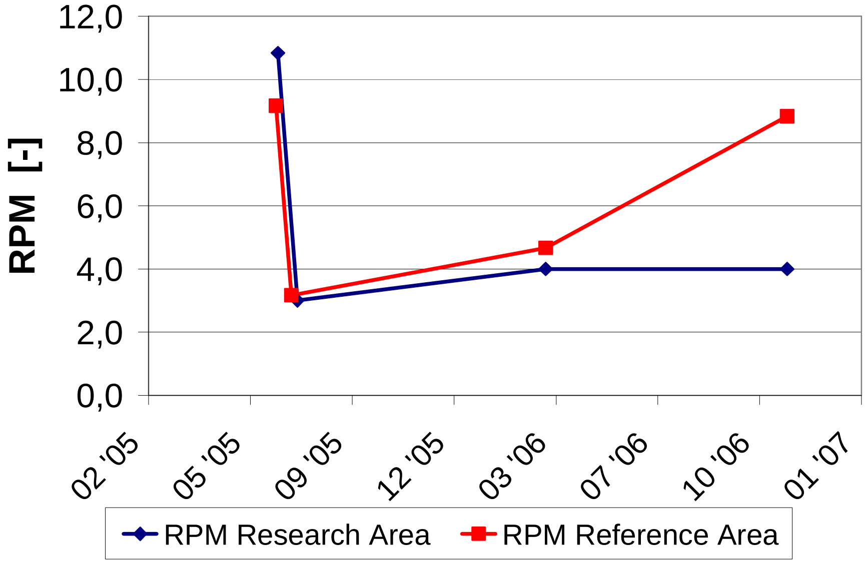


Fig 3-14 Average RPM in the Research and the Reference Area during the whole research period, based on the data from Table 3-10

The standard deviations for each measuring point are graphically presented in Fig 3-15. The large standard deviation in the Reference Area is mainly caused by the very low values of RPM-location 1, that cannot be explained with the available data. Ignoring this location entirely makes the image clearer, but doesn't change the general trends. The average in the end situation is even higher as in the -1 situation and the standard deviation decreases to 0,6, making the distinction with the value in the Research Area very significant.

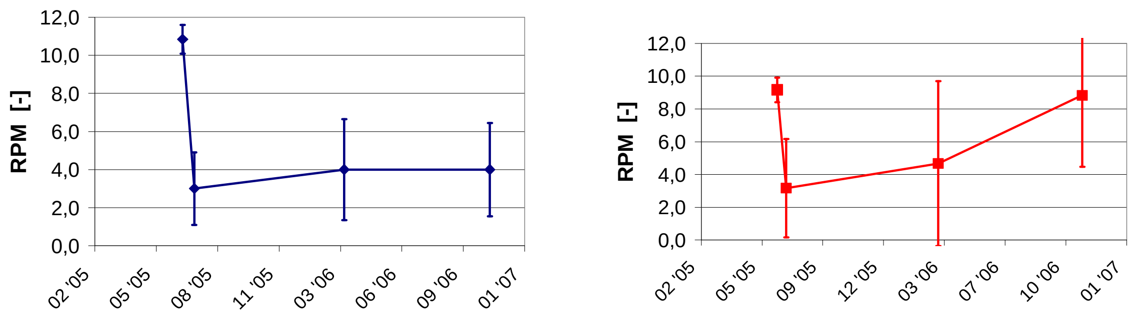


Fig 3-15 Average RPM with the error bars indicating the standard deviation. Left the Research Area and right the Reference Area, both based on the data tabulated in Table 3-10.

The shape of line with average RPM's confirms the hypothesis that the RPM was largely improved (decreased) as a result of cleaning and that the recharging with sediment that would increase the RPM again was less in the Research Area than in the Reference Area. This hypothesis was graphically shown in Fig 1-8 in section 1.6. Taking into account the standard deviation in the results, the significance of the conclusion still holds true, though there is an overlap in results. From the average data on RPM and the standard deviation it can be concluded that the increase in the RPM in the Research Area has come to a stop what would mean that the accumulation of particles was not measurable with the RPM.

The cleaning frequency for the Reference Area would be, based on these data, once every 1,5 tot 2 years, while the cleaning frequency for the Research Area would be indefinite. For the Research Area the average RPM stays constant and the standard deviation is declining.

3.4.5 Total sediment analysis

After the year period both networks were cleaned with a unidirectional dedicated flushing program. The flushed water was monitored for volume and turbidity. Next to that samples were taken from the first pipe turnover to analyse TSS, VSS, Fe, and Mn levels. The parameters turbidity and TSS were interrelated as is shown in Fig 3-16. The relations with other parameters were not made in the analysis.
The cleaning that was done prior to the experiment was not analysed in this way.

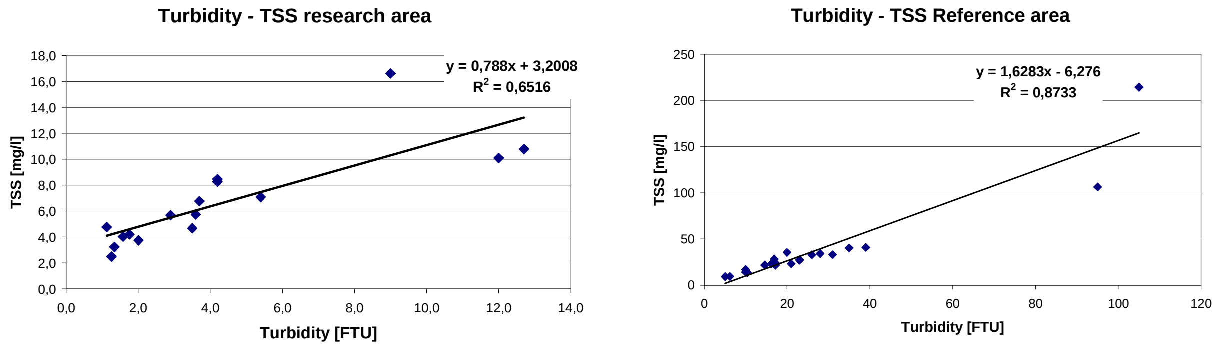


Fig 3-16 Relation Turbidity – TSS for in samples of the flushed water

The relations presented in Fig 3-16 were used to transform the turbidity of the flushed water into suspended solids. With the measured volume flow the absolute amount of suspended solids removed with the flushing can be detrmined.
The relation FTU-TSS was different for both areas, and in the Research Area the relationship was not as strong as in the Reference Area: R^2 0,652 versus R^2 = 0,873. The turbidity-TSS relationships were different for the sediments found in the Research Area and the Reference Area which was demonstrated in the slope of the lines: 0,788 versus 1,628. Sediment in the Research Area has less light scattering characteristics and may be of a different composition.

All the pipes with a diameter larger than 75 mm were flushed and the results of the flow measurements calculated to the total removed amount of TSS and the relative amount of TSS per meter of pipe. The results are summarised for the Research Area in Table 3-11 and for the Reference Area in Table 3-12. The layout of the flushing program is presented in Fig 3-17 and Fig 3-18.

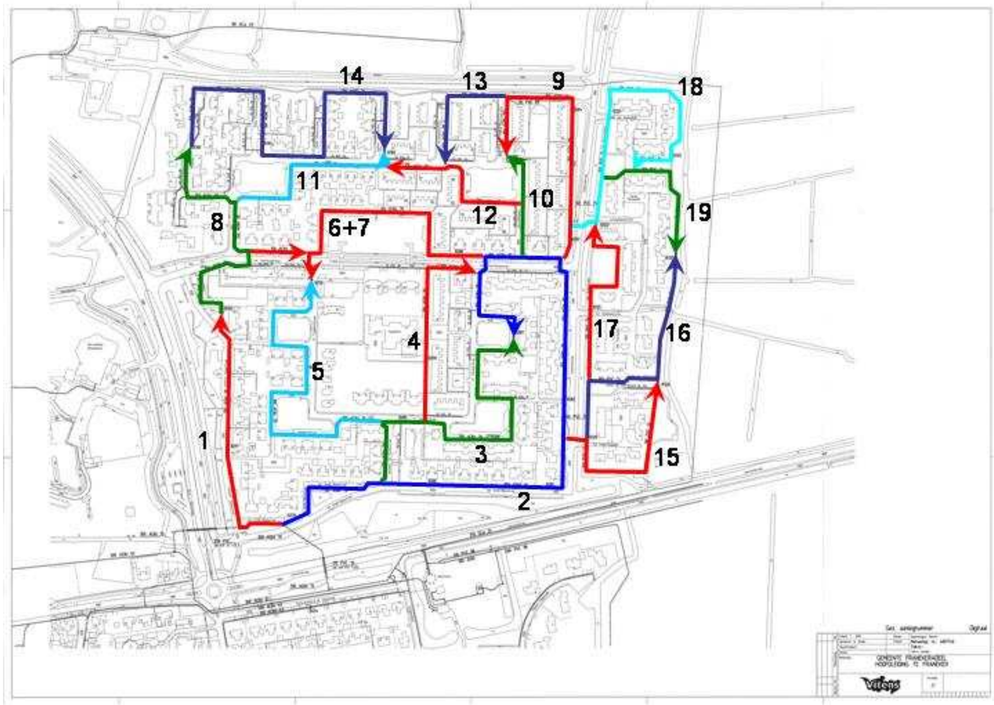


Fig 3-17 Layout of flushing program Research Area. The actions are performed in the numbered order. The smaller diameter pipes (less than 90mm) are not flushed.

Table 3-11 Analysis data from flushing Research Area

action nr	diam [mm]	length [m]	material	Velocity [m/s]	removed turbidity volume [ftu*m3]	removed TSS [gr]	removed TSS per meter [mg/m]
action 1	150,0	350	AC	1,26	41,74	36,09	103,1
action 2	150,0	850	AC	1,40	126,86	103,16	121,4
action 3	100,0	500	AC	2,12	33,42	29,54	59,1
action 5	100,0	500	AC	2,12	15,70	15,57	31,1
action 6 + 7	150,0	450	AC	1,41	66,77	55,81	124,0
action 8	150,0	350	AC	1,41	39,42	34,26	97,9
action 9	150,0	340	AC	1,41	4,94	7,09	20,9
action 11	100,0	240	AC	2,12	53,62	45,45	189,4
action 12	100,0	200	AC	2,12	5,28	7,36	36,8
action 13	100,0	265	AC	2,12	5,02	7,16	27,0
action 14	100,0	500	AC	2,48	39,67	34,46	68,9
action 15	147,6	400	PVC	1,46	116,79	95,23	238,1
action 16	147,6	150	PVC	1,46	12,00	12,66	84,4
action 18	147,6	575	PVC	1,46	32,54	28,85	50,2
action 19	147,6	170	PVC	1,46	11,65	12,38	72,8

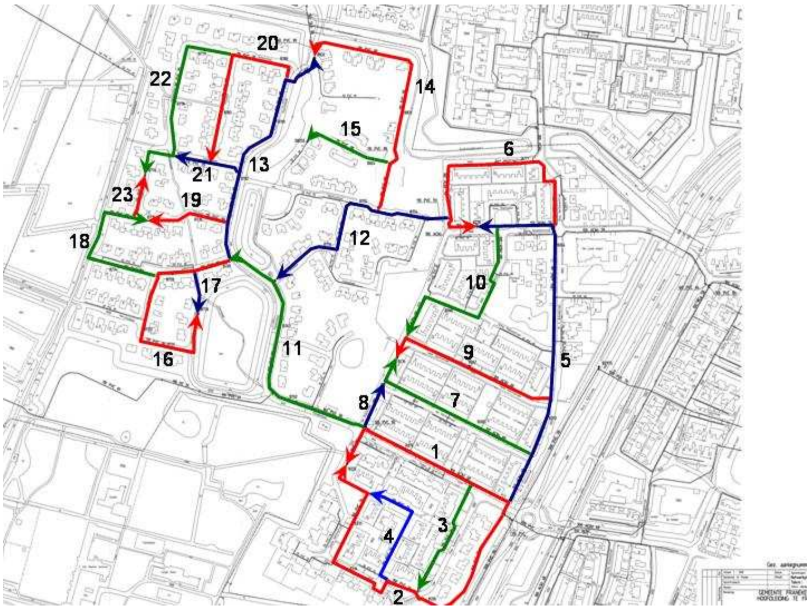


Fig 3-18 Lay out of flushing program for Reference Area. The actions are performed in the numbered order. The smaller diameter pipes (less than 90mm) are not flushed.

Table 3-12 Analysis data flushing Reference Area

action nr	diam [mm]	Length [m]	material	velocity [m/s]	removed turbidity volume [ftu*m3]	removed TSS [gr]	removed TSS per meter [mg/m]
action 1	122,5	520	AC + PVC	1,65	617,83	999,74	1922,6
action 2	147,6	400	PVC	1,14	558,79	903,60	2259,0
action 3	101,6	160	PVC	2,06	73,33	113,13	707,1
action 4	101,6	140	PVC	2,06	69,18	106,36	759,7
action 5	150,0	520	AC	1,38	390,24	629,16	1209,9
action 6	100,0	350	AC	2,12	187,75	299,44	855,5
action 7	100,0	240	AC	2,12	458,14	739,71	3082,1
action 8	100,0	64	AC	2,12	5,24	2,26	35,3
action 9	100,0	240	AC	2,12	75,32	116,36	484,8
action 10	100,0	280	AC	2,12	53,43	80,72	288,3
action 11	147,6	400	PVC	0,97	180,62	287,83	719,6
action 12	101,6	400	PVC	2,06	168,61	268,27	670,7
action 13	147,6	320	PVC	0,97	279,18	448,32	1401,0
action 14	101,6	330	PVC	2,06	210,88	337,09	1021,5
action 15	67,8	330	PVC	2,31	67,61	103,81	314,6
action 16	101,6	330	PVC	1,03	33,12	47,65	144,4
action 17	101,6	36	PVC	1,03	42,71	63,27	1757,4
action 18	101,6	205	PVC	2,06	28,50	40,14	195,8
action 19	101,6	105	PVC	2,06	105,60	165,68	1577,9

At actions 20-23 the turbidimeter was malfunctioning

The totals for both areas are tabulated in Table 3-13.

Table 3-13 Summary of sediment data for flushing Research and Reference Area

	Total length flushed [m]	removed TSS [gr]	Removed TSS per meter [mg/m]
Research	5840	525,08	89,9
Reference	5370	5752,52	1071,2

For the Research Area supplied with UF-water, the relative amount of sediment removed was 89,9 mgr/m and for the Reference Area this was 1071,2 mgr/m. From the pipes in the Reference Area almost 12 times more sediment was removed than from the Research Area. Given all the possible errors in the measurements this can be considered as a significant difference.

The samples have also been analysed for Fe, Mn and VSS. To calculate the balance of the samples Fe is assumed to be present as Fe(OOH) and Mn as MnO_2. Fig 3-19 and Fig 3-20 represent both the relative composition (left chart) and the absolute composition of the sediment.

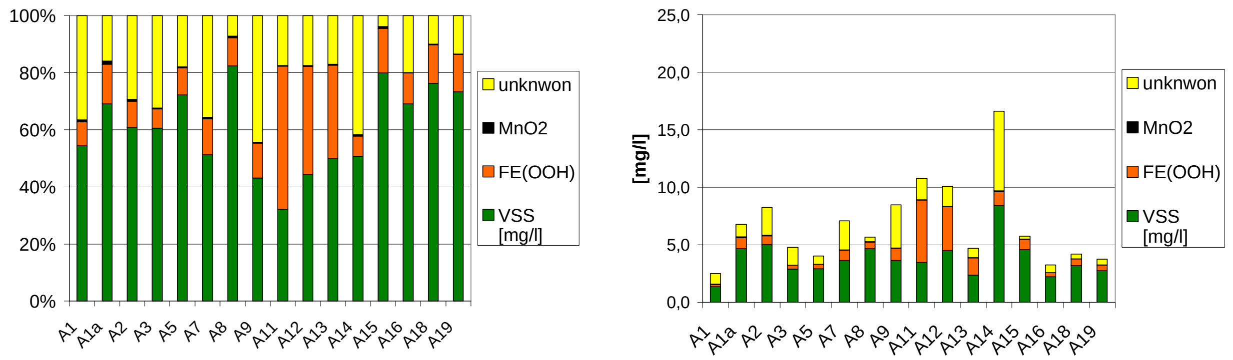


Fig 3-19 Composition of flush samples from Research Area: left the relative compostion and right the absolute composition

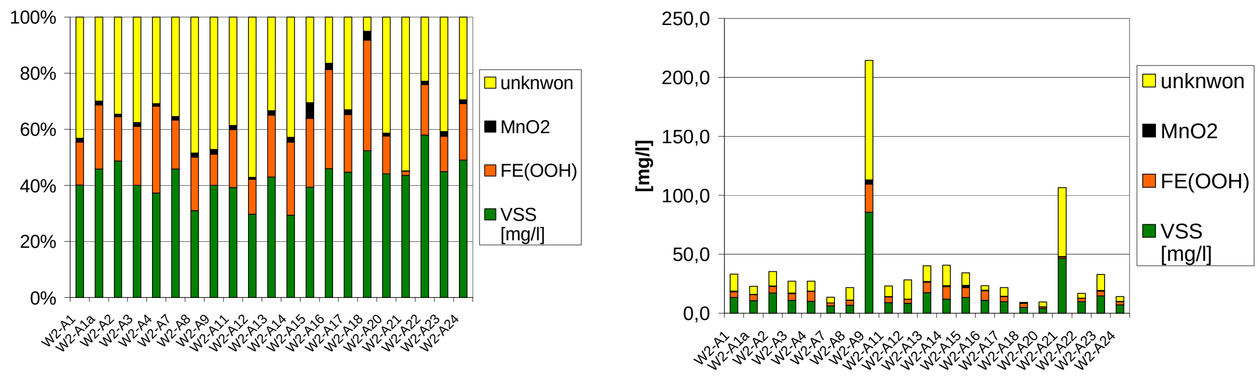


Fig 3-20 Composition of flush samples from Reference Area

The data on the relative composition is tabulated in Table 3-14 and graphically presented in Fig 3-21. The average VSS level of the samples from the Research Area was 60,6%, while the average VSS level of the samples from the Reference Area was 42,5%. The TSS-Turbidity relations also differed between both areas, especially expressed in the slopes of the lines in

Fig 3-16. Taken into account the standard deviation these results suggest that the sediment in the Research Area is of a more organic nature and could be formed by a biological process in the Research Area.

Table 3-14 Relative composition of sediments

	Research Area		Reference Area	
	Average [% of TSS]	St Dev (n=16)	Average [% of TSS]	St Dev (n=21)
VSS	60,6	14,7	42,5	7,1
FE(OOH)	16,4	12,5	20,0	8,5
MnO2	0,5	0,3	1,6	1,1
Unknown	22,5	12,5	35,9	12,1

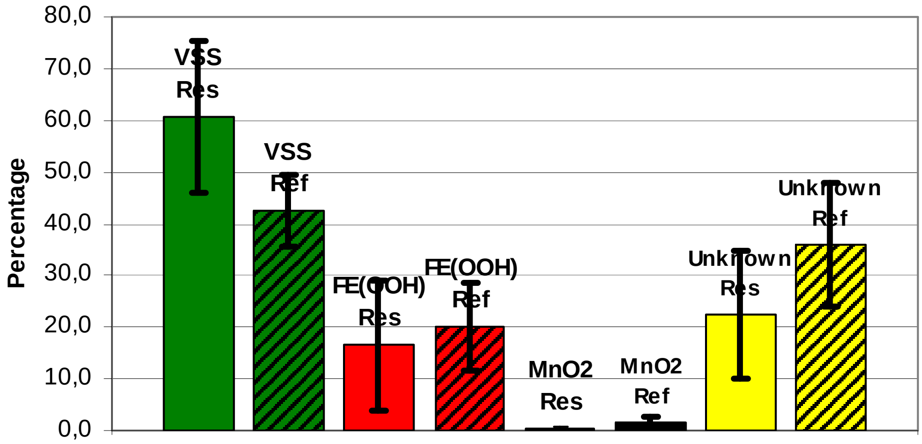


Fig 3-21 Relative composition of samples taken during flushing of the Research and the Reference Area. The error bars indicate the standard deviation interval.

3.4.6 Hemoflow measurements

Analyses of the pre-concentrated Hemoflow samples are shown in Table 3-15. The correction is based on the ICPMS scan that analysed all minerals (
Table 3-16). Based on that the correction for the TSS-analysis is determined.

Table 3-15 Summary of Hemoflow samples

Sample	Filter volume [l]	Sample volume [l]	Damp rest [mg]	Corr [mg]	TSS absolute [mg]	VSS Absolute [mg]	TSS [µg/l]	VSS [µg/l]	VSS %
2/11/2006 - 3/11/2006	1.926	0,910	359,45	243,99	115,46	43,17	59,95	22,42	37,39
3/11/2006 - 6/11/2006	3.613	0,695	302,33	208,76	93,56	20,61	25,90	5,71	22,03
6/11/2006 - 10/11/2006	1.942	0,960	364,80	274,87	89,93	19,34	46,31	9,96	21,50
10/11/2006 - 13/11/2006	6.897	0,645	364,43	194,06	170,36	44,49	24,70	6,45	26,11
Total	14.378				469,31	127,61	32,64	8,88	27,2
	UF-water								
21/11/06 - 27/11/06	*2872*	*0,815*			*1*[1]		*1,2*		

Table 3-16 Correction for solubles in TSS

Sample	Ca org [mg/l]	sample [mg/l]	corr [mg/l]	Mg org [mg/l]	sample [mg/l]	corr [mg/l]	Na org [mg/l]	sample [mg/l]	corr [mg/l]	K org [mg/l]	sample [mg/l]	corr [mg/l]	tot corr [mg/l]
2/11/2006 - 3/11/2006	33,0	26,7	66,6	9,7	8,2	20,2	74,0	72,3	177,8	2,0	1,4	3,5	268,1
3/11/2006 - 6/11/2006	33,0	27,8	69,3	9,7	9,1	22,4	74,0	83,0	204,3	2,0	1,7	4,3	300,4
6/11/2006 - 10/11/2006	33,0	25,8	64,3	9,7	8,4	20,7	74,0	80,3	197,6	2,0	1,5	3,7	286,3
10/11/2006 - 13/11/2006	33,0	27,9	69,6	9,7	9,1	22,4	74,0	83,1	204,5	2,0	1,8	4,4	300,9

The difference in TSS concentration for the various periods can be attributed to the varying turbidity and particle content of the water. In the period 13-10-2006 to 10-11-2006 the particle content of the treated water was measured at the entrance of the Reference Area. This has a good similarity with the water that was sampled at the water tower. Due to practical reasons it was not possible to install a Hemoflow where the particle counter was located. The sample of outflow of the UF installation did not contain any TSS as could be expected.

Fig 3-22 gives the calculated particle volume now including the 1-2 µm ranges.

[1] This is below the minimum required mass of 2,5 mg to produce accurate values

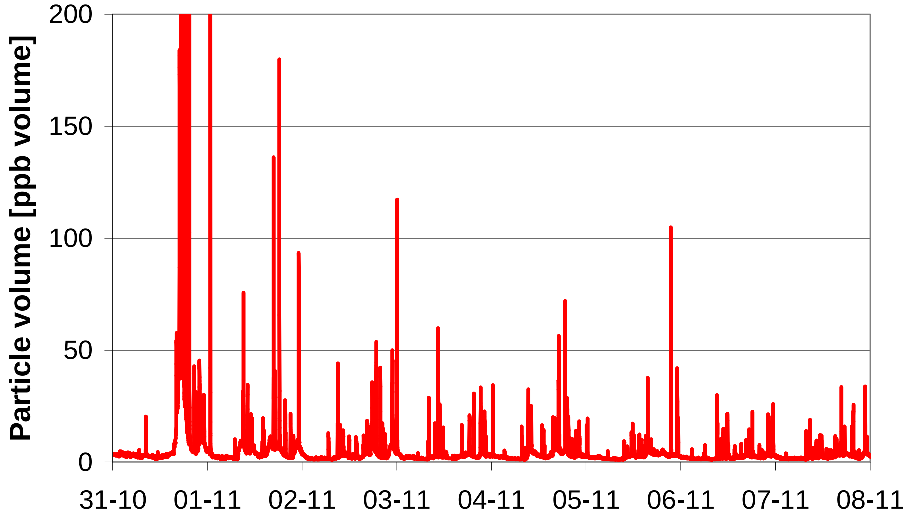


Fig 3-22 Total particle volume at the start of Reference Area during Hemoflow sampling period

The average sediment load at the Reference Area, according to the Hemoflow concentrated sample was 32,64 µg/l. Cleaning the system showed that 5753 gr solids were removed with flushing. These solids were accumulated in the period between the two cleaning procedures at on 24 June 2005 and again on 13 November 2006, which is a period of 507 days. The average daily consumption per capita in the Reference Area was 118 lpppd and 2,7 persons per connection. In total this means that in the 507 days 83.996 m^3 had entered the Reference Area with a TSS of 32,64 µg/l. This is a TSS load of 2743 gr. With flushing, in total 5753 gr TSS was removed. However, the Hemoflow filters themselves probably retained an amount of TSS that was not taken into account with these measurements. The difference in colour of a clean and used filter shows that the amount of solids can be substantial (Fig 3-23).

Fig 3-23 Picture of Hemoflow filters pre- and post-sampling. The increased colour suggests that some of the particles are left in the filter, thus clouding the accuracy of the sample

As the total amount of TSS found in a sample is in the order of 100 mg, it could well be imagined that the coloured material also contains an amount in this order. An error factor in the order of 2 can be expected, making it difficult to draw conclusions from these measurements. The measurements are reported here because the methodology can still be applied if the filter itself is also analysed for suspended solids. In this case, however, that could not be done because the filters were already too tampered with and partly destroyed.

3.5 Discussion

Not surprisingly all measurements show that the sediment load at the Research Area was significantly less than at the Reference Area. Theoretically the sediment load to the Research Area should be negligible, but in practise it turns out that even in the outflow of the UF-reservoir some calculated particle volume was present. These particles were probably picked up in the UF-reservoir. Though the reservoir had been cleaned before being taken into service, it obviously contained some particles that settled in the first pipe of the network. The Hemoflow measurement that had been done on the outflow of the UF-installation shows that the actual amount of TSS was virtually negligible. The total amount of loose deposits removed from the Research Area at the end of the measuring period was less than 8% of the amount removed from the Reference Area. The main difference between the two systems was the particle volume in the water that was supplied, since materials that were used and the diameters that were applied are the same. The UF-treatment only removed the TSS from the water, but left other components as AOC and other soluble substances unchanged. The chemical properties of the water were identical.
The average VSS level of the samples flushed from the Research area was 60,6%, while the average VSS level of the samples flushed from the Reference Area was 42,5%. In the Research Area 89,9 mg/m of loose particles were flushed out and in the Reference Area 1071,2 mg/m. Those percentages applied to the deposits would indicate that the corresponding organics are 54,0 mg/m and 450,0 mg/m respectively.
In the Reference Area 27% of the incoming TSS was VSS (Table 3-15). If this percentage is applied to the sediments found that would account for 121 mg/m of the total 450 mg/m found in the Reference Area. If the VSS is considered to be the product of biological regrowth then the net growth in the Reference Area results in 329 mg/m organic matter and in the Research Area in 54,0 mg/m organic matter, a significant difference of a factor of more than 6. The UF-membrane treatment doesn’t take out the AOC, as this is soluble. Other biological properties of the water such as colony counts are not considered in this study, but with these results it is plausible that the amount of particles in the water and those accumulated in the network promotes biological regrowth, because they offer a service to grow on (van der Kooij, 2000; Huck and Gagnon, 2004).

In the Research Area a sediment amount has been removed that is equal to a sediment rate of 89,9 mg/m while in the Reference Area a sediment rate of 1071,2 mg/m was found. (Carriere et al., 2005) report sediment rates varying from 30 to 24500 mg/m and (Barbeau et al., 2005) gives values of 260 to 410 mg/m. These sediment rates, however, were based on samples taken under different and non-uniform circumstances that can lead to both over- and underestimating of the ‘real’ sediment rate.
In earlier research sediment rates in small diameter pipes are found in the order of 3,9 to 247,3 mg/m. In a cast iron network that is supplied with water treated with slow sand filtration as the finishing treatment step, sediment rates were found ranging from 9,1 to 63,3 mg/m (Vreeburg et al., 2007).

The Surf-90% and Surf+90% values of the particle volume of the Reference and Research Areas show that the major part of the particle loading was done in a relatively short time. The actual particle load to the system is also dependent on the volume flow. In the Reference Area the high peaks were often the result of resuspension, since they coincided with the periods of increased demand, which would also indicate that a larger load was being transported. To make the mass balance, a real-time measurement of the volume flow is necessary, but difficult to measure. The development of a stochastic end-use model (Blokker and Vreeburg, 2005; Blokker et al., 2006) will facilitate this largely.

The traditional, and legally obliged, weekly or daily sample analysis of the treatment plant is not sufficient to detect the short peaks in turbidity or particle load. As is seen from the Surf-90% and Surf+90% values, an important part of the sediment load was realised in a short period that would have been easily missed in a sampling program. Continuous monitoring for analysis of network processes is applied longer (van den Hoven and Vreeburg, 1992). Continuous monitoring for a few days at the treatment plant together with analysis of the frequency distribution, the 90/99,5 percentile ratio, the average particle load and the Surf-90% and Surf+90% are a better way to evaluate the operation of the treatment process.

According to the RPM measurements the Reference Area was back on the starting level after the period of 507 days or roughly one-and-a-half years. During the same period the sediment build-up in the Research Area was less than 8% of this resulting in a significant lower RPM. The measurements of the actual particle load through particle volume concentration and the Hemoflow measurement are not absolute, but show a plausible and significant difference in favour of the Reference Area. Based on the RPM measurements, it is plausible that the cleaning frequency of the Research Area is much longer than that of the Reference Area. In fact the cleaning frequency could be argued indefinite because the average RPM stays constant after 6 months and the standard deviation decreases. However, the average increase of 1 point RPM in one-and-a-half year would imply that the initial level of 11 would be reached in 12 years. This would mean an increase in cleaning frequency with a factor of 8, practically interpreted as a factor 5 to 10.
The beneficial effect for the operation of the network is obvious in terms of saving cleaning costs. Next to that, the overall level of the discolouration risk is lower. The theoretical effect of improved treatment on the cleaning frequency will result in a longer period with lower discolouration risks (Fig 3-24).

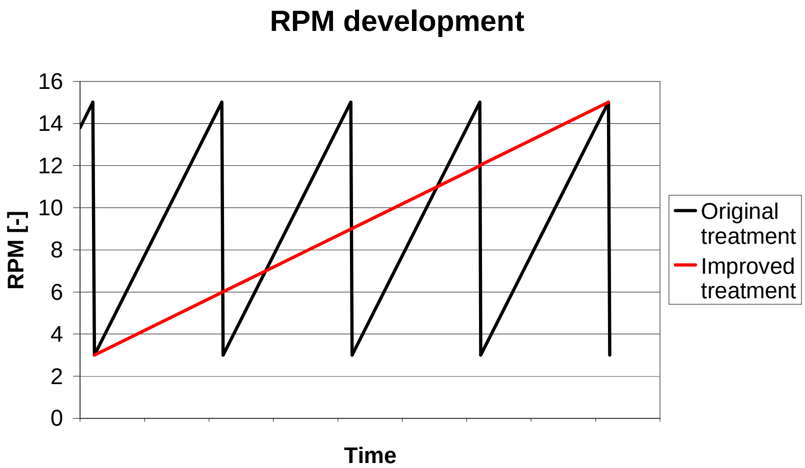


Fig 3-24 Theoretical RPM development with improved treatment

The effect is not only the prolonged interval between two cleaning actions, but also the lower level of RPM over a longer time. The RPM does not exceed the maximum level in both cases shown in Fig 3-24, but complaints still can occur. This is depending on the thresholds in the ranking table but also on the severity of the hydraulic disturbance. With the improved treatment, the overall discolouration risk is lower over a longer time resulting in less complaints.

3.6 Conclusions

From the research on the effect of particles in the drinking water in this particular study, it can be concluded that:

- The particle load from the drinking water recharges the system to the initial RPM level within one-and-a-half years. In this situation the cleaning frequency would be twice in three years.
- In the supply area of a conventional groundwater treatment plant the loose deposits in the network originate mainly from the treated water.
- The growth of the organic material in a network is related to the total amount of particles in the water and the deposits in the pipes. In this case the increase in organic material was 6 times more in the presence of particles than in their absence.
- Particle-free water prolongs the cleaning interval in this case by a factor of 5 to 10.
- The major part of the particle load in the network happens in a relatively short time. Avoiding these peaks reduces the particle load considerably.
- Particle counters are powerful tools to analyse particle-related processes in drinking water distribution network
- The performance of treatment on particle load cannot be characterised with daily samples. At least a continuous monitoring of turbidity is necessary, but preferably continuous particle counts over several days. Results are interpreted with the cumulative frequency distribution and the Ratio 90/99,5, the average and the Surf+90% values.

4 Velocity-based self-cleaning residential drinking water distribution systems

4.1 Introduction

The design of drinking water distribution systems (DWDS) has historically been a matter of skilled labour rather than clever design, mostly dominated by the fire fighting demand and the intuitive need for looping to sustain the continuity of supply. Without explicit knowledge about the adverse effects of low velocity and long residence times, the importance of quantity and pressure supersede the importance of water quality. However, with growing insight into the effect of velocity on the build-up of sedimentary layers, together with the awareness that discolouration problems are not exclusively related to cast iron, more attention to the detailed design of the DWDS is necessary.

Aesthetical water quality problems such as discoloured water occur when loose sediments in the DWDS resuspend and reach the customer in concentrations that can be visually noticed and may lead to complaints. Regardless of the origin of the particles, the net result of regular deposition and resuspension is the accumulation of loose particles. In conventional distribution networks the velocities are low, in the order of a few centimetres per second or even stagnant (Blokker et al., 2006), because the design is mostly dominated by the fire flow demands that supersede the drinking water demand by several times (Snyder and Deb, 2003). It is hypothesised that drinking water distribution systems with regularly occurring higher momentary velocities do not experience an accumulation of particles. Consequently, the discolouration risk will be low and the networks can be considered self-cleaning. The regularly occurring high velocity resuspends the particles that have accumulated in periods of low velocities which typically occur during the night.

In this chapter the design principles of self-cleaning DWDS are developed and discussed. The effects of the wide implementation of these principles on the newly laid networks in the Netherlands are demonstrated. The design principles are restricted to DWDS that are dominated by connections with a household demand pattern because, in the conventionally designed DWDS, the discolouration problem actually leads to customer complaints. The main practical issues involved in supplying sufficient fire fighting flows, allocating valves to sustain supply continuity, and impacts on costs are addressed.

4.2 Self-cleaning velocity

4.2.1 Gravitational settling

The particle-related processes described in Chapter 1 (Fig 1-5) suggest that a bottom layer of particles results from gravitational settling. Settling and resuspension theories are described by the formulas of Stokes and Shields. Berlamont has developed a Shields-based theory on the resuspension of particles and experimentally arrived at a formula that describes the critical velocity for settling and resuspension of material in sewer pipes (Equation 1) (Berlamont and van Goethem, 1984).

$$(u_*)_{cr} = \sqrt{\alpha_{Berlamont}\, g \frac{(\rho_s - \rho_w)}{\rho_w} d_s}$$ *Equation 1*

with

$(u_*)_{cr}$	:	Critical velocity [m/s]
$\alpha_{Berlamont}$	:	Berlamont coefficient (α=0,8 for resuspension) [-]
ρ_s	:	Density of particle [kg/m^3]
ρ_w	:	Density of water [kg/m^3]
d_s	:	Diameter of particle [m]

The relation between the critical sheer stress velocity u_* and the average velocity v in the pipe is described with Equation 2:

$$v = \frac{u_{*cr}}{\sqrt{\lambda/8}}$$ *Equation 2*

with

$(u_*)_{cr}$	:	Critical velocity [m/s]
v	:	Average velocity [m/s]
λ	:	Darcy-Weisbach friction factor

With a course estimation of $\lambda = 0.02$, representative for a pipe with a diameter of 100 mm and a Nikuradse roughness of 0,1 mm gives a relation of $v=20 \cdot u_{*cr}$

Typical particles found in the drinking water networks have sizes that vary between 1 and 25 µm (Boxall et al., 2001; Verberk et al., 2006). Though densities are difficult to establish exactly, it is plausible that they are probably very low as they are from an organic origin or from iron hydroxides (Gauthier et al., 1996; Gauthier et al., 2001). Fig 4-1 shows the relationship between critical velocities for resuspension following the empirical Berlamont formulas in a density range from 1050 kg/m^3 tot 2600 kg/m^3. The low density represents the flocky material formed by organic material and iron flocs, and the high density is typically sand.

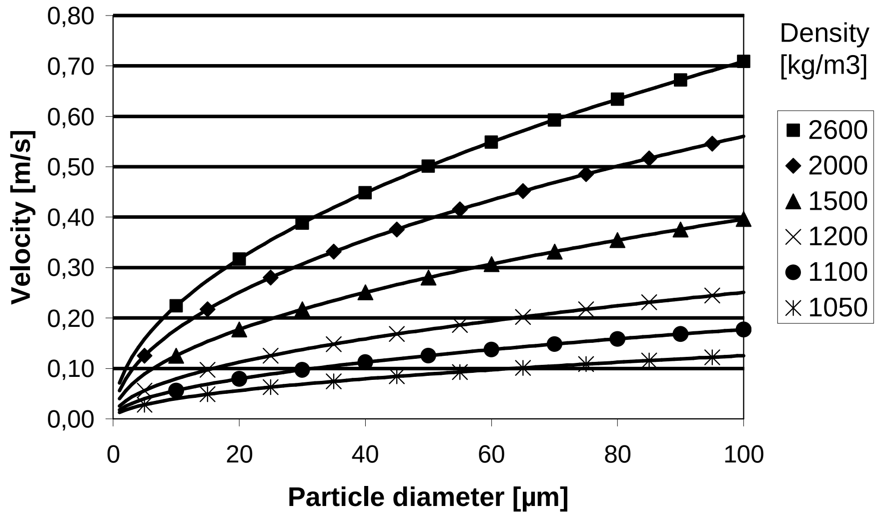


Fig 4-1 Critical resuspension velocities assuming α=0,8 and λ=0,02

Under normal circumstances, velocities in conventional distribution networks are a few centimetres per second (Blokker et al., 2006), which would explain the settling of material according to Berlamont. However, values of 0.05 m/s will be regularly exceeded, thus theoretically resuspending sediment with diameters larger than 10 to 20 µm and with low densities. Similarly Boxall et al (2001) arrived at the same conclusion that practically all particles should stay in suspension and they proposed that particles are caught in a cohesive layer for which the strength is determined by the maximum velocity that occurs in the network. This can explain the fact that smaller and lighter particles are found in the network than theoretically could be expected.

4.2.2 Influence of turbophoresis

As mentioned in the previous section, particles found in the DWDS cannot have settled only due to gravitational settling according to Stokes' law and resuspension according to Berlamont. The particles have been trapped, somehow, in a layer near the wall preventing them from resuspending. This phenomenon can be explained by using the theory of turbophoresis as first proposed by Caporaloni et al (1975). The main driving force that causes a flux of particles towards the pipe wall depends on the gradient of turbulence in the area near the pipe wall. The larger this gradient (i.e., the higher the velocity), the larger the turbophoretic force driving the particles to the near wall, will be.
A graphic presentation of the turbulent diffusion and turbophoresis is given in Fig 4-2.

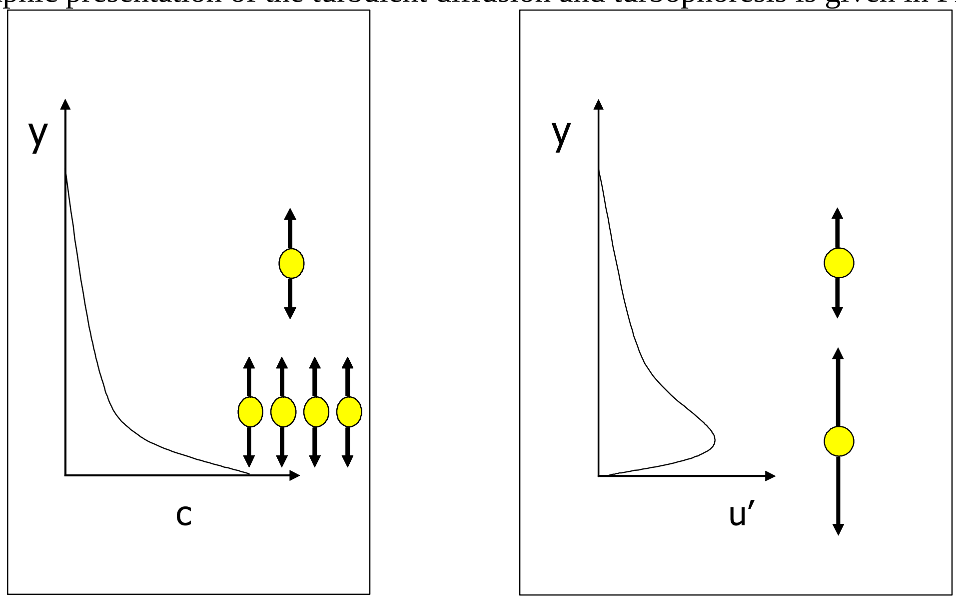


Fig 4-2 Turbulent diffusion (left) and turbophoresis (right)

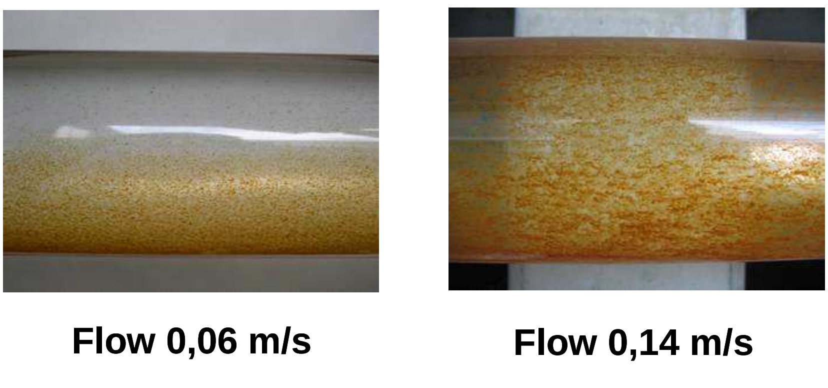
The y-axis represents the distance called near-wall distance Y to the actual pipe wall in both figures. The x-axis in the left graph represents the concentration c of particles. Turbulent diffusion depends on the concentration gradient, meaning that the transport is in the direction of the $-\frac{dc}{dy}$. The driving force is dependent on the concentration differences, which are represented in the left graph.
In the right graph of Fig 4-2 the x-axis is the turbulent fluctuation on the average velocity in the pipe. Because these fluctuations decay to zero near the wall, the near-wall turbulence is highly non-homogeneous, with a gradient in turbulence intensity as a function of the near-wall distance. When a particle with some inertia is thrown into the region with lower turbulence intensity near the wall, it is more likely that it will stay there than that it will make

the return journey away from the wall, as it has to overcome the turbophoretic force. The turbophoresis is dependent on the inertia of the flow, and the transport is in the direction of $-\frac{du'}{dy}$, meaning that the driving force is largest in the vicinity of the wall. Once particles are near the wall, it is difficult to escape again, because the turbulence gradient is highest in that area and drives the particles to the wall again. This explains Boxall's finding that particles seem to stick to the wall but can also be removed when higher velocities are applied.

The principle of turbophoresis is mostly applied to air flow with particles and is not usually used to describe water flows. Young et al. (1997) introduced turbophoresis in pipe-flows and showed it to be a dominant process for particle settling in the near-wall region. In an extensive literature review (Sippola and Nazaroff, 2002) it is concluded that turbophoresis has not been frequently recognised in the literature, even though it was proven to be a dominant transport mechanism in turbulent (air) flows for some inert particles near walls.

The turbophoresis effect was probably observed in a test rig with transparent pipes flowing through with water and a high concentration of iron hydroxide flocs which shows agreement with the theory of turbophoresis (Lut, 2005). To investigate the process of loss of material from the bulk fluid to the pipe wall, a 6 m. long pipe test facility was built out of 100mm internal diameter Perspex pipe. The facility was run in a pressurised re-circulating mode with a downstream flow control. The source water was initially dosed with iron chloride to a concentration of 10 mg/l iron corresponding to a turbidity of 10 FTU. After 4 days of recirculation the turbidity had dropped below 0.5 FTU, resulting in an accumulation on the pipe walls, as shown in Fig 4-3, for the section of the pipe where the flow is stable and unaffected by curvature or entry / exit conditions. What is particularly remarkable about what is shown in Fig 4-3 is that at low velocities only the lower half of the pipes accumulates iron flocs, which is consistent with gravitational settling. The higher velocity lets flocs accumulate over the complete pipe perimeter. However, the size of the flocs formed will be a function of the flow regime and the relatively pure iron hydroxide flocs formed are likely to be larger than the size of particles typically seen in discoloured water samples. This adds to the inertia of the flocs that are involved in the turbophoresis.

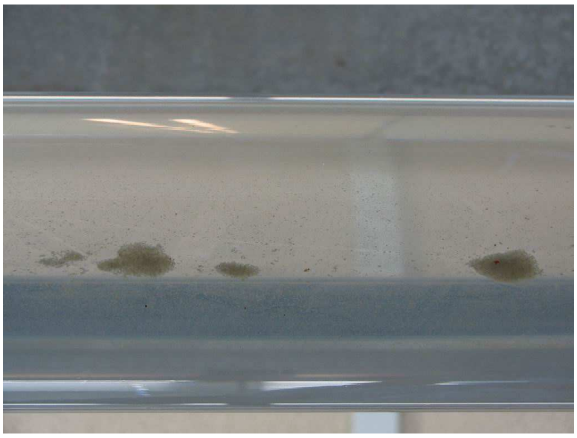
Pipe diameter 100 mm



Flow 0,06 m/s **Flow 0,14 m/s**

Fig 4-3 Photographs of accumulated material within a Perspex pipe loop after 4 days re-circulation with high concentrations of ferric chloride (10 mg/l)

The circumferential settling can also be observed in the same test rig, but now loaded with heavier kaolinite particles as shown in Fig 4-4. The left picture shows the sediment from a bottom-up view, the right picture is again the side view. The particles now only rise halfway up the pipe.



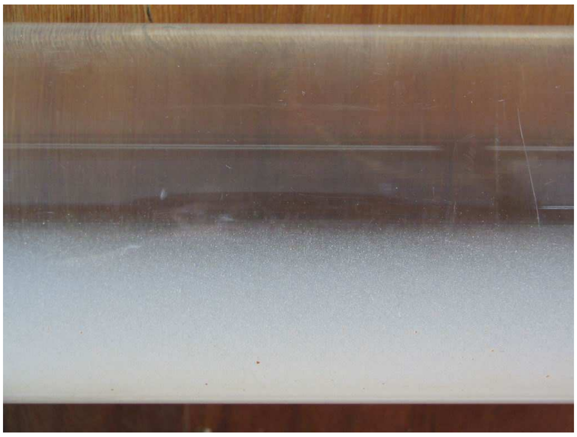


Fig 4-4 Results of same test rig, loaded with kaolinite particles. Left 0,06 m/s (bottom view) right with 0,14 m/s

This phenomenon is in accordance with turbophoresis(Young and Leeming, 1997): particles are transported from the bulk fluid to less turbid regions near the wall where they can be trapped in cohesive layers. With higher velocities, the gradient is greater because the turbulence at the pipe wall must always be zero, resulting in a larger force driving particles from the centre to the wall of the pipe. In light of this theory, it can be suggested that at a flow rate of 0.14 m/s the turbophoresis force exceeds the gravitational force, resulting in a uniform supply of material at the pipe surface, while at 0.06 m/s the gravitational force is dominant.
The trapping of the particles on the complete pipe wall is, following the turbophoresis phenomenon, a dynamic process that should stop when the water flow stops. This was not explicitly studied in the experiment, because after stopping of the flow the installation was drained and prepared for new experiments. The impression however was that the particles did not immediately detach from the wall implying that also some cohesion to the pipe wall aplys a role. This is consistent with the cohesive layer theory (Boxall et al., 2001).
The time scale of the experiment was a few days, which is too short to develop a biofilm. It can, however, be imagined that turbophoresis transports particles to the wall and in this way incorporates them more easily in the biofilm.

4.2.3 Self-cleaning velocity: a decision

Particle settling and resuspension processes are very complicated processes and are not easily described in mathematical terms. Strictly reasoned from the Stokes/Berlamont approach, a progressive relationship between the velocity and the settling/resuspension would be expected. With the phenomena observed in the test rig (Fig 4-3 and Fig 4-4), the relationship is obviously not only progressive, but probably will experience a kind of hysteresis.
The RPM-procedure resuspends a sediment layer using a disturbance of the velocity of 0.35 m/s (Vreeburg et al., 2004a). Outcomes of these RPM's show that 0.35 m/s does resuspend particles. Moreover, lab experiments (Beuken, 2001) showed that resuspension occurs at velocities in the range of 0,2 to 0,4 m/s. These experiments consisted of loading a 6 meter pipe with a synthetic sediment and real sediments and, after settling overnight, resuspend it with a certain range of velocity. The velocity was increased gradually until resuspension of the sediment could be observed visually. Due to time constraints the experiments were limited, but the conclusion of resuspension at these values could be drawn.

Based on these observations, from a practical point of view it is hypothesised that a velocity of 0,4 m/s would be the threshold for self-cleaning networks. This velocity is applied in several networks and the effects were tested in these full scale networks.

4.3 Methodology

4.3.1 Design principles of drinking water distribution systems

The DWDS considered here are dominated by connections with a typical household demand in residential areas, which is the majority of the public drinking water supply. In areas dominated by industry, the demand patterns are more specific and require a tailored design, depending on local circumstances and requirements.

Historically, the most important design criteria for DWDS are the minimal pressure, the continuity of supply and the fire flow demand. Without a velocity criterion, application of these criteria result in the design of a looped network consisting of pipes with diameters that are sufficient to supply the dominant fire flow demands. During normal demand situations, the pipes experience low velocities, because pipes are dimensioned for the required fire flows that are much higher than the drinking water demand. These networks are commonly found in all water companies (Snyder and Deb, 2003). Blokker (Blokker et al., 2006) showed that maximum velocities in this type of network are in the range of a few centimetres per second. In a representative case study, she demonstrated that residence times may amount to 48 hours in an isolated, 500- connection supply area.

For the self-cleaning new design, a minimal velocity is added as a design criterion which is defined as a velocity that occurs daily for at least a few minutes and is capable of resuspending particles that were allowed to settle during low velocity periods. The velocity criterion changes the layout of the network radically to a branch-type network with unidirectional high velocities. The branch-type network will demand another approach towards continuity of supply and fire flow requirements; criteria that are intuitively better served with looped networks. These design criteria will be reviewed separately showing that it is possible to make networks that both supply the required fire flows and have a high continuity of supply and still sustain sufficient velocity to prevent accumulation of particles.

4.3.2 Minimal pressure

Minimal pressure in a network in the Netherlands is generally set at 200 kPa at the connection point of a house or dwelling. This pressure in combination with a well designed in house plumbing system ensures that taps on the third floor of buildings can be supplied without extra pressure boosting in the installation. For normal flow conditions this minimal pressure is kept as a design value for new networks, however in fire flow situations pressure may be lower.

4.3.3 Continuity of supply: valve location and size of cut-off sections

Continuity of supply in a looped distribution network is largely determined by the location and functionality of valves and not so much by the structure of the network in loops (Trietsch and Vreeburg, 2004; Trietsch and Vreeburg, 2006). Supply to individual connections is interrupted when a part of the network is cut off for maintenance or repair reasons. In conventional looped networks, isolation of a cut-off section takes at least two valves, but in practise an average of three to four valves is needed for a single isolation. If the reliability of the valves is assumed to be 90% (Rosenthal et al., 2001), a successful shutdown of a four-valve section has a probability of $0.9^4=0.66$. If the isolation could be done with only one valve, the probability of a successful shutdown would increase to 0.9. An international questionnaire on valves, answered by 16 utilities in Europe, USA, Australia and Japan, shows 897,471 valves on 111,570 kilometres of main (Rosenthal et al., 2001). This means 8 valves per kilometre of main. Assuming 4 valves for one cut-off section and 10 metres of main for one connection, this results in an average cut-off section size of 50 connections.

The size of the cut-off section or the number of connections in a cut-off section is a design criterion that, however, is not quantified in many cases. It is remarkable that within the aforementioned questionnaire no water company reported explicitly that there is a set number of connections per cut-off section as a design criterion.

The addition of the velocity as a design criterion changed the layout of the network: self-cleaning high velocity networks are characterised by a branched structure with a downstream declining diameter that sustains a daily occurring high velocity of at least 0.4 m/s. The length or size of the branches is determined by a maximum number of properties that can be cut off simultaneously for repair or maintenance reasons. For the initial design criterion for self-cleaning networks, the maximum cut-off number is set at 100. Though in practise, water companies in the Netherlands often use higher numbers up to 200 in the application of the new design rules. For proper application of the design rules, the definition of a fixed number is essential though it can be company specific. An extra criterion can, for example, be that the length of the cut-off section is maximised.

Every cut-off section is designed as a branch with a unidirectional flow that upstream is connected to a looped structure of trunk mains. This is counter intuitive towards the inherent continuity of supply of a completely looped system. The single connection of 100 property lines, however, does not compromise the continuity of supply and even offers a higher continuity of supply than a looped connection. Next to the more reliable shut off, the actual isolation time is shortened when fewer valves have to be exercised. Reinstallation is also quicker and has fewer error possibilities: in normal practise, the cut-off section stays isolated on a single connection until the hygienic safety is proven by lab samples. After at least 24 hours all valves have to be relieved, which is a source of errors for valves left shut in the system.

Overall, it can be concluded that the single attached cut-off section offers a better continuity of supply than the multi-valve sections in conventional looped networks. This is provided that the cut-off section is attached to a looped main structure that has strategically located valves. The continuity of supply can be further increased if the connection of the branched cut-off section is backed up with two extra valves in the main pipe. Failure of one valve, which has a probability of 10%, induces the use of the two back-up valves without affecting more connections and keeping the failure confined to the one cut-off section (Trietsch and Vreeburg, 2006). The total probability of a successful shut off of the section is then $0.9 + 0.1*0.9*0.9 = 0.98$.

4.3.4 Velocity for self-cleaning

The velocity in a household demand dominated DWDS will vary over the day from complete stagnation during night hours to a high velocity during typical peak hours in the early morning. During warm weather situations, a second and even higher peak may occur in the evening (de Moel et al., 2006). In order to be self-cleaning the velocity in the peak hours should be capable of resuspending the particles that have settled during the low flow or stagnation periods.
As is explained in the paragraph ‘Self-cleaning velocity’, the self-cleaning threshold velocity is set at 0,4 m/s.
The new branch-like structure of a network decreases the real extremes in velocity that typically cause discolouration incidents. The real extremes are caused by incidents such as pipe breakage which cause very high flows during the bleeding of the break. Looped distribution network will act as a bypass for the compromised transport network, causing higher velocities than with the normal maximum demand. In a branched layout, the distribution network cannot act as a transport network and thus the maximum velocities will only occur in high demand situations.

4.3.5 Demand estimation

Estimation of the demand of individual connections and groups of connections for the design of the network is based on the so-called $q\sqrt{N}$ method. This method has been used over the last decades in the Netherlands to design house plumbing installations and is laid down in the National Standard for general requirements for water supply installations (NNI, 2002). Though widely accepted and documented in design manuals, an original description of the method could not be located. The core of the method is that the nature and the size of the tapping points on a single line determines the maximum flow in a square-root relationship. Every type of tapping point is credited with Tapping Units equal to 0.083 l/s (300 l/h). A toilet cistern is, for example, typically 0.25 TU, and a kitchen sink tap has 4 TU’s. Table 4-1 gives a TU count for a typical Dutch, single family home. Tapping units can be obtained from the manufacturer of the appliances based on the maximum flow that the appliance can process.

Table 4-1 Typical house installation

Tap point per house	Number of TUs
Toilet cistern tap1	0.25
Toilet washbasin1	0.25
Toilet cistern tap 2	0.25
Toilet washbasin 2	0.25
Kitchen sink	4
Dishwasher	4
Bath/shower mixer tap	4
Washbasin mixer tap (bathroom)	1
Wash basin tap (bedroom)	4
Washing machine tap	4
Total per house	22

Maximum flow at the beginning of the installation at the connection point to the DWDS is

$$q_{\max} = TU\sqrt{N_{TU}} \qquad \textit{Equation 3}$$

with

q_{max} : Maximum flow [l/s]
TU : Tapping Unit @ 0.083 l/s
N_{TU} : Number of tapping units in the installation

For the design of the cut-off section the total section is considered to behave as a single installation, which allows calculation of the maximum flow at any point with any combination of inner installations.

$$q_{\max} = TU\sqrt{\left(\sum_{i=1}^{n} z_i N_{TUi}\right)} \qquad \textit{Equation 4}$$

with

n : Number of types of inner installations
z_i : Number of properties with N_{TUi} tapping points
$N_{TU\,I}$: Number of tapping points

An example of the maximum flow for several types of houses is given in Fig 4-5

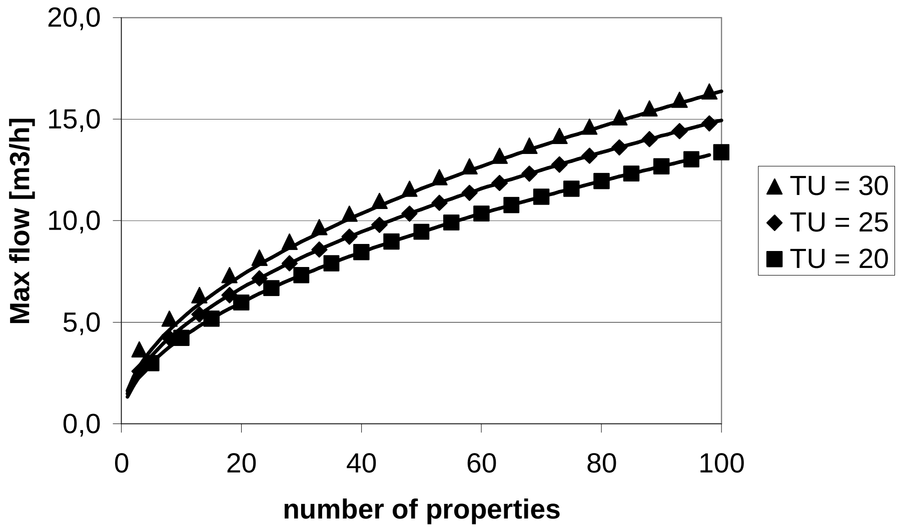


Fig 4-5 Maximum flow calculated with $q\sqrt{N}$ for various types of inner installations characterised with the number of Tapping Units (TU)

As a design criterion the maximum demand should cause a velocity of at least 0.4 m/s to induce the self-cleaning effect. The limit for the maximum velocity is determined by the available head at the beginning of the line or by an absolute value set for other reasons, e.g., prevention of a water hammer. Table 4-2 summarises a design table for a typical number of property connections. The maxima and minima are related to the typical diameter, in this case PN10 PVC.

Table 4-2 Design criteria PVC PN10 pipes

Outside diameter [mm]	Inside diameter [mm]	Min number of houses TU = 20 v_{min} = 0.4 m/s	Max number of houses TU = 20 v_{max} = 1.5 m/s
32	28.8	0	7
40	36.2	1	17
50	45.2	3	42
63	57.0	8	106
75	67.8	15	213
90	81.4	31	442
110	99.4	70	983

4.3.6 Fire fighting demand

The historic decision of public water supply to provide fire flows has had a significant impact on the design of the DWDS. Fire fighting demands vary between 30 m^3/h to 240 m^3/h, albeit without much explanation or factual support. The flow is supplied by hydrants with 80 to 100 meter interspaces based on the requirement that every building has a hydrant within a radius of 40 to 50 meters. Throughout North America and many European countries, there are no formal laws requiring water utilities to provide fire flows. It is generally up to the local municipalities to adopt ordinances to provide them. The lack of uniform rules led to the large variety in required fire flows. Conventional DWDS are therefore heavily dominated by high fire flows and hydrant spacing (Snyder, 2002).

Incorporating the fire flow demand within the self-cleaning high velocity network concept requires intensive communication with the fire fighting departments and organisations of fire fighters during the design process. For the Netherlands this has resulted in a guideline that allows a fire flow of 30 m^3/h, which was laid down in a national model agreement for municipalities and fire fighting services (Esveld, 1996). The main argument to revise the required fire flow was that the original higher demand (60 to 90 m^3/h) was based on outdated fire fighting practises and building codes. In the Netherlands modern post-1950 buildings meet such fire resistance codes so that high flows from the public drinking water networks are not required anymore for the initial first attack of the fire brigade. This facilitates the development of so-called 30 m^3/h fire hydrants in the periphery of the pipes. Next to the lower flow, pressure requirements could also be lowered as the hydrant flow is repressurised by fire trucks. For back up, and if more flow is needed, ‘larger’ fire hydrants can be found in the surrounding network.

A dual pressure situation can now be considered in the design: the maximum flow for drinking water purposes with a minimum effective pressure of 200 kPa in the pipes, and a maximum flow for fire fighting demand with a minimum effective pressure of 5 kPa in the pipes. The minimum pressure in the network depends on the hydraulic resistance in the hydrant-standpipe-hose combination. This possible low pressure during fire flows could endanger the safety of the drinking water because of the back siphoning of the water and re-entrance of leakage water. The use of a fire hydrant to the full fire flow extent, however, is very rare and in those cases, post-incident measures should be taken. In practice however, this is seldom necessary.

4.4 Application of high velocity design principles in the Netherlands

The design guidelines for self-cleaning networks were formally introduced in the Netherlands in 1999 (Boomen and Vreeburg, 1999). The introduction was accompanied by a big campaign of introductory workshops with individual water companies that proved to be very useful to train designers in the use of the new rules. This changed the face of new networks to branched-type small pipe networks. A typical example of a new network is shown in Fig 4-6.

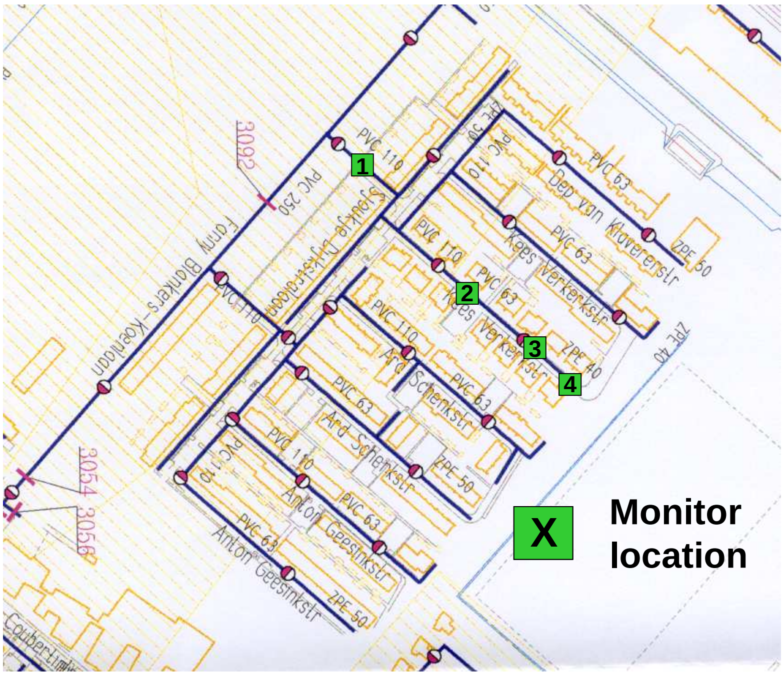


Fig 4-6 Example of new high velocity network. The monitor location are used to measure the effect of the new lay out on water quality (see section 4.5.3)

The complete section is connected to the main looped 250 PVC structure at location “1” and is composed of unidirectional PVC Ø110 and Ø 63 pipes that end in Ø 40 mm ZPE pipes between locations 3 and 4. Note that the ends of the pipes are not connected to loops, which would be typical for a conventional network. The red/white circles represent fire hydrants that can supply 30 m^3/h and can be connected to a Ø 63 PVC pipe. All hydrants are marked as 30 m^3/h hydrants, but the ones on pipes with diameters above 100 mm are capable of supplying 60 m^3/h or even more. The principle and technical layout of this connection is shown in Fig 4-7. Applying a number of different diameters allows for more flexibility in design, but more complexity in the actual making and maintaining of the networks.

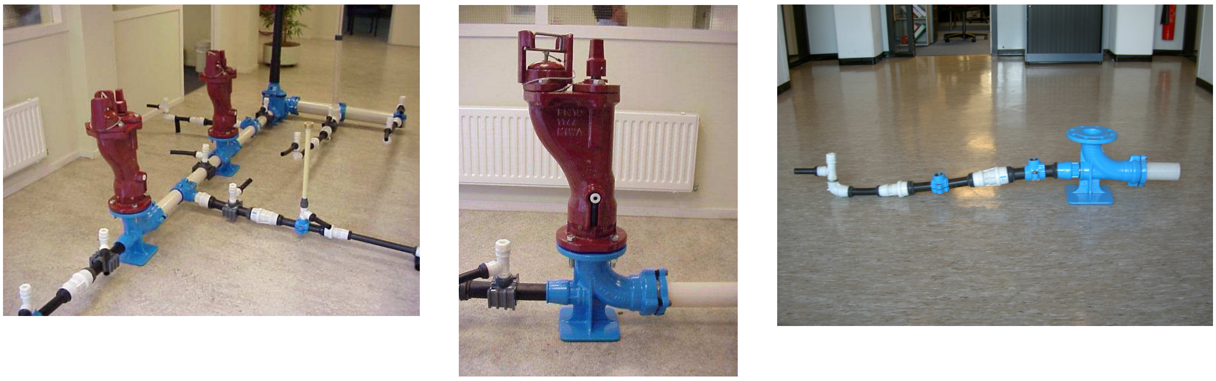


Fig 4-7 Example of technical layout of new network and newly designed 30 m³/h fire hydrant

Note especially in Fig 4-7 the details of the last connection and the hydrant that also allows for a decline in the diameter of the pipe. This equipment is specially designed for these purposes.

Since the formal introduction of the new design standards, the Dutch water companies have gradually applied these guidelines, and the general characteristics of the newly laid networks have slowly changed, as is shown in Fig 4-8. The figures originate from a questionnaire that was answered by 80% of all water companies in the spring of 2005 and was aimed at the analysis of the diameter and length of pipes that are typically used in new networks.

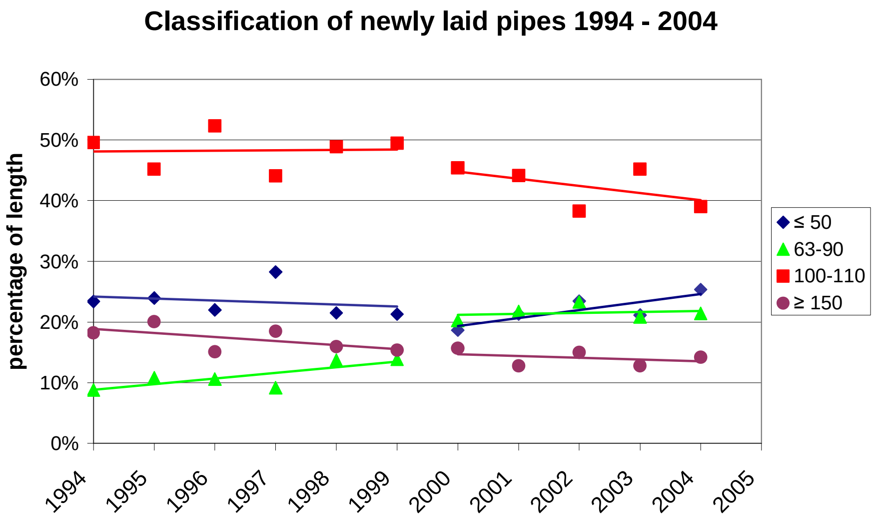


Fig 4-8 Classification of newly laid pipes in the Netherlands. Introduction new design guidelines in 1999

The total length of new pipes considered in the questionnaire was 4500 km, of which 2400 km was laid in the period 1994-1999 and 2100 km in the period 2000-2004. The segment of 110/100 mm pipes significantly decreased from 55% to less then 45% and the amount of small diameter pipes (≤ 50 mm) increased. A more or less abrupt change can be observed in the 63-90 range, which can be explained by the addition of 30 m³/h fire hydrants that can be connected to Ø 63 mm PVC pipes.

4.5 *Water quality effects*

4.5.1 Experimental setup

The hypothesised effect of regular high velocities in networks is that it prevents particles from accumulating into layers of loose deposits. To verify this effect, an experiment was set up in the supply area of one treatment plant. Three different types of DWDS were isolated and during a certain time the particle content at different locations was measured. The characteristics of the three DWDS differ in layout and size of the pipes. The first area is a conventional DWDS that is representative for most of the existing networks in the Netherlands with a looped layout and dominated by larger diameter pipes. The second network can be characterised as a first-generation high velocity network. That means that the majority of the pipes are branched, but that the diameters are not fully tuned to the high velocities, especially at the end of the branches. The third DWDS complies fully with the design rules for high velocity networks, which means that the ends of the branches are also downsized to diameters of 40 mm.

4.5.2 Materials and methods

In each of the areas at four locations the water quality was measured with particle counters. Two Pamas Waterviewer particle counters were available, the first of which was continuously located at the entrance of the network and the other was alternated between each of the other three locations. The particles were counted in the ranges 1-2, 2-3, 3-5, 5-7, 7-10, 10-15 15-20 and > 20 µm. Measuring frequency is 2 minutes and the equipment is calibrated following manufacturers instruction. With reference to the particle-related processes shown in Fig 1-5, the first measuring location is the incoming water, the other three are the outgoing water. For the branched-type networks, the first location is the start of a cut-off section and for the looped network it is the supposedly-preferred supply point for the area. In the last case, the other connection points to the loops are considered to be less important for the daily supply of the network. The differences in particle volume at the various locations will lead to conclusions towards the self-cleaning capacities of the considered DWDS.

4.5.3 Three areas

Area 1: conventional DWDS

Fig 4-9 presents a map of the conventional DWDS with the measuring locations.

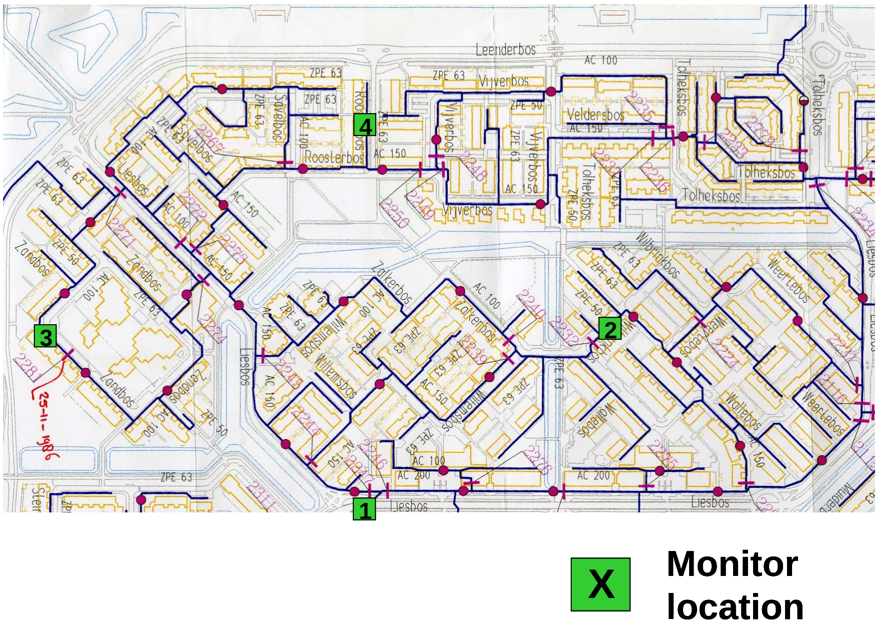


Fig 4-9 Area 1: conventional DWDS. Locations 1, 2, 3 and 4 are measuring locations for the particle counters

As the network is looped there are several feeding points though the one at location 1 is considered to be the most important. Location 2 is at the opposite location of location 1 on the same Ø 150 mm AC loop. Location 3 is at the 'top' of a Ø 100 AC mm loop with shuttling water. Location 4 is at a dead end with a diameter Ø 63 mm PVC and with 8 houses downstream.
The network was mainly constructed in 1986 with a proximally 650 connections and was not systematically cleaned for 13 years.

Area 2: first-generation high velocity network
The map for the second area is given in Fig 4-10.

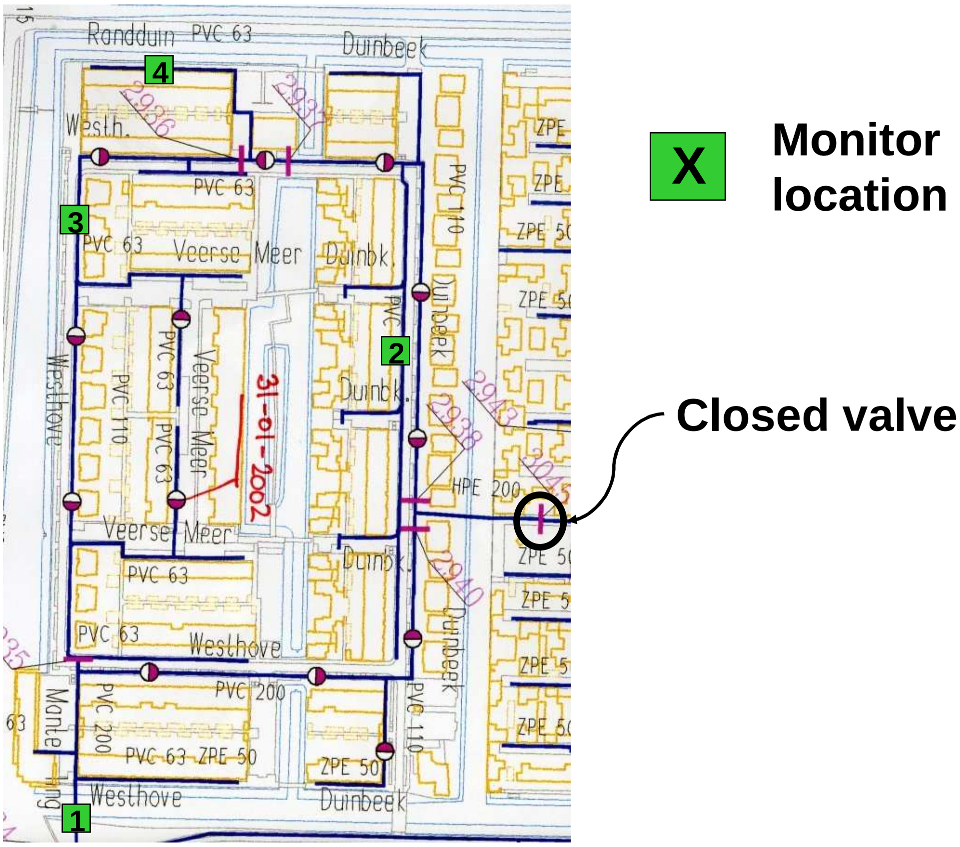


Fig 4-10 Area 2: First generation high velocity network. Locations 1, 2, 3 and 4 are measuring locations for the particle counter; the circled valve is closed to isolate the branch

This network has a central loop with some smaller branches with 373 connections in total and was constructed in 2002. Location 1 is now the forced entrance of the network and is equipped with a flow meter; the valve at the second connection point is closed, indicated by the red circle right in the picture. Location 2 is a branch with 30 connections downstream, location 3 is on the Ø 110 mm ring, and location 4 at a Ø 63 mm dead end. The network was constructed in 2002 and the material used is exclusively PVC PN10 with external diameters Ø 63 and Ø 110 mm.

Area 3: High velocity network

This is the network that is represented in Fig 4-6 that was used to illustrate a new network. This network was constructed in 2004 using the high velocity design criteria to its full extent. In the branch 157 connections are served and it starts at location 1 with a particle counter and a flow meter. Locations 2, 3 and 4 are on one end that goes down to a 40 mm pipe (location 4). Location 2 serves 12 connections through a Ø 63 mm PVC PN10, location 3 serves 6 locations through a Ø 50 mm PVC PN10 and location 4 supplies 2 connections with a Ø 40 mm ZPE pipe.

4.5.4 Results of water quality analysis

The measurements were done in the period form June 1 to July 6 2006
The results of the particle counters are transformed into calculated particle volumes, assuming that the particles in a certain range are spherical and have a diameter that is the averaged of the boundaries of the range. From the calculated volumes a distribution frequency and a table

of 90, 95, 98, 99 and 99,5 percentile values are made. Also, the ratio between the 90 and 99,5 percentile is calculated. For each combination of location 1 (incoming water) and the other locations (2, 3 and 4), separate graphs of the calculated volume and the distribution frequency are made, as well as a summary table with the percentile values. For Area 1 (conventional DWDS, Fig 4-9), the results for the calculated particle volume and the distribution frequency are presented in Fig 4-11 to Fig 4-16 and Table 4-3. Results of Area 2 (first-generation high velocity DWDS, Fig 4-10) are presented in Fig 4-17 to Fig 4-22 and summarised in Table 4-4. Finally, the results of Area 3 (high velocity DWDS, Fig 4-6) are presented in Fig 4-23 to Fig 4-28 and summarised in Table 4-5.

Results Area 1

For Area 1 (i.e., the conventional DWDS), the incoming average of the calculated particle volumes is less than the outgoing calculated particle volumes. The 90/99.5 ratio of the incoming water is relatively low. For locations 3 and 4 the 90/99.5 ratio increases; especially for location 4, this increase indicates that the extreme values are less. For all locations the absolute values of the percentiles increase, location 4 showing the greatest increase. Location 4 can be considered a mixture of a classical dead end leading to a hydrant and a flowing end as meant in the high velocity networks. The measuring period coincided with some high temperatures (extreme for the Dutch climate), which resulted in relatively high velocities in the pipes. The frequency distributions of the network locations (2,3 and 4) are all to the right of the frequency distribution of the incoming water. The Surf-90% and Surf+90% (see section 2.2.3) show that more than 40% of the average incoming particle load is entering in 10% of the time. Outgoing, the Surf+90% values are less, indicating a higher base level of outgoing particle volume.
The conclusion is that the net particle volume is removed from the network during the high velocities due to the extreme weather conditions. This means that there was particle volume accumulated in the network.

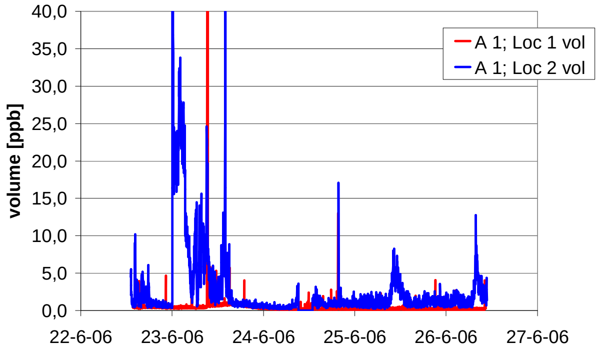


Fig 4-11 Particle volume Area 1, Loc 1 and 2

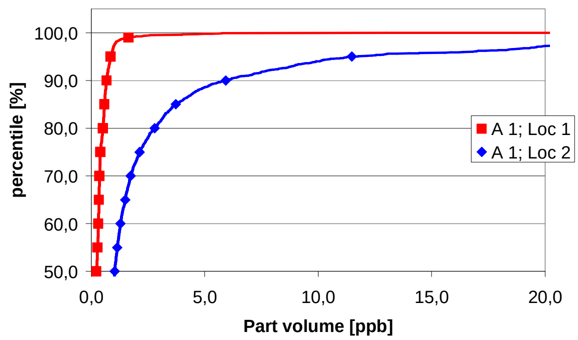


Fig 4-12 Frequency dist Area 1, Loc 1 and 2

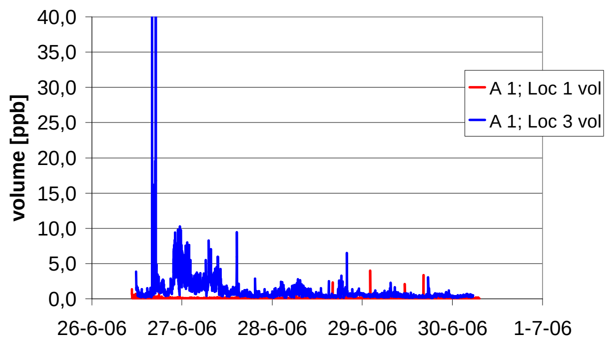


Fig 4-13 Particle volume Area 1, Loc 1 and 3

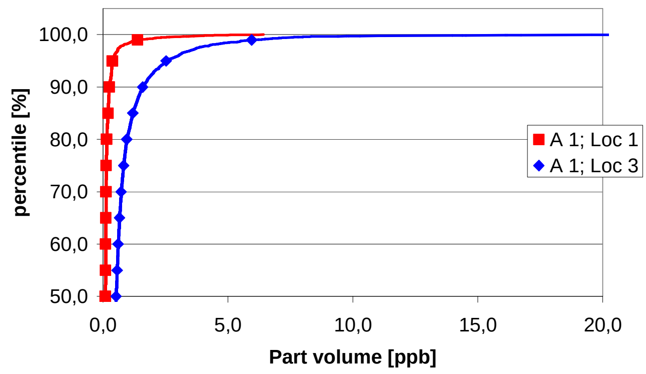


Fig 4-14 Frequency dist Area 1, Loc 1 and 3

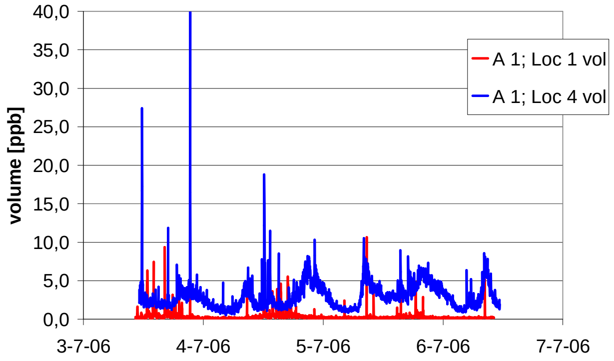


Fig 4-15 Particle volume Area 1, Loc 1 and 4

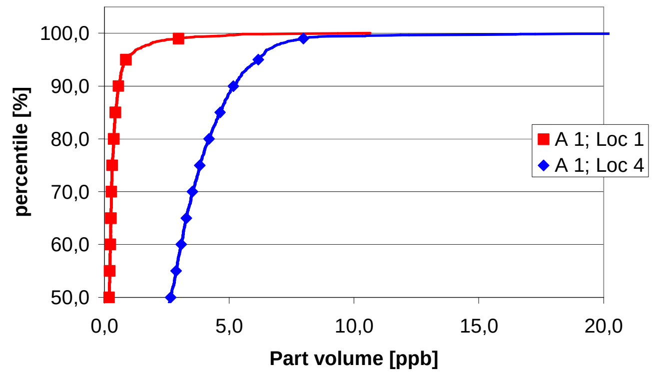


Fig 4-16 Frequency dist Area 1, Loc 1 and 4

Table 4-3 Frequency percentiles and curve characteristics Area 1

Frequency Percentile	A 1; Loc 1	A 1; Loc 2	A 1; Loc 1	A 1; Loc 3	A 1; Loc 1	A 1; Loc 4
	[ppb]	[ppb]	[ppb]	[ppb]	[ppb]	[ppb]
90,0	0,67	5,93	0,25	1,59	0,56	5,17
95,0	0,85	11,49	0,38	2,53	0,87	6,17
98,0	1,09	22,35	0,83	4,29	1,86	7,04
99,0	1,64	25,85	1,38	5,95	2,97	7,98
99,5	2,63	28,63	2,46	7,62	4,61	10,45
ratio 90/99,5	0,26	0,21	0,10	0,21	0,12	0,49
Average [ppb]	0,38	2,82	0,17	0,88	0,34	3,04
surf-90%	58,4%	42,6%	56,5%	58,2%	57,7%	77,4%
Surf+90%	41,5%	57,4%	43,4%	41,7%	42,3%	22,6%

Results Area 2

For Area 2, the first-generation high velocity DWDS, the frequency distributions show a different image. In the low percentiles, incoming and outgoing are practically the same. In the higher percentiles there is deviation, meaning that the particles are removed in peaks, also as a result of the relatively high velocities in the network due to the extreme weather conditions. The peaks come regularly in the late afternoon/early evening, which is consistent with the high demand during hot days. At locations 2 and 3 this influences the shape of the frequency distribution curve because of the high and long peak. The Surf-90% and Surf+90% values show that most of the particle volume is removed in short peaks. They indicate that in these locations, sediment has accumulated that is now resuspended as a result of the high velocities.

In location 4 of Area 2, comparable to location 4 of Area 1, there is a net accumulation taking place as the frequency distribution of the incoming concentration is higher than that of the outgoing one. During the measuring period for location 4 a maximum of the extreme weather conditions occurred, also causing the incoming particle load to be higher as result of high velocities in the total network. With these high concentrations, however, a net accumulation occurred, as is shown in the average value of the calculated particle volume. The 90/99,5 ratio and the Surf-90% and Surf+90% values show that the shape of the curve is practically the same, indicating that this pipe accumulates sediment and acts like a classic dead end. It also shows the relative small effect of a dead end: sediment accumulates, but extreme velocities are rare because the maximum flow is determined by the few houses that are connected. Because the pipe is not looped, disturbances that occur elsewhere in the network do not influence this location.

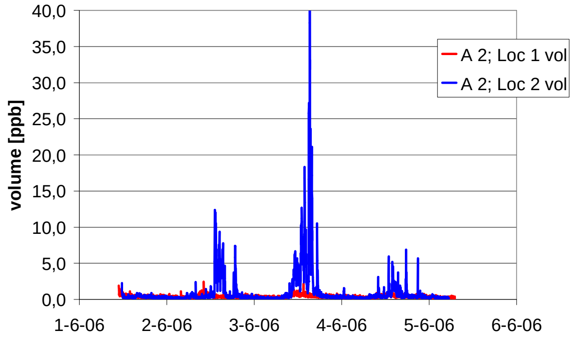


Fig 4-17 Particle volume Area 2, Loc 1 and 2

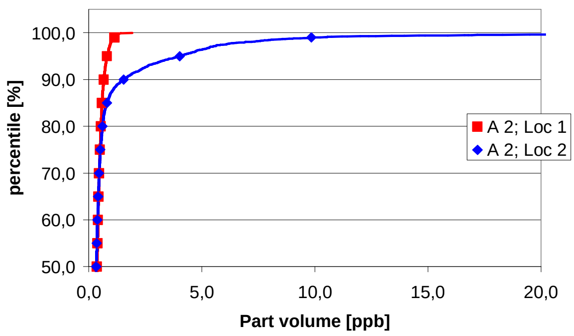


Fig 4-18 Frequency dist Area 2, Loc 1 and 2

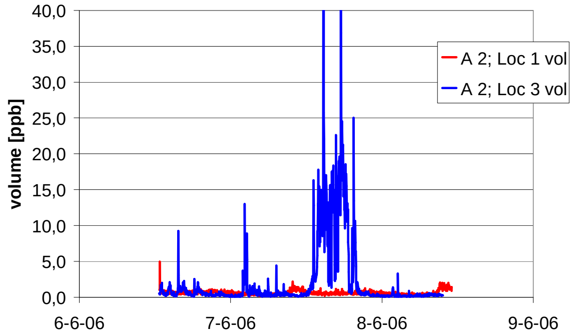


Fig 4-19 Particle volume Area 2, Loc 1 and 3

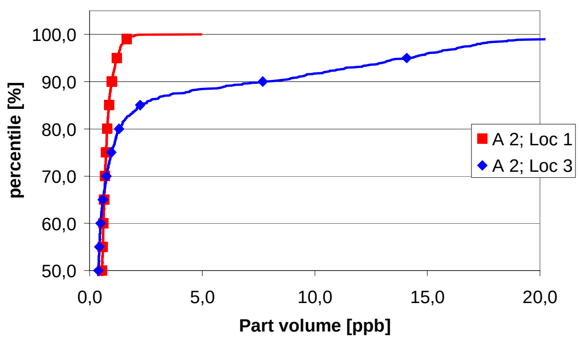
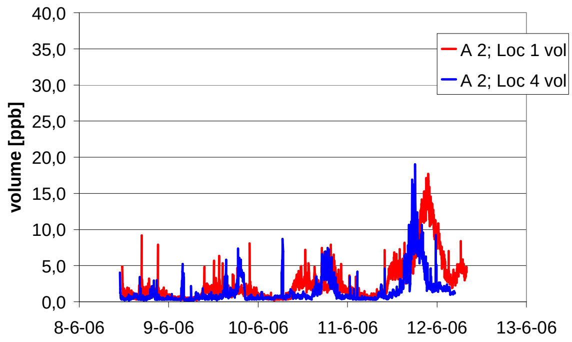


Fig 4-20 Frequency dist Area 2, Loc 1 and 3



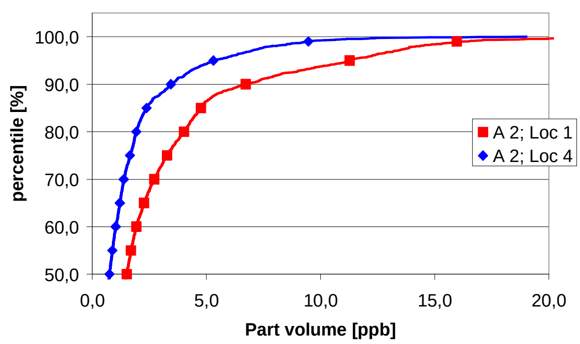


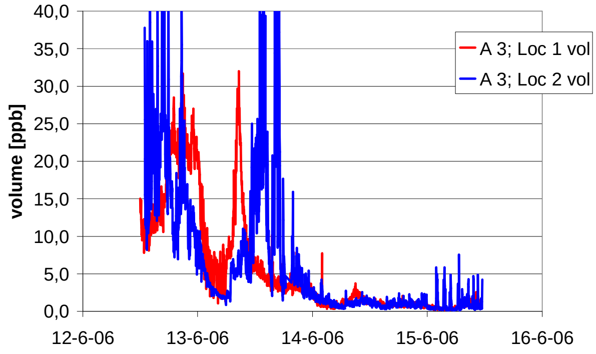
Fig 4-21 Particle volume Area 2, Loc 1 and 4 *Fig 4-22 Frequency dist Area 2, Loc 1 and 4*

Table 4-4 Frequency percentiles and curve characteristics Area 2

frequency percentile	A 2; Loc 1 [ppb]	A 2; Loc 2 [ppb]	A 2; Loc 1 [ppb]	A 2; Loc 3 [ppb]	A 2; Loc 1 [ppb]	A 2; Loc 4 [ppb]
90,0	0,66	1,55	0,98	7,68	6,72	3,44
95,0	0,80	4,02	1,21	14,08	11,29	5,30
98,0	0,98	6,91	1,45	17,35	14,19	7,83
99,0	1,13	9,84	1,65	20,42	15,97	9,47
99,5	1,24	16,79	1,80	22,59	18,85	10,97
ratio 90/99,5	0,53	0,09	0,55	0,34	0,36	0,31
Average [ppb]	0,41	0,89	0,62	2,14	2,81	1,48
Surf-90%	79,1%	37,1%	79,1%	29,0%	57,9%	58,3%
Surf+90%	20,9%	62,8%	20,8%	70,9%	41,6%	41,6%

Results Area 3

Area 3, the high velocity area also, experiences a period of high particle volume coming into the network during the period that locations 1 and 2 are monitored (Fig 4-23). In this case however, the frequency distributions of both the incoming and the outgoing water are practically the same, as are all the other values. This indicates that the high sediment load can be absorbed without a net settling of material: the self-cleaning effect as it is hypothesised. Also, in the other locations, the distribution frequencies are practically the same for the values below the 90%. The Surf-90% and Surf+90% show that the outgoing water has more spikes in concentrations of calculated particle volume than the incoming water. With almost identical average values of the calculated particle volumes, this also indicates the self-cleaning effect as it is hypothesised.



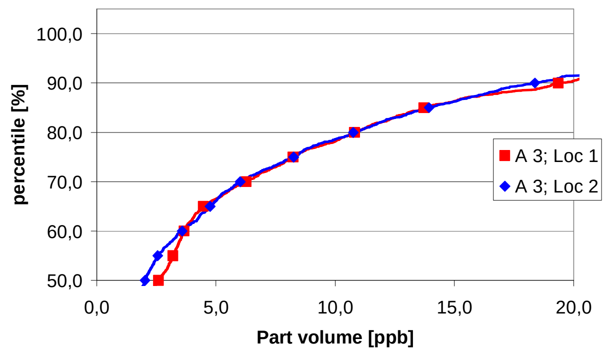
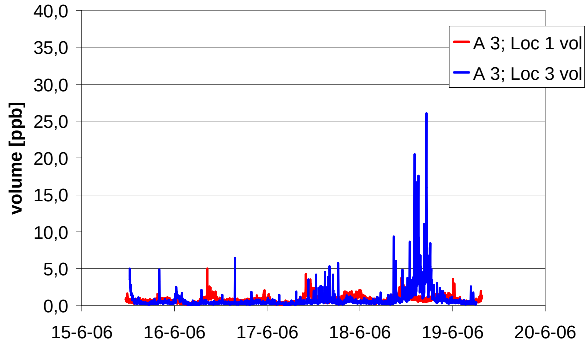


Fig 4-23 Particle volume Area 3, Loc 1 and 2 *Fig 4-24 Frequency dist Area 3, Loc 1 and 2*



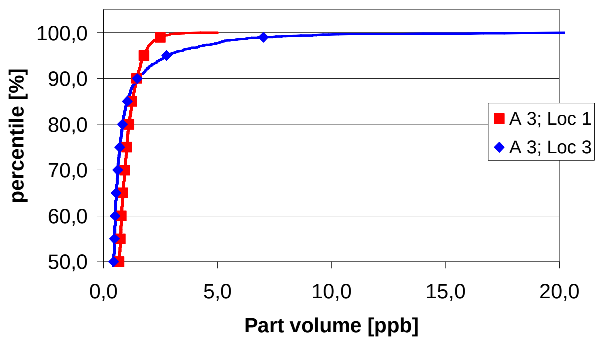
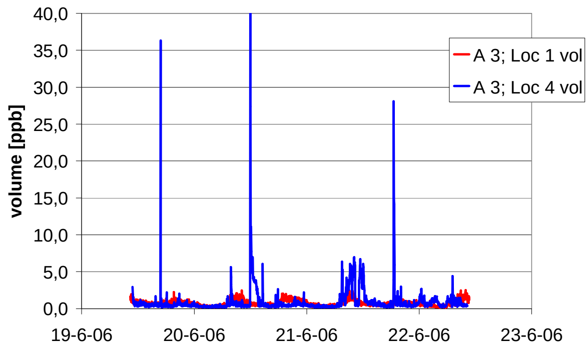


Fig 4-25 Particle volume Area 3, Loc 1 and 3 *Fig 4-26 Frequency dist Area 3, Loc 1 and 3*



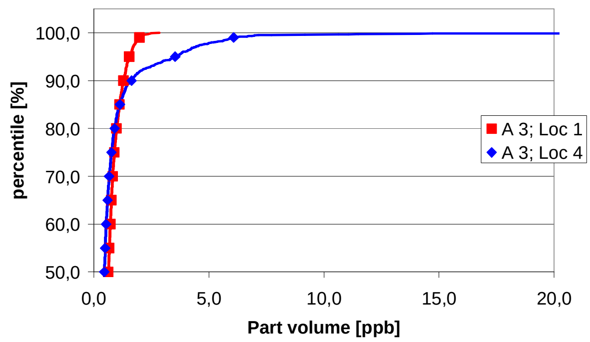


Fig 4-27 Particle volume Area 3, Loc 1 and 4 *Fig 4-28 Frequency dist Area 3, Loc 1 and 4*

Table 4-5 Frequency percentiles and curve characteristics Area 3

Frequency percentile [%]	A 3; Loc 1 [ppb]	A 3; Loc 2 [ppb]	A 3; Loc 1 [ppb]	A 3; Loc 3 [ppb]	A 3; Loc 1 [ppb]	A 3; Loc 4 [ppb]
90,0	19,35	18,37	1,46	1,48	1,29	1,65
95,0	22,30	25,11	1,79	2,77	1,54	3,54
98,0	24,83	35,94	2,16	5,19	1,82	5,20
99,0	26,41	41,46	2,49	7,02	1,99	6,08
99,5	27,33	48,55	2,81	9,35	2,14	7,16
99,9	29,67	62,79	3,61	17,58	2,46	27,39
Ratio 90/99,5	0,71	0,38	0,52	0,16	0,60	0,23
Average [ppb]	5,85	6,32	0,82	0,84	0,73	0,90
Surf-90%	60,7%	54,5%	76,6%	53,0%	77,8%	51,4%
Surf+90%	39,2%	45,2%	23,3%	46,9%	22,2%	48,6%

During the measuring period there were some extremely hot days. The total volume supplied by the pumping station that supplies the research area, amongst others, is presented in Fig 4-29, together with the measuring periods in the three areas. Also in the same graph the temperatures in the areas are given on the right axis (KNMI, 2006). There is a clear relationship between the temperature and the total volume supplied. As the temperature is extreme, also the daily volumes are extreme and concurrently the velocities and flows in the network. Especially during the measuring periods in Areas 2 and 1, the extremes are high.

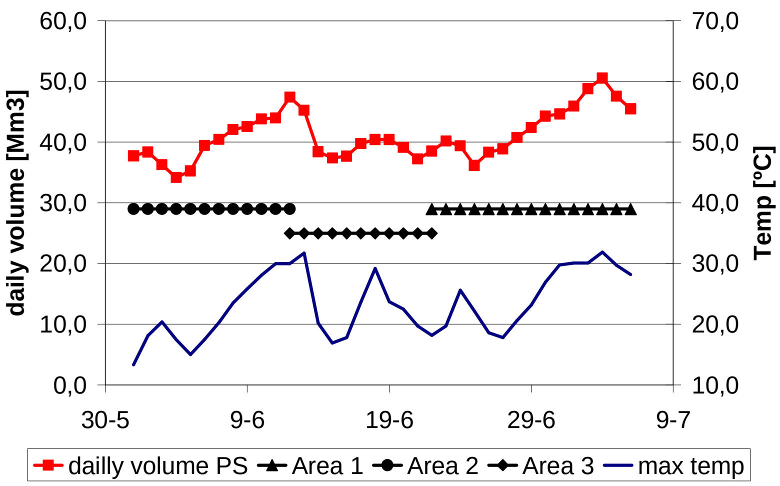


Fig 4-29 Daily volume supplied by the pumping station that supplies the Areas 1, 2 and 3 during the measuring periods. The measuring periods are indicated by the straight lines in the graphs, The daily volume is given on the left vertical axis and the maximum temperature on the right vertical axis.

4.6 The self cleaning effect

To illustrate the self cleaning effect in Area 2 and Area 3 and to distinguish them from Area 1 a further analysis is made of the frequency percentiles. A ratio is determined from the equivalent percentile values of the distribution frequencies of the locations that are simultaneously monitored.

$$\text{Ratio} = \frac{\text{X\% percentile value loc 1}}{\text{X\% percentile value loc Y}} \qquad \textit{Equation 5}$$

The ratio between the 20% to 99,5% percentiles are calculated for the incoming location (location 1) and the 20% to 99,5% percentiles at the other locations (Y= 2, 3 and 4) and graphically represented for Areas 1 and 2 in Fig 4-30 and for Area 3 in Fig 4-31.

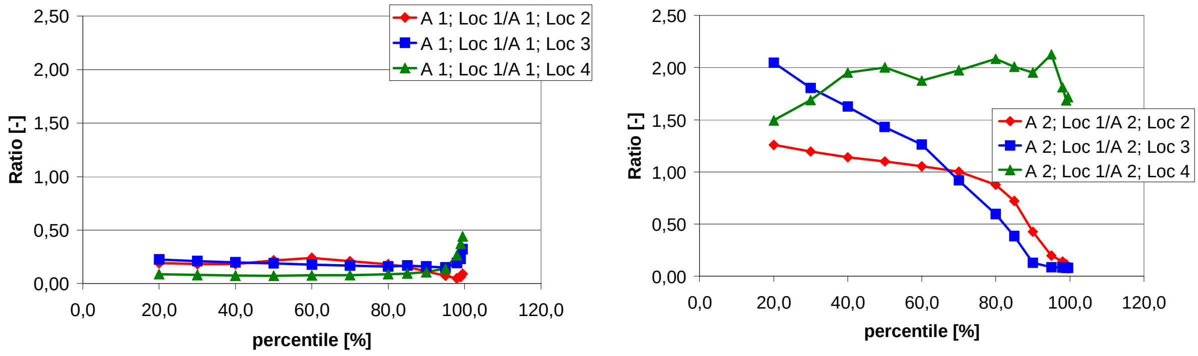


Fig 4-30 Left: Ratio percentiles Area 1(conventional DWDS); Right: Area 2 (1st generation High Velocity DWDS)

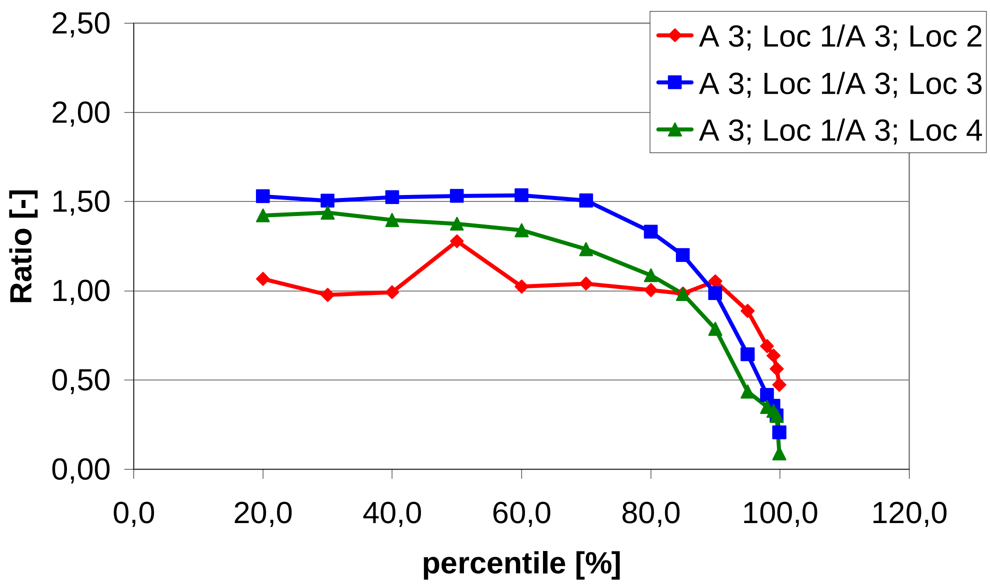


Fig 4-31 Ratio Percentiles Area 3: High velocity DWDS

These graphs show the ratio between the distribution curves and illustrate how the frequency distribution curves relate to each other. If the ratio is above 1 than the value of that particular percentile measurement of the 'incoming' water is higher value than that of the 'outgoing' water. When the ratio is below 1, then the outgoing value is higher than the incoming value. The ratio curves for all locations in Area 1 are completely below 1, indicating that the percentile values of the outgoing water are consistently higher than the percentile values of

the incoming water. This shows that over the whole measuring time there was a net removal of sediment.
The location 2 and 3 of Area 2 show a ratio curve that crosses the value of 1, indicating that in the lower percentiles there is a net settling of particles that are removed in the higher percentiles. This shows that over a part of the measuring period there was a net settling and over a part of the period there was a net removal of sediment.
Location 4 of Area 2 is constant above 1 indicating a net settling over the whole measuring period, which is explainable because that location is a true dead end.
For Area 3, the high velocity DWDS all curves cross the value of 1 in the higher percentiles and have in the lower percentiles values moderately to slightly above 1.

When the ratio curve crosses the value of 1 during the measuring period this indicates that a part of the period there was a net settling and a part of the period there was a net resuspension of sediment. This is the self cleaning effect, because the settling and resuspension are in balance over a period of a few days. During the measuring period of Area 1 there was an extreme situation towards the demand in the network. The measuring was done in the hottest period in the summer of 2006, which led to an extreme high demand (see Fig 4-29). The Area 1 was not cleaned for a long period so the conclusion is plausible that the area was saturated with sediment in such a way that even a small increase in demand resulted in a resuspension of sediment albeit not leading to turbidity on complaint level (>10 FTU).
The locations 2 and 3 of Area 2 can also be considered as true self-cleaning because the percentile ratio curve crosses the value of 1 during the measuring period. Though the network is constructed in 2002 and was not cleaned until the measuring in 2006, there was not so much sediment accumulated to show the same effect as the locations in Area 1. The network in these locations is also self-cleaning over the measuring period. Location 4 in Area 2 is a true dead end that even in the high demand situation sediment is accumulated. It also shows that the accumulation of sediment in a dead end does not have to lead to discolouration events because extreme velocities are almost impossible in the normal situation.
For all locations in Area 3 the ratio curve crosses the value of 1 during the measuring period, even in the extreme circumstances that are comparable to those during the measuring period in Area 1. This shows that the settling and resuspension are in balance during the measuring period of a few days and the complete network can be considered as self cleaning.

4.7 Discussion

Adding the velocity criterion to the design criteria leads to a DWDS that has a very different layout as a transport network: branched instead of looped. During workshops with designers to teach the new rules, this layout turned out to be a confusing point. In the conventional approach to DWDS there is no difference between the layout of the transport and the distribution system: both are looped. The looping in the transport system is mainly to ensure the reliability of the supply to larger areas. Failure of a transport pipe doesn’t affect individual connections directly, but it affects areas that are known as nodes in a network model with a concentration of connections. Failure of a distribution pipe, however, affects the connections directly and the looping doesn’t prevent the interruption of supply to the connections of the failing cut-off section.
The second confusion for network designers is that the branching of pipes leads to ‘dead ends’. However, the alleged dead ends in the new designs are in fact ‘flowing ends’ with a high velocity. The conventional dead ends are the extensions of pipes that are necessary for supplying fire demands to remote areas. A pipe with very little or no regular demand is extended to allow for a fire hydrant leading to a true dead end with water with very long residence times and no velocity. One of the pitfalls of the new design is the extension of

branched pipes to accommodate high fire flow demands; this can only be done if there is enough regular demand to ensure a high velocity in the pipe.

Theoretical design trials showed that the new design would potentially be 10 to 20% cheaper in investment costs. This motivated the water companies to initiate a discussion with the fire fighting departments to evaluate the conventional fire fighting demand of 60 m^3/h or more. Though the fire fighting demand has been reduced to 30 m^3/h for large areas, it is still one of the main issues in meetings with fire fighters. The new design made water companies aware of the role they should play in planning new building areas.

The measurements presented show that the hypothesised effect on water quality actually does occur and that the water quality benefits are realistic. It also shows that the detailing of the design is especially important. The first-generation network showed that the effects are easily compromised and that the rules should be strictly applied. Now that the networks are actually in place, it allows for the evaluation of the design criteria that leads to adjustments of the design rules. Several companies have adjusted the $q\sqrt{N}$ method of estimating maximum flows, because, as it turns out, actual flows are lower than estimated. This can be explained with the fact that the number of people living in a house has decreased while the number of tapping points has increased. As the method probably originates from a period in the '50s of last century, the simple square-root function is not accurate any more. Blokker developed the SIMDEUM model that shows good results in estimating the demand (Blokker et al., 2006). For a first setup, however, the $q\sqrt{N}$ -method is sufficient.
The water quality measurements in Area 3 showed that the self-cleaning effect is real and sufficient to keep the pipes clean. However, in reality the intended velocity of 0.4 m/s is probably not reached as indicated by some trials with the SIMDEUM model in this location and the experience of other flow measurements. The conclusion is that the self-cleaning effect is reached if the network is designed with an over-estimation of the flow, but also with a higher design velocity than required. It is the combination of the $q\sqrt{N}$ -method with the design velocity of 0,4 m/s that results in a self-cleaning network.
Adjusting the design velocity to a more realistic value also requires an adjustment of the demand estimation to the same level of realism. This requires a more sophisticated estimation method that can be offered in further development of the SIMDEUM model.

The discussion with the fire departments on the use of the drinking water network for supplying fire fighting water focussed attention on the more general aspects of fire safety in buildings, from a drinking water perspective. It turned out that residential sprinklers may well be a meaningful alternative, as was shown in Scottsdale, Arizona, USA (Ford, 1997). The challenge is to incorporate these new insights in fire safety into the new networks without affecting the velocity criteria. This is easier in areas with a high domestic demand. In Europe the household demand varies from 90 litres per person per day (lpppd) in the eastern part of Germany to over 200 lpppd in the south of Europe. When the basic per capita per day demand is higher, probably also the peak demand will be higher. That will result in higher peak volume flows and a possibility to apply larger pipes. In combination with a lower fire flow demand, more flexibility in the network is available to incorporate the hydrants but also to accommodate residential sprinkler systems. For that reason, the American networks, with individual demand of over 300 lpppd help the possibilities become evident.

The new design rules led to networks that have less length of pipes: loops are not closed anymore, leading to at least a 10% reduction in pipe length. Next to that, fewer valves are

necessary because of rational valve location (Trietsch and Vreeburg, 2006) and less material is used in the smaller pipes. In the Dutch situation with soft sandy soils, the application of smaller pipes also reduced the overall cost of pipe-laying because the material component in the total cost is considerable. In more complicated situations with rocky soils, the relative part of the material costs in the total cost can be less. For the Dutch situation the new rules lead to networks that turn out to be 20% cheaper in investment costs compared to conventional networks. The cost savings on maintenance through flushing are not considered, as they may vary locally depending on the water quality. If the water quality is good, i.e., has a low sediment load to the network, the maintenance costs are already low compared to networks that are supplied with a higher sediment load (Chapter 1).
An overall cut in construction costs by 20%, however, was a very strong motivator for the water companies to do pilot studies with the new design rules, which led to the degree of implementation that is proven by the statistical data on network materials (Fig 4-8).
The introduction of the design rules requires a determined effort by many parties. During introductory workshops at water companies, strong resistance was felt against the new rules, especially with the introduction of what was perceived as numerous dead ends. Because of the very complicated particle-related processes in the network, as explained earlier, pilot trials are crucial to investigate the effectiveness of the new approach. This circle of discussion can only be broken with pilot projects that show the possibilities of the approach and demonstrate the improved water quality and the saving in construction costs.

An important point of discussion during implementation of the new design rules in water companies is the uncertainty of the development in water demand in the area that is supplied with the new network. Commonly this uncertainty is covered by an over-dimensioning of the network: “just to be sure”. This, however, compromises the high velocity design rules in the very essence and should be avoided. This is possible is the extension is considered in more detail and analysed on how it would affect the branches that are typical for the high velocity network. Substantial extensions such as new streets and other new housing developments will lead to new branches that can be connected to the main structure. This main structure will mostly have some flexibility that allows for extra branch connection, because it is looped. The diameter is mostly 150 mm or higher and the capacity range of such a pipe is relatively high. Minor extensions such as a few new houses can lead to a problem when they can only be connected to a branch and especially when this is more at the end of the branch. In that case it is necessary to also enlarge pieces of the branch more upstream. In practice however, there is no data available on how often an extra capacity in the network is used within a period of ten to twenty years. During the implementation period with the Dutch water companies that started in 1999, the growing awareness of the influence of velocity on water quality has mostly led to the choice not to build redundant capacity but to extend networks when the expansion actually is needed and take the extra costs for the replacement of pieces of the existing network.

The costs saving, however, were not the original driver to establish new design rules, but the anticipated improvements of water quality and the reduction of the discolouration risk. In the chronological order, first the new design rules were applied resulting in high-velocity networks and only then was it possible to verify the effect. The comparative particle count measurements in the three different areas did show that the hypothesised effect of self cleaning could actually be observed. Although the circumstances in which the measurements took place were not representative for a longer period they did convincingly show the self cleaning effect. The extra presentation of the ratios of percentile values for the different locations are clear on the self cleaning effect.

The analysis of the effects on water quality were focussed on the accumulation of particles in the network and the effect on the aesthetical water quality. Biological quality, however, will probably also be improved, following the proposition to regard the network as a bioreactor (Huck and Gagnon, 2004). In this approach the contact time and contact area are important factors. With the new design the contact time will be decreased. Firstly because the volume of the network decreases, decreasing the average residence time in the network. Secondly because the flow direction is unidirectional avoiding stagnant water in the periphery of looped networks. Blokker showed that residence times in a simple 500 connections network can go up to 48 hours (Blokker et al., 2006). In a 500 connections newly designed network the average residence time is actually the average demand over the volume of the network resulting in residence times of maximal several hours.
The relative contact area in the newly designed network will increase linearly with a decreasing diameter. A decrease in diameter with a factor of 2 will double the contact area, but decrease the volume and consequently the residence time with a factor of 4. The overall effect will be that the biological regrowth processes will decrease with a positive effect on biological water quality.

4.8 Conclusions

Resuspension of accumulated particles in drinking water distribution systems (DWDS) is the main cause for customers complaints to the water company about the water quality. Preventing the particles from accumulating in these DWDS can be achieved by high velocities in the pipes.
The research on the effects of particle volume concentration in three different areas shows that the basic hypothesis for self-cleaning networks, that daily peak demands can be used to prevent particles from accumulating, is confirmed.
The high velocities during the peak demands resuspend the particles that were settled during the periods of low velocities on a daily base. In the conventional networks it turns out that, under high demand situations, also part of the sediment is removed, but that the level of accumulation is higher and a net accumulation still occurs.
The design method based on an empirically determined peak velocity of at least 0,4 m/s in combination with a demand estimation based on the $q\sqrt{N}$ -method was implemented in the Netherlands in 1999. In reality the actual self cleaning velocity is probably lower as well as the actual maximum demand. Implementation of a more realistic self cleaning velocity is therefore only effective if the demand estimation is brought to the same level of realism as is proposed in the SIMDEUM model (Blokker and Vreeburg, 2005).
Since the introduction of the new design rules a gradual change in the distribution of pipe diameters in new networks shows that the design method has been applied and has led to considerable savings in construction costs, and it is still sustaining sufficient fire flows that meet the modern requirements.
Cost calculations show that residential networks designed with the new design rules are around 20% cheaper than conventional networks, depending on local circumstances.

5 Cleaning of networks

5.1 Introduction

Aesthetic water quality problems, such as discoloured water, occur when loose sediments in the drinking water distribution system resuspend and reach the customer in concentrations that can be visually observed with the naked eye and subsequently may lead to complaints. One of the actions to prevent complaints is to limit the amount of resuspendable sediment in the network. This is done by regularly removing the layer of loose sediments in a network. The layer is built up by the processes shown in Chapter 1 (Fig 1-5). Usually these problems are dealt with by cleaning the networks using such techniques as unidirectional flushing, pigging or water/air scouring aimed at removing the accumulated sediments from the pipes (Antoun et al., 1999).

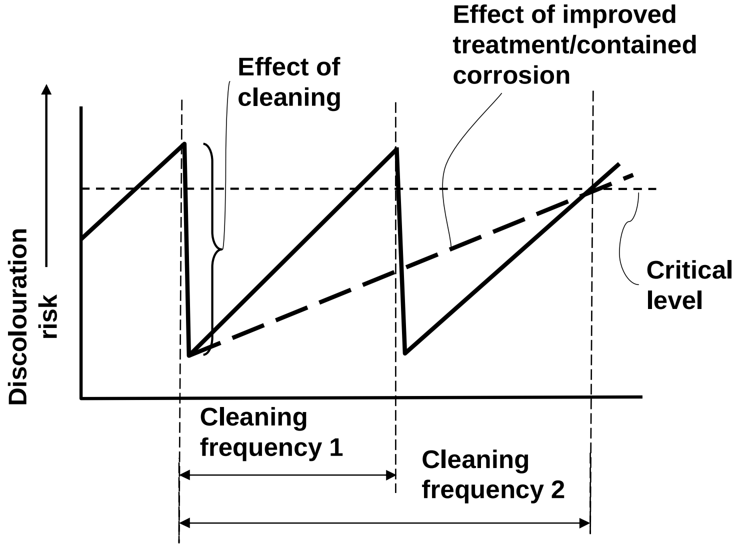


Fig 5-1 Cleaning frequency

If the sediment layer is removed, but the other particle-related processes as presented in Fig 1-5 are left unchanged, the build up will start again and cleaning will be necessary within a certain period of time. This is shown graphically in Fig 5-1, in which the vertical axis is the discolouration risk that can be determined with the RPM (paragraph 2.3.2) and the horizontal axis, the time. The effect of cleaning is such that the layer of sediment is removed and no or much less loose sediment is available for resuspension. The (re)charging of the system with particles continues, however, and after a while cleaning is necessary again. Limiting the recharging of the system by improved treatment, for instance, will result in a lower cleaning frequency (dotted line in Fig 5-1).

Though cleaning the network is a good method to manage the sediment layer in the network, it has a bad operational image. The reasons for this could well be the problems that are associated with conventional flushing as a customer inconvenience, seemingly a waste of water and sometimes adverse effects resulting in increased customer complaints. Without proper pre- and post-assessment of the need for cleaning and the effect of cleaning, it is easy to misinterpret the recurrence of complaints. This recurrence may be caused by insufficient cleaning leaving the discolouration risk at the same or only slightly lower level. It can also be caused by a rapid recharging due to insufficient treatment.

In this chapter the operational requirements and possible effects of several cleaning methods are described and discussed. The efficiency of the methods and the relevance of the operational requirements are illustrated by some experiments.
For analysis of the discolouration risk, the RPM was applied. In the earlier experiments only continuous turbidity measurements were used because at that time the RPM was not developed yet.

5.2 Cleaning methods

To clean pipes, several techniques are used that share the same historical development based on practical experience and a subjective appreciation of the results. Three methods are most commonly used in pressurised drinking water distribution systems:

- Flushing with water
  Water is flushed through a pipe with a certain high velocity. The increased sheer stress resuspends the loose sediments and removes them with the flushed water;
- Water/air scouring: Flushing with water and injected air
  Pressurised air is injected into the water flow causing extra turbulence and thus extra sheer stress to resuspend the loose material that is removed with the water;
- Pigging
  Soft or hard pigs with a diameter equal to or slightly larger than the diameter of the pipe to be cleaned are introduced into the pipe and pushed through the pipe with water pressure. The pig scrapes the loose sediment off the wall and carries it to the outlet. Often more than one pig is used.

Other methods, like high pressure jetting or mechanical scraping, are used in pipes that are taken out of service and are not pressurised. These methods are not widely applied to remove sediments on a regular base and thus fall out of the scope of this study that concentrates on practical methods.
The cleaning of networks involves skilled labour rather than scientific analysis. Despite this, however, there is little awareness of common operational conditions under which the methods are most effective. Extensive research incorporating a water company enquiry in the UK and USA (Friedman et al., 2003) learned that not all companies had a regularly scheduled flushing program: 20 out of 23 responses in the USA and 9 out of 15 responses in the UK responding in having one. Of those companies responding, 17 in the US and 5 in the UK evaluated the results of the flushing. There was also a wide range in how the cleaning programs were conducted. This confirms that there is little shared knowledge about the optimal operational conditions to get efficient removal of sediments.
Also in evaluating the removed deposits, as is done by various researchers (Gauthier et al., 2001; Zacheus et al., 2001; Torvinen et al., 2004; Barbeau et al., 2005; Carriere et al., 2005) there is no uniformity in sampling methods with respect to velocity and volume flow to obtain the deposits. The lack of insight in the operational requirements led to an underestimation of the complexity of flushing and a negative image of flushing, because of problems that resulted from what is called ‘conventional flushing’ (Antoun et al., 1997). The problems are the increased number of customer complaints during and immediately after the flushing and a minimal short-lived water quality benefit, but also a potential for increased coliform occurrences.

5.3 Water flushing

5.3.1 Introduction

Water flushing is the most common and longest applied method for cleaning networks (Antoun et al., 1999; Friedman et al., 2003). The principle of the method is that an increased

water flow causes an increased velocity in the pipe which leads to an additional sheer stress on the loose deposits in the pipes. These loose deposits are whirled up and removed by the flushed water. The extra flow is most commonly induced by opening a hydrant and blowing off the extra water. Despite the long history, there is little known about the operational requirements for effective flushing.
Conventional flushing is the approach used by most utilities in the UK and USA (Friedman et al., 2003). This was also the method mostly applied in the Netherlands until 1990. Conventional flushing is defined as opening hydrants in a specific area of the distribution system until pre-selected water quality criteria are met, mostly by a visual assessment of the turbidity. Though this seems to be a clear definition, in practice there is little uniformity in the application of conventional flushing. This makes that it hardly can be seen as a standard method. There are even examples that the end point of a flushing, expressed as a certain turbidity of the water, is reached by slowly decreasing the velocity of flushing by gradually closing the hydrant, even compromising the little operational criteria. This makes conventional flushing an ineffective method resulting in the problems mentioned in the previous section.
In the 1990s at several locations the conventional flushing was refined to unidirectional flushing (Oberoi, 1994; Slaats et al., 2002). Advantage of the unidirectional flushing is that a more clear operational guideline is applied that leaves less room for ambiguity. The basic characteristic of unidirectional flushing is that the flush flow is directed in one direction through manipulation of valves aimed at reaching a maximal velocity. Sometimes dedicated flushing points are applied instead of hydrants to reduce outflow resistance in the hydrants resulting in higher flows in the pipes. The benefit of the unidirectional flushing is that it requires careful planning to meet the operational parameter of unidirectional flow.

The intended effect of flushing is the increase in flow resulting in increased sheer stress that resuspends the sediments. For an effective flushing requirements must be set to the velocity and the amount of water flushed. Next to that also requirements should be set on the planning of the order of flushing to prevent recontamination of cleaned pipes. In other words: Water used for flushing should be conveyed of clean pipes. These requirements will be described in the following paragraphs and illustrated with experiments.

5.3.2 Minimum velocity

The value set as minimum velocity in several studies is 1,5 – 1,8 m/s (Brashear, 1998; Slaats et al., 2002). The reasoning behind this is partly the theoretical approach based on Shields' and Stokes' equations, though this leads to lower velocities than 1,5 m/s e.g. (Boxall et al., 2003). For the other part, the value of 1,5 m/s is based on practical experience that these velocities are significantly above the normal daily maximum velocities. With 1,5 m/s all the sediment that could cause discolouration would then be removed. Finally the velocity of 1,5 m/s is well attainable, taking into account a number of valve manipulations.
The flow resistance of a hydrant-standpipe-hose combination often limits the maximum capacity to 60 to 90 m^3/h. At this flow the resistance approaches the available head in the network. Typically, the hydrants are installed on 100 mm pipes and a 60 m^3/h flow pushes 2,1 m/s through those pipes. Normally the velocity in those pipes are in the order of a few centimetres per second (Blokker et al., 2006).

Apart from the theoretical approach looking at the possibilities of obtaining a velocity of 1,5 m/s, a practical approach is to see what happens if the velocity is not reached. The adverse effects of low velocity are best demonstrated by the negative effects of conventional flushing as mentioned by(Antoun et al., 1997): the increase of complaints after flushing. This effect can be explained with the results of turbidity measurements during an experiment with two flushing velocities (Fig 5-2). In this experiment in a 3 inch cast iron pipe first a flushing/disturbance was applied with a velocity of 0,4 m/s. The pipe was isolated by closing a valve near the hydrant that was located in such a way that a unidirectional flushing was caused. Turbidity was monitored with the Sigrist KT65, earlier described (Chapter 2) with a measuring frequency of 10 minutes. The experiment was performed in 1998.

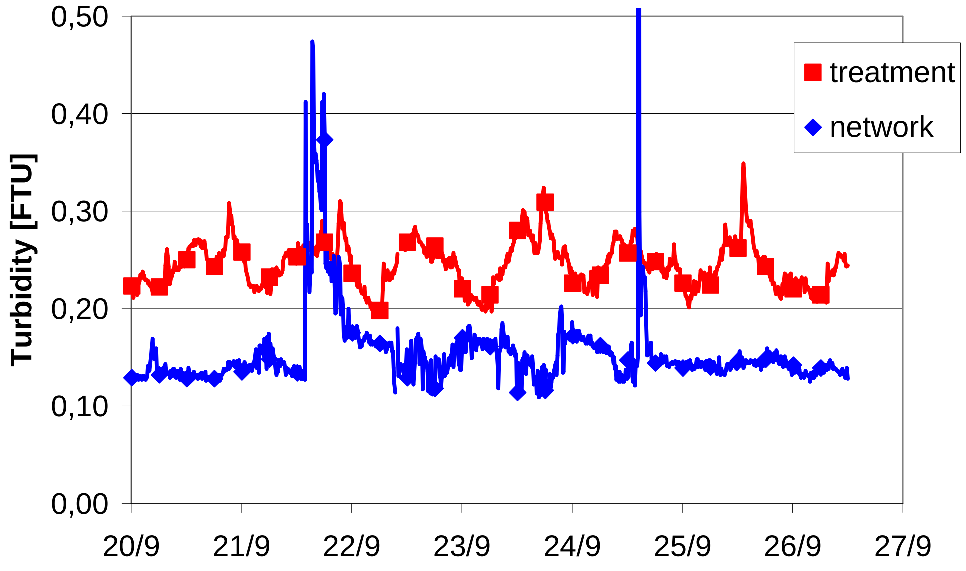


Fig 5-2 Influence of flushing velocity on sediment mobility; first flushing on 21/9 was with velocity less than 1,5 m/s (at least 0,4 m/s), second flushing with at least 1,5 m/s.

The monitoring started on 20 September and there was a distinct difference between the turbidity at the treatment plant and at the research location in the network. This difference indicated that the network was being loaded with particles. The turbidity at the monitoring location was stable until the disturbance/flushing on 21 September. The stability in turbidity indicated that the layer of sediment was not mobile under normal flow circumstances. In the early afternoon of 21 September, a disturbance/flushing took place with a flow of 6,5 m^3/h. Because the pipe was an old 3” unprotected cast iron pipe the velocity is as least 0,4 m/s, but presumably higher because the effective flow area can be decreased by corrosion products. The turbidity following the disturbance is higher than the turbidity prior to the disturbance. It took almost 24 hours before the turbidity resided again, though not to the starting level. After that, the turbidity pattern had more variation than the pattern before the disturbance. In the afternoon of 24 September the hydrant was opened again, but now intentionally causing a unidirectional flushing with a velocity of at least 1,5 m/s (flow is 27 m^3/h) under the same unidirectional conditions as the original flushing. The total amount flushed during the second flushing was three times the pipe content and the water was conveyed through clean pipes (a clear water front was used). The turbidity during the opening of the hydrant was high, but only for a short period. Directly after closing the hydrant, the turbidity dropped again to the initial level and stayed stable.
This indicates that the flushing/disturbance with a lower velocity mobilised the sediment without actually removing it completely. During the disturbance, though, a part of the sediment was removed. The flushing with high velocity removed the sediment instantaneously and limited the customer inconvenience resulting from the flushing to the actual flushing time.

These results lead to the conclusion that the velocity of 1,5 m/s is sufficient to remove the drinking water sediments in this case, while the lower velocity had an adverse effect. Though this is only one experiment, the results make the adverse effects of conventional flushing understandable: Too low velocity disturbs and mobilises the sediment without actually removing it.
The operational requirement for velocity in water flushing is set on 1,5 m/s amongst others based on this experiment. It must be noted that lower velocities could also effectively remove sediment, but with the experience gained over the years (see also section 5.3) it can be said that it is universally applicable for many network and treatment combinations. Moreover, the awareness that too low velocities can have adverse effects because of the mobilisation of the sediment, attributed to the acceptance of this requirement for minimum velocity for flushing.

5.3.3 Flushed volume

The materials that must be removed from the pipe are the loose particles. With a sufficient increase in velocity, the relevant particles will resuspend and be flushed out with the water flow. In principle all particles will be resuspended immediately after the sheer stress from the increased velocity is exercised. The depth of the particle layers is in the order of some micrometers or millimeters (Vreeburg et al., 2007) that would indicate that a scouring effect is only necessary during a short period. The velocity profile over the cross-section of a pipe with a turbulent flow is not linear. These two effects make that if just a bit more than the volume of the pipe is flushed no additional sediment would be removed and that the minimal flushed volume should be more than one time the volume of the pipe to be cleaned

An example of many experiments in pipes in the Netherlands is presented in Fig 5-3. It shows that the nature of the sediment removed was actually loose deposits that resuspend immediately and completely when the sheer stress was increased. Fig 5-3 shows the measurements of the turbidity of the flushed water during a flushing of a 400 mm AC pipe with a velocity of 1,5 m/s. The distance between the flushing point and the clear water front is 3600 meter and the flushing is unidirectional from the clear water front to the flushing point. Turbidity is measured with a Dr Lange Ultraturb described earlier with a measuring frequency of 1 minute. The theoretical first turnover was around 40 minutes, which was confirmed during the experiment with the sharp decay of turbidity. The exponential further decay can be explained by the non-linear velocity distribution: the lower turbidity is the mixing of the fluid layers closer to the wall that move slower than the bulk of the fluid.

The drop in turbidity during after ten minutes of flushing and gradual increase again after 25 minutes of flushing are probably caused by the irregular nature of the deposits in the pipe. The layer of sediment is not distributed evenly over the complete pipe. What causes this irregularity cannot be explained by the experimental results. It is speculated that this could be caused by the horizontal level profile of the pipe that can have a slight ‘top’ half way the pipe. The pipe is laid in a flat terrain, but underground level differences of e few tens of centimetres could have caused this.

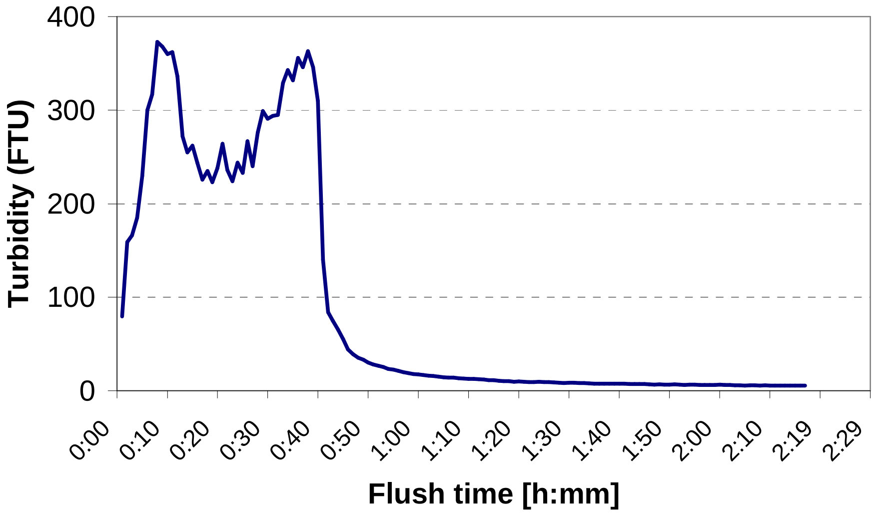


Fig 5-3 Turbidity measurements at the flushing point of a 300 mm AC pipe, length 3600 m, volume flow 680 m^3/h (1,5 m/s). Horizontal axis is the time elapsed after the start of the experiment

As said these kind of observations (a sharp drop in turbidity after one turnover of the pipe) is observed in many experiments of which this is one example. It leads to the second operational condition for water flushing to remove the sediment effectively that the flushed volume should be at least two times the content of the pipe to overcome the velocity dispersion. In the Netherlands the first set of operational requirements stated a refreshment rate of 3, but in practice it was sufficiently proven with measurements as presented in Fig 5-3 that two turnovers are enough as the water clears fairly well after two refreshments.

5.3.4 Clear water front

The turbidity trace in Fig 5-3 shows the effect when the water used for flushing itself is not clear enough. First, this clouds the effective end point of a flushing when this is defined in the form of a turbidity threshold. Carriere (2005) explicitly reports a stop criterion for flushing of 1 FTU, but that on several locations, values below 5 FTU could not be reached. Probably the effective removal of sediment from the pipe has stopped, but the water used for flushing has a turbidity of 5 FTU. The second effect is that with the flushing 'up-stream-sediment' is carried to the flushing location, already partly loading the pipe with sediment. A third effect is that the 'upstream-sediment' is resuspended with velocities lower than the threshold velocity of 1,5 m/s, with effects as shown in Fig 5-2.

This sets the third operational requirement to effective water flushing: the water used to flush the pipes should come from pipes with no resuspendable sediment. This concept is called "Working from a clear water front."

5.3.5 Discussion of water flushing

The effect of water flushing has long been underestimated because of the negative effects of the so-called conventional flushing. Much of the negative effects mentioned by (Antoun et al., 1997), such as an increased number of customer complaints during and immediately after implementation of flushing and a minimal, short-lived water quality benefit, can be explained when the operational requirements are considered, or rather the lack of operational requirements. The complaints during and immediately after the actual flushing could be caused by the low velocities that are the result of randomly opening fire hydrants. The effects of the increased velocity can be felt in the vicinity of the flushed hydrant, but also in more remote areas where the sediment is disturbed. The mobilisation of sediment, as shown in Fig 5-2, explains the complaints that are seemingly delayed. Unidirectional flushing is recognised as a good technique, but guidelines are mainly driven by "good management practices" (Friedman et al., 2002) to minimise the costs of a flushing program rather than to maximise the effect.

The operational requirements should all three be met for an effective cleaning of the network. Moreover, it forces the operators to make minute plans for the flushing involving valve exercising. Implementation of the three requirements gives a relatively simple framework to make flushing plans.

A widespread misunderstanding is that water flushing uses large amounts of water. However, a good flushing plan uses only at maximum three times the volume of the network. For example, in the Netherlands this volume can roughly be related to the yearly demand in a network. The total length of the network in the Netherlands is 110.000 km (Geudens, 2006). There are no sufficient data on the diameter distribution in the network, but a fair estimation would be an average diameter of 150 to 200 mm. The average daily consumption in the Netherlands is $3{,}0 \cdot 10^6$ m^3. This gives an average residence time of 15 to 27 hours. Three times the volume of the network is equal to 45 hours of average demand or 0,53 to 0,94% of the total yearly demand. A complete flushing program once every 3 years would add 0,18 to 0,31% to the Unaccounted For Water on a yearly basis.

5.4 Water/air scouring

5.4.1 Introduction

Water/air scouring or air scouring was developed in part because of the seemingly insufficient results of the conventional flushing programs. The method is based on injecting pressurised air into the water flow to create more turbulence and scouring stresses to resuspend the sediments (Fig 5-4). Another reason for developing this method was that more aggressive cleaning would not only remove mobile sediments, but also the more firmly attached voluminous corrosion products. The two-tiered goal in that case is not only the removal of loose deposits, but also the reinstatement of the hydraulic capacity. The claimed extra benefits of water/air scouring compared to conventional flushing are that it would take less water and the efficiency of sediment removal would be better.

Operational requirements are the ratio of air/water, the minimum flow of water and the air injection regime. The requirement of a clear water front is also applicable for water/air scouring. The required volume is determined by the flow needed to release the injected air out of the system. In the 1990s in the Netherlands the principle of water/air scouring was applied by several water companies, but varying operational criteria were used without an objective base. The definition of the best operational requirements was based on limitations of air compressors or on the appreciation of the strength of the outflow. One of the requirements, for instance, was that the injection of air should be done intermittently, which was based on the observation that a continuous air injection blocks the water flow. Later experiments learned that this blockage was caused by the pressure of air being too high, resulting in too great an air flow at the injection point as result of expansion, based on the Boyle-Gay-Lussac Law. The highly pressurised air expanded in the lower pressure in the pipe and blocked the water flow towards the flushing point. The solution was to limit the air pressure, which required adjusting the equipment.

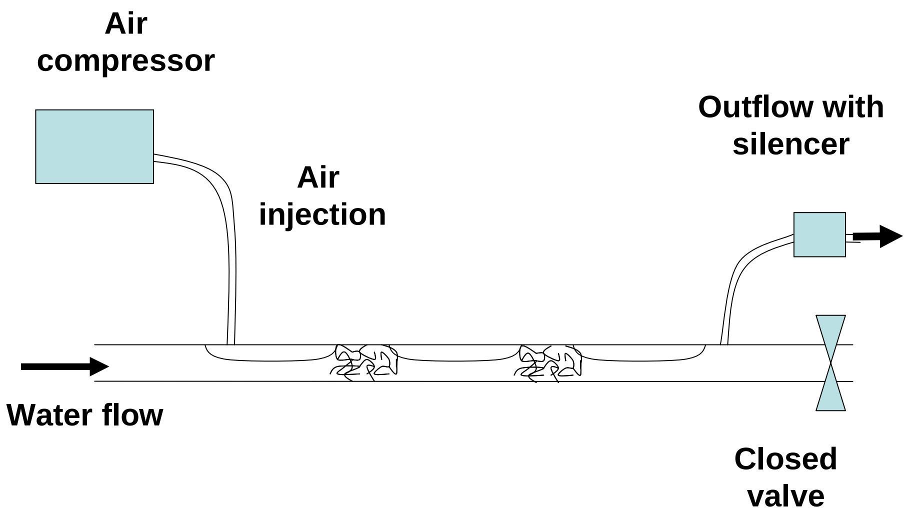


Fig 5-4 Principle of water/air scouring

5.4.2 Operational requirements of the experimental setup

In 1997 a test rig was build at the WL|Delft Hydraulics to determine the optimal operational requirements for water/air scouring. The setup of the experiment is sketched in Fig 5-5. The goal of the experiment was, firstly, to visually appreciate the phenomena that occur in the water/air mixture and, secondly, to determine the operational conditions that allow for an effective cleaning.

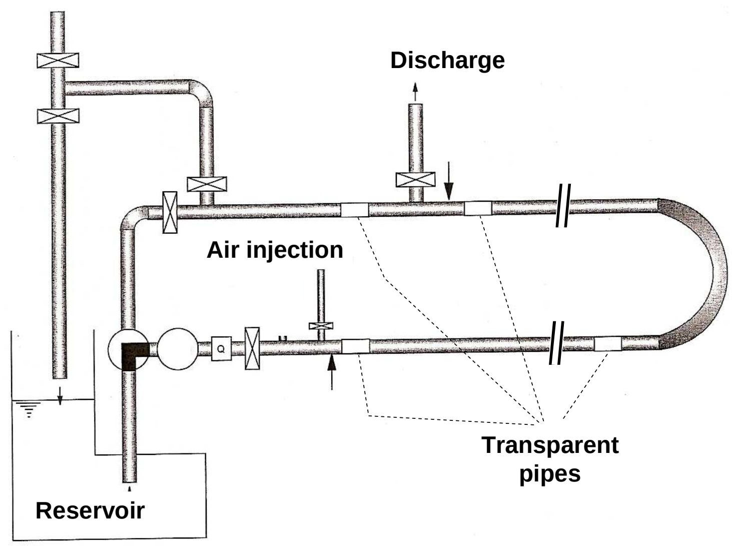


Fig 5-5 Experimental setup of water/air scouring test rig

Several experiments were conducted varying the air/water ratio, the initial velocity of the water and the injection pressure of the air (Pothof, 1998). For optimal operational conditions, the following boundaries were determined:

- Initial velocity of the water in the fully-filled pipe is 0,5 m/s
- The air/water flow ratio is 1:1
- The pressure of the air is 0,5 bar above the water pressure.

When these requirements are met, a stable system of large air bubbles separated by areas of very high turbulence is formed in the pipe. Directly downstream of the injection point, a stable flow of a half-filled pipe and a large air pocket is formed, as is shown in the video capture in Fig 5-6. (The camera view is in the direction from the discharge point towards the air injection point as shown in Fig 5-5.) In the left picture the upper transparent pipe element is just downstream of the injection point. The upper half of the pipe is filled with the injected air forming the first air pocket (Fig 5-4). The second, lower transparent part is at the end of the 400 meter pipe of the test rig (Fig 5-5) and shows the high turbulence area that can resuspend the sediment and which separates the two air pockets. These areas of high turbulence are baptised 'hydraulic pigs' referring to pigging as the cleaning method. They are formed at the end of the first air pocket at the injection point and propagate through the pipe. The right picture shows the same test run, but now in both locations are only the air pockets and the water. After the injection has stopped the pipe will be refilled, driving out the large pockets of air.

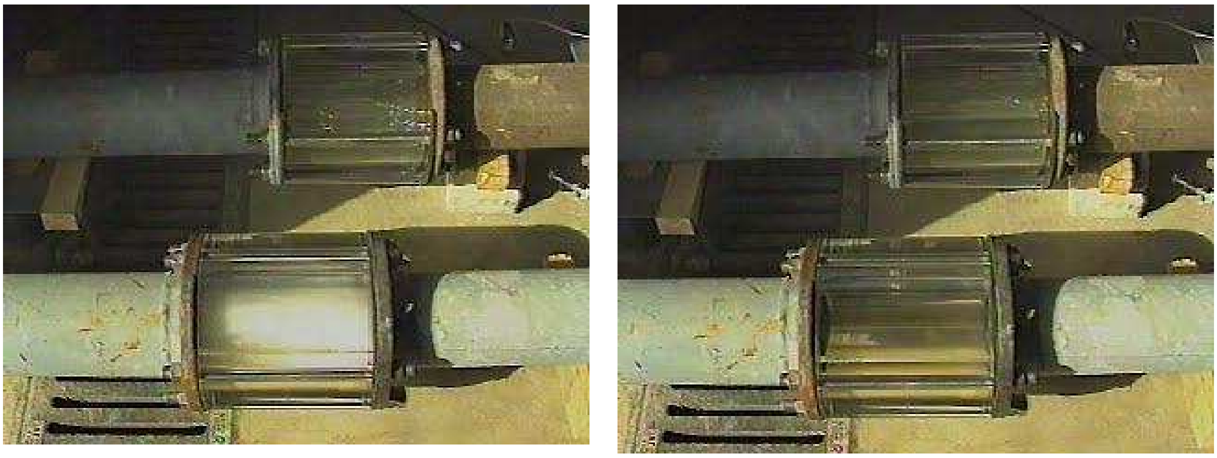


Fig 5-6 Video capture of air/water test rig

Based on the results of the test rig, the phenomenon of water/air scouring can be described as follows. An initial flow of 0,5 m/s is created by opening a flushing point. Air is injected at a preset flow and pressure resulting in an air/water flow ratio in the pipe of 1:1. In the pipe a relatively stable system of air pockets separated by highly turbulent areas is installed. The end point of a flushing action can also be determined with a turbidity assessment of the flushed water. The air in the water should be released before measuring the turbidity.
After stopping the injection of air, the pockets of air are flushed out of the system. The flushing point can be closed when air is no longer released from there.

5.4.3 Discussion of water/air scouring

The development of water/air scouring for cleaning networks was initiated by the unsatisfactory results of the conventional water flushing. The lack of water quality improvement by conventional flushing was one reason, but also the need to use less water was an argument to look at alternatives.
The momentary water flow needed for water/air scouring is obviously lower then the flow needed for water flushing, as the resuspension force is not the sheer stress of the water, but the areas of high turbulence between the larger air pockets. The total volume needed to clean the pipe is, however, at least two pipe volumes. The first pipe volume is removed from the pipe with the air and replaced by the air/water mixture. After stopping the air injection, the air has

to be removed from the system. The tests have proven that the removal of air is difficult with the low velocity of 0,5 m/s and air is probably left in the system. This rejects the claimed benefit of less water use with respect to water flushing, though it could be true with respect to conventional flushing.
The requirement of a clear water front is still recommended in this situation, but less stringent as in the case of water flushing. As the water flows here are lower, also the upstream disturbance is less, and the danger of unplanned disturbances and entrainment of upstream sediment is less.
Application of water/air scouring needs two access points: one to inject the air and the other one to release the water/air mixture. This means more disruption of the system. Moreover, the injection point also introduces a possible external contamination of the water. The air is compressed by a mechanical compressor and precautions should be taken to prevent oil or other contamination.
The claimed ability to remove more firmly attached corrosion products is not reliably measured in any experiment in this study. However the effects on turbidity are measured in experiments described later. From those experiments it can be concluded that the hard scales on corrosion products are damaged and probably at least partly removed.

5.5 Pigging or swabbing

5.5.1 Introduction

Pigging or swabbing is based on the introduction of a pig or swab, typically foam-like or plastic, in a pipe and pushing this through the pipe using water pressure. Pig originally stands for Pipeline Inspection Gauges and were first used in the oil and gas industry to clean and inspect pipe-lines. The squealing sound the abrasive pigs made also contributed to the naming of the pigs. The pig usually has a diameter larger than the diameter of the pipe, so there is a high sheer stress along the pipe wall scraping the loose deposits and attached products from it. The debris is pushed out of the pipe with the pig. The left side of Fig 5-7 shows some examples of pigs used in cleaning drinking water pipes. They come as soft sponge-like material, and also coated with a sealant preventing the pigs to get soaked or with additional swabs as seen on the large horizontal pig. On the right side of Fig 5-7, the dedicated outlet of a pigging action is shown.

Fig 5-7 Left, some examples of pigs used in pipe cleaning; right, the outlet of pigs with a dedicated flushing point

Pigging was developed as an alternative to conventional cleaning methods, with the aim of using less water and having a greater efficiency in removing semi-attached or cohesive layers. Sometimes the removal of biofilm is advertised as one of the benefits of the method.

Applying the pigs in encrusted cast iron pipes requires firm pigs. The soft pigs will get damaged and small parts of the pigs will stay in the pipes. That can lead to the clogging of water meters and taps.

5.5.2 Operational requirements

The application of pigging as a routinely applied cleaning method is mostly restricted to larger diameter pipes, typically above 300 mm. General operational requirements are not available but often tailored to the particular situation with regard to the number and type of the pigs. The common element is that usually several pigs are launched within short intervals and that, after the removal of the last pig, some time is allowed for refreshing the water in the pipe. There are no requirements for a set velocity of the pigs, which make the requirement for a clear water front less important. The total volume of water used is at least two volumes of the pipe: at least more than one volume is needed to let the pigs pass the pipe and at least one complete volume for refreshment after the last pig is removed.

5.5.3 Discussion pigging

Pigging is a costly method compared to the other methods. It requires a very careful operation, because the introduction of the pigs introduces a risk for contamination that is larger than the risk associated with water/air scouring . Pigs can be introduced under pressure with special equipment (Fig 5-8) though this requires special appliances to the pipes.

Fig 5-8 Launce installation for pigs in a pressurised pipe

The pigs themselves should be disinfected very well and are disposable, which adds to the total costs of the method. Because experimental data are lacking, it is difficult to give an opinion on the efficiency of the method. For removal of loose deposits it is not likely that the method is very effective, as the loose deposits are light and should be resuspended in a liquid phase. If the deposits are merely pushed forward the amount of material in front of the pig will probably soon become too large to be effectively transported and the pig can start rolling over the sediment. It could also occur that the deposits are pushed into the spaces between joints and other spaces. Visually, the method appears to be effective as the amount of material coming out of the pipe just in front of the pig is very concentrated (Fig 5-7).
Pigging in larger transport mains, above 300 mm, is sometimes applied as alternative for water flushing when the minimum velocity for flushing cannot be reached or if the volume flows associated with water flushing cannot be properly discharged of at the flushing point.

The cleaning efficacy of the pigs in that case is probably higher than flushing with too low velocity, but should be assessed with pre and post measuring of the RPM.
Another application of pigging is in charging new transport main with diameters larger than 300 mm. The pig is introduced in the empty pipe and pushed forward with the water and consequently filling the pipe. The pig is now not explicitly used to clean the pipe but to see if any large object is left in the pipe, which is not seldom the case.

5.6 Case study: water flushing

5.6.1 Introduction

Since the end of the 1990-s a number of water companies have adopted the planned unidirectional flushing as an adequate way of cleaning the network. A case study in the city of Venlo described in this section shows the results of such a program. The city of Venlo is part of the network of the Water Company Limburg and is supplied by the treatment plant Hooger Heide. The water quality data from this pumping station is given in Table 5-1, based on the reports in REWAB.

Table 5-1 Water quality data from treatment plant Hooger Heide (Source REWAB)

		2001			2002			2003		
parameter	Unity	avg	min	max	avg	min	max	Avg	min	max
Turbidity	FTE	0.48	0.07	2.40	0.31	0.06	1.40	0.25	0.11	0.63
pH	[-]	7.63	7.55	7.85	7.56	7.20	7.80	7.73	7.50	7.83
Hardness	mmol/	1.27	1.24	1.30	1.27	1.20	1.36	1.35	1.28	1.52
Iron	µg/l	53	30	170	30	20	40	23	10	40
Manganese	µg/l	<10	<10	<10	<10	<10	<10	<10	<10	<10
SI	[-]	-0.15	-0.15	-0.15	0.01	-0.16	0.28	0.04	-0.12	0.13
EC	mS/m				25.4	21.5	27.5	28.4	24.5	32.0

		2004			2005		
parameter	Unity	avg	min	max	avg	min	max
Turbidity	FTE	0.16	0.12	0.29	0.17	0.09	0.42
pH	[-]	7.78	7.56	7.88	7.70	7.58	7.85
Hardness	mmol/	1.35	1.34	1.37	1.44	1.34	1.61
Iron	µg/l	<20	<10	20	<10	<10	20
Manganese	µg/l	<10	<10	<10	<10	<10	<10
SI	[-]	0.11	0.04	0.18	0.04	-0.12	0.16
EC	mS/m	29.3	28.0	30.5	32.8	28.5	36.5

The water quality data shows that the treatment process based on traditional groundwater treatment through aeration followed by rapid sand filtration, has been working adequately when evaluated on average values. They all meet the Dutch water act standards. Starting over the course of 2003, the pumping station at Hooger Heid is supplying a mixture of water treated with the traditional groundwater treatment and drinking water from a new treatment plant Heel. The Heel-water is supplied directly into the clear water reservoir of the pumping station Hooger Heide through a separate transport system. This means that the original groundwater treatment plant has a lower production and that the deficit is supplied with 'better' drinking water.
From the clear water reservoir the water is pumped into the distribution system. The mixing improved the overall water quality significantly, as is shown by the average turbidity, iron and

manganese levels. This is especially expressed in the decreased maxima of the values; given the fact that extremes are only rarely spotted during a sampling program, this shows the improvement of the water quality.
The network is partly made of cast iron and steel and partly of asbestos cement as is shown in Fig 5-9. The network was cleaned during 2002 in several stages, starting with the larger pipes and progressing through the distribution network to be sure of the clear water front.

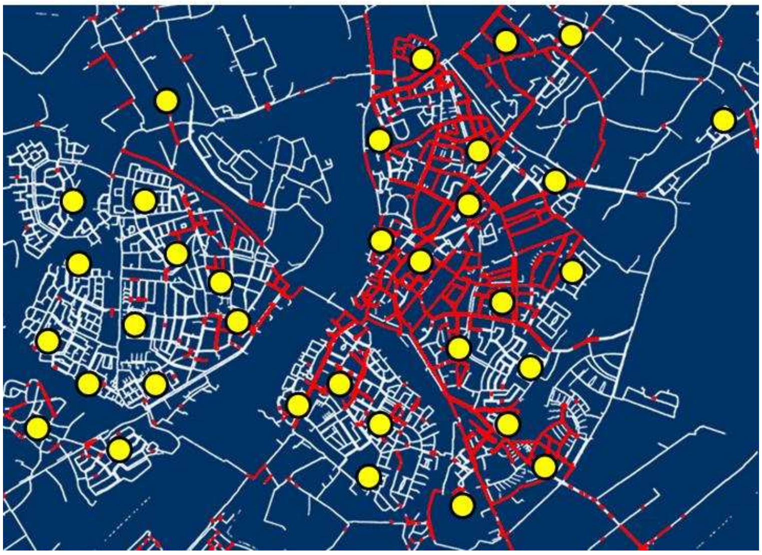


Fig 5-9 Network in the city of Venlo. The red lines are cast iron or steel, the blue lines are pipes made of non-ferrous material, primarily asbestos cement. The yellow circles represent the RPM-locations.

5.6.2 Materials and methods

Results of the cleaning were measured with an adjusted RPM procedure(Vreeburg et al., 2004a). In total 34 measuring points were selected (Fig 5-9), of which 15 were located in unlined cast iron and steel pipes and the other 19 in the other materials. The RPM procedure was adjusted for the length of the disturbance period: 5 minutes instead of 15 minutes. The main reason was that this shorter disturbance required a shorter length of isolated pipe, 105 meters instead of 315 m, which made it easier to process the actual measurement. Fewer valves had to be closed, and often it was sufficient to close only one valve in the neighbourhood of the hydrant used.
The monitoring of the resettling time was stopped after 30 minutes, which limited the overall time needed for the measurement. With this procedure it was possible to perform one measurement in one-and-a-half hours and to do 5 to 6 per day.
The measuring equipment was a Dr Lange Ultraturb turbidimeter built in a dedicated RPM tool that also had a transparent part to visually appreciate the turbidity of the water. Data were immediately processed in a laptop computer mounted in a special van (Fig 5-10).

The ranking table for the RPM was also adjusted: the maximum and average turbidity were ranked and so was the resettling time (Table 5-2) in 5 categories. The RPM ranking ranges from 0 to 12 points in this system. The boundaries for the RPM ranking are chosen in such a way that the ranking is sensitive in the lower ranges.

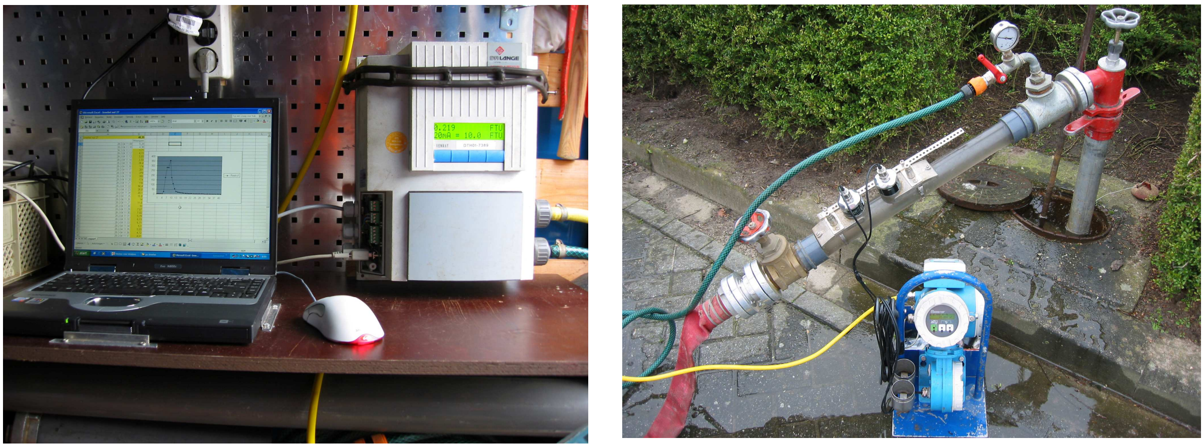


Fig 5-10 Measurement equipment for RPM

Table 5-2 Ranking table with adjusted RPM-ranking Venlo

Points	0	1	2	3	4
Max 5 minutes [FTU]	<2	2-10	10-25	25-50	>50
Average 5 minutes [FTU]	<2	2-10	10-25	25-50	>50
Resettling time [min]	<5	5-15	15-25	25-30	>30

Not all 34 locations were monitored regularly. At 28 location the RPM's were measured prior to cleaning and 29 locations also post-cleaning, enabling the evaluation of the efficacy of the cleaning. To follow the refouling of the system, during three years (2002, 2003 and 2004) all locations were monitored, in 2005 18 locations, and in 2006 only three locations. The main reasons for the decreasing number of measurements were due to financial and planning constraints.

5.6.3 Results

One example of the actual turbidity measurements during the RPM procedure in an AC pipe shows the basic information; represented in RPM, the picture is clearer and shows the typical saw tooth that is hypothesised (Fig 5-11).

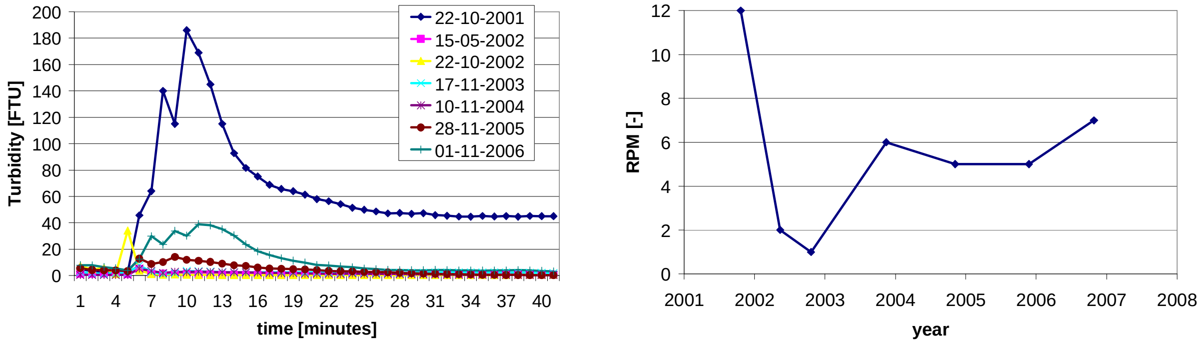


Fig 5-11 Left: Turbidity data for an AC location in Venlo; Every line represents a RPM experiment repeated on a yearly base. Right the same turbidity data translated into RPM-ranking

The individual data show that the cleaning was effective. The RPM dropped from the maximum of 12 in this case to a value of 2 and even to 1 after a short while. It seems that the recharging of the system was relatively quick, but that it stabilised afterwards, though with an increase again towards the end of the measuring period.

Similar data can be shown for a location on a cast iron pipe (Fig 5-12). Compared to the AC location, the cleaning at the CI-location was not as effective as it was in the AC location and also the recharging of the system seemed to have a more constant and higher rate.

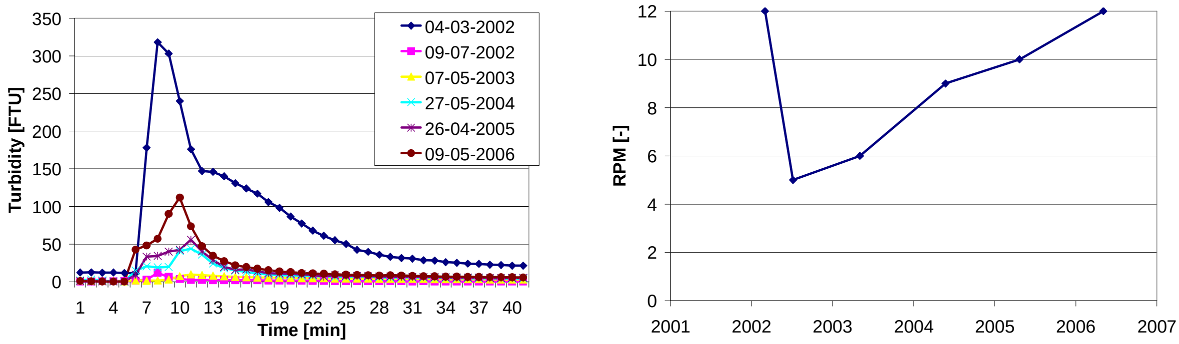


Fig 5-12 Left: Turbidity data for a CI location in Venlo; Every line represents a RPM experiment repeated on a yearly base. Right the same data translated into RPM-ranking

The individual cases showed some variation and are not always as clear cut as the examples shown. A reason for this could be that the locations are not varied, but every RPM is measured at the same location. The cleaning effect of the RPM in itself (see also section 2.3) interferes with the outcomes, probably causing the RPM even to decline if the period between two measurements is too close to each other. When the locations are averaged, however, the overall picture is consistent with the hypothesised pattern. The average of the RPMs (Fig 5-13) has the saw tooth shape, suggesting a cleaning frequency for this area once every four to five years. The error bars in Fig 5-13 are one standard deviation large, showing the increase in variation of measurements as the time progresses. The small standard deviation for the last measurement can be explained with the fact that this only concerns three measurements. The data on the measurements are also tabulated in Table 5-3.

Table 5-3 RPM-data on cleaning with water flushing

	date	RPM	date	RPM	date	RPM	date	RPM	date	RPM	date	RPM	date	RPM
Loc 1 non-Fe	11-10-01	11	08-05-02	2	22-10-02	4	17-11-03	3	17-11-04	2	28-11-05	3	01-11-06	9
Loc 2 CI	06-08-01	11	08-05-02	5	22-10-02	3	17-11-03	7	17-11-04	9	11-11-05	9	01-11-06	7
Loc 3 non-Fe	22-10-01	12	15-05-02	2	22-10-02	1	17-11-03	6	10-11-04	5	28-11-05	5	01-11-06	7
Loc 4 non-Fe	12-07-02	11	16-10-02	3	16-05-03	6	27-05-04	11	26-04-05	10	09-05-06	8		
Loc 5 non-Fe	08-11-01	12	22-08-03	2	09-03-04	0	09-03-05	5	30-03-06	6				
Loc 6 non-Fe	28-06-02	9	12-07-02	6	16-05-03	6	27-05-04	6	13-05-05	9	09-05-06	4		
Loc 7 CI	04-03-02	12	09-07-02	5	07-05-03	6	27-05-04	9	26-04-05	10	09-05-06	12		
Loc 8 CI	08-11-01	10	09-07-02	5	07-05-03	2	27-05-04	3	26-04-05	4	10-05-06	3		
Loc 9 non-Fe	06-11-01	10	14-10-02	3	19-05-03	6	27-05-04	7	26-04-05	9	09-05-06	12		
Loc 10 non-Fe	06-11-01	11	15-10-02	3	19-05-03	8	27-05-04	7	26-04-05	7	09-05-06	7		
Loc 11 CI	06-08-02	12	16-10-02	3	19-05-03	3	28-05-04	10	13-05-05	7	10-05-06	9		
Loc 12 CI	05-11-01	11	13-11-02	3	02-06-03	7	27-05-04	9	13-05-05	12	09-05-06	12		
Loc 13 non-Fe	06-08-02	12	22-11-02	0	02-06-03	4	28-05-04	8	13-05-05	11	10-05-06	12		
Loc 14 CI	05-11-01	12	03-06-03	8	19-12-03	7	03-12-04	11	29-11-05	9	30-11-06	3		
Loc 15 CI	20-11-01	8	04-06-03	4	19-12-03	9	03-12-04	11	28-11-05	9	29-12-06	12		
Loc 16 CI					26-01-04	11	03-12-04	10	28-11-05	12	29-12-06	7		
Loc 17 non-Fe	06-08-02	11	29-11-02	3	21-10-03	4	06-12-04	4	28-11-05	3	30-11-06	11		
Loc 18 CI	06-08-01	12	22-05-03	2	19-12-03	8	03-12-04	11	29-11-05	12	30-11-06	12		
Loc 19 CI	01-11-01	11	22-05-03	6	18-12-03	8	03-12-04	8	29-11-05	6	29-11-06	5		
Loc 20 CI	24-02-03	10	23-09-03	5	09-03-04	5	09-03-05	5	30-03-06	6				
Loc 21 CI	07-08-02	12	20-06-03	10	02-02-04	12	09-03-05	12	06-02-06	12				
Loc 22 non-Fe	14-11-02	8	08-01-04	1	30-06-04	5	11-07-05	4	18-07-06	6				
Loc 23 non-Fe	23-10-01	9	21-08-03	8	02-02-04	5	09-03-05	8	06-02-06	2				
Loc 24 CI	27-11-01	12	30-10-03	0	28-05-04	5	18-05-05	4	17-07-06	9				
Loc 25 non-Fe	06-08-01	8	11-09-03	3	08-03-04	3	14-03-05	4	10-03-06	3				
Loc 26 non-Fe					26-01-04	9	09-03-05	7	10-03-06	7				
Loc 27 non-Fe					27-01-04	5	14-03-05	8	30-03-06	6				
Loc 28 CI	08-10-01	11	19-12-04	1	02-06-04	8	08-06-05	7	17-07-06	8				
Loc 29 non-Fe			02-12-03	4	02-06-04	3	08-06-05	8	17-10-06	8				
Loc 30 non-Fe			02-12-03	6	02-06-04	3	21-10-05	2	17-07-06	11				
Loc 31 non-Fe	20-11-01	10	08-01-04	2	30-06-04	5	11-07-05	3	17-07-06	10				
Loc 32 non-Fe	30-10-01	12	11-09-03	4	09-03-04	6	09-03-05	2	30-03-06	3				
Loc 33 non-Fe	30-10-01	12	21-10-03	1	02-06-04	0	08-06-05	6	18-07-06	11				
Loc 34 non-Fe			27-01-04	4	30-06-04	5	11-07-05	5	18-07-06	8				
Avg Cast Iron/Steel	14-01-02	11,0	02-05-03	4,2	08-11-03	6,6	07-11-04	8,0	06-11-05	9,0	07-08-06	8,4	01-11-06	7,0
Avg non-ferrous	02-02-02	10,6	13-04-03	3,2	11-11-03	4,4	03-12-04	5,8	02-12-05	6,7	24-04-06	7,8	01-11-06	8,0
Average RPM	24-01-02	10,8	22-04-03	3,7	10-11-03	5,4	22-11-04	6,8	21-11-05	7,7	21-06-06	8,1	01-11-06	7,7

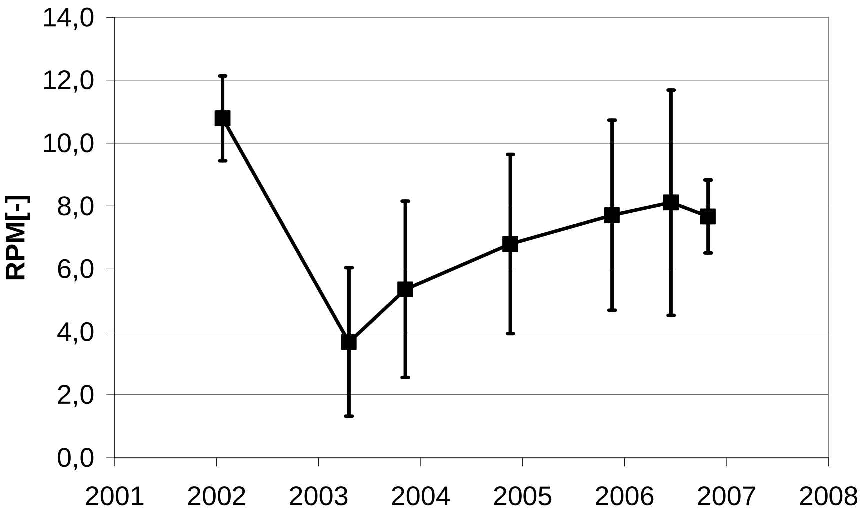


Fig 5-13 Averaged RPM's at 34 locations pre- and post-cleaning with water flushing. Error bars at one standard deviation

By further dividing into the material categories of cast iron/steel and non-ferrous the RPM reveals more discerning, as is shown in Fig 5-14. Considering the averages the cast iron network was not cleaned as thoroughly as the non-ferrous material and seems to recharge more rapidly. However, this was only in the beginning of the post-cleaning period while the recharging rate in the period 2004-2006 was the same. The last three values in 2006 should be interpreted with some care, because they are based on a lower number of actual samples, for example, the last cast-iron RPM value was based on one measurement only.

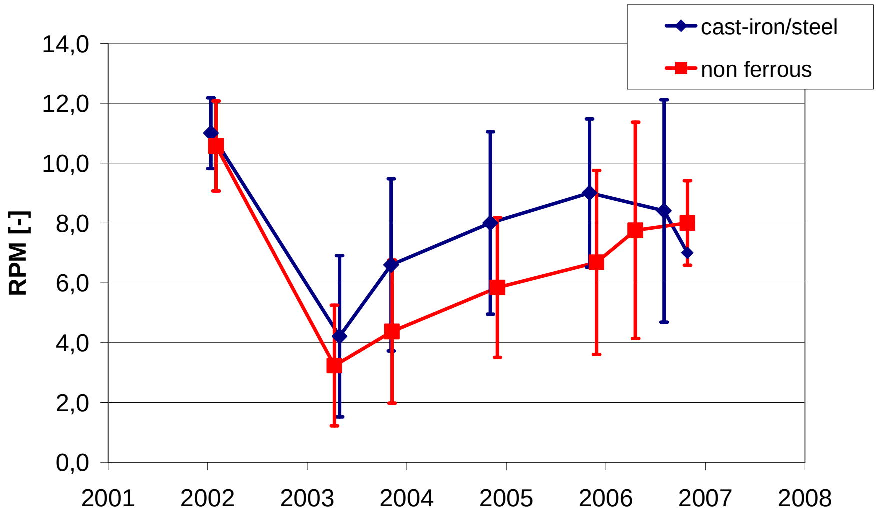


Fig 5-14 Average RPM's, separated into ferrous and non-ferrous pipe materials; error bars set at one standard deviation

The cast iron and the non-ferrous material deteriorate in the same rate, though the increase in RPM in the beginning is somewhat different. This shows that the resuspendable sediment does not originate from the cast iron exclusively. It could be argued that the more rapid increase in RPM in the cast iron network is caused by a combination of corrosion and loading from the treatment plant. This cannot be confirmed because the material causing the RPM is not analysed. Probably that would not have rendered much more information because in both cases a form of iron is the dominant element. After the initial corrosion this process has stabilised again and the deterioration is primarily caused by particles from the treatment. (see also section 3.5). This leads to the observation that cleaning of cast iron initially (re)starts the corrosion process because probably the protective scale of corrosion products is damaged. After the corrosion scale is stabilised again the dominant charging process is the particle load from the treated water.

5.6.4 Discussion

Flushing with water reduced the RPM significantly in this demonstration case. Also the recharging of the system is clearly shown in the increase of the RPM value. Because of the adjusted RPM, the information on the actual turbidity associated with discolouration risk is somewhat limited, but the overall effects are clear enough. Though the initial RPM pre-cleaning is almost at maximal value, the effect of the absolute maximal turbidity is even more illustrative. The average of the maximum turbidity during the disturbance pre-cleaning is almost 300 FTU, while the average of the maximum turbidity during the disturbance post-cleaning is 13.2 FTU (Table 5-4). The last measurement in 2005 gives a value of 74 FTU for the average maximum turbidity.

Table 5-4 Turbidity values for RPM measurements

	24/1/ 2002	22/04/ 2003	10/11/ 2003	22/11/ 2004	21/11/ 2005
Av. Max turb. [FTU]	297,2	13,2	26,1	43,4	73,6
Av. Max turb CI [FTU]	346,1	14,2	44,2	60,2	121,2
Av. Max turb non-CI [FTU]	248,3	12,3	11,9	30,1	36,0
Av. av. Turb. [FTU]	181,1	6,6	16,3	25,5	43,9
Av. av. turb CI [FTU]	212,3	9,0	28,5	38,2	73,7
Av. av. turb non-CI [FTU]	149,9	4,7	6,7	15,4	20,4

For post-cleaning, the average max turbidity decreased to 13,2 FTU, but especially the average turbidity has dropped to 6,6 and that is well below the visibility threshold of 10 FTU (Slaats et al., 2002). If only the non-ferrous material is considered, the value drops even lower to 4,7 FTU.

With this case study, the effect of cleaning in ferrous materials was also disclosed. The efficacy of the cleaning was marginally less than in non-ferrous materials, which is plausible looking at the inner surfaces of a corroded cast iron pipe (Fig 5-15). Hydraulic scouring will be influenced by the irregular surface of the layer of corrosion products. In a smooth pipe the loose deposits will be more easily resuspended than on a rough surface. This effect is showed for the different materials in the post-cleaning averaged RPM (Fig 5-14). The recharging of the pipes with sediment leading to increased RPMs was of the same nature, though it looked, in the beginning, that the increase in RPM was more rapid. This rapid increase can be caused by the sediments that were not effectively removed but were remobilised due to the lower velocities in the irregular surface "holes". The effective velocity in the shadows of those "dunes" was probably less than 1,5 m/s, causing the mobilisation of the particles (Fig 5-2). Another plausible explanation is that the corrosion process was re-initiated because of (mild) damage of the protective hard scale of the corrosion products.
The effect that the measuring of the RPM has on the amount of sediment because some of it is removed with the disturbance of the velocity leads to an underestimation of the actual RPM. In the comparison in the particular case study this is of little importance because the locations were all measured with proximally the same intervals.

5.7 Case study on aggressive cleaning of cast iron pipes through pigging

5.7.1 Introduction

Unlined cast iron pipes are historically suspected to be the main cause of discolouration. Consequently, cleaning these pipes has received a lot of attention. Cleaning a cast iron pipe, however, is a delicate procedure. The inside of an encrusted cast iron pipe, as is shown in Fig 5-15, showing that corrosion products can block the hydraulically-effective cross-section of a pipe. The build up of a layer of corrosion products has been studied by several authors. A comprehensive description of the corrosion scale layer is given by (Sarin et al., 2004). Iron corrosion scales consist of porous deposits, with a dense shell-like layer near the top of the scale enveloping a soft porous core. The porous core has a high percentage of ferrous phases (e.g. $Fe(OH)_2$ and $Fe\ CO_3$), which can readily dissolve to produce large quantities of Fe^{2+} which can be oxidised to Fe^{3+} and flocculate to small iron hydroxide particles. In the presence of carbonates, the Fe^{2+} can also form $FeCO_3$ that adds to the hard layer confining the soft deposits and protects it against further corrosion (Rostschutzschild or protective layer). The capacity of the scale to release iron ions is much less than the porous media. Damage of the scale will increase the release of iron consequently leading to elevated levels of iron and also turbidity. When the water has sufficient carbonates the protective layer will repair itself in due time because of the formation of $FeCO_3$.

Fig 5-15 Picture of the inside of a 100-year-old cast iron pipe

Cleaning cast iron pipes has the danger of damaging the relatively hard protective scale. Should that happen, the stabilised corrosion process starts again or is at least intensified.

5.7.2 Experimental setup

In the cast iron network in the city of Tilburg a part of that network was cleaned using pigs. The cleaning of the pipes was done using several kinds of hard foam pigs. At least three types of pigs were used for each line.
In the cast iron network 5 locations were selected at which turbidity was continuously monitored and RPM was determined (Fig 5-16).

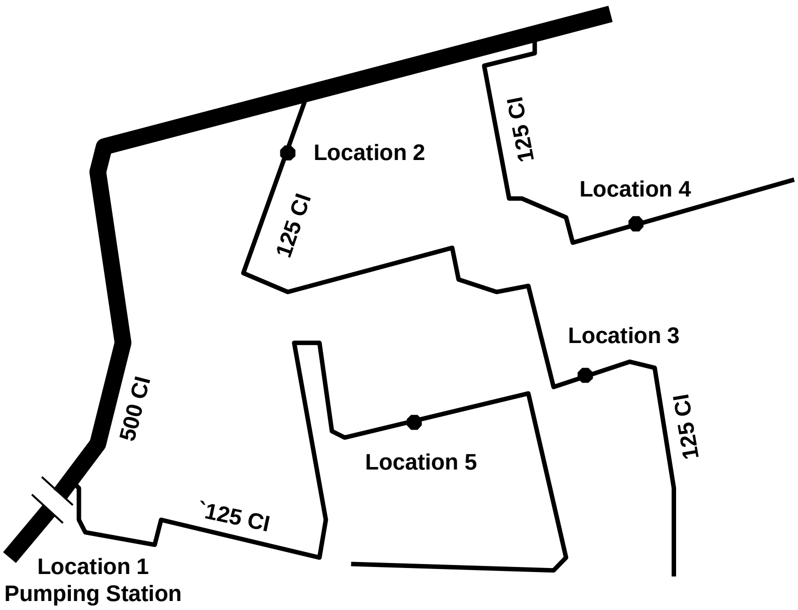


Fig 5-16 Monitoring locations in cast iron network Tilburg

At location 1 the turbidity at the treatment plant was recorded. Location 2 was close to the main transport system at which the possible changes in turbidity between the treatment and this location were recorded. Both locations were used to monitor the incoming water in the sense of the particle-related processes model. Locations 3, 4 and 5 recorded the changes in turbidity after the transport through the cast iron pipes.

5.7.3 Materials and methods

Turbidity was measured using a Sigrist KT65 turbidity monitor. This is a white light turbidimeter that measures the scattering of light at a 90^o-angle. The monitor does not work following the ISO-7027 standard, but was chosen for its robustness and capability to be used in an experimental environment.

Pre cleaning in the period 26 April – 3 May 2000 the turbidity was monitored at all locations; on 2 May the RPM was determined at locations 2, 3, 4 and 5. Post-cleaning, the measurements were repeated in the period 13-20 June 2000 with RPM at 15 June 2000. Approximately a year after cleaning, the measurements were repeated from 1-8 May with RPM on 4 May 2001 and identified as second post cleaning.

The ranking table for the RPM is given in Table 5-5. This ranking is quite different from the other ranking used in this chapter (Table 5-2). The relatively low values are connected to the Sigrist equipment that records relatively low values. A number larger than 2,3 FTU could be any value because it would be above the threshold of the equipment, which was set at 2,4 FTU.

Table 5-5 Ranking table RPM with Sigrist KT65

	0	1	2	3
Absolute max first 5 min	<0,5	0,5-1,5	1,5-2,3	>2,3
Average first 5 min	<0,5	0,5-1,5	1,5-2,3	>2,3
Absolute max last ten min	<0,5	0,5-1,5	1,5-2,3	>2,3
Average max last ten min	<0,5	0,5-1,5	1,5-2,3	>2,3
Time to clear	< 15 min.	15-30 min	30-45 min	>45 min

5.7.4 Results

The results of the Resuspension Potential Method are summarised in Table 5-6

Table 5-6 Results RPM analysis

Location	Pre-analysis					tot	Post-analysis					tot	Post-2 analysis					tot
Loc 2	0	0	0	0	3	3	0	0	0	0	0	0	0	0	0	0	0	0
Loc 3	3	2	2	1	3	11	3	2	3	1	0	9	1	1	1	1	1	5
Loc 4	3	2	3	1	3	12	3	2	1	1	0	7	1	1	1	1	0	4
Loc 5	1	1	3	1	3	9	3	2	1	1	3	10	2	1	1	0	3	7
					Average	8,8					Average	6,5					Average	4

From these results one sees that the need for cleaning was not very high because the average RPM was 8,8, though the average was clouded by location 2 that hardly had any resuspendable sediment. The effect of cleaning on the other three relevant locations that are more in the heart of the network (3, 4 and 5) was not large. The average RPM dropped a few points, but after a year the turbidity dropped even further.

The turbidity measurements during the pre-cleaning period are presented in Fig 5-17.

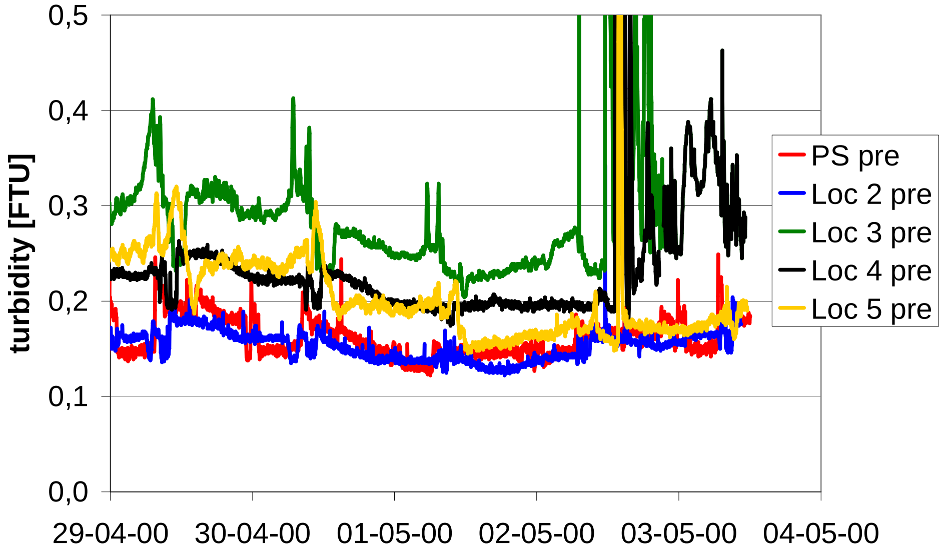


Fig 5-17 Turbidity measurements during pre-cleaning period; on 2 May the RPM is determined

The turbidity increased in the cast iron network. The irregularities on 2 May were caused by the RPM measuring. For the post-cleaning periods, the results are presented in Fig 5-18 with, on the left, the turbidity of location 2 as the incoming water and location 3 as a trajectory. The right graph shows the results from location 2 in combination with locations 4 and 5 and is on a larger scale to show all the relevant values.

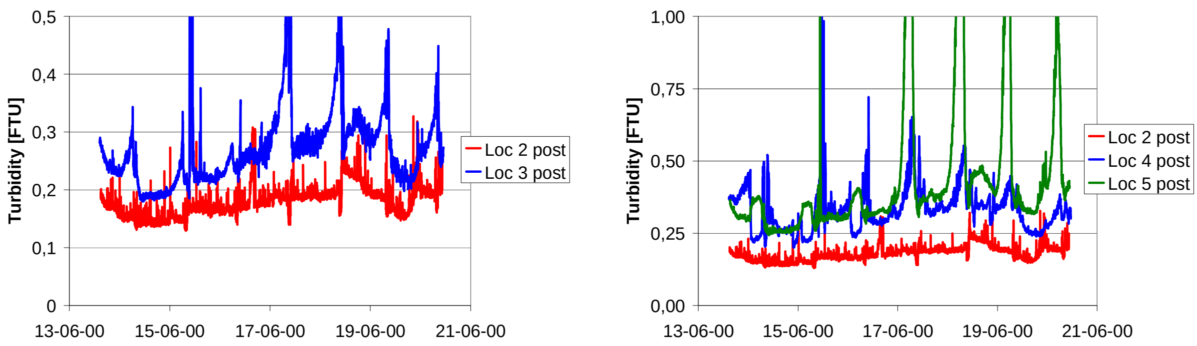


Fig 5-18 Turbidity post-cleaning; location 2 serves as incoming water; in all the locations the typical corrosion pattern can be observed

Locations 3, 4 and 5 showed a turbidity pattern that was not present in the pre-cleaning period and was also not recognisable in location 2. This changed pattern indicates that the particles that caused the turbidity were produced within the pipe by corrosion. This pattern can be explained by an active corrosion process. The process produced particles with a constant rate but during the night hours the residence time of the water was longer, allowing more time for the particles to be picked up. During the day, when residence time decreased, the time for picking up particles was less and the turbidity dropped accordingly.

The results for the second post-measurement, almost a year after the actual cleaning, are represented in Fig 5-19.

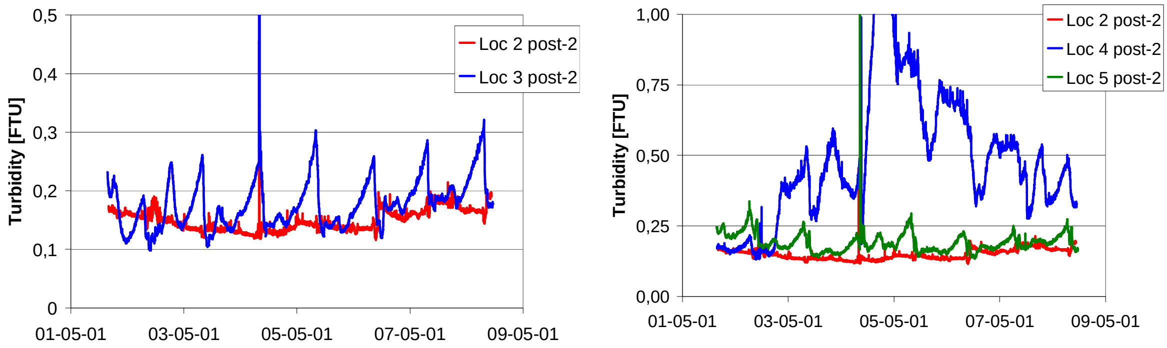


Fig 5-19 Turbidity second period post-cleaning; location 2 serves as incoming water

The regular patterns were still recognisable, indicating that the process had not stabilised yet. Location 4 showed a relatively high turbidity pattern that cannot be explained.

5.7.5 Discussion

The RPM results show that cleaning with pigs was not very effective in this case. The average RPM value dropped, but not very much, and after a year the average RPM level dropped without an active cleaning program. What this continued drop in RPM was caused by is not clear. The negative effect of damaging the stable layer caused the turbidity to follow a certain pattern, which is consistent with an active corrosion process. This means that a particle production process had been initiated. The production process, however, was not enough to contribute to a layer of loose sediment, as the average RPM value did not increase but rather decreased.

In fact, several phenomena were disclosed with this measuring campaign.

- An active corrosion process can be measured with the help of continuous turbidity measurements. The pattern is very typical with a slow rise in turbidity during times of stagnation, usually the night hours in the network with a household-dominated demand. The turbidity drops sharply with an increase in the velocity in the morning hours as a result of the increased demand in that period.
- The particles produced by the active corrosion process don’t necessarily contribute locally to layers of loose sediment.
- Pigging damages the stable and hard layer on top of the corrosion products as described by Sarin et al, (2004) and will take some time to stabilise again.

5.8 Discussion of cleaning methods

5.8.1 General

Cleaning networks is the main reactive measure network operators take when confronted with discolouration events. Historically, hydrant flushing or conventional flushing without any guidance of flow, was the only means that was available and that did not work well. With knowledge of the role of particles in discolouration and the importance of velocity, it is now clear that conventional flushing cannot work properly and can even have an adverse effect. However, if only appreciating the colour of the water being flushed with the conventional procedure, it seems as if the flushing procedure is effective in removing sediments. This shows the persistent misconception that the efficacy of a cleaning method can be judged by appreciating the water being flushed. The conclusion that conventional water flushing is not effective is valid, though.

The ensuing search for more aggressive and more effective methods to clean pipes was therefore aimed at the wrong criteria. However, it logically led to procedures such as water/air scouring and pigging. Water/air scouring produced a lot of violence at the outlet point and measures had to be taken to prevent damage resulting from darting hoses. Pigging produced very dirty water (Fig 5-7) in relatively short plugs. The effect of these methods were judged conventionally on the basis of the colour of the water that was flushed out and not so much on the total amount of sediment removed or, more importantly, on the amount and mobility of sediment that was left behind.

Introduction of the RPM as a measuring method enabled an unbiased pre- and post-assessment of the discolouration risk and made it clear that the efficacy of any cleaning method is better appreciated by analysis of the sediment that was left behind instead of analysis of sediment removed. The analogy used was that you cannot see how clean a table gets if you are underneath it and can only see the dirt that falls off during cleaning.

Anecdotal in this aspect is the setup of the test rig for water/air scouring. The setup of the test rig was motivated by the desire to put an end to the discussion of what was actually the hydraulic process in the pipe that was responsible for the cleaning. The standing practice was to inject air intermittently while operating the compressor at full throttle. The test rig (Fig 5-6) disclosed the actual process that required a constant air injection aimed at the water flow with only a mildly above-normal pressure. That was more or less contradictory to the standing practice and effectively changed the working procedure, but it also led several companies to leave the water/air scouring method and proceed to the planned unidirectional flushing as a more effective method.

For pigging, two key projects played an important role that demonstrated less an inefficiency of the method but more the importance of understanding the fouling process. The first was the disappointing long-term effect of an extensive pigging project. A relatively large area was cleaned up to the distribution pipes using a clear water principle and a pigging procedure that allowed multiple pigs to clean the pipes. Obviously this was a very costly project that involved a lot of late night work to avoid customer disruption as much as possible. After a year, however, complaints started again and an analysis began to more closely examine the cause of the recurrent complaints. It turned out that the particle load at the treatment plant was the major source of particles and that the pigging was effective in removing the sediments, but that the cleaning frequency was about a year. (The picture of the black bath used in the introduction was actually taken in that supply area.) Regular cleaning with planned and unidirectional water flushing turned out to be a much more cost-effective way of managing the discolouration problem, and it also started an evaluation on how to improve the performance of the treatment plant.

The second key project was the evaluation of cleaning trunk mains with pigs by a water company. Turbidity was monitored at several locations pre- and post-cleaning, but RPMs were not possible. At that time the RPM was not often tested and confidence in the method was not high. Besides that, there were time constraints. Based on the turbidity measurements, an argument could be made that only one out of three cleaning actions was effective. Closer analysis showed that the pig in that case was moved through the network at a velocity close to 1,2 m/s. The conclusion was that the actual cleaning was done by the high water velocity rather than the scouring of the pig.

A number of projects that were done within consultancy constraints showed that alternative methods to planned unidirectional water flushing were more costly and not more effective.

5.8.2 Cleaning of cast iron

In both cases cleaning of unlined, old cast iron or steel pipes showed to be complicated. In both cases it is plausible that post cleaning the corrosion process is activated, resulting in production of particles. More aggressive cleaning using higher velocities, water/air scouring or even pigging showed that these procedures probably damage the inner hard scale on the corrosion products, reactivating the corrosion process and releasing the contained layers of soft deposits. The reactivation leads to an increased internal production of particles measurable through increased turbidity. This clearly indicates the operational dilemma for the cleaning of cast iron pipes. Cleaning with water flushing has a positive effect and recharging to the initial level of RPM directly after cleaning more rapid than with other materials, but this effect stabilises after one to two years. More aggressive cleaning leads initially to a better effect, but reactivates the corrosion process measurable in increased levels of turbidity and releases soft deposits contained by the corrosion scales thereby affecting both the pipe wall and the water quality.

5.9 Conclusion regarding cleaning methods

Cleaning is the most appropriate short-term response to discolouration problems. The available methods are water flushing, water/air scouring and pigging.
Water flushing is the most effective method when strict operational requirements are met for velocity, flushed volume and order: 1,5 m/s, 2 to 3 pipe volumes and a clear water front.
RPM is very suitable to evaluate the need for and result of cleaning when applied pre- and post-cleaning.
Cleaning cast iron pipes has an initial deteriorating effect on the corrosion that is difficult to prevent.

6 Particle composition and hydraulic behaviour: case studies

6.1 Introduction

Particle composition and hydraulic behaviour vary across different drinking water networks. The particle composition at any given place in the network depends on the initial composition at the treatment plant, the net result of particle-related processes in the network as presented in Fig 1-5 and the residence time in the network. New particles can be produced by chemical and biological processes or flocculated from smaller particles to larger ones. The hydraulic behaviour that transports the particles is dependant of the size and density of the particles.

A drinking water network can roughly be separated into a transport and a distribution network (see also chapter 1). The hydraulic circumstances in terms of flow velocity and shear stress differ in transport systems and in distribution systems because of the difference in function between the two.
The function of a transport system is to convey relatively large quantities of water from one place to another without actually supplying water directly to connections on the pipes. Often transport pipes are connecting lines between production plants and reservoirs or distribution systems. The velocities in these transport pipes don't vary on a minutes base and are relatively high, up to 0,5 to 1,0 m/s occurring on a daily base.
The function of a distribution network is to actually supply the water to the installations directly connected to the pipes. The demands and subsequently the velocities in these pipes vary much more. In conventional networks the absolute velocities are low (Blokker and Vreeburg, 2005; Blokker et al., 2006).
In practice it is difficult to make a clear cut distinction between the transport and distribution system as they are mostly interconnected. Sometimes the transport pipes are part of a separate dedicated transport system, which enhances the distinction, but in the Netherlands mostly the transport system and the distribution system are integrated.

In this chapter three case studies are described that explore the composition and hydraulic behaviour of particles in drinking water distribution systems. Within the case studies two new analysis methods are explored to characterise the composition and behaviour of the particles. One approach explores the feasibility of a pre-concentration technique to analyse the composition of very low concentrations of particles normally occurring in drinking water. It is called the Time Integrated Large Volume Sampling (TILVS). The other approach aims at a hydraulic characterisation of sediments flushed out of distribution systems using a standard Jar Test Equipment (JTE).

The first two cases deal with typical transport systems. One of them is a low pressure transport system supplied with water with low particle content originating from water abstracted from an artificially recharged aquifer. The total transport distance was in total 23 kilometres. This transport system is used to feed a balancing reservoir and subsequently the flow was rather constant and the variation in velocities was low. In this case study the new TILVS methodology has been tested.
The other case also dealt with a large transport system, but now incorporated within a distribution system. This means that the pressure is high and that flow is more dependent on normal demand because there are no balancing reservoirs within the system. The flow situation is thus less clearly defined resulting in more variation in velocity than in the low pressure dedicated system. The treatment process of the water is based on a surface water

treatment that just prior to the measurements was extended with a UV-disinfection and a granular activated carbon filtration. The particle load of this surface treatment is also assumed to be low, using these additional steps.

The last case study deals with a dedicated project to identify the hydraulic characteristics of drinking water sediments using the standard Jar Test Equipment. The hypothesis is that sediments can be characterised using the hydraulic capabilities in a JTE experiment. Within the case study sediments from several treatment plants are analysed that include samples from the distribution network that is supplied with the water from the aforementioned low pressure transport system. Information about the hydraulic characteristics obtained from the JTE experiments in combination with the elemental composition of the sediments and their probable origin will enable more dedicated measures to prevent and control discolouration problems.

6.2 Case1: Low pressure transport system

6.2.1 Introduction

In the low pressure transport system of Amsterdam Water Supply a case study was performed on the behaviour of particles. The goals of the study were to gain more understanding of the behaviour of particles in water with a presumed low particle load in a large transportation system and to evaluate the method of TILVS in combination with particle counters (Chapter 2).
With respect to the particle-related processes (see Fig 1-5) in this case the particle load of the water should be very low because the final treatment step is slow sand filtration. Theoretically this water type could be compared with water treated with ultra membrane filtration with regard to the particle load. Corrosion of cast iron is excluded because the transport system did not have unlined cast iron but was made of cement mortar lined steel and concrete pipes dating from the early 1960-s. The condition of the lining is unknown, but there is no indication that it is not in good condition. Only appliances such as valves are made of cast iron, but do not contribute significantly to the total length of the system.

6.2.2 Material and methods

Characteristics of the Amsterdam water supply system
For the city of Amsterdam potable water treatment occurs at two locations, Leiduin and Weesperkarspel. This case study deals with the experimental results obtained from the low pressure transport system that connects the Leiduin treatment plant to the main distribution pumping stations in the city of Amsterdam.
Drinking water from the Leiduin treatment plant has undergone a number of treatment steps. The source water is surface water from the river Rhine that is pre-treated by flocculation with iron chloride, settling in open basins, followed by rapid sand filtration. Then the pre-treated water is transported over 60 km to a dune area where it is infiltrated. After an average of three months' residence time in the dunes the water is abstracted via wells and canals and transported in an open system to the Leiduin treatment plant. The infiltration in the dune area turns the water into groundwater or at least donates it groundwater characteristics such as a constant temperature, anaerobic water and low dissolved iron and manganese. At the Leiduin treatment plant the water is aerated, rapid sand filtered, ozonated, softened, filtered over activated carbon and finally slow sand filtered. Caustic soda is used for the softening. The distributed water has a very good quality (Table 6-1). Though on average the AOC level is

higher than the recommended 10 µg/l (van der Kooij, 1992) the water can be qualified as biological stable and is according to Dutch tradition distributed without a disinfectant residual.

Table 6-1 Water quality analyses at treatment plant Leiduin (REWAB, 2003)

Parameter	Unit	Min	Average	Max
Turbidity	NTU	< 0.1	0.12	0.36
Temperature	ºC	1.5	13.5	25.1
pH	-	7.96	8.37	8.69
SI	-	0.3	0.48	0.7
Dissolved Oxygen	mg/L	5.9	9.1	12.4
DOC	mg/L	0.9	1.1	1.4
AOC	µg/L	4.2	13	30
Aeromonas (30 ºC)	n/100 mL	0.5	21.2	1100
Chloride	mg/L	75	93	112
Iron	mg/L	<0.01	<0.01	0.13
Manganese	mg/L	0.000	0.000	0.002
Aluminium	µg/L	< 3	3.7	4
Magnesium	mg/L	8.8	9.9	10.9
Calcium	mg/L	39	43	47
Copper	µg/L	0.3	0.9	2
Barium	µg/L	13.8	16.3	18.6
Zinc	µg/L	0	0.13	0.7

Fig 6-1 represents the transport system from Leiduin to the two main distribution pumping stations at Haarlemmerweg and Amstelveenseweg. From the Leiduin treatment plant several main lines transport the drinking water to the western side of the city of Amsterdam. Most water goes through the Haarlemmermeer intersection. Afterwards the main splits into two lines: Amstelveenseweg North and Amstelveenseweg South before reaching the Amstelveenseweg storage reservoir (Fig 6-1). The North line is somewhat shorter than the South line and has part of its supply diverted to the distribution system at the booster station at Osdorp. The transport mains that are monitored are made of concrete and cement mortar lined steel and the total distance between Leiduin and Amstelveenseweg is about 23 km. The system is operated with gradual velocity changes to feed the clear water balancing reservoirs at Haarlemmerweg and Amstelveenseweg. Sampling taps at all locations were directly connected to the transportation mains.

At Amstelveenseweg two sampling points were available: one at the North line and one at the South line. During the first period of measuring the particle counter was connected to the South line. For the particle counting during the TILVS measuring at location Amstelveenseweg the particle counter was connected to the sampling tap at either the North or the South location.

Fig 6-1 Map of the sampling sites used in the study. Leiduin is the treatment plant location. Sampling also occurred at Haarlemmermeer and Amstelveenseweg on both the North and South lines. The distance between Leiduin and Amstelveenseweg is 23 km.

Sampling strategy and analytical methods

Two TILVS units (see paragraph 2.4.2) were deployed at the Leiduin treatment plant in parallel and ran in conjunction with a particle counter for 10 days. The experimental setup of the TILVS in combination with the particle counter is drawn schematically in the right of Fig 6-2. The TILVS units were then moved to Amstelveenseweg reservoir and run separately on the North and South lines for 5 days. Both units were run with a particle counter. The units were then run simultaneously on the South line followed by the North line so that replicate filter samples could be collected for further chemical analysis. One sample was for elemental analysis while the second sample was for volatile suspended solids analysis (VSS).

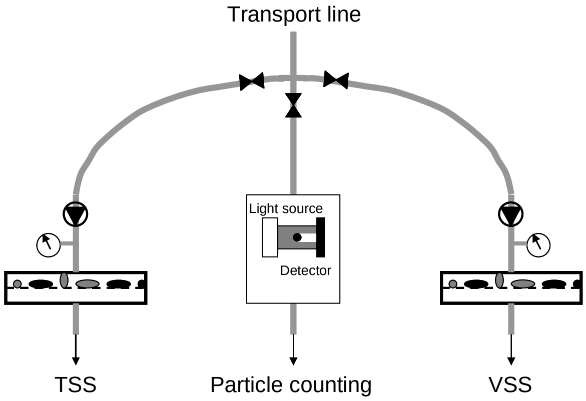


Fig 6-2 Left photo: TILVS-unit. Right drawing: Schematics of the experimental TILVS-PC setup used.

The particle counters used were Met One PCX units with 32 channels of one µm band width ranging from 1 to >31 µm. The particle counters were calibrated at the same time and ran simultaneously in the laboratory and at the Leiduin location to compare the magnitude of the signal from each unit. The results were in reasonable agreement showing peaks at the same time and having an average difference less than 10 % in total particle counts.

The TILVS filters were analysed gravimetrically for total suspended solids (TSS) after drying in a 105ºC oven and cooling in a desiccator. Any samples with a mass below 2.2 mg were

excluded from gravimetric and further analysis and considered to be below the detection limits.
In the test case the two filters were 'harvested' from each run. One of them was used to determine the Volatile Suspended Solids by burning the filter at 450^{o}C (APHA, 1998) and weighing the ashes. The second filter was used for the gravimetric analysis followed by a destruction and ICPMS analysis for all the metals. In this way the kations of the sediments were determined and by presuming their most likely anions a mass balance of the sediment was made. Inorganic analysis was undertaken by the Kiwa laboratory at Nieuwegein and involved microwave digestion and analysis of the digest filtrate. The samples were digested using 4 ml of 65% HNO_3 (Merck Suprapur) made up to 100 ml using metal-free water and digested for 30 minutes. ICP-MS was used to analyse for K, Mg, Ca, Si, Mn, Al, Cu, Ba and Zn. Iron was analysed by flame-AAS. Four filter blank samples were also analysed. Insolubles were negligible.
Eventually only four duplicate samples were fit for further analysis. Other samples were not viable due to a malfunction or leakage in the filtration units or too low mass on the filters. During the tests several modifications were made to the filter unit and the overall operation of the equipment.

6.2.3 Results

Particle counting
The calculated particle volume and frequency distribution of the volumes (see paragraph 2.2.5) is presented in Fig 6-3. The percentile ratios and other curve characteristics are summarised in Table 6-2 and Fig 6-4.

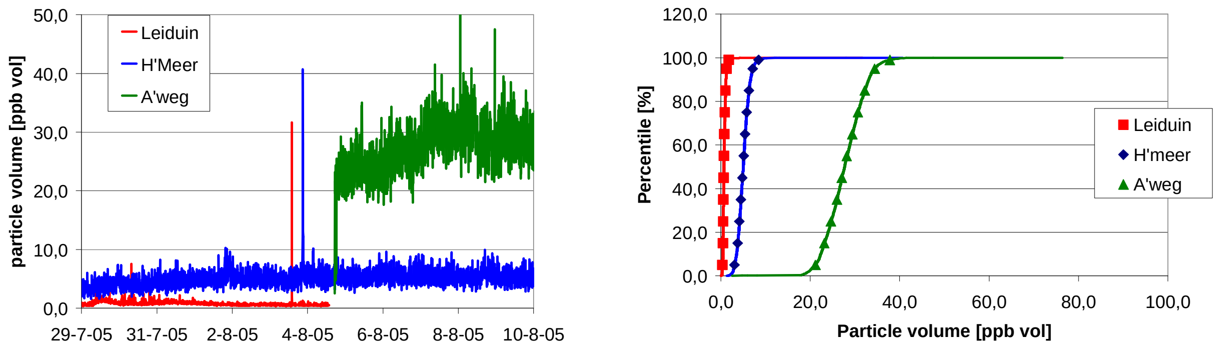


Fig 6-3 Calculated particle volumes in low pressure transport system and frequency distribution. The AW location is the North location

Table 6-2 Frequency percentiles and curve characteristics

Frequency percentile [%]	LD [ppb]	HM [ppb]	AW South [ppb]
90,0	1,10	6,60	33,06
95,0	1,28	7,12	34,41
98,0	1,51	7,83	36,23
99,0	1,77	8,44	37,79
99,5	2,20	9,15	38,97
99,9	4,19	12,14	41,50
ratio 90/99,5	0,50	0,72	0,85
average [ppb]	0,74	5,02	27,71
surf -90 [%]	78,7%	85,0%	87,3%
surf +90 [%]	21,3%	15,0%	12,7%

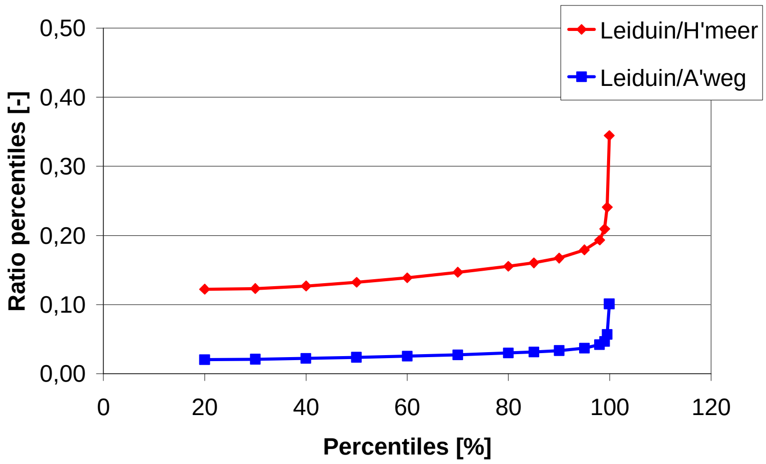


Fig 6-4 Ratio of percentiles values at the two network locations and the treatment locations. The A'Weg location is the South location

The input at the Leiduin location was very comparable with the particle load and the stability as was produced by the UF-installation that is described in paragraph 3.2. The extreme peak on the evening of August 3, 2005 was probably caused by an incident with pumps that could not be traced in the operational records. The peak was clearly recognisable at the second location, which made it possible to determine a residence time of 9,5 hours between the two locations.
The total particle volume increased in the flow direction. The pattern stayed very stable, as can be seen from the 90/99,5 percentile ratio and the Surf-90% and Surf+90%. Also the graph with the percentile ratios shows that the increase was a true growth of particle volume and not a resuspension. The flow through the pipes was rather constant as it was used to fill a balancing reservoir and no obvious resuspension peaks could be measured.

Particle-size distribution changed during the transport as is shown in Fig 6-5. There was a clear increase in the number of larger particles. For the presentation the average was taken for the 12 measurements from 10:00 to 11:00 at August 3 and August 7 respectively. As the variation in time was not very high (Fig 6-3) this can be assumed representative for the long-term process.

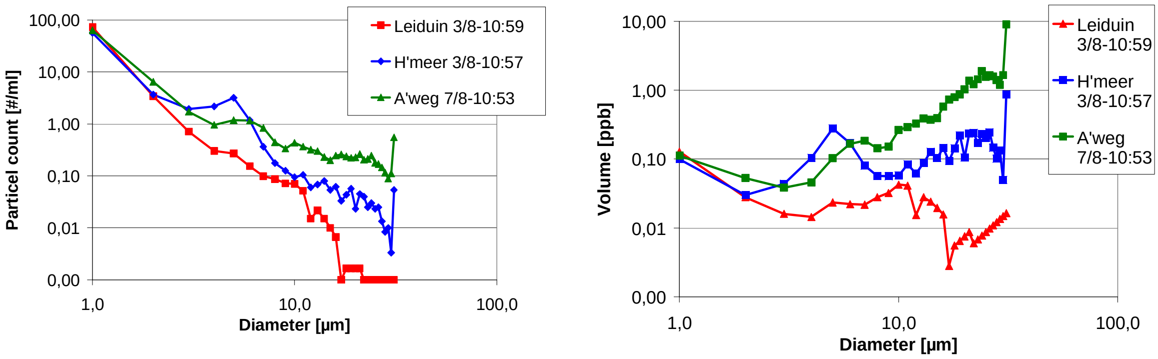


Fig 6-5 Particle- size distribution and volume distribution at LD, HM an AW South locations. Size distribution based on an average of 12 samples during one hour

Unlike what has been observed so far, there were particles larger than 31 µm that obviously dominated the particle volume. Those particles make up to 30% of the total particle volume.

TILVS and particle composition

The composition of particles was derived using TILVS. At the Leiduin treatment plant even after filtering 191 L (Table 6-3), a mass above the detection limit could not be collected, indicating a TSS less than 13 µg/L. At Leiduin predominantly small particles (1 µm) were present and these particles contribute very little to the mass (Fig 6-5).

At Amstelveenseweg samples up to 112 L were filtered with a maximum mass of 6.6 mg retained on the filter. The Amstelveenseweg South line had an average TSS of 58 µg/L, while the Amstelveenseweg North line had an average TSS of 41 µg/L. Statistically there is no significant difference between the concentration of the two lines, though the particle counts over the sampling period also had a lower volume of particles in the Amstelveenseweg North line (Fig 6-6 and Table 6-4). As the TSS and the particle count measurements were independent of each other it might be indicative of a real difference.

Table 6-3 Total mass, TSS, VSS and elemental composition of particulate material in the Leiduin transport system. The first part gives the absolute mass per filter, the second part represents the concentration in the water. The six samples from Leiduin were too small to analyse properly.

Location		Leiduin 6 samples	AW South 23/8 # 1	AW South 23/8 # 2	AW South 24/8 # 2	AW South 24/8 # 1	AW North 25/8 # 2	AW North 25/8 # 1	AW North 26/8 # 2	AW North 26/8 # 1
Filtered Volume	[l]	48 - 191	95,8	95,8	92,4	92,4	79,6	79,6	112,4	112,4
(TSS)	[mg]	0 - 1,2	3,2	5,8	6,2	6,6	3,1	3,9	4,2	4,2
(VSS)	[mg]	-	-	1,4	-	3,6	-	0,6	-	1,6
Fe(OOH)	[µg]	-	522,5	-	1017,5	-	539,3	-	683,7	-
MnO2	[µg]	-	24,9	-	61,7	-	25,7	-	31,6	-
AL(OH)3	[µg]	-	145,9	-	370,5	-	166,1	-	208,7	-
Ca(CO)3	[µg]	-	388,5	-	658,9	-	433,8	-	414,6	-
SiO2	[µg]	-	243,8	-	553,1	-	262,5	-	300,0	-
Unknown (incl. VSS)	[µg]	-	1874,5	-	3538,3	-	1672,6	-	2561,3	-
(TSS)	[mg/l]	<13	33,4	60,5	67,1	71,4	38,9	49,0	37,4	37,4
(VSS)	[mg/l]	-		14,6		39,0		7,5		14,2
Fe(OOH)	[µg/l]	-	5,5	-	11,0	-	6,8	-	6,1	-
MnO2	[µg/l]	-	0,3	-	0,7	-	0,3	-	0,3	-
AL(OH)3	[µg/l]	-	1,5	-	4,0	-	2,1	-	1,9	-
Ca(CO)3	[µg/l]	-	4,1	-	7,1	-	5,4	-	3,7	-
SiO2	[µg/l]	-	2,5	-	6,0	-	3,3	-	2,7	-
Unknown (incl. VSS)	[µg/l]	-	19,6	-	38,3	-	21,0	-	22,8	-

The calculated particle volumes from the concurrent particle count measurements at the two North and South locations showed a slightly different image than in the first measurement period. The North location now had a higher average (42,2 ppb versus 27,7 ppb during the first period) and the South line had an even higher average (62,6 ppb) (Table 6-4).

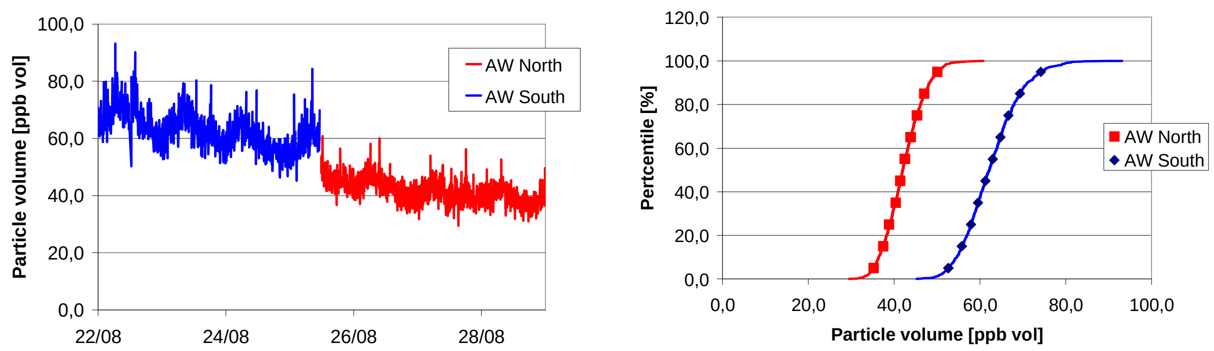


Fig 6-6 Calculated particle volume and frequency distribution during TILVS sampling at AW North and AW South

The frequency distribution and the curve characteristics showed a rather constant level without many spikes, which could be expected because the flow in the pipes was filling a balancing reservoir. Especially at the South location, but also to a lesser degree in the North location during this measuring period a daily repeating pattern could be observed. This is very like the turbidity pattern associated with corrosion described in paragraph 2.2.3 and 5.7. Corrosion is in that case assumed to be a process that releases particles at a more or less constant rate. As result of this constant release the turbidity rises according to the residence

time. The longer the residence time, the higher the turbidity. The change in the particle volume as observed at the Amstelveense Weg South and North was probably also caused by a constant process, with associated dependency of the level of particle volume and residence time or, more precise, contact time. The elemental analyses of the TILVS samples give more information on the composition of the particle sample and the underlying particle production mechanism, which in this case cannot be a classic corrosion due to the lack of ferrous material.
Secondly there appears to be an overall decreasing trend in the calculated particle volume concentrations that is consistent at both locations. This cause of this is unknown. During the measuring time there were no anomalies reported within the treatment process, though it is most likely treatment related. This typical pattern cannot be recognised in Fig 6-3 that presents data from the same location but a few days earlier.

Table 6-4 Frequency percentiles and curve characteristics

Frequency percentile [%]	AW North [ppb]	AW South [ppb]
90,0	48,27	70,95
95,0	50,05	74,16
98,0	51,84	78,53
99,0	53,10	79,86
99,5	55,38	81,55
99,9	58,08	90,07
ratio 90/99,5	0,87	0,87
average [ppb]	42,23	62,58
surf -90 [%]	88,0%	87,9%
surf +90 [%]	12,0%	12,0%

At Amstelveenseweg VSS was highly variable, ranging from 15-55% of the TSS with an average of 33% (Table 6-3). From the elemental metal analysis and the VSS an approximate mass balance can be made. It was assumed that Fe was present as FeOOH, Mn as MnO_2, Al as $Al(OH)_3$, Ca as $CaCO_3$ and Si as SiO_2. Although this is probably an oversimplification of the possible compounds present, these are the likely dominant forms and have been previously used for such calculations (Gauthier et al., 1996; Gauthier et al., 2001). It was not possible to calculate a mass balance for the Leiduin location because of the low amount of mass retained on the filters.
Because the samples were taken in duplo and one filter is analysed on TSS and elemental analysis and the other on VSS, these results have to be combined to make the mass balance. To convert the VSS-level from the one sample to the other, the VSS of the one sample are calculated as percentage of the TSS and this percentage is applied to the other sample (Fig 6-7 and Table 6-5).

Table 6-5 Elemental analysis and VSS as percentage of TSS

	Location	AW South 23/8	AW South 24/8	AW North 25/8	AW North 26/8	average
(VSS)	% of TSS	24,1%	54,5%	15,4%	38,1%	33,0%
Fe(OOH)	% of TSS	16,3%	16,4%	17,4%	16,3%	16,6%
MnO2	% of TSS	0,8%	1,0%	0,8%	0,8%	0,8%
AL(OH)3	% of TSS	4,6%	6,0%	5,4%	5,0%	5,2%
Ca(CO)3	% of TSS	12,1%	10,6%	14,0%	9,9%	11,7%
SiO2	% of TSS	7,6%	8,9%	8,5%	7,1%	8,0%
Unknown	% of TSS	34,4%	2,5%	38,6%	22,9%	24,6%

The unknown components account on average for almost a quarter of the mass. This is in the same range of the measurements of Gauthier, who found the undetermined components range up to 20% of the mass on the filter (Gauthier et al., 1996; Gauthier et al., 2001). Regardless of the level of TSS, the relative contribution of the metals measured was very consistent, making up 42% of the overall mass regardless of the variation in VSS and whether samples were taken from the North or South line (Fig 6-7 and Table 6-5).
The results of the elemental analysis show that there must be a large increase in the concentration of particulate metals in the transportation system. The increase showed also in the colour of the filters, which at Leiduin had a very pale grey colour after sample collection whereas the filters at both Amstelveenseweg sample locations had a red/brown colouration.

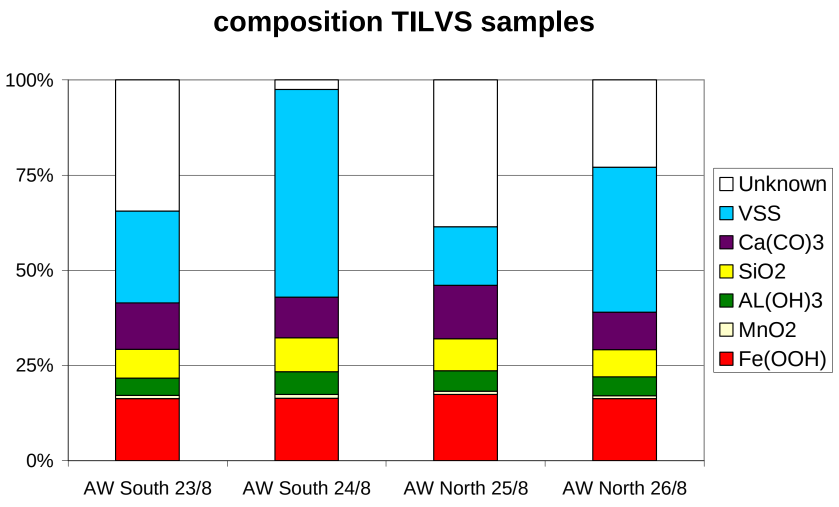


Fig 6-7: Elemental and VSS contribution to total mass of particulate material. Each analysis consists of two filter, VSS as percentage of TSS is applied to the second filter.

Though the original composition of the particulate matter at the treatment plant Leiduin was not sufficiently known because the mass on the filters was too little, it can be said that there was a true increase in particulate matter. A comparison with the composition of the water at Leiduin as analysed in the routine sampling program (Table 6-1) was not possible because this concerns all the solute and particulate forms of the elements. The increase in mass was at least with a factor 2,5 to 5,5 and was probably not confined to one process.
The most plausible explanation for the increase in the iron content in the samples is an ongoing flocculation in the transportation pipes because there is almost no unlined iron or steel in the system. The increase in the particulate form of silicium, aluminium and calcium is

also very constant and is not present in these quantities in the incoming water can plausibly be explained by leaching of the cement mortar lining. Leaching can nearly always be expected in cementitious pipes (van den Hoven and Vreeburg, 1992).
The increase in VSS or organic fraction is probably caused by a biological process.

6.2.4 Discussion

In the Leiduin system an increase in the particle volume was measured by the particle counters. This increase in particle loading was also measured concurrent with the particle counters operated TILVS that showed an increased mass collected on the membrane filters. The calculated particle volume concentration increased by a factor of 37 and the TSS levels increased with a factor 2,5 – 5,5. This discrepancy between the growth factors is significant but both indicate a significant growth of particulate matter.
The increase in calculated particle volume was dominated by a few larger particles in the range >31 µm. For the calculation the average diameter is set on 31,5 µm and even then the dominance is evident. It is, however, questionable whether these larger particles can still be considered as purely spherical. If not, which is likely because of the formation of flocks, than the influence on total particle volume is overestimated. Taking this overestimation in account than the growth in particle volume and particulate mass would be more in accordance.

As indicated in Fig 6-7 and Table 6-5 the VSS fraction of the TSS ranged from 15 to 55%. The organic material in the TILVS samples is an important but variable component with an average of almost 25%. This may simply be an artefact of the low masses retained on the filter reducing the accuracy and reliability of the VSS measurement. However, it also seems likely that the biological contribution will be irregular over time. It could also be an artefact of the sampling point, being a tap at the pipe wall. The water was drawn from flow lines at the wall, which can be influenced by the biofilm and the turbophorectic effect that traps particles at the wall. Loose parts of the biofilm can disturb the VSS measurement. The VSS range found in this study is lower than the range found by Gauthier (Gauthier et al., 2001) for the >5µm fraction in Nancy, France and Montreal, Canada. In these studies VSS varied from 40-76% of the TSS. The lower VSS found in this study is likely to be attributed to the low AOC concentration of the Leiduin water which is a result of the extensive treatment used including the artificial ground passage that provides ground water characteristics to the water. On occasion visual inspection of the filters from the TILVS found the presence of nematodes, which would have made a large contribution to the VSS and would also indicate that there was still some biological growth occurring despite the low AOC concentration.
The fact that the filters used in this study have a pore size of 0,45 µm versus the >5 µm analysed by Gaulthier is probably not an explanatory factor. According to the particle counts the range from 1-5 µm account for a few percent of the total particle volume (Fig 6-5). Especially the larger volumes found at the network locations are dominated by the particle above 5 µm.

Iron was the element that was the highest contributor to particulate mass making up 16% of the material, next to the unknown fraction (Table 6-5). Although the percentages may vary, iron has commonly been found to be the element of greatest proportion in pipe deposits (Gauthier et al., 1996; Smith et al., 1997; Gauthier et al., 2001; Zacheus et al., 2001).
The TSS concentrations increase by at least a factor of 2,5 to 5,5 during transport from the treatment plant at Leiduin to the distribution reservoir at Amstelveenseweg. The increase of particulate iron and manganese may indicate a classic flocculation of these metals, which are present in the infiltrated water and removed almost completely in the treatment process. It is unlikely that the increase in the amount of particulate iron would be due to corrosion of iron,

as the pipes are made of concrete or are cement lined, with only cast iron fittings, such as valves. The concentration of elemental iron in the clear water at the treatment plant is reported as less than 10 µg/l, so post-flocculation of soluble iron could account for the increase in particulate iron of 5,5 to 11,1 µg/l present as Fe(OOH).
Physiochemical processes have been found to be dominant close to the treatment plant (Sly et al., 1989). It is possible that the small lightweight particles that leave the treatment plant act as a catalyst for further particle formation and growth due to their large surface area (Stumm and Morgan, 1996). Pipe loop experiments (Clark et al., 1994) have demonstrated that particles get larger during transport.

Decades of water transport could have created a layer of sediment in the main. Disturbance of the layer adding to the particle volume increase is not likely, however, considering the results of the particle counts which did not show the typical resuspension characteristics. Moreover, the flow in this pipe is relatively constant which would lead to equilibrium between resuspension and settling rather than a net resuspension.

A biological process has had a contribution because the VSS levels also grew. The average VSS level over the four samples amounted to 18,5 µg/l, while the original total TSS-level was < 13 µg/l. A nematode on one of the filters was visible. The biological process was not studied further, as there were no biological analyses as the ATP-level or HPC determined. The contribution, however, was probably not very large as the AOC-level (on average 13 pg/l) of the water is very well in the neighbourhood of the level set for biologically stable water (<10 pg/l) (van der Kooij, 1992).

The growth of the particulate matter during transport through the system as well as the growth in calculated particle volume cannot be explained with a single process. In this case three contributions can be recognised based on the average composition of the particulate matter on the TILVS filters:

- The particulate form of iron is 16,6 % of the TSS and is most probably caused by post flocculation.
- The particulate form of aluminium, calcium and silicium is 25,7% of the TSS, most probably caused by leaching of the cementitious pipes;
- The VSS is 33,0% of the TSS, but is highly variable and is most probably caused by a biological process.
- The remaining fraction is 24,6% of the TSS and of unknown cause.

The differences in the contribution of the three processes to the composition of the TSS was not significant, indicating that there was no dominant process that can account for the major part of the growth, but that the three processes have comparable contributions in this particular case.

6.2.5 Conclusions regarding the low pressure transport system

The growth of the calculated particle volume concentration and the TSS concentration in this low pressure transport system was equally caused by precipitation and flocculation of dissolved iron to particulate iron, leaching of particulate aluminium, silicium and calcium from the cementitious pipe material and biological growth. Eventually these processes increased the particle volume load to the distribution network measured in TSS with a factor of at least 2,5 to 5,5 with possible effects in that distribution system.
TILVS is a new tool in distribution water quality research and has proven to be useful to pre-concentrate particulate material to characterise the composition of particles in low concentrations in drinking water. The trial showed that further development of the TILVS

sampling protocol is necessary to enhance the reliability and reproducibility of the method. The combination of particle counting and TILVS enables conclusions on the composition, origin and changes in time and place of particulate concentration in the distribution system. The specific case of the Amsterdam low pressure transport system showed that during the residence time in the transport network the particle volume of the water increased significantly.

6.3 Case 2: High pressure transport system

6.3.1 Introduction

To study the behaviour of particles in a high pressure drinking water transport network, water from a surface water treatment plant was measured during several weeks at several locations, at the treatment plant itself and at two distant locations in the transport network. Similar to the previous case the treated water was assumed to have a low particle load to the system, at least at the time of measuring. A few weeks before the measurements, the treatment process was extended with UV disinfection and activated granular carbon filtration as final step, which probably had an effect on the total particle load.
The goal of this case study was to analyse the changes in particle volume and composition in this water type during transport through a high pressure transport network using particle counters only. The high pressure transport system is part of the total drinking water network and has more velocity variation than a dedicated system that supplies a balancing reservoir as the low pressure transport system in the previous case study.

6.3.2 Materials and methods

The treatment process used on the water that was monitored in the high pressure transport network was a multiple-barrier system. It started with storage in basins with a residence time of five months and a chemical dosing to soften the water. After low pressure transport the water entered the actual treatment process by passing micro sieves. The water was then coagulated and clarified in a settling process. Secondary coagulation was followed by rapid sand filtration, UV-disinfection and activated granular carbon filtration. After storage the water was pumped directly into the transport and distribution system. UV-disinfection and activated granular carbon filtration were installed just prior to the measurement period to replace the former dosing of the disinfection residual.
The three monitor locations are schematically presented in Fig 6-8. The first location was the Treatment plant right after the pumps. In other words, it was “At the fence of the treatment plant”. What looks like a single line connecting the treatment plant to the second location is in reality a looped system that serves a large area directly without balancing reservoirs. All the transport lines are made of cement mortar lined steel pipes with diameters varying from 1200 to 800 mm at the second monitor location. The distance between the treatment location and the second location is 30 km.

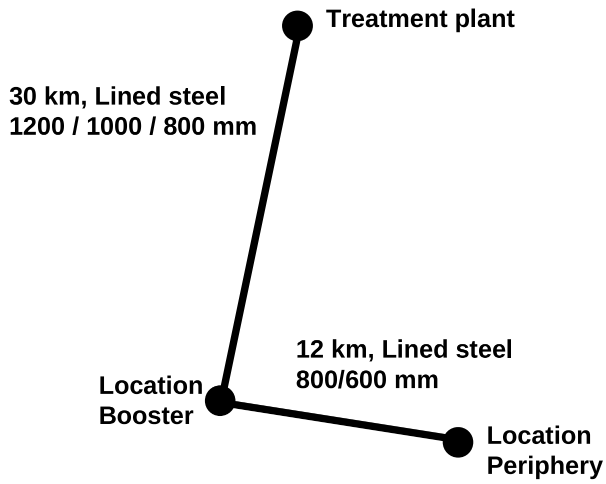


Fig 6-8 Schematic presentation of the monitoring locations

The second location was a booster station that periodically boosted the water pressure by one bar or less to prevent low pressures in the periphery location. The booster pumps only worked during times of high demand and were bypassed when not in operation. The actual sampling point at the second location was at the lower pressure side or suction side of the pump.
The third location was in the periphery of the transport network and basically serves the distant locations. It was connected to the second location by a single pipe with a diameter of 600 mm. connected to the booster station by a pipe with a diameter of 800 mm. All these pipes are also made of cement mortar lined steel; the total length of the pipelines is 12 km.

Water quality was monitored using MetOne PCX particle counters installed at the three locations with a dedicated sampling point at the top of the pipes. The particle counters worked in multiple ranges, with a 1 µm width. Two of them started from 1 µm and measured in 32 ranges with the last range >31 µm. One particle counter started from 2 µm and measured in 31 ranges with the last range >31 µm. The particle volumes are calculated ignoring the 1-2 µm range.
The data from the particle counters are presented as was clarified in paragraph 2.2.5.

6.3.3 Results

The calculated volumes from the particle counts and the frequency distributions are presented in Fig 6-9 and Table 6-6.

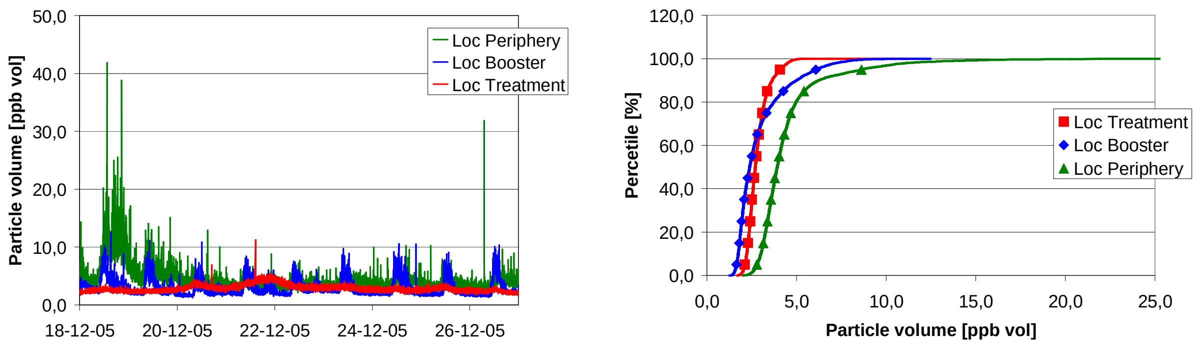


Fig 6-9 Calculated particle volume and cumulative frequency distribution of the high pressure transport network

The frequency distribution shows that the level of particles increased in the flow direction.

The second location shows a dual effect: below the 70-percentile value the particle volumes at the booster location are lower than those of the treatment plant, indicating a net deposition of the particles. Above the 70 percentile, the particle volume at the booster location is higher than at the treatment. The calculated particle volume at the location Treatment Plant was constant and had no relation with the pattern observed at the Booster location nor the Periphery location.
The frequency distribution of the third location clearly shows an increased level of particle volume and also a more shallow distribution indicating more variation and peaks. The 90/99.5 percentile ratio for the first two locations was high, indicating a stable particle volume level. This ratio was lower for the third location (Table 6-6).

Table 6-6 Frequency distribution percentiles

Frequency Percentile [%]	Loc Treatment [ppb]	Loc Booster [ppb]	Loc Periphery [ppb]
90,0	3,62	4,98	6,21
95,0	4,07	6,08	8,61
98,0	4,48	7,18	11,10
99,0	4,68	7,88	13,77
99,5	4,84	8,44	16,17
99,9	5,25	10,30	21,93
ratio 90/99,5	0,75	0,59	0,38
Average	2,81	2,89	4,47
Surf-90%	85,2%	78,0%	78,5%
Surf+90%	14,8%	22,0%	21,5%

The averages and the Surf-90% and the Surf+90% confirm the image: The Surf+90% value at the Booster location was higher than at the Treatment location indicating higher peaks at the Booster location. The higher average at the Periphery location and the higher Surf+90% value showed the net removal of particle volume. Production of particles was also possible but less likely looking at the Surf+90% value that also showed that peaks occurred and were thus resuspension driven.

Compared to the UF water in paragraph 3.4 (Fig 3-7 and Fig 3-8) the total average particle load was a factor of 4 to 5 higher with a similar steady pattern. The ratio 90/99,5 of the UF water was lower, but the Surf-90% and the Surf+90% values were in the same order. Compared to artificial groundwater based multi barrier treatment with a finishing step of slow sand filtration in the previous case (location LD in Table 6-2) the average particle volume concentration was also a factor of 4 higher but had a steadier pattern as shown in the ratio 90/99,5 and the Surf-90% and Surf+90% values.

The Booster location showed a regular pattern that was dependent on the operation of the booster pumps that caused resuspension. A closer detail of the Booster location particle volume concentration trace, in combination with the velocity at the booster station, showed clearly the relation of the velocity and the particle volume concentration (Fig 6-10). This relation shows that the loose particles were resuspended with every increase in velocity caused by a switch of the pump, also indicating a level of resuspendable sediment in the pipes.

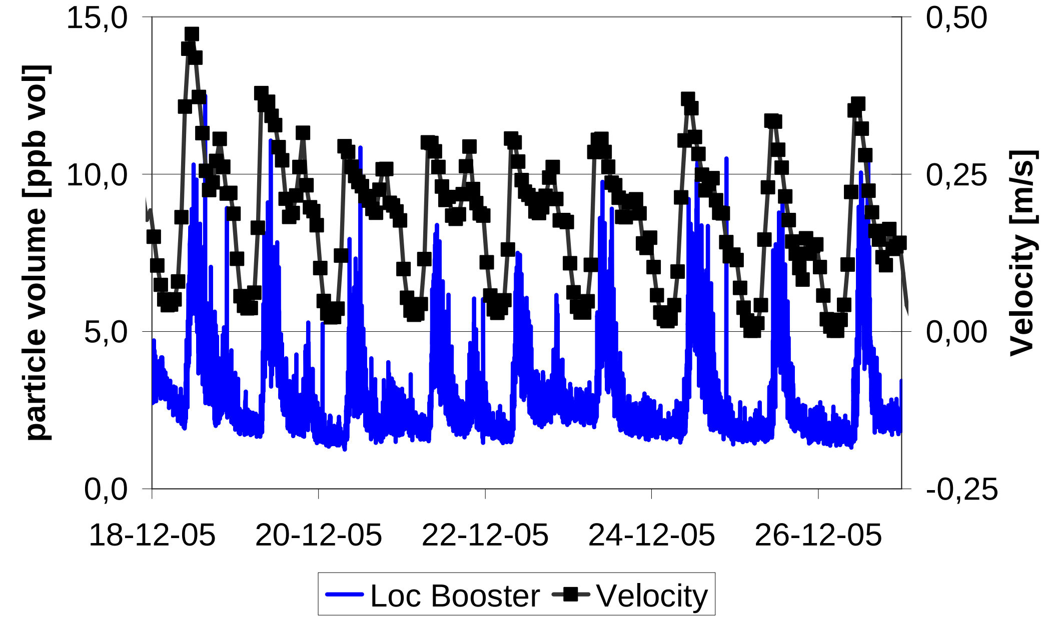


Fig 6-10 Velocity and calculated particle volume at location Booster. The relation between the two traces is evident

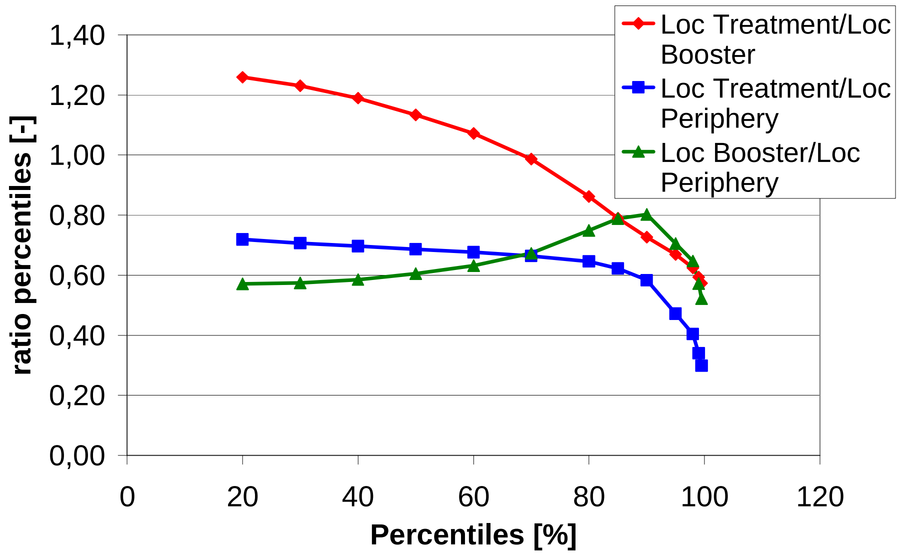


Fig 6-11 Percentile ratios for the combinations of measuring locations

The presence of resuspendable sediment is confirmed by the graph showing the percentile ratios (Fig 6-11). Between the location Treatment and location Booster the percentile ratio line crosses the value of 1 showing that there was equilibrium between the incoming and outgoing volume concentration. With the pattern evident in Fig 6-10 it is clear that this had to do with the resuspension of sediment.

The percentile ratio between the Treatment location and the Periphery location shows a net removal of sediment as the incoming calculated particle volume concentration was consequently lower than the outgoing. Given the new treatment, it is plausible that this had to do with the removal of old sediment. Also the ratio line of the locations Booster and Periphery shows a net removal.

The estimated residence time between the Treatment location and the Booster location was more than 24 hours. The variation in particle volume concentration at the Treatment location was too low to recognise a pattern at the Booster location to verify this estimation.

For the 25-percentile and 98-percentile value the particle size distribution for the three locations was compared. Based on the particle size distribution the particle volume was calculated, excluding the 1-2 µm range because this was not available on the third particle counter.
The particle distribution and the particle volume distribution over the diameter ranges are shown in Fig 6-12 and Fig 6-13. The total particle volumes are summarised in Table 6-7.

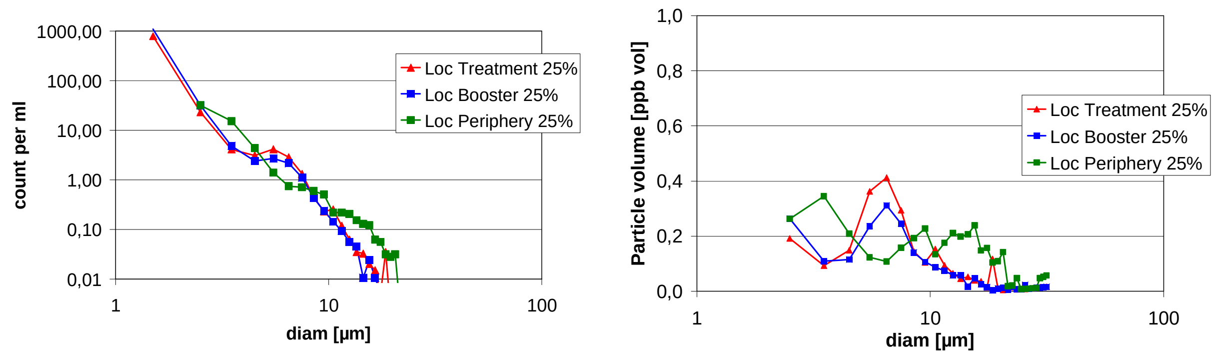


Fig 6-12 Particle size distribution of 25-percentile measurement and volume distribution per diameter range for the three measuring locations

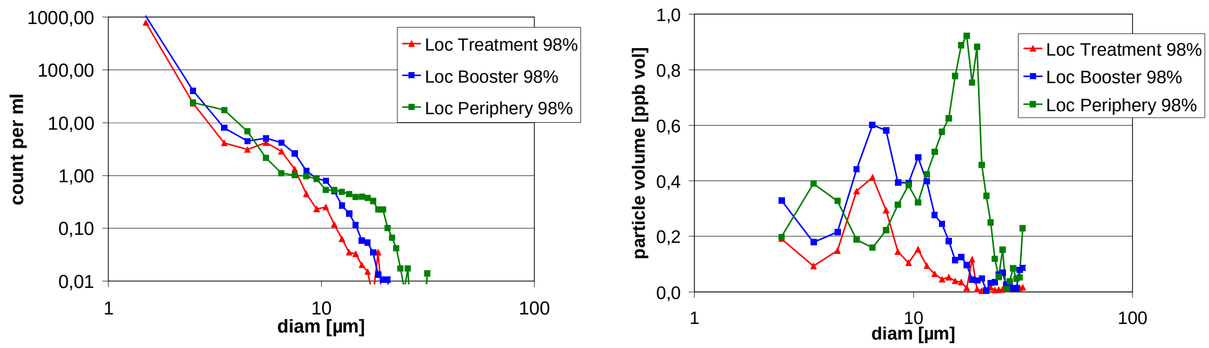


Fig 6-13 Particle size distribution of 98-percentiel measurement and volume distribution per diameter range for the three measuring locations

Table 6-7 Particle volumes 25 percentile and 98 percentile particle volume

	Particle volume	
	25% percentile [ppb vol]	98% percentile [ppb vol]
Loc Treatment	2,50	4,84
Loc Booster	2,07	5,65
Loc Periphery	3,74	10,70

The increase in volume in the 98%-percentile values was different for the location Booster and location Periphery with respect to the particle size distribution and volume at the Treatment location. At the Booster location all the ranges were higher, but stayed in the same ratio to the distribution of the Treatment location. At the Periphery location there was a shift

in the particle size distribution with an increase in the 2-4 µm range, a decrease in the 5-9 µm range and an increase in the 5-30 µm range. The increase in the larger diameter particles accounted for the majority of the increase in the calculated particle volume in that location.
In the low 25%-percentile counts the particle distribution of both the measurements at location Treatment and location Booster had the same shape and were almost identical, while at the location Periphery there were more larger particles. This indicates that the particles at the Periphery location had a different composition and might originate from older sediment.

6.3.4 Discussion

The particle load resulting from the treated water in this system was a factor of 4 to 5 higher than was measured in the UF treated water (paragraph 3.4) and the slow sand filtration treated water of the previous case study. Compared to the loading of the part of the network in the Reference Area discussed in paragraph 3.4 the loading is a factor of 2 to 3 lower and much less 'spikey' meaning a higher Surf-90% value and a lower Surf+90% value.
The treatment process at the Treatment location was recently adjusted and extended with a UV-disinfection and activated carbon filtration. Especially this last extra filtration step could have an effect on the calculated particle volume and the particle size distribution of the water. Though this cannot be verified with the measurements in this series, it can be seen that the particle size distribution in the first and largest part of the transport network does not change significantly.
Clearly there was sediment in the first part of the network as can be seen from the reaction of the particle volume concentration on the velocity at the Booster location. Increasing velocity resuspended material causing increased particle volume and also the ratio of the range of percentile values of the treatment location and the booster location show that net sediment is reuspended. The origin of the sediment could be the 'old' treatment process, but the characteristics of the particle size distribution of the resuspended particles were relatively the same as that from the treatment.
In the present situation between the Treatment location and the Booster location it seems as if there is no production of particles. The characteristics of the frequency distribution are the same as with the self-cleaning networks. This would suggest that the transport system is self-cleaning and that no accumulation is happening. The velocity in the network is actually within the ranges for the self-cleaning threshold for distribution networks, but for larger pipes the consequent shear stress is lower.

Towards the third location there was a change in the particle content of the water, both in volume and in particle size distribution. There are more large particles that consequently contribute to a higher degree to the particle volume. In this case there were no data available on the chemical and biological composition of the particle volume that could have contributed to a better understanding of the water quality changes.
From the average values of the calculated particle volume it is feasible that relatively more sediment was transported out of the system than that was put in. There are a few possibilities for the origin of the sediment that was removed. It could be production from a biofilm or other biological process, leaching of cement (there is no unlined cast iron) or flocculation of smaller particles to larger particles that have accumulated.

The pattern of the particle volume concentration at the Periphery location, if observed on a smaller scale, had a pattern that is consistent with a normal demand pattern as was also observed at the Booster location (Fig 6-10). The almost identical Surf-90% and Surf+90% values show also that the patterns were similar. The base level of the Periphery location was higher, though, than the base levels at the other two locations. Together with the change in particle size distribution this points to the resuspension of particles that probably originated from the ‘old’ treatment. Other production processes like flocculation, leaching or a biological process are less likely because production didn’t occur in the first part of the network with longer residence times as is observed in the low pressure transport network fed with artificial groundwater-based drinking water. The resuspension of ‘old’ particles would be the most logical explanation, especially when the change in treatment process is taken into account.

6.3.5 Conclusions with regard to high pressure transport network

The particle volume loaded on the network from this multiple barrier surface water treatment water is a factor of 4 to 5 higher than with the artificial ground-water based treatment process finished by slow sand filtration in the previous case study. During the transport through the network the particle composition and particle volume concentration didn’t change significantly. In the periphery of the network the particle composition and volume changed, indicating old sediment being resuspended.
Elemental analysis is necessary to draw final conclusions on the particle-related processes that dominate this network, though it is not likely that particles are produced either by a physical/chemical process or by a biological process.

6.4 Case 3: Composition and hydraulic behaviour of drinking water distribution systems sediments

6.4.1 Introduction

Discoloured water is caused by the resuspension of deposited sediments as a result of an increase in the flow velocity of the water in the pipes. The composition and the hydraulic characteristics of the layer of deposits or particles determine how high the discolouration risk is. In this case study the composition and the hydraulic characteristics of some drinking water distribution sediments were examined.
Particles in the water are subject to several forces. Gravitational and turbophorectic forces drive the particles to the pipe wall while a drag force initiated by the flow velocity will resuspend them. The strength of a particle layer or its resistance to resuspension is dependent on both the composition of the sediment and the hydraulic forces. Boxall suggested that layers formed in flows causing high average shear stress will be stronger or have a higher resistance against erosion than layers formed in pipes with flow patterns that cause lower shear stresses (Boxall et al., 2001).
Several studies into the composition of loose deposits in drinking water systems show various outcomes (Gauthier et al., 2001; Zacheus et al., 2001; Torvinen et al., 2004; Barbeau et al., 2005). All concluded that the major components of the sediments were organic matter and that the larger part of the inorganics was formed by (hydrated) iron hydroxides. About the origin and development of the particles no study is conclusive.
For a good understanding of the origin and accumulation of the sediments not only the chemical and biological composition are important, but also the hydraulic properties. Within the framework of this case study both the composition and hydraulic behaviour of sediments from different distribution systems were studied. The hydraulic behaviour, or more specifically the resuspension characteristics of the samples were explored using the conventional Jar Test Equipment (JTE). The goal was to investigate the applicability of the standard JTE for the hydraulic characterisation of loose deposits in networks and the influence of the chemical composition on those hydraulic characteristics. The main advantage of the JTE would be that with the simple and already standardised test a representative characteristic can be developed that can qualitatively relate different types of deposits to hydraulic conditions and chemical composition.

6.4.2 Materials and methods

Field sampling

For this research several sediment samples were taken out of four different distribution systems in the Netherlands that are supplied with different types of water. (Table 6-8)

Table 6-8 Sampling location characteristics

Location	Pipe material	Internal diameter [mm]	Flushing velocity [m/s]	No of samples	Treatment Location
Berlicum	AC	150	2.0	3	Nuland/Loosbroek
Rosmalen	PVC	101.6 (110 ext)	1.2	4	Nuland
Koudum A	PVC	101.6 (110 ext)	1.5	1	Spannenburg
Koudum B	PVC	101.6 (110 ext)	1.5	3	Spannenburg
Koudum C	PVC	101.6 (110 ext)	1.5	2	Spannenburg
Amsterdam A	CI	101.6 (4”)	0.6	1	Leiduin
Amsterdam C	CI	76.2 (3”)	1.2	1	Leiduin
Amsterdam D	CI	127 (5”)	0.5	1	Leiduin
Amsterdam B	CI	101.6 (4”)	1.0	1	Weesperkarspel
Amsterdam E	CI	250	1.4	1	Weesperkarspel
Amsterdam F	CI	101.6 (4”)	0.4	1	Weesperkarspel

Samples were taken during the unidirectional flushing with an intended velocity of 1,5 m/s in isolated pipes made from various materials. Actual velocities were calculated based on the measured flow rates. Samples were taken during the first turnover of the pipe content; firstly because then more of the sediments could be recovered (paragraph 5.3, (Vreeburg, 1996)); secondly because then definitely only sediments present in the pipe itself are sampled. Longer flushing times could drag sediment from upstream pipes into the sampling location, unless these pipes were also cleaned and could be considered a so-called clear water front (paragraph 5.3 and (Schaap and Vreeburg, 1999)).
At locations Berlicum, Rosmalen, Koudum B and C multiple samples were taken during the first turn-over. Samples were identified with the town name, an optional letter indicating more locations in one town and a number, indicating more samples at one location during the first turn-over. (For example: Koudum B-2 is the second sample at the B-location in Koudum).
All locations except location Rosmalen were flushed within the routine cleaning procedures of the companies. Location Rosmalen was recently flushed, but incorporated in the sampling because of persistent brown water complaints at two specific connections on the flushed pipe.
Three of the Amsterdam locations were supplied with water from the treatment plant Leiduin and transported through the low pressure transport system that was the subject of research in the first case study.

The average composition of the water leaving the treatment plants is given in Table 6-9, based on the routine samples taken within the legal water quality analysis program for the year 2004. (REWAB, 2004). The Leiduin data for 2003 are given in Table 6-1.

Table 6-9 Average composition of the treated water at the different feeding points

		Nuland				Loosbroek				Spannenburg			
		n	avg	min	max	n	avg	min	max	n	avg	min	max
Turbidity	NTU	52	0,38	0,12	0,78	52	0,13	0,06	0,26	52	0,21	<0.05	1,30
Temperature	ºC	13	12,2	8,0	14,9	13	12,4	11,1	13,0	54	12,0	10,0	13,0
pH	-	52	7,78	7,69	7,88	52	8,03	7,82	8,13	52	7,55	7,40	7,65
SI	-	4	0,58	0,57	0,59	4	0,88	0,83	0,99	52	-0,20	-0,33	-0,04
Dissolved Oxygen	mg/L	52	9,8	9,1	10,7	52	11,7	9,5	12,3	52	6,1	4,6	7,6
DOC	mg/L									4	7,5	7,3	8,1
KMnO4	mg/l O2	4	2,8	2,5	3,3	4	2,0	1,9	2,3				
Aeromonas	n/100 mL	52	40	0	92	52	0	0	3	16	10	6	29
Chloride	mg/L	52	108	86	130	4	53	52	54	4	30	30	31
Iron	mg/L	52	0,04	0,02	0,06	52	<0.01	<0.01	0,01	12	0,03	0,02	0,08
Manganese	µg/L	52	<10	<10	<10	52	<10	<10	<10	12	<10	<10	<10
Aluminium	µg/L	1	<5	<5	<5	1	7	7	7	4	<5	<5	<5
Magnesium	mg/L	4	9,6	9,4	9,8	4	10,0	9,8	10,0	52	9,4	8,2	10,0
Calcium	mg/L	4	83,0	82,0	85,0	4	77,0	74,0	79,0	52	31	25	41,0

		Leiduin				Weesperkarspel			
		n	avg	min	max	n	avg	min	max
Turbidity	NTU	729	<0.10	<0.10	0,27	397	<0.10	<0.10	0,88
Temperature	ºC	725	12,4	5,9	19,4	364	12,1	3,7	22,3
pH	-	728	8,39	8,11	8,79	364	8,04	7,79	8,60
SI	-	206	0,48	0,24	0,73	53	0,29	-0,05	0,62
Dissolved Oxygen	mg/L	78	9,7	6,1	12,9	52	7,8	5,7	10,9
DOC	mg/L	51	1,1	0,6	2,0	51	3,2	2,6	4,2
KMnO4	mg/l O2								
Aeromonas	n/100 mL	94	2	<1	14	52	3	<1	34
Chloride	mg/L	710	99	89	109	13	78	72	81
Iron	mg/L	103	<0.01	<0.01	0,03	51	<0.01	<0.01	0,09
Manganese	µg/L	104	<10	<10	<10	12	<10	<10	<10
Aluminium	µg/L	52	5	<3	23	4	7	<3	12
Magnesium	mg/L	104	10,0	9,2	11,0	51	6,6	5,8	7,0
Calcium	mg/L	208	4,3	3,9	4,8	53	49,0	46,0	52,0

At the treatment plants Nuland, Loosbroek and Spannenburg groundwater is treated with aeration and rapid sandfiltration in one or two stages. Leiduin and Weesperkarspel are multi barrier treatment plants that treat surface water. For the Leiduin location the water is captured from the Rhine River, coagulated and rapid sand filtered. Then the water is infiltrated in the dune area and retrieved again after an average of three months residence time. Post treatment comprises aeration, rapid sand filtration, and ozonisation followed by pellet softening and activated carbon filtration and finally slow sand filtration (see also paragraph 6.2.2). The Weesperkarspel plant looks much like that, albeit that the source water is retrieved as seepage water from a polder and not infiltrated The treatment is a true surface water treatment. The water is coagulated, then stored in a reservoir with a detention time of three months and then rapid sand filtered. The post-treatment also consists of ozonisation, pellet softening and activated carbon filtration, and finally slow sand filtration.

Water and sediment composition

The water samples with sediments were analysed on TSS and VSS using the AWWA standard method (APHA, 1998) filtering the water samples with sediments through rinsed, dried and weighed 0,45 μm filters. The filters were again weighed after drying at 105°C to determine the TSS level and again after burning at 450°C to determine the VSS level. The sediment from the samples taken at the Berlicum and Rosmalen location were analysed on Fe with AAS; the Fe concentration of the Koudum and Amsterdam samples were determined within the ICP-MS scan, as was the Mn concentration of the Berlicum and Rosmalen samples. Turbidity of the samples was measured with a Dr Lange LTP4/LPV159.

Hydraulic behaviour

The hydraulic behaviour of the sediment was evaluated with the conventional Jar Test Equipment (JTE). The goal was to link the impellor velocity to the turbidity at the middle and bottom level in the beaker representing some hydraulic properties of the sediment (Fig 6-14). The turbidity both at the bottom and the middle of the jar was measured with a HACH 2100N lab turbidity meter using dedicated siphons (Fig 6-15) to take samples at the different levels.

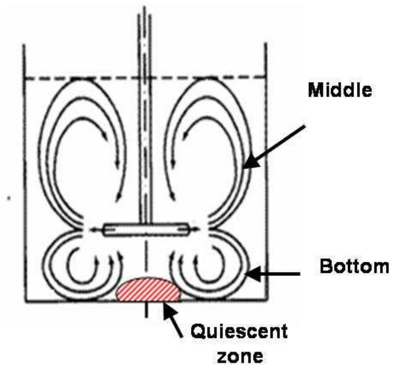


Fig 6-14 Secondary flow pattern jar

Fig 6-15 Siphon in jar test equipment

Originally, the standard jar test was used in treatment-related research to optimise dosing rates and the mixing intensity for coagulation/sedimentation/flotation. Since the introduction of the mean velocity gradient G (Camp and Stein, 1943) several researchers have studied the velocity field in the jars and the applicability of a general parameter to represent the mixing conditions (Cheng et al., 1997). The overall conclusion was that the flow in the jars is a complex system in which the mixing intensity is a function of the impellor speed and location. Turbulence is consequently not homogeneous throughout the beaker. The secondary flow pattern as is hypothesised in Fig 6-14 shows that at the middle level the direction of the flow is upward and at the bottom level downward. Light particles will not be able to 'escape' from the bottom turbulence because of the downward flow causing a difference in turbidity at the middle and the bottom level.

The beakers of the JTE were filled with 1,8 litres of the stirred samples from the distribution networks and left to settle for at least 6 hours for complete settlement. The stirring rate in the jars was varied in a range of 10, 15, 20, 30, 45, 60, 90, 120 and 150 RPM.

The 6-hour settling time frame was determined with pre-experiments on artificial sediments consisting of a kaolinite mixture and a ferric hydroxide flock mixture.
The objective was to determine the critical stirring rate (CSR) defined as the stirring rate at which the turbidity in the jar reaches a constant level: the CSR was reached if an increase in the stirring rate doesn't increase the turbidity of the sample.
At the CSR there will be a bit of sediment in the quiescent zone as can be seen in Fig 6-16. In the second jar the CSR is reached, still leaving some sediment at the bottom. Increasing the stirring rate beyond the CSR resuspends this last bit, but doesn't increase the turbidity significantly.

Fig 6-16 Bottom view Amsterdam C after 10, 30, 60 and 90 RPM

The samples were all tested within 2 to 5 days after the actual sampling. To find out whether compacting or any other physical change would occur, the Koudum and Amsterdam samples were retested after 4 to 5 weeks of storage in the lab without any special preservation measures to assess the effect of ageing on the sediments.

6.4.3 Results

Water and sediment composition
Chemical analyses of the samples are given in Table 6-10.
The amount of sediments (TSS) and turbidity at the Berlicum and Rosmalen location were significantly lower than at the other locations. Location Berlicum proved to be relatively clean, and also showed low turbidity during flushing. The location Rosmalen was flushed four months prior to the experiment because of persistent discolouration problems at two specific connections. The low turbidity and TSS levels showed that the cause of this problem could not be found in the sediment from the network. An explanation for the problems is still being researched. At the other locations the TSS and turbidity levels were higher indicating more loose sediments and a greater need for cleaning.

Table 6-10 Analysis results

	Sample ID	TSS [mg/l]	VSS [mg/l]	VSS%	Turbidity [FTU]	Mn [µg/l]	Fe [mg/l]	Al [mg/l]	Si [µg/l]	v flush [m/s]	diam [mm]
Berlicum	BE 1	11,0	4,9	44,55	26,0	280	2,1	0,0	8,9	1,2	150,0
	BE 2	10,0	5,0	50,00	10,7	260	2,0	0,0	9,6	1,2	150,0
	BE 3	10,0	6,2	62,00	11,3	285	1,9	0,0	9,0	1,2	150,0
	BE 4	14,0	4,4	31,43	9,8	380	1,9	0,3	9,6	1,2	150,0
Ros-malen	RO 1	2,8	2,0	71,43	2,3	130	0,4	0,2	10,3	2,0	101,6
	RO 2	3,9	2,0	51,28	4,0	160	0,7	0,0	8,8	2,0	101,6
	RO 3	13,0	5,9	45,38	12,0	520	2,8	0,1	9,8	2,0	101,6
Koudum	KO A	325,0	135,0	41,54	290,0	2000	51,7	0,9	12,9	1,5	101,6
	KO B-1	83,0	35,0	42,17	72,0	500	13,7	0,3	3,3	1,5	101,6
	KO B-2	430,0	180,0	41,86	390,0	3200	74,6	0,9	9,8	1,5	101,6
	KO B-3	200,0	91,0	45,50	143,0	1900	29,3	0,4	5,4	1,5	101,6
	KO C-1	58,0	26,0	44,83	39,0	400	8,2	0,2	1,9	1,5	101,6
	KO C-2	275,0	125,0	45,45	216,0	1900	41,3	0,9	8,7	1,5	101,6
A'dam Leiduin	AM A	12,0	3,6	30,00	16,0	100	2,2	0,0	0,2	0,6	101,6
	AM C	36,0	6,5	18,06	68,0	200	8,0	0,1	0,5	1,2	76,2
	AM D	130,0	21,0	16,15	260,0	700	42,2	0,2	1,8	1,4	127,0
A'Dam WK	AM B	40,0	10,0	25,00	57,0	500	10,1	0,1	0,6	1,0	101,6
	AM E	5,5	0,8	14,55	5,0	0	1,2	0,0	0,1	0,5	250,0
	AM F	32,0	7,4	23,13	59,0	200	8,8	0,1	0,5	0,4	101,6

The Koudum location had the highest sediment load and the Amsterdam LD location had a moderate load. The relation between the flushing velocity and the TSS level in the sample showed that with lower velocities significantly lower TSS levels in the flushed water were obtained (Fig 6-17).

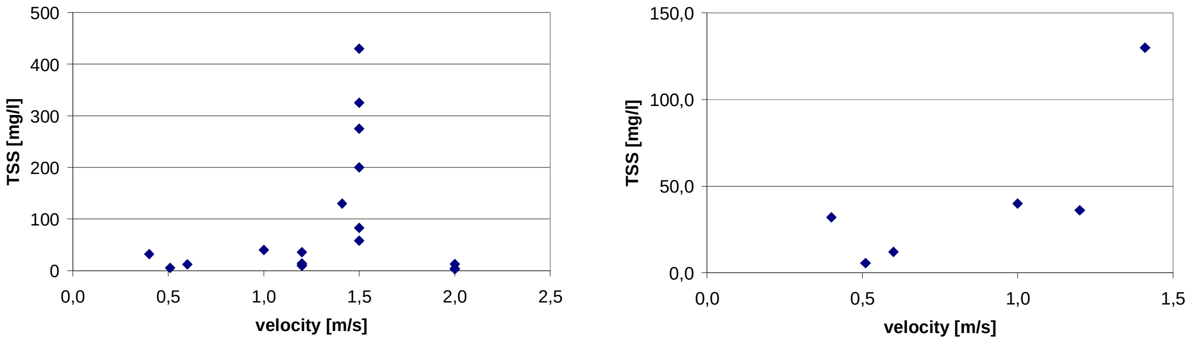
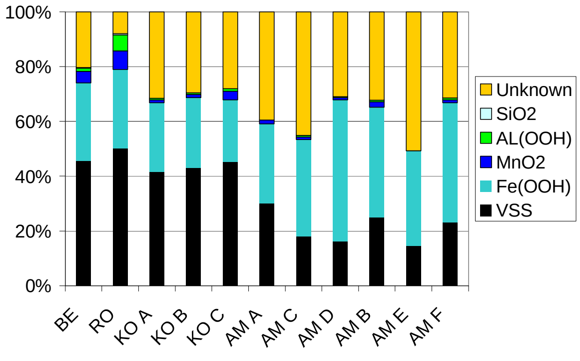


Fig 6-17 Velocity versus TSS in all flushed samples (left) and in Amsterdam location (right)

At the Amsterdam locations the relationship between the actual velocity and the TSS-level was very clear, as is shown in the right graph of Fig 6-17. This hints to the practical value of the guideline that is applied in the Netherlands to flush with a minimum velocity of 1,5 m/s. (section 5.3)(Vreeburg, 1996; Schaap and Vreeburg, 1999). Because the samples were taken in the same distribution area, the actual sediment levels should be within the same order. With different velocities however different amounts of sediment were removed with an increase at the high velocity. The amount of data is however too little to draw significant conclusions.
At the Koudum location with the same velocity and in the same type of pipe (PVC), different levels of TSS were found in the flushed water.

The relative composition of the samples can be calculated if the Fe is assumed to be present as Fe(OOH), Al as AL(OOH), Mn as MnO_2 and Si as $Si0_2$. For the locations with multiple samples an average was taken (Fig 6-18)



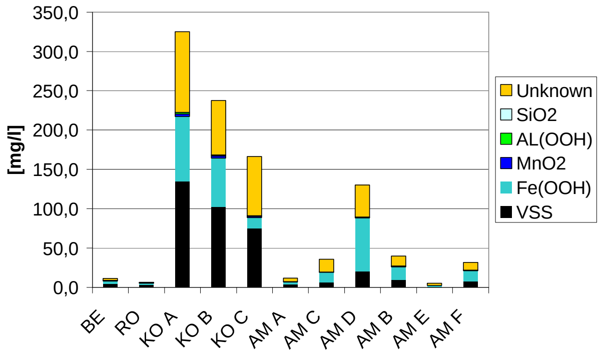
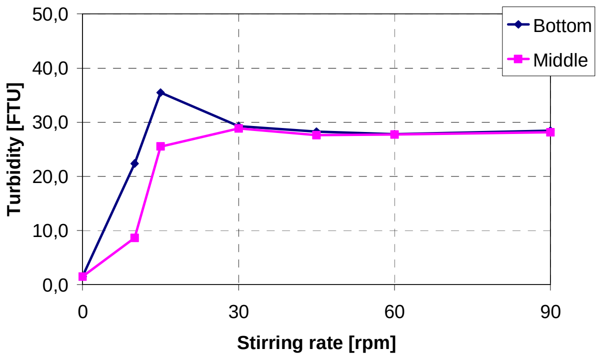


Fig 6-18 Relative (left) and absolute (right) composition sediments in the network

There were distinct differences in the compositions of the samples. Remarkable is that the relative VSS-component in the Amsterdam location was significantly lower than at the other sites. The average Fe(OOH) content of the Amsterdam samples was 39% against 27% in the other samples. The main difference was that the Amsterdam samples were taken out of cast iron pipes and the others from PVC and AC.
The absolute levels of the most prominent components (VSS and Fe(OOH) content) showed the iron level in the Koudum locations to be higher than the Amsterdam location despite the lack of cast iron in the Koudum network.

Hydraulic behaviour
Fig 6-19 shows the typical results for the Koudum C-1 and Amsterdam B locations.



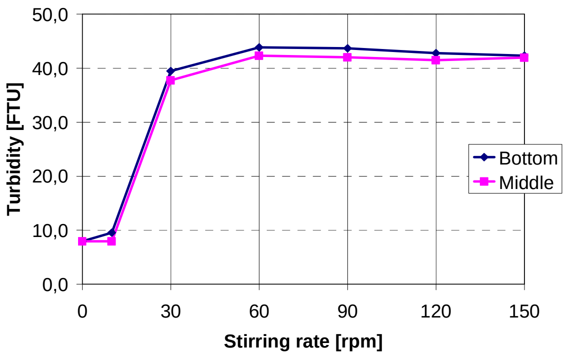


Fig 6-19 Turbidity in Koudum C-1(left) and Amsterdam B (right) samples with differences between the bottom and middle positions

For all the Koudum samples there were distinct differences between the middle and bottom position of 25 to 30%. For the Amsterdam locations this difference was much less in the order of a few percent. This was, however, consistent for all these samples. In the Berlicum and Rosmalen samples there were no clear differences between the two positions.

Fig 6-20 shows the results of the low-turbidity samples from Berlicum and Rosmalen, in one graph, next to the high turbidity samples from Koudum and Amsterdam for the determination of the CSR. The CSR can be read from the figures at the point that the turbidity line reaches a plateau.

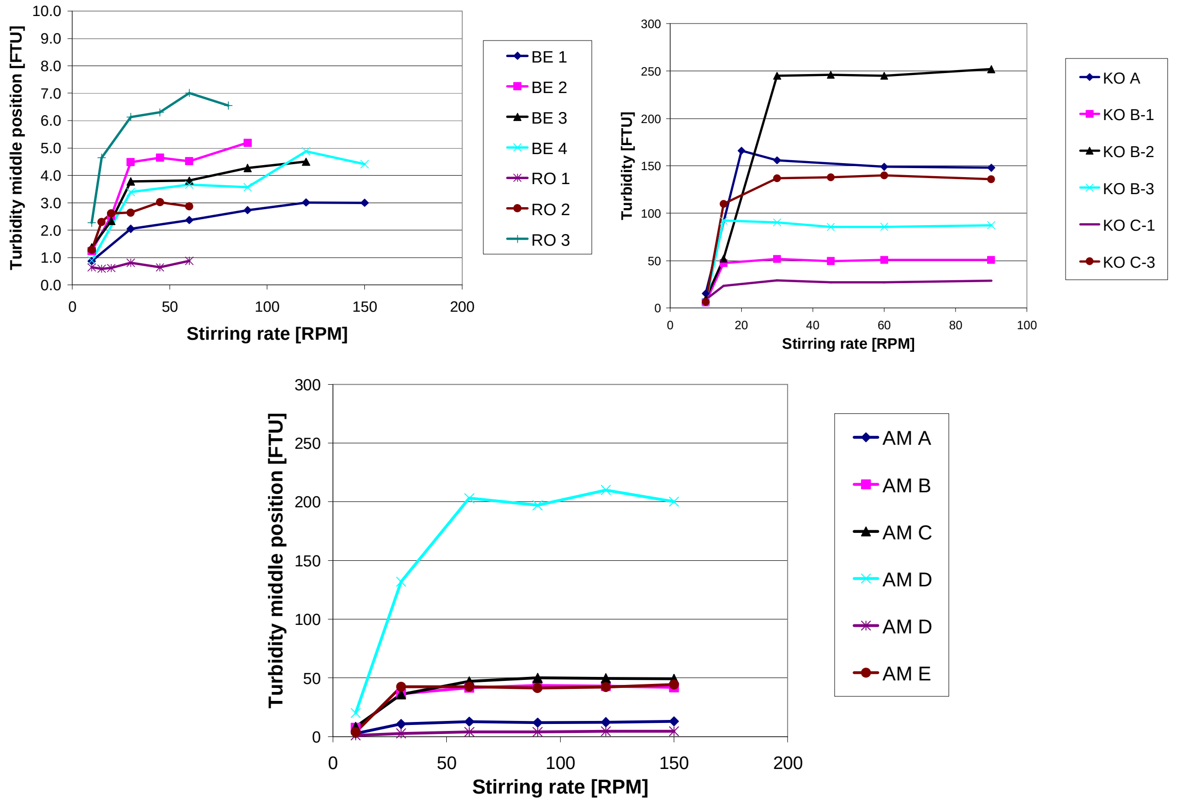


Fig 6-20 Critical Stirring Rate results from jar test of all samples separated for the Rosmalen and berlicum locations (left upper graph), the Koudum locations (right upper graph) and Amsterdam locations (lower graph)

The results show that except for the Amsterdam-D location the sediment was in full resuspension at 20-30 RPM. According to the definition of the Critical Stirring Rate, this means that despite the different actual velocities with which the sediments were flushed out, the critical stirring rate was around 30 RPM.
The JTE experiments for some of the samples were repeated after unconditioned storage for 5 weeks. The results for the Koudum B-2 are shown in Fig 6-21 for both the middle and bottom position. As can be seen, the results were identical, which is also representative for the other stored samples.

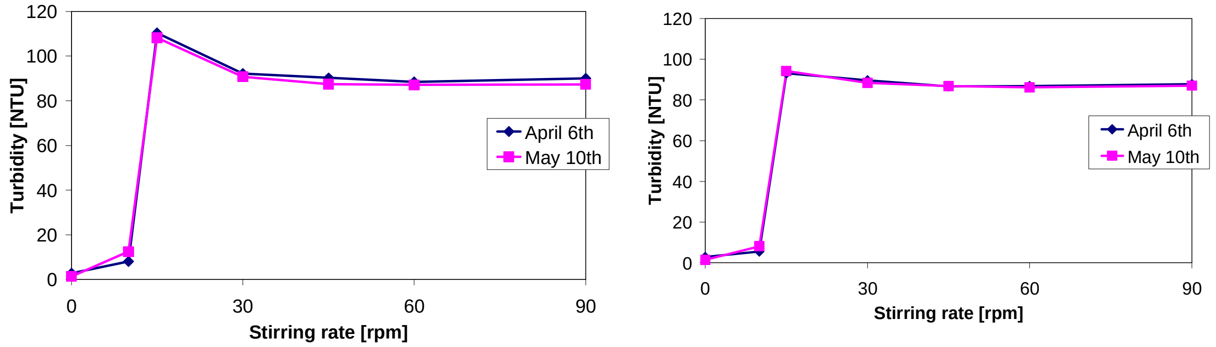


Fig 6-21 Koudum B-2 samples before and after 4-week storage. Left the results from the bottom jar position; right those from the middle jar location

6.4.4 Discussion

The main components of the sediments in the samples are organic matter and iron hydroxides, which is in accordance with many other researches (Sly et al., 1990; Gauthier et al., 2001; Zacheus et al., 2001; Gauthier et al., 2003; Barbeau et al., 2005; Carriere et al., 2005). The absolute levels of sediments are also in the range reported by these researches, though the Rosmalen and Berlicum levels are low. In the study of Barbeau (Barbeau et al., 2005) TSS levels of 10 to 15 mg/l during the first turnover are showed but they claim that typical levels are in the range of over 200 mg/l. In experiments described here this whole range of values was actually measured (minimum 2,8 mg/l and maximum 430 mg/l).

The Amsterdam samples taken at the Leiduin locations (AM-A, -C and -D) had a slightly higher iron content than the samples of the particulate matter that were taken during the first case study at the Leiduin treatment location itself. The increase could be the result of the prolonged residence time and the further flocculation of the material, but could also be iron picked up from corrosion products. The iron could partly be present in the crystalline form and partly in the flocculant form. This would indicate that although the samples from the cast iron pipes have relatively high iron content, not all the iron originates from the corrosion of the pipe, but could also for a significant part originate from the treatment plant.

The results of the chemical analysis were slightly different from those quoted in literature. (Carriere et al., 2005) reported an Fe(OOH) content of 38-72% in the deposits and a VSS of 14-24%. In our experiments these levels were 23 to 46% for the Fe(OOH) and 15 to 50 % for the VSS. It should be noted that the drinking water in the Netherlands is not chlorinated, which could account for the higher relative levels of VSS. The absolute levels, however, were still low. It is remarkable that the relative iron content in the cast iron location (Amsterdam locations) was higher compared to the Koudum locations, but the absolute levels are lower. Corrosion was very likely the source of the iron at the cast iron location. Apparently, the iron source in the Koudum case was from the treatment while in the Amsterdam case also the pipes themselves were a source. This was supported by the fact that the Fe-concentration at the treatment plant in Spannenburg is much higher than in Leiduin (0.03 mg/l vs less than 0.01 mg/l, Table 6-9). In earlier research it was shown that post-treatment flocculation of Fe can contribute significantly to the amount of particles and sediment in the network (Paragraph 6.2).

In the PVC locations in Koudum as well as in the other AC and PVC location, the origin of the iron must have been the treatment plant because there was no cast iron in the network.

The depth of the sediment layer can be calculated when an average density of the sediments is assumed at 1050 kg/m^3, and also assumed that sediments were settled at 50% of the lower half of the pipe over a width of $0{,}25*\pi*D$. The results of the test rig (see section 4.2.2) show that sediments can settle over the complete circumference of the pipe wall. The assumption that in this case the sediment settles on 50% of the circumference is done, because the velocities in a distribution network are mostly very low (Blokker et al., 2006). Table 6-11 gives an overview of the calculated layer depths. This table also presents the sediment rates, which are the amount of sediment from the flushed water evenly distributed over the pipe length.

One study on sediment rates reported values from 30 to 24500 mg/m (Carriere et al., 2005) and another gave values of 260 to 410 mg/m (Barbeau et al., 2005). These sediment rates, however, were based on longer flushing times than required to get one pipe turnover. Consequently, the rates are higher because upstream sediment was also entrained in the pipe. Carrière explicitly reports a stop criterion for flushing of 1 FTU, but that at several locations values below 5 FTU could not be reached, meaning a constant sediment entrainment at the sampling location. This leads to an overestimation of the sediment rate in the considered pipe.

Table 6-11 Calculated sediment layer depth

	Diam [mm]	TSS [mg/l]	VSS [mg/l]	sed layer depth [µm]	sed rate [mg/m]
BE	150,0	11,3	5,1	16	3,93
RO	101,6	6,6	3,3	6	5,00
KO A	101,6	325,0	135,0	314	247,28
KO B	101,6	237,7	102,0	230	180,83
KO C	101,6	166,5	75,5	161	126,68
AM A	101,6	12,0	3,6	12	9,13
AM C	76,2	36,0	6,5	26	48,69
AM D	127,0	130,0	21,0	157	63,30
AM B	101,6	40,0	10,0	39	30,43
AM E	250,0	5,5	0,8	13	0,69
AM F	101,6	32,0	7,4	31	24,35

The method of harvesting the sediments from the system was not uniform in the studies on sediment composition. Barbeau (2005) used unidirectional flushing with a velocity of 1,8 m/s, while Zacheus (2001) and Torvinen (2004) used the pigging method. Gauthier(2001) and Carrière(2005) flushed with several flushing velocities (0,65 to 2,3 m/s). Also, the flushing time was not defined; Barbeau(2005) used a set point of 25 minutes which, in that case, meant at least 5 turnovers of the pipe. Considering the diameters of the pipes (203 mm), this must have had an effect upstream of the pipes.
The technique for cleaning and sampling will influence the composition of the deposits present in the water samples. Moreover, the pigging procedure was not uniform for the number of pigs used per length of pipe nor for the velocity with which the pigs propagated through the system. Pigs will detach cohesive layers, but probably have a limited transport capacity for the sediment (Vreeburg, 1996). At longer lengths the pigs will roll over the sediment instead of transporting it, and then sediments come out in short plugs just before the pig emerges. Consequently the results varied widely in an absolute sense (Zacheus et al., 2001) and did not correlate to the type of water or pipe material. The general conclusion that soft deposits play a mayor role in the microbial growth in a network, however, holds true considering the VSS level of the samples.

Though not intended the samples at the Amsterdam location were flushed with different velocities. This clearly showed that with the low velocities also less sediment is removed with the flushing. The sharp rise in the removed sediment with increasing velocity at 1,4 m/s showed the validity of the threshold of 1,5 m/s that is used in the Netherlands. It also showed the aforementioned influence of the flushing velocity on the measured level of sediments.

The sediment layer depth proved to be very thin, but this layer already caused high turbidity when resuspended. This could also be observed in the jars during the experiments, where after settling only thin layers of sediment could be observed. In practise this limits the value of visual observations through laparoscopic inspection of pipes for sediments.

In the Koudum locations under relatively identical conditions different amounts of sediment were removed during flushing. All the locations are part of conventional ‘in-the-street’ networks that were strongly looped and dimensioned to satisfy the fire demand that largely exceeded the actual drinking water demand (Snyder and Deb, 2003). Actual flow velocities in the network were difficult to determine either by measuring or modelling (Blokker and Vreeburg, 2005). Differences in velocity are a plausible explanation for the differences in sediment layer next to the possible influence of level differences in the pipes. A difference in level of the pipe bottom introduces a gravitational force parallel to the axis of the pipe. Through a bed load transport the sediment can move to the lowest point in the pipe. These variations in velocity and level are subtle and never taken into account in the design process of the network.

The JTE test on the sediment from the Koudum locations showed the anticipated difference in turbidity at the middle position and the bottom position for sediments that have a certain amount of light components. The sediments from the Amsterdam location did not show this difference, neither did the samples from the Berlicum and Rosmalen locations. The difference in light and heavier components was primarily in the iron content of the sediment. If the iron originated from the treatment plant or was a product of post-coagulation, then it would be of a flocculant nature and would have low density. This was the case in the Koudum samples. If the iron originated from corrosion, which was more likely at the Amsterdam locations, than the iron would be in the crystalline form, such as goethite (Alpha-FeOOH) with a higher density, which would explain the JTE results.

The repeated tests on the stored samples after four weeks showed that the age of the sediment did not influence the JTE results or the composition in heavier or lighter material. Within the sensitivity of the JTE, the reaction was identical, as is shown in Fig 6-19. This indicates that the compacting of sediment did not take place within this period of a few weeks. As the samples were stored in 20 litre jerry cans with a height of about 40 cm, the sediment layer at the bottom of the can was much thicker than the layer in the network and that did not have an influence on the compacting. Obviously, possible chemical or biological processes during the storage also did not change the characteristics of the sediment.

The data on the water quality at the treatment plants indicated some differences in water quality especially on the iron and manganese levels. The levels at Nuland and Loosbroek were higher than the levels at the Leiduin and Weesperkarspel treatment plants, but that did not result in very different levels of sediment. The levels at Spannenburg were the highest. During other research at this plant it turned out that the backwashing program had a significant influence on the turbidity and consequently the iron content of the water. The spikes in turbidity, mostly with a duration of one to two hours, proved there was a large loading on the system which showed in the high sediment levels found in the pipes.

The critical stirring rate (CSR) did not vary significantly across the samples. This showed that the JTE is not sufficiently sensitive towards characteristics of the beginning of resuspension. This was already expected from the pre-experiments with synthetic sediments, but confirmed in the tests with the actual sediment. All the sediments were resuspended at an impeller speed of 30 RPM. As the original objective of the JTE was to get an optimal mixing it is not surprising that at the lower rates a good resuspension could already be reached. This might be influenced, for instance, by taking out the baffles from the jars, which would get a more uniform flow pattern at the bottom in such a way that the beginning of movement would be observed better. This would, however, compromise the uniformity of the JTE. Another possibility would be to vary the stirring rate in smaller steps from 1 to 30 RPM.

The experiments with the JTE had an explorative nature to see if there was a clear difference in sediments from different treatment plants and pipe materials. As the CSR did not vary significantly across the samples, this shows either that there is not much difference between the hydraulic capacities of the different samples of that the JTE is not the right instrument to assess these differences. Because the shear stresses and the velocity distribution over the geometry of the jars is not uniform this should be explored in more detail before the conclusion can be positively made about the appropriateness of the JTE for this purpose.

6.4.5 Conclusions case sediment analysis

The major components of distribution system drinking water sediments are from organic matter and iron hydroxides. They account for 45 to 75% of all the sediment in the considered samples, which is similar to other studies. The JTE did not conclusively proved to be a good tool to determine the hydraulic capacities of the sediments, but it has potential that should be further explored. Judging the difference in turbidity at different levels in the jars gave qualitative information about the density of the sediments and led to conclusions about the origin of the iron content being the lighter flocculant form (treatment) or the heavier crystalline form (corrosion).
The JTE showed that hydraulic capacities of the sediment, within the range the JTE can test, did not change after storage of the samples during several weeks. Considering the time consuming nature of the test procedure this makes it possible to test multiple samples that were taken in relatively short time, for instance during a flushing program.

The absolute sediment levels drawn from one system can be largely influenced by the flushing velocity applied, as was observed in the Amsterdam case. At the velocity of 1,5 m/s considerably more sediment was flushed out than with lower velocities. Velocity and pipe level play an important though subtle role as shown by the Koudum case in which under seemingly similar circumstances different levels of sediment were found.
Iron originating from a conventional groundwater treatment plant can accumulate to higher levels of iron in flushed samples than iron levels as a result of corrosion.

6.5 Conclusions

The (in)stability of the water with regard to the particulate matter has a large influence on the growth of the particulate matter in the distribution system. The difference in particle volume concentration growth between the two transport systems studied, was mainly determined by this stability, resulting for one system in a growth of particle volume concentration with a factor of 32 and for the other system to negligible growth of particle volume concentration. The surface-water based treatment produced more stable water with respect to particle volume than the artificial-groundwater based treatment. In the latter case the production of particles was not significantly dominated by one process. Flocculation, leaching and biological growth seemed to account equally for the increase in particulate matter. The increased iron levels could also be retrieved from the sediments found in the distribution network, though this was based on only a few samples.

The method of TILVS and the application of the Jar Test Equipment are promising measuring methods that characterise the composition and type of sediment.

7 Summary and conclusion

7.1 Historical development

The drinking water distribution system has long been the working field of mere technicians. The design and construction of the network is considered to be more a matter of skilled labour and less that of engineers and scientists. The explanation for this might be the perception that the network is a relatively simple technical construction with the first priority being to supply enough transport capacity and integrity to fulfil the basic goal of a water company: ‘supply sufficient water in sufficient quantities’. The network was long considered to play a minor role in the total scheme of water quality management, and the focus on capacity led to a situation, especially for the distribution network, of over-dimensioning, resulting in long residence times and even to the stagnation of flows.

Historically, over-dimensioning the distribution network was primarily caused by the interdependency of the drinking water distribution system with the supply of water for fire fighting. The need to supply fire flows dominates the dimensions of the pipes leading to a network that is grossly over-dimensioned with respect to the actual drinking water supply. The appearance of the network, though, does not suggest this over-dimensioning. Pipe diameters of 100 to 150 mm don’t appear to be too big. Moreover, the general perception is that the investment costs of the pipes are only partly determined by the material costs, making a more detailed design not worth the trouble.
A second important reason for the over-dimensioning of the distribution network is that a distribution network’s layout design is based on the design principles for reliability of a transport network. The pipes should be looped to guarantee a ‘double-sided’ supply in case of failure. The goal is to increase the continuity of the supply or in more common words: “If the water cannot come from one side, there is an alternative on the other side.” For a transport network, this effectively increases the continuity of supply because there are usually no direct connections to the network. Failure of a pipe does not affect directly the water supply, but only the transport capacity. Transport networks should therefore be designed in such a way that, in case of the failure of a pipe, enough water can still be supplied to centres of demand (Vreeburg et al., 1994).
For a distribution network, however, the supply connections are made directly to the pipes. Failure of a pipe, consequently, stops the supply of drinking water to those connections and the looping of the distribution network does not increase the continuity of supply.

The water quality issue discolouration that actually is connected to the role of the network is, however, over- simplified: discolouration is caused by corrosion of unlined cast iron pipes. Within the context of the first historical networks this is a logical presumption, because, then, networks were exclusively made of cast iron. One image of a heavily encrusted inner pipe wall explains this simplification of the problem. The search for new materials that would not corrode led to the use of asbestos cement and PVC as pipe material, but these new materials did not solve the problem.

As was said previously, the distribution network is a technician’s world which is reasonable given the complexity of the construction of early networks. Making joints and handling the heavy cast iron pipes demanded craftsmanship. Some examples are given in the pictures of Fig 7-1 showing the manufacturing of a lead joint in a cast iron pipe, explaining the naming of the profession ‘plumber’ in English and ‘loodgieter’ in Dutch.

Fig 7-1 Pre-1940 pipe laying with lead joint

Water quality changes and hydraulics have more aspects than the technicians and plumbers are trained for. Easy solutions for complex water quality issues, such as conventional flushing, did not work satisfactorily and discolouration problems were considered almost unavoidable.

At the start of the 1990s the problem of discolouration, however, got attention in the research program of the Dutch Water Companies (BTO, an acronym for the Dutch “BedrijfsTakOnderzoek”, the joint research program of the Dutch water companies). The direct cause of the attention was what resulted from a research project initiated by the water company in the city of Eindhoven (then, NRE, Nutsbedrijf Regio Eindhoven, but now merged into the company Brabant Water) in which continuous monitors were used to explore the cause and nature of discolouration problems. This research resulted in the first setup of what eventually led to the model of particle-related processes, as is shown in Fig 1-5 (Chapter 1). Counter measures that were introduced in the course of 1992 were the systematic cleaning of the network based on the principles of good water flushing (1,5 m/s, three times the turnover volume pipe and a clear water front) and a raised awareness of the treatment operators on the effects of peaks of turbidity. This awareness initiated some game-like competition between operators to achieve the best and most stable turbidity in the outgoing water. As the water company had a very good registration system of complaints, the effects could be monitored with the complaint registration system, which resulted in a spectacular drop in complaints, as is shown in Fig 2-18.

The development of a motorised wrench to exercise valves that could be mounted on a bike is anecdotal. The bike mounts were a convenient way to operate the many valves that were necessary to guide flows to hydrants during the flushing program. Bikes were much easier to move around than cars. This is an example of the willingness of the company to go beyond the historical developments, and it also ‘infected’ some of the other companies to adopt the results of the research as well. Water Company South Kennemerland (now part of the PWN Water Company) was one of the first to adopt the new concepts for a network design for branched-type networks.

The results of the research projects within the framework of the BTO on discolouration were very quickly disseminated in the peer-to-peer groups that worked around the research groups known as the ‘Regional Distribution Groups’. Originally five groups, each with about ten companies convened three to four times per year to discuss operational issues and research results. The groups were technically-orientated and solution-focused. The relatively simple model of particle-related processes appealed to their common sense. The translation of this model into an effective set of rules for water flushing that is potentially as effective as other much more costly methods, like water/air scouring and pigging, enhanced the acceptance of the particle concept significantly.

Practitioners such as network operators are by nature flexible and can adopt to new concepts easily, especially when those concepts turn out to be effective and cost saving. The same practical nature is, however, a drawback because attention to scientific proof is not always a priority. The idea “It works, doesn’t it?” is also the reason why historical mistakes were allowed to continue for a long period.

In almost every water company, there used to be a separation between the production department and the distribution department. The research on discolouration started in the network and the success of the cleaning programs, together with the new design rules, proved that the discolouration problem could be managed within the boundaries of the network. The cleaning programs are, however, a true end-of-pipe solution that can be thought of as dealing with the symptoms. High-velocity networks are better geared to prevent discolouration and are a more sustainable solution.
The separation between the production and distribution departments frustrated, for a while, the next step in the prevention of discolouration by limiting the introduction of particles in the first place. In the joint research program of the Dutch water companies (BTO), the Delft Cluster Research Program and in the research program of the Delft University of Technology room was made to get a more scientific foundation for the empirical outcomes but also to explore the possibilities of preventing the introduction of particles.

7.2 Measuring methods

For this research, several new strategies of measuring were developed, of which the continuous measuring of particle-related parameters like turbidity and particle counters are the most important. Though the continuous monitoring of turbidity is the method that has been used the longest (van den Hoven and Vreeburg, 1992), the application was not common within the drinking water network. The quality of the drinking water leaving the treatment plant was primarily determined with the obligatory sampling program. Increasingly, however, water companies used the continuous monitoring of turbidity at pumping stations to measure the effect of the operation of the treatment process on water quality.
Recently, particle counters have become available, which has enabled a more quantitative measurement for the calculation of the particle volume entering the network and its development within the network. The calculated particle volume, together with the particle size distribution as applied in this thesis, gives information on what happens to the particles in the network and where the possibilities to reduce the particle load are.
To calculate the particle volume the particle counts were sufficiently fine-meshed and there was no need for modelling as Ceronio (2005) developed. Their work showed that particles larger than 30 µm were rare and that within the 32 available 1 µm size ranges all relevant particle could be counted. This made further modelling needless, as the actual data could be used. The large quantities of data were quite manageable within current spreadsheet programs. The approach to use modelling to fingerprint the origin of the particles is not further explored in this thesis.

With the results and success of the continuous monitoring at the treatment plant and in the network, a discussion has started on the added value of permanent water quality monitors in the network. This discussion is inspired by the on-line pressure monitoring that is applied to manage the pressure situation in the network. The comparison of these quantity-based pressure measurements with the quality-based turbidity/particle count measurements is not simple.
Water quality monitors will not function as operational tools for immediate intervention as the pressure monitors do. If, for instance, pressure unexpectedly drops in the network, typically an acute cause can be found and dealt with. Particle counters or turbidity monitors in the network, however, give information on the regular processes that cannot be interfered with at

the time of measuring. Moreover, effects can only be analysed over an extended measuring period and data have to be processed prior to interpretation. Then, for instance, a regular loss of particles can be identified but not immediately be influenced with a specific action. Continuous monitoring at the treatment plant is useful to make operators aware of the 'normal' low peaks in turbidity or particle counts. Each peak adds to the sediment layer in the network, and every peak that can be avoided delays the accumulation. The monitors provide information that allows operators to be more alert and to avoid the peaks from happening in the future. This differs fundamentally to on-line pressure management.
Water quality monitors like turbidimeters and particle counters in the network are best employed for a dedicated objective, for instance to analyse the RPM or to see how the particle content of the treated water changes during the transport and distribution. In discussions with network operators on the costs and amount of equipment, the end-of-the-day advice is to have three to four mobile systems that are flexible and easy to handle. Conscience application of this equipment to answer concrete questions is more effective than rigid application of fixed monitors without a dedicated purpose.

The Resuspension Potential Method (RPM) has proven to be a powerful tool to analyse discolouration risk in a network. The flexibility in the ranking system makes it possible to tune the sensitivity and range to the needs of specific systems. The ranking system used in the Venlo system (see sectionh 5.7) is different from that of the Franeker case in Chapter 1. In both cases the RPM serves as a comparative tool to measure the effects of (remedial) actions. The Venlo case, discussed in section 5.7, is part of a larger analysis system that, in total, contains 695 locations in an 8500 km network monitored on a regular basis. The goal of the analysis system is to pinpoint areas with the highest discolouration risk. These high ranking areas are cleaned in the order of declining averages of RPM so the limited cleaning resources can be optimally applied. The cleaning resources are limited by the manpower available to perform the dedicated flushing actions.
In the Franeker case the objectives were to see how long it would take to recharge the system to the initial discolouration risk level and to evaluate the difference between the two systems. These objectives demanded a lower absolute turbidity threshold in the RPM ranking than the Venlo case. It can also be imagined that in the Venlo case the ranking system was gradually adjusted to lower absolute turbidity levels, because the overall discolouration risk decreased as a result of the optimal use of the cleaning resources.
The RPM analysis typically demands a flexible system of equipment that can be applied to hydrants. In both aforementioned cases, small trucks were designed that are dedicated to this type of work and are operated by trained staff. In this way it is possible to perform 4 to 6 measurements per day per truck. It is clear that with every RPM a certain amount of sediment is removed, influencing the 'virginity' of the measuring location. This matter is not evaluated thoroughly within the scope of this study, but there are indications that in low loading systems the recharging of the system is so low that a yearly repetition of the RPM is sufficient to remove the accumulated particles from that year. In the Venlo case obviously the accumulation is enough to fill the gap of the previous RPM. It could, however, also mean that the accumulation in the rest of the network is actually higher than would follow from the RPM locations analysed. For that reason RPM analysis requires a number of locations at which measurements should alternate.

The possibilities of concentration methods like the TILVS and the Hemoflow are explored in this research and they prove to be promising but not yet fully developed. They allow for a closer analysis of the particle process, but are not yet reliable enough to be used on a routine base. The strength of the methods is that the pre-concentration is a relatively straightforward

procedure. The weakness is that the circumstances in which the measurements are done are often not very favourable. Moreover, the methods are time consuming and require a fairly steady process or a sufficiently high enough number of analyses to give sound information. Unreported in this research are several trials with the TILVS that were unsuccessful because of difficulties with the equipment and an obvious lack of experience. Also, the actual amount of material caught on the filter paper was frequently too low to be processed (less than 2,5 mg).
Based on these experiences, four new pieces of TILVS equipment have been made that can be used in future research, so the method can be further developed.
The Hemoflow was only used during the last period of the experiment with the particle-free water in the distribution network. Due to inexperience, the analysis made on the concentrated samples was not adequate. Also, it seems that a lot of the material to be concentrated was left in the filter itself. The colour changed during the analysis period, suggesting some material was left in the filter. As the total mass of the particulate matter was in the order of 100 mg, it is plausible that a similar amount could have been left in the filter compromising the accuracy of the measurement.
Despite these disappointments, the strength of the methods was that they allowed for a more accurate analysis of the particle load and that they combined with a relatively simple sampling procedure that didn’t require highly trained personnel.

The setup of the research on the effect of particles in treated water by isolating two areas in a real network has proven to be very successful. It allowed for the analysis of the effect of one factor, particle load, while keeping all the other relevant factors like pipe material and hydraulics, relatively constant. This approach can also be applied to other dedicated research in the network, for instance, the effect of cast iron on water quality by isolating a cast iron network and a non-cast iron network with the same dimensions in one supply area. The biggest problem in the research was keeping the isolated areas undisturbed during the analysis period. The major success factors were patience and good registration, and there were unique opportunities for objectified analysis of different items involved in the discolouration processes.

The use of test rigs and artificial pipe loops was limited in this study. The first priority was to understand how processes run in the actual situation. Mimicking the real situation in pipe loops was tried in a test rig, for which some pictures are shown in section 4.2. The results of these test rig experiments were disappointing. The phenomena to be studied, accumulation under different hydraulic circumstances, are difficult to translate to test rigs. The test rig that was used to analyse the phenomena of water/air scouring, however, was successful because it had the dimensions of a real-life system.
The further development of test rigs, therefore, can be of major importance for the further understanding of micro-mechanisms in the processes in the network. The eye-opening effect of particles settling over the complete circumferential of the pipe urgently requires further investigation to get more knowledge on the micro world of particle settling in the vicinity of the wall. It will also have an effect on the study of biofilms and their growth in the network.

7.3 Treatment

The effect of particles in drinking water can well be observed in the propagation and development of peaks in turbidity or particle counts in the network. The shape of the peak fingerprints an amount of water that subsequently can be followed in the network. First, residence times in the network can be precisely determined (see section 2.2.4). Secondly, the

changes in water quality can be analysed specifically, and other parameters can also be continuously monitored.
Turbidity is not a quantitative measurement, meaning that it cannot be used to determine a particle load. Particle counters and the calculated particle volume give more quantitative information. The presentation of measurements using the frequency distribution of the calculated particle volume concentration, together with the average value, the percentile ratios and the Surf-90% and Surf+90%, proves to be powerful in characterising the patterns. The overall effect of turbidity and calculated particle volume in the network can conveniently be seen using these parameters, as is shown in sections 2.2.5 and 3.4. Though these parameters look very promising and useful in interpreting the measurements and disclose the particle-related processes, a lot of experience still has to be gained. Combining the particle counts and the elemental analysis of the water could be an especially fruitful possibility deserving further exploration.

Within this thesis two types of treatment were analysed directly that proved that the particle volume concentration at the treatment plant was not the only parameter that could account for the particle volume in the network. In the artificial groundwater based multi-barrier treatment (section 6.2), post-flocculation of iron hydroxide caused a considerable particle load in the distribution network as did the leaching of the cementituous pipe material and a biological growth process. The surface water based multi-barrier system produced a more stable type of water that did not put an extra particle load to the distribution network after the transport. The initial particle volume at the surface water treatment plant was a factor 4 higher, 0,74 ppb versus 2,81 ppb. At the end of the transport system the average particle volume of the artificial groundwater based system grew to a level of 27 to 62 ppb, while the average particle volume in the surface water treatment based system stayed constant in the first part of the system and grew to a level of 4,47 ppb in the second part.
The stability of the water towards post-flocculation-potential is not a single parameter but will be an important indicator for the particle load to or in the network. The total iron and manganese level are not sufficient as they indicate the soluble as well as the particulate form. The soluble fraction can grow to a particulate form that adds to the sediment load of/in the network Theoretically a TILVS-analysis distinguishes the particulate fraction of the iron and the soluble fraction and could function as part of the parameter set that characterises the post-flocculation-potential of the water. There is, however, still a lot of development and validation of these parameters necessary to get suitable target values.

Using the Surf-90% and the Surf+90% shows that a large part of the particle load in the system occurs in a relatively short time. Sometimes, in less than 10% of the time more than 50% of the particle volume is loaded on the system (Surf+90% > 50%). A suggestion for improving the sediment load of the treatment without adding new treatment steps is to concentrate on decreasing the peaks. This means decreasing the Surf+90% or increasing the Surf-90%. Suggested values for the Surf+90% are less than 15% and subsequently for the Surf-90% is more than 85%. Illustrative are the particle count figures as shown in Fig 2-9 and in the turbidity graph in Fig 3-4. These graphs show that the peaks in turbidity and particle counts are mostly connected to the backwashing of filters. Those peaks decrease in the network, indicating a loss of particles that accumulate in the pipe and constitute a discolouration risk. A first step in improving the treatment process is to decrease these short period loads that are responsible for high Surf+90% values. In section 3.4 the effect of peaks is shown for a more distant location (start of the Reference Area), but it is also clear that peaks contribute greatly to the load in the system.

The case study on the composition and hydraulic behaviour of sediment (Chapter 1) confirmed the role of treatment in loading the network with particles versus the role of corrosion of unprotected cast iron. The sediment levels in a cast iron pipe with a low particle load treatment plant were lower than the levels in non-cast iron pipes supplied by a higher particle load treatment plant under comparable hydraulic circumstances. Though the low level particle load at the treatment plant (which was the groundwater based multi-barrier system discussed previously) increased considerably in the low pressure transport system, still the absolute levels of particulate iron were low compared to the load in the Reference Area discussed in Chapter 1.

This thesis concentrated primarily on the behaviour of particles and did not consider the biological implications. Research into the relationship between turbidity, particle counts and *Cryptosporidium* levels showed that this relationship is not uniform (Huck et al., 2002). Especially during suboptimal situations, there is a relationship between particle counts and *Cryptosporidium* removal. Turbidity was a less effective parameter to monitor the filter performance because it was less directly related to *Cryptosporidium* removal.
This indicates that optimising a treatment process on particle counts probably also has an effect on the biological stability of the water.

Further research into the effect of particles in the treated water and their fate in the network is highly recommended to get more insight into how to prevent the loading in the distribution system and possibly improve it’s biological stability.

7.4 High-velocity networks

Within the model of particle-related processes, control of the accumulation of particles by controlling the velocity is one of the ways to manage the discolouration risk. Velocity should be controlled in such a way that the particles, settled during low demand periods, are resuspended during high demand periods, preferably on a daily base. Effectively, that would mean that on a daily base no net accumulation of particles will occur.
Though it is a logical consequence of the model, it was difficult to persuade water companies to adopt new design rules that would lead to high-velocity networks. A main reason for that is that the consequence of the new design is a network that is branched instead of looped. This is counter intuitive to the common concepts for networks that prescribe looping for reliability and forbid dead ends for water quality reasons. The basic misconception is that there is a difference between transport and distribution networks. The reliability concept for transport networks (Vreeburg et al., 1994) directly translated to distribution networks doesn’t lead to reliable distribution networks.
For the reliability of transport networks, the pipes should be looped to guarantee a ‘double-sided’ supply in case of failure. The goal is to increase the continuity of supply or in more common words: “If the water cannot come from one side, there is an alternative on the other side”. For a transport network this effectively increases the continuity of supply because usually there are no direct connections to the transport pipes. Transport networks should be designed in such a way that in case of the failure of a pipe enough water can still be supplied to centres of demand (Vreeburg et al., 1994).
For a distribution network, however, the supply connections are made directly to the pipes. Failure of a pipe consequently stops the supply of drinking water to those connections and the looping of the distribution network does not increase the continuity of supply. Increasing the reliability of the network has to be translated into increasing the supply continuity. Once that switch is made, it is clear that the time of interruption is a decisive factor and not the looping.

As a spin-off, the location of valves in existing networks must be analysed critically, resulting in higher reliability, or continuity of supply, with fewer valves (Trietsch and Vreeburg, 2006).

The presumed adverse effects of dead ends on water quality are mainly based on so-called classic dead ends that connect a distant fire hydrant to the network. For the fire hydrant a relatively large pipe is needed, causing long residence times in this pipe as there is little actual demand. This will have a negative effect on water quality. The dead ends in the new networks are better described with the term ‘flowing ends’. The pipes are dimensioned on the actual water demand in such a way that a velocity of 0,4 m/s will occur once a day, removing the accumulated sediment.

The demand situation in a distribution network is highly variable over a 24-hour period. For the design of distribution pipes, detailed insight into the patterns of an individual connection is necessary. This insight was lacking at the time the design rules were implemented. To overcome this lack of knowledge, a practical solution was found by using the design rules for inner installations: the $q\sqrt{n}$ approach. As is described in Chapter 1, this design leads to a network that is self-cleaning within the time scale the research was done. Further development of a stochastic demand allocation model shows that actual demands are underestimated towards the $q\sqrt{n}$ estimation (Blokker et al., 2006). Still, the network that has been designed proved to be self-cleaning, obviously leading to the conclusion that the velocity required to remove the sediment on a daily basis is lower than the estimated 0,4 m/s.
Further development of the SIMDEUM model (Blokker et al., 2006) will give increased insight into the actual demands and velocities in the network without the need for meticulous measuring, which is virtually impossible in real time. However, with this knowledge, also a better estimation of the real resuspension velocity is necessary so the design can be optimised. The conclusion up until now, though is that the combination $q\sqrt{n}$ – 0,4 m/s gives a self-cleaning network and can be applied as such.

From the perspective of fire fighters, the hydrants in the drinking water network are a “natural source” of water. This is understandable given the historic situation that fire fighting and drinking water belonged to the same municipal department, Public Services. This situation however stopped the need for critical analysis of this water source for a long time. An international survey on the standards and legislation on fire flow supply (Snyder and Deb, 2003) showed that formal arrangements are lacking, but also general insight into the actual need is rare. In discussions with fire fighters in the Netherlands, it became clear that even the 30 m^3/h is still twice as much as is actually used for the first attack of the fire.
The new design rules forced water companies and fire fighting departments to sit down and discuss the requirements for protecting people from fires. In those discussions the added value of sprinkler installations was brought up for the protection of typical houses. The test case in Scottsdale, Arizona, is very convincing for the added value of domestic sprinklers (Ford, 1997). However, the possible impact of domestic sprinklers is still underestimated and under-researched.

The original goal of the new design rules was to improve the water quality and to reduce the discolouration risk. This original goal has been almost completely surpassed by the economical effects and the sometimes emotional discussions with both the water companies and the fire fighters. However, the comparative measurements as presented in section 4.5 convincingly show that the hypothesised self-cleaning effect can actually be observed. In the aforementioned emotional discussions, this has almost degraded to a minor argument, but should still be a major driver to improve the design of the distribution networks.

The new approach for designing distribution networks showed that the conventional approach is hardly a systematic procedure. Practically, one can say that the development of a distribution network has not changed significantly since the beginning of the public drinking water supply. Though the original goal of the new design was to improve the water quality, it turned out to be the start of a critical review of the design principles for drinking water distribution networks. Wider application of these new design rules in, for instance, the rehabilitation of networks will require adjustments, but potentially can save 10 to 20% on investment costs. Applied to the UN Millennium Development Goal to reduce by half the proportion of people without sustainable access to safe drinking water, this would mean that this can be done with less money or reduce the proportion even further with the same amount of money.

7.5 Cleaning methods

Conventional flushing was historically the most important answer to local discolouration incidents, next to the rehabilitation of unlined cast iron pipes. Chronologically, cleaning of networks was the first aspect of the particle-related model that was researched. With the introduction of the continuous monitoring of turbidity and the RPM, tools became available to evaluate the effect of cleaning of networks (Vreeburg, 1996).
The adverse effect of a too low velocity on the mobility of sediment could explain why conventional flushing is counter effective, as is the aggressive cleaning of unlined cast iron pipes. This demystification of the effects of cleaning, together with the development of clear rules for effective cleaning through flushing, upgraded the cleaning methods from low-end work to a high-end design process. Several water companies spend as much as one or two man-years towards the development of flushing plans. These plans are based on all-pipe network models that could rather easily be extracted from Geographical Information Systems. The advantage of this application of network calculations is that it is not necessary to model the normal demand precisely. The flushing volume is so much higher than the normal demand that the latter can be neglected within the calculation.

The initial base for 1,5 m/s as a minimum threshold for water flushing was dictated by the need for a practical guideline. An important argument was its practical feasibility without the need for large investments and indications that it would work (Fig 5-2). Together with the requirement for a clear water front, it proved in practise to work very convincingly for the practitioners. A noteworthy anecdote was the flushing action of a Ø400 PVC pipe with a length of 5 km. Shortly after midnight, technicians started flushing, and in ten minutes the flushed water was so turbid that they sceptically commented on the time that would be required for the water to clear: at least to sunrise. After exactly one pipe turnover, approximately one hour, the turbidity dropped sharply and after two hours (two turnovers) the water was completely clear. These results were very quickly communicated across informal circuits and the requirements for flushing were rapidly adopted and applied.

One of the traditional disadvantages of water flushing is the seeming waste of water. This is, however, the image of conventional flushing in which a fire hydrant is opened and let run for as long as it takes for the water to clear. With the effects mentioned on turbidity in adjacent pipes, this can take a long time, indeed, leading to an extensive waste of water. With the planned unidirectional flushing, 300 metres of pipe can be refreshed three times within 10 minutes, or two times within 6,5 minutes. The use of water is limited to a maximum of three times the contents of the network. The actual period with an increased turbidity during flushing is limited for the example pipe of 300 metres to 6 to 10 minutes.

A good flushing plan uses at maximum three times the volume of the network. For the Netherlands' situation this means that a complete flushing of the entire network demands 0,53 to 0,94% of the total yearly demand. The network in the Netherlands does not different from any other drinking water network, which means that this figure can be applied to many networks.

With the RPM it can also be made clear that the efficacy of a cleaning action is not so much the amount of sediment taken out of the pipes, but the amount of sediment that is left in the system. Pigging is often intuitively considered an effective method, based on the extremely dirty water that emerges from the pipe just before the pig comes out of the pipe. But if this amount is divided over the total volume of the pipe, the result would not be that spectacular. The research also showed that most alternative methods to water flushing use more water instead of less.

The effect of cleaning pipes can have an effect on the biofilm in the pipes, though that aspect has not been included in this study. The flushing of pipes has primarily concentrated on the removal of loose sediment and that probably will not damage the biofilm itself, as it is attached to the wall. Mechanical cleaning methods such as pigging will affect the biofilm, though it probably is difficult to actually remove the sticky film. On the other hand, the effect of removing a biofilm does not have a lasting effect if nothing is changed in the Biofilm Formation Potential of the water (van der Kooij et al., 2003). The biofilm is probably relatively quickly recharged again. In a biofilm monitor, the development of a biofilm is relatively fast up to a certain equilibrium level in a few months (van der Kooij et al., 2003).

7.6 Conclusions

The objective of this thesis was to analyse the particle-related processes involved in the generation of discolouration problems in the network. To this end, new measuring methods have been developed such as continuous monitoring of turbidity and particle count, the Resuspension Potential Method (RPM), and the Time Integrated Large Volume Sampler (TILVS). With these new methods the discolouration problem could be seen as related to loose deposits in the network. The incidental resuspension of accumulated loose particles is the main cause of discolouration events in the network. The origin of the particles is mainly the treated drinking water, followed by processes in the network like post-flocculation, corrosion and leaching and biological growth and regrowth.
Irrespective of the cause of the particles the accumulation to layers of loose deposits can initiate water quality problems. Managing the accumulation is possible through managing the velocity in the pipes and through removing the loose deposits through effective cleaning.

As remedial actions to manage and control the discolouration risk in the network, three stages of measures have been identified:

- Prevent particles from entering the network or being formed within the network. Particles in the treated water are the main source of accumulated deposits in the network, especially the particles that are released to the network due to variations in the operation of treatment processes. There are several ways to limit particle release into the network, depending on the dominant cause:
    - Improve the treatment system: aimed at the removal of particles, for instance with the addition of an extra polishing/filtration step.
    - Improve the existing treatment process in such a way that peaks are avoided which may lead to a substantial reduction of particle load on the distribution system.
    - Adjust the treatment process to decrease the corrosivity or the post-flocculation.

The parameters suitable to monitor the particle load from a treatment plant to a distribution system are the Surf-90% and Surf+90% together with the average of the calculated particle volume. Based on the experience up till now a Surf-90% of 80 to 85% or higher of the average particle volume characterises a stable operated treatment process. A goal for an average particle volume alone is not enough, but should be accompanied with parameters that characterise the post-flocculation potential of the water, the biofilm formation potential (van der Kooij et al., 2003) and the corrosion-index (van den Hoven and van Eekeren, 1988). The total iron and manganese content are preferably accompanied by a TILVS-analysis to determine the particulate fraction after sufficient validation of these parameters for this purpose.

- Prevent particles from accumulating
  Particles accumulate specifically in the distribution network as a consequence of the low velocities. Conventional distribution networks are designed to supply a high volume flow for fire fighting. Reduction of this fire flow, together with a new approach to network design with branched sections and declining diameter to sustain a velocity of 0,4 m/s once a day, based on demand estimation following the $q\sqrt{n}$ methodology, results in self-cleaning networks that do not accumulate loose deposits.
- Remove accumulated sediment
  The regular cleaning of networks prevents the accumulation of sediment to unacceptable levels. The critical level of accumulation can be measured with the Resuspension Potential Method that is also used to evaluate the efficacy of the cleaning method. Flushing with water under the operational conditions of 1,5 m/s (i.e., two to three turnovers of the pipe content and a clear water front) is effective to remove the deposits that cause the discolouration risk.

7.7 Recommendations for future developments/research

The research described in this study discloses some of the particle-related processes in the network and shows that unconventional approaches to the distribution system led to new operational guidelines. Those new guidelines are controversial against the common belief, up until now:

- Discolouration is mainly caused by particles from the drinking water itself versus the common belief that unprotected cast iron is the main source for discolouration problems.
- Cleaning with water flushing is effective under strict operational circumstances versus ineffective conventional flushing.
- Distribution networks should be shaped as branched networks with a declining diameter and high velocities versus looped networks with large diameter pipes.

The application of the new operational guidelines demands a lot of flexibility for all parties involved with the designing, building and operation of networks. But, it also demands more proof and substantiating to facilitate the actual implementation. The following recommendations are seen as the most important fields for future research:

- Analysis of the effect of improvement of treatment either by adding new steps or by better operation of existing treatment steps:
  As this study shows it is likely that distribution problems can be prevented or at least contained if the treated water meets certain standards. A new definition of particulate water quality towards distribution stability must be developed analogous to the biological stability (van der Kooij et al., 2003) or chemical stability (van den Hoven and van

Eekeren, 1988), including an on-line and mobile measurement technique that can be applied both at the treatment plant as well as roving through the distribution system.

- Development of water quality models that deal with particles:
  Presently the limits of water quality modelling are determined by the accuracy of the hydraulic water movement calculation. Development of the model SIMDEUM as proposed by Blokker et al (2005) will bring the water quality modelling a step further and will enable more accurate water quality modelling.
  This new base for water quality modelling should also extend the model parameters beyond the commonly modelled disinfectant residual. Obviously particle movement is one of the new parameters, but also modelling DOC, AOC, iron and manganese will enhance the possibilities for network maintenance.
- Development of test rigs for closer analysis of the micro-processes that govern the settling and resuspension of particles in the piped network. A challenge in developing pipe rigs is to either accelerate or concentrate the processes in order to get a reasonable time frame for test procedures.
- Further development and validation of measuring methods like the TILVS and Hemoflow:
  The concentration methods are vital for establishing more knowledge on the composition of particulate matter in the water and the effects on the build-up of sedimentary layers. The methodology must be standardised and provided with a good process description. In due course a database can be filled with the elemental composition of particulate matter in different water types and the effects of that in the network.
  The combination of the TILVS together with the total iron content of the water could be a suitable parameter to monitor the post-flocculation-potential of the water.
- Development of alternatives for fire flows through hydrants:
  Hydrants are still dominant in the design of networks, but their actual contribution to fire safety is relatively small. Discussions with fire fighters sparked interest in the total concept of fire prevention and pro-action, in which water obviously plays an important role. As water specialists, we have an interesting new field to explore to optimise the role of the public drinking water supply in the total fire prevention and pro-action scheme. The domestic sprinkler, for instance, is a proven powerful tool to enhance fire safety, but has a very bad image with the water companies because of the requirements that industrial sprinklers impose on the network. The main development should be in the direction of developing sprinklers that need less water than the present ones to be applied on a large scale. This can save potentially 90% of all fire damage costs and can save up to 40 to 50 lives per year, not to mention many casualties.
- Bolder and more flexible operation of networks:
  Incorporating the velocity requirement in the design of distribution networks opened new ways of approaching the distribution concept. It invoked an awareness of the role of valves in the continuity of supply (Trietsch and Vreeburg, 2004; Trietsch and Vreeburg, 2006). The possibilities of introducing the velocity criterion into the transport network as well as strategic valve locations should be explored along with possibilities for more active control of the hydraulics in the network with, for instance, remote controlled valves and pumps to ‘keep the water moving’.

8 Samenvatting en conclusies

8.1 Historische ontwikkeling

Drinkwaterleidingnetten zijn altijd het domein geweest van de fitters. Het ontwerpen en bouwen van leidingnetten wordt meer beschouwd als een vakmanschap dan als een werkgebied voor ingenieurs en wetenschappers. De verklaring hiervoor kan zijn dat een leidingnetwerk meestal gezien wordt als een relatief eenvoudige technische constructie met als eerste prioriteit het voorzien in voldoende en betrouwbare transport capaciteit om het doel van een drinkwaterbedrijf te verwezenlijken, namelijk: “Het leveren van water in voldoende hoeveelheid”. Lange tijd is verondersteld dat het netwerk slechts een kleine rol speelt in het hele proces van waterkwaliteit en de focus op capaciteit heeft ertoe geleid dat met name het distributienetwerk is overgedimensioneerd met als gevolg lange verblijftijden en zelfs stilstaand water.

Historisch gezien is de overdimensionering van het distributieleidingnet voornamelijk veroorzaakt doordat de levering van bluswater afhankelijk was van de drinkwatervoorziening. De noodzaak om bluswater te leveren bepaalt grotendeels de afmetingen van de leidingen en dat leidt tot een leidingnet dat behoorlijk is overgedimensioneerd voor wat betreft de vraag naar drinkwater. Als je een netwerk bekijkt, lijkt het echter helemaal niet zo overgedimensioneerd. Leidingen met een diameter van 100 tot 150 mm zien er niet geweldig groot uit. Bovendien lijkt het zo dat de investeringskosten voor leidingen slechts voor een deel worden bepaald door de materiaalkosten waardoor het niet de moeite waard lijkt om een gedetailleerd ontwerp te maken.
Een tweede belangrijke reden voor de overdimensionering van het distributienetwerk is dat de layout van een ontwerp is gebaseerd op de ontwerpprincipes voor leveringszekerheid van een transportnetwerk. Daar moeten leidingen vermaasd worden aangelegd om een tweezijdige voeding te garanderen voor het geval een calamiteit optreedt. Het doel is om de continuïteit van de drinkwatervoorziening te vergroten of met andere woorden: “Als het water niet van de ene kant kan komen, dan komt het wel van de andere kant.” Voor een transportleidingnetwerk is dit daadwerkelijk zo omdat daarop meestal geen directe aansluitingen zijn aangebracht. Het uitvallen van een leiding zal dus geen effect hebben op de levering aan de aansluitingen maar wel op de transportcapaciteit. Transportleidingnetten moeten daarom zo zijn ontworpen dat als een leiding wordt afgesloten er voldoende water kan worden getransporteerd naar zwaartepunten van verbruik breekt (Vreeburg et al., 1994) en het vermazen van het transportnet vergroot hierdoor daadwerkelijk de leveringscontinuïteit.
In een distributienetwerk worden de aansluitingen echter wel rechtstreeks op de leidingen gemaakt. Het afsluiten van de leiding betekent dan dat de levering van drinkwater aan die aansluitingen ophoudt en dat betekent dat een vermazing van het leidingnet de continuïteit van de levering niet vergroot.

Het waterkwaliteitsaspect dat wel is verbonden met de rol van het leidingnet, het bruin water probleem, wordt echter te sterk versimpeld: bruin water wordt veroorzaakt door het roesten van gietijzeren leidingen. Binnen de context van de eerste historische leidingnetten is dit wel een logische veronderstelling omdat toen de leidingnetten alleen maar van gietijzer waren gemaakt. Eén foto van het inwendige van een sterk gecorrodeerde en aangegroeide leiding verklaart deze vereenvoudiging ook wel. De zoektocht naar nieuwe materialen die niet roesten heeft geleid naar het gebruik van asbest cement en PVC als leidingmateriaal, maar klaarblijkelijk hebben deze nieuwe materialen het bruin water probleem niet opgelost.

Dat het netwerk het terrein van de fitters is, is verklaarbaar als je kijkt naar de complexiteit van het aanleggen van de vroegste netwerken. Het werken met de zware gietijzeren leidingen en het maken van verbindingen is vakwerk. Een paar voorbeelden daarvan worden getoond in Fig 7-1 die het maken van een loodstriktouw verbinding in tussen gietijzeren pijpen laten zien. Dit verklaart ook de naam 'loodgieter': het vak bestaat letterlijk uit het gieten van vloeibaar lood dat slechts gedaan kan worden door een vakman.

Fig 8-1 Het aanleggen van een gietijzeren leiding en een loodstriktouw verbinding voor 1940

De waterkwaliteitsveranderingen en de hydraulica zijn elementen waar een fitter of een loodgieter niet voor zijn opgeleid. Eenvoudige oplossingen voor gecompliceerde water kwaliteitsprocessen, zoals het conventioneel spuien, werkten niet voldoende en bruinwater problemen werden daarom als min of meer onvermijdelijk gezien.
Aan het begin van de jaren negentig van de vorige eeuw kwam het bruin water probleem in de belangstelling van het BedrijfsTakOnderzoek (BTO) van de Nederlandse waterleidingbedrijven. De directe aanleiding voor die belangstelling was het resultaat van een onderzoeksproject dat was gestart op initiatief van het waterleidingbedrijf van Eindhoven (de toenmalige NRE, Nutsbedrijf Regio Eindhoven, nu onderdeel van het bedrijf Brabant Water) waarin met behulp van continue monitoren onderzoek was gedaan naar aard en oorzaak van de bruin water problemen. Het resultaat van dit onderzoek was een eerste opzet van wat later het model van deeltjes gerelateerde processen zou worden zoals dat is weergegeven in Fig 1-5 (Hoofdstuk 1). De operationele maatregelen die werden ingevoerd in de loop van 1992 waren het systematisch schoonmaken van het leidingnet door middel van goed water spuien (met de randvoorwaarden van 1,5 m/s, 2 tot 3 maal verversen van de leidinginhoud en een schoonwaterfront) en het bewust maken van de bedrijfsvoerders van de zuivering van de effecten van pieken in de troebelheid. De bewustwording van het effect van pieken heeft een soort van wedstrijd doen ontstaan tussen de verschillende bedrijfsvoerders wie de meest stabiele troebelheid van het uitgaande water weet te bereiken. Omdat het bedrijf een zeer goede klachtenregistratie had, konden de effecten worden gemeten hiermee worden gemeten en deze lieten een spectaculaire daling van het aantal klachten zien, zoals weergegeven in Fig 2-18. Anekdotisch is in dit geval de ontwikkeling van een gemotoriseerde brandkraan sleutel om de afsluiters te bedienen die op een speciale fiets kon worden vervoerd. De gemotoriseerde sleutel was zeer geschikt om de vele afsluiters te bedienen tijdens de spuiprogramma's en fietsen waren veel gemakkelijker om mee rond te rijden dan auto's in de spuiwijken. Het is daarmee een voorbeeld van de bereidheid van het bedrijf om buiten de paden van de historische ontwikkeling te gaan en het 'infecteerde' daar ook andere bedrijven mee om de resultaten van het onderzoek te accepteren en toe te passen. Zo was het Waterleidingbedrijf Zuid Kennemerland (nu onderdeel van PWN Waterbedrijf) een van de eerste bedrijven die de nieuwe ontwerprichtlijnen voor vertakte leidingnetten toepaste.

De resultaten van de onderzoeksprojecten van het BTO werden zeer snel verspreid in de professionele netwerkgroepen die rondom het onderzoek waren geformeerd, de zogenaamde

Regionale Distributie Groepen (RDG's). Oorspronkelijk ware dit vijf groepen, ieder bestaand uit ongeveer 10 bedrijven die drie tot vier keer per jaar bijeen kwamen om operationele zaken te bespreken en ook de onderzoeksresultaten te beschouwen. De groepen waren gericht op de techniek van de waterleiding en voor oplossingsgericht. Het relatief simpele model van de deeltjes gerelateerde processen sloeg goed aan bij de sfeer in de groepen. De vertaling van het model in een set van effectieve randvoorwaarden voor water spuien waarmee deze methode potentieel net zo effectief, zo niet effectiever, is als duurdere methoden zoals water/lucht spuien en proppen vergrootte de acceptatie van het deeltjes concept aanmerkelijk.

Praktijkgerichte mensen zoals distributie medewerkers zijn van nature geneigd om nieuwe concepten te accepteren, zeker als die effectief en kostenbesparend blijken te zijn. Diezelfde praktijkgerichtheid kan echter ook een nadeel zijn omdat er niet heel veel belangstelling is voor wetenschappelijke bewijsvoering. Het principe: “Het werkt toch goed?” is ook de reden waarom historische vergissingen lange tijd kunnen blijven bestaan.

Bijna ieder waterleidingbedrijf kende een scheiding tussen de afdeling Productie en de afdeling Distributie. Het onderzoek naar aard en oorzaak van bruin water is gestart vanuit de distributie en het succes van de schoonmaakprogramma, samen met de nieuwe ontwerprichtlijnen bewezen dat het bruin water probleem kan worden beheerst binnen de grenzen van het leidingnet. Schoonmaakprogramma's zijn echter echte end-of-pipe oplossingen en zijn eigenlijk symptoombestrijding. De hoge snelheid leidingnetten zijn meer gericht op voorkomen van het probleem en daarmee een meer duurzame oplossing. De scheiding tussen de afdelingen Productie en Distributie heeft de volgende stap in het voorkomen van bruin water door het voorkomen van de zuivering zeker tijdelijk gefrustreerd. In het gezamenlijk BedrijfsTakOnderzoek (BTO) van de Nederlandse waterleidingbedrijven, in samenwerking met Delft Cluster en het onderzoekprogramma van de Technische Universiteit Delft is er ruimte gemaakt om meer wetenschappelijk bewijs te krijgen voor de empirische uitkomsten, maar ook om de mogelijkheden te onderzoeken voor het voorkomen van de deeltjesbelasting van de zuivering.

8.2 Meet methoden

Voor het onderzoek zijn verschillende nieuwe meetstrategieën ontwikkeld, waarvan het continu meten van de deeltjes gerelateerde parameters zoals troebelheid en deeltjes tellingen de meest belangrijke zijn. Hoewel het continu meten van de troebelheid reeds lang bekend was (van den Hoven and Vreeburg, 1992), is de toepassing ervan niet wijd verspreid in het drinkwater leidingnet. De waterkwaliteit af pompstation werd voornamelijk vastgesteld met het wettelijk verplichte bemonsteringsprogramma. In toenemend mate echter, wordt continue meting van de troebelheid op pompstations toegepast om de effecten van maatregelen in de bedrijfsvoering van de zuivering op de waterkwaliteit te meten.

Sinds kort zijn deeltjestellers beschikbaar gekomen die het mogelijk hebben gemaakt om een meer kwantitatieve bepaling te doen door het berekenen van het deeltjesvolume in het water dat het leidingnet inkomt en de ontwikkeling daarvan in het leidingnet zelf. Het berekende deeltjesvolume te samen met de deeltjesgrootte verdeling, zoals toegepast in dit proefschrift, geven informatie over wat er gebeurd met de deeltjes en waar de mogelijkheden liggen om de deeltjesbelasting te verminderen.

De deeltjestellingen waren voldoende fijnmazig om het deeltjesvolume te berekenen en er was geen noodzaak om de deeltjesgrootte verdeling te modelleren zoals Ceronio (Ceronio and Haarhoff, 2005) voorstelt. Hij laat zien dat deeltjes groter dan 30 µm zeldzaam zijn en dat daarom binnen de 32 beschikbare 1 µm ranges alle relevante deeltjes kunnen worden geteld.

Dit maakt verder modellering overbodig omdat gebruik kan worden gemaakt van de echte aantallen. De grote hoeveelheden meetgegevens waren goed hanteerbaar met de huidige spreadsheetprogramma's. De mogelijkheden om met behulp van de modellering een vingerafdruk te maken van de deeltjes en daardoor hun herkomst te bepalen is niet nader onderzocht in dit proefschrift.

Het succes en de resultaten van de continue metingen bij de zuivering en in het leidingnet heeft een discussie gestart over de toegevoegde waarde van permanente waterkwaliteitsmetingen in het leidingnet. De discussie is geïnspireerd door de continue drukmetingen in het leidingnet. De vergelijking van deze kwantiteit gerelateerde druk metingen met de kwaliteit gerelateerde troebelheidmetingen en deeltjes tellingen is echter niet eenvoudig.
Waterkwaliteitsmetingen kunnen in tegenstelling tot kwantiteitsmetingen niet dienen als een operationeel instrument om direct in te grijpen. Als bijvoorbeeld de druk in het leidingnet plotseling daalt dan zal daar een acute oorzaak voor kunnen worden gevonden en een actie worden ondernomen om het probleem te verhelpen. Deeltjestellingen en troebelheid metingen in het net geven echter voornamelijk informatie over de reguliere processen waarop niet kan worden ingrepen op het moment dat wordt gemeten. Bovendien moeten gegevens over langere tijd worden bewerkt en geanalyseerd voordat zinnig kan worden ingegrepen. Er kan dan bijvoorbeeld worden geconstateerd dat er een verlies van deeltjes in het leidingnet optreedt dat echter niet met een directe actie kan worden beïnvloed. Continu meten bij de zuivering is wel nuttig om de bedrijfsvoerders bewust te maken van het optreden van 'normale' pieken in de troebelheid of de deeltjes tellingen. Iedere piek draagt bij aan de sediment laag in het netwerk en met iedere piek die kan worden voorkomen wordt de opbouw van de sedimentlaag vertraagd. De meetinstrumenten stellen de bedrijfsvoerders in staat om alerter te zijn om toekomstige pieken te voorkomen. Dit verschilt fundamenteel met de on-line druk metingen,
Waterkwaliteitsmetingen zoals troebelheid en deeltjes tellingen kunnen het best worden ingezet met een specifiek doel, bijvoorbeeld om de opwervelingspotentie te meten of om te zien hoe de deeltjes in het water af pompstations zich gedragen tijdens transport en distributie. Bij discussies met netwerk beheerder over de kosten en het aantal instrumenten is het uiteindelijk advies meestal om drie tot vier mobiele monitors te maken die flexibel en eenvoudig hanteerbaar zijn. Het is effectiever om specifieke vragen te beantwoorden met een geplande inzet van de apparatuur dan om een aantal monitors in het net te fixeren zonder een gericht doel.

De Opwerveling Potentie Methode (OPM) is een krachtig instrument gebleken om het bruin water risico in een leidingnet vast te stellen. De flexibiliteit van het beoordelingssysteem van de metingen maakt het mogelijk om de gevoeligheid van de metingen zo in te stellen als noodzakelijk is voor de toepassing. Daarom is het beoordelingssysteem in de Venlo-studie (zie paragraaf 5.6) anders dan dat in de Franeker studie van hoofdstuk 3. In beide gevallen wordt de OPM gebruikt als een middel om de effecten van ingrepen te bepalen. De Venlo-studie, zoals besproken in paragraaf 5.6 is een onderdeel van een groter analyse systeem waarmee in totaal 695 locaties over 8500 km leidingnet regelmatig worden gemeten. Het doel van het systeem is om gebieden met een hoog bruin water risico aan te wijzen. Deze hoog risico gebied worden vervolgens schoongemaakt op volgorde van een aflopend gemiddeld bruin water risico, waardoor de gelimiteerde mogelijkheden voor schoonmaken optimaal kunnen worden ingezet. De mogelijkheden voor schoonmaken worden beperkt door de hoeveelheid mankracht die beschikbaar is om de specifieke schoonmaakacties uit te voeren.

In het geval van de Franeker-studie was het doel om te zien hoelang het zou duren tot weer hetzelfde bruinwater risico zou zijn bereikt om op die manier het verschil tussen de beide systemen te bepalen. Voor dit doel was het noodzakelijk om een lagere absolute troebelheid te gebruiken in het beoordelingssysteem voor de OPM dan in het Venlo-studie. Het is ook voorstelbaar dat het beoordelingssysteem van in de Venlo-studie langzaam wordt bijgesteld naar lagere absolute troebelheidswaarden, omdat het algemene bruin water risico geleidelijk zal afnemen vanwege het optimaal gebruik van de schoonmaakmogelijkheden.
Voor het meten van de OPM is een flexibel meetopstelling noodzakelijk die kan worden aangesloten op brandkranen. In de beide genoemde studies was de apparatuur ingebouwd in fitterswagens die speciaal zijn aangepast voor dit soort werk en worden bediend door goed opgeleid personeel. Daardoor was het mogelijk om 4 tot 6 metingen per dag uit te voeren.
Het is duidelijk dat met iedere OPM meting een zekere hoeveelheid sediment wordt verwijderd, waarmee de ‘maagdelijkheid’ van de meetlocatie wordt aangetast. Hiermee is niet altijd rekening gehouden in deze studie, maar er zijn wel aanwijzingen dat in laagbelaste systemen de deeltjes accumulatie zo laag is dat het jaarlijks herhalen van de OPM voldoende is om al het sediment te verwijderen. In de Venlo-studie is dit klaarblijkelijk niet het geval en is de accumulatie genoeg om het gat van de OPM te vullen. Dit zou echter ook kunnen betekenen dat de werkelijke OPM in de rest van het systeem hoger is dan volgt uit de analyse van de gemeten OPM. Daarom is het noodzakelijk dat de OPM wordt uitgevoerd op een aantal vergelijkbare locaties die regelmatig worden afgewisseld.

In de studie beschreven in dit proefschrift zijn de mogelijkheden concentratiemethoden als de TILVS en de Hemoflow nader bekeken en ze zien er veelbelovend uit, maar zijn nog niet volledig uitontwikkeld. De methoden maken het mogelijk om de deeltjes processes beter te bekijken, maar zijn nog niet betrouwbaar genoeg om routinematig te gebruiken. De kracht van de methoden is dat het pre-concentreren een relatief eenvoudige methode is. De zwakte is daarentegn dat de omstandigheden waarin de metingen worden gedaan vaak niet erg gunstig zijn. Bovendien vragen de methoden nogal wat tijd en daarmee een redelijk regelmatig proces of veel metingen om zinvolle informatie te krijgen. Verschillende proeven met de toepassing van de TILVS zijn niet nader beschreven omdat het resultaat onvoldoende was vanwege problemen met de apparatuur en gebrek aan ervaring. Daarnaast was de hoeveelheid afgefilterd materiaal vaak te weinig (minder dan 2,5 mg) en een betrouwbare analyse te doen. Met de ervaring die hiermee is opgedaan zijn vier nieuwe en aangepaste TILVS apparaten gemaakt die kunnen worden ingezet in nieuw onderzoek zodat de methode verder kan worden ontwikkeld.
De Hemoflow is alleen gedurende de laatste fase van het experiment met het deeltjesvrije water in het leidingnet gebruikt. Als gevolg van de onervarenheid is de analyse van de geconcentreerde monsters niet goed gedaan. Daarnaast bestond de indruk dat veel van het materiaal dat moest worden geconcentreerd ook in het filter zelf achterbleef. De kleur van het filter veranderd gedurende de analyse periode wat erop duidt dat er materiaal achter bleef. De totale hoeveelheid materiaal in het geconcentreerde monster was inde orde van 100 mg en het is mogelijk dat ongeveer eenzelfde hoeveelheid materiaal ook in het filter was achtergebleven. Hiermee wordt de nauwkeurigheid van de meting sterk beïnvloed.
Ondanks deze teleurstellingen is wel duidelijk dat de kracht van de methode is dat een betere analyse kan worden gemaakt van de deeltjesbelasting en dat de monstername procedure relatief eenvoudig is en dat geen speciaal hoog opgeleid personeel noodzakelijk is

De onderzoeksmethode om het effect van deeltjes in drinkwater te bekijken door het isoleren van twee vergelijkbare gebieden in een echt leidingnet is heel succesvol gebleken. Het was mogelijk om het effect van één parameter te bekijken, namelijk deeltjesbelasting, terwijl alle

andere parameters zoals leidingmateriaal en hydraulica relatief constant bleven. Deze methode kan ook gebruikt worden bij ander gericht onderzoek in een netwerk zoals bijvoorbeeld naar het effect van gietijzer op de waterkwaliteit door twee gebieden van ongeveer gelijke omvang in een leidingnet te isoleren waarvan de ene uit gietijzeren leidingen bestaat en de andere uit niet-gietijzeren materiaal.
Het grootste probleem in het onderzoek was het ongestoord en geïsoleerd houden van de beide gebieden Geduld en goede registratie waren de belangrijkste succesfactoren maar het resultaat was dat er unieke gelegenheid was om verschillende aspecten die te maken hebben met bruin water objectief te kunnen analyseren.

Proefleidingen en testinstallaties zijn slechts beperkt gebruikt in het beschreven onderzoek. Het primaire doel was om te begrijpen hoe processen lopen in de werkelijkheid. Er is geprobeerd om deze werkelijkheid na te bootsen in een proefleiding installatie waarvan enkele foto’s zijn getoond in paragraaf 4.2.2. De resultaten van deze installatie waren teleurstellend. Het eigenlijk proces, het accumuleren van deeltjes onder verschillende hydraulische omstandigheden, laat zich moeilijk vertalen naar een proefinstallatie. De installatie die was gebruikt om het water/lucht spuien in beeld te brengen was wel succesvol, omdat deze installatie de afmetingen had van een echt leidingsysteem
Voor het begrijpen van de micro-mechanismen in het leidingnet kan het verder ontwikkelen van proefinstallaties toch van belang zijn. Het opmerkelijke effect dat deeltjes niet alleen op de bodem van de leiding bezinken maar hechten aan het volledige wandoppervlak maakt het dringend noodzakelijk om meer onderzoek te doen naar de micro wereld van deeltjes bezinking in de nabijheid van een (leiding) wand. Ook onderzoek naar de groei en ontwikkeling van biofilms zou in een dergelijke installatie nader kunnen worden bekeken.

8.3 Zuivering

De verspreiding en verandering van pieken in de troebelheid of de deeltjes tellingen in het leidingnet laten goed het effect van de zuivering, of beter het effect van deeltjes in het drinkwater, op het leidingnet zien. Door de piek wordt een hoeveelheid water van een vingerprint voorzien die gevolgd kan worden in het leidingnet. Allereerst kan de verblijftijd precies worden bepaald (zie paragraaf 2.2.4). Daarnaast kunnen de veranderingen in de waterkwaliteit worden bepaald, ook op andere parameters dan de troebelheid.
De troebelheid kan niet gebruikt worden om de deeltjes belasting te bepalen omdat het geen kwantitatieve parameter is. Deeltjes tellingen en het daaruit berekende deeltjesvolume geven meer kwantitatieve informatie. Het presenteren van de meetgegevens met behulp van de cumulatieve frequentieverdeling van het berekende deeltjesvolumen samen met de gemiddelde waarde daarvan en de Surf-90% en de Surf+90%, blijkt een krachtige manier te zijn om patronen te karakteriseren. Het totale effect van de troebelheid en het berekende deeltjesvolume kan gemakkelijk worden gezien, zoals is gedemonstreerd in paragraaf 3.4. Hoewel deze parameters er veelbelovend uitzien en nuttig waren om de metingen te interpreteren en de deeltjes gerelateerde processen te begrijpen, moet er nog veel ervaring worden opgedaan. De combinatie van het meten van het deeltjesvolume en het analyseren van de watersamenstelling verdient zeker nader onderzoek.

In het beschreven onderzoek zijn twee typen zuivering bekeken en is aangetoond dat het deeltjesvolume in het water af pompstation niet de enige parameter is die het deeltjesvolume in het leidingnet zou kunnen verklaren. Nacoagulatie van ijzerhydroxide tijdens het transport veroorzaakte een aanzienlijke deeltjesbelasting voor het distributieleidingnet dat werd gevoed met water met meerdere behandelingsstappen wordt geproduceerd uit kunstmatig grondwater. Ook uitloging van de cementlaag in de leiding en een biologisch groeiproces veroorzaakte een

deeltjesbelasting. (paragraaf 6.2). De oppervlaktewater zuivering in meerdere stappen produceerde stabieler water dat geen extra deeltjesbelasting vormde voor het distributienet. De concentratie van deeltjesvolume in het oppervlaktewater was direct na de zuivering een factor 4 hoger dan dat van het kunstmatige grondwater, 0,74 ppb versus 2,81 ppb. Aan het einde van de transportleiding was het gemiddelde deeltjesvolume van het kunstmatige grondwater gestegen tot 27 to 62 ppb, terwijl dat van het oppervlaktewater constant bleef in het eerste gedeelte van het leidingnet en tot 4,47 ppb groiede in het tweede gedeelte.
De stabiliteit van het water voor wat betreft de uitvlokking-potentie wordt niet bepaald door een enkele parameter, maar is wel een belangrijke indicator voor de deeltjesbelasting van het leidingnet. Het totaal ijzer en mangaan gehalte is niet voldoende omdat dit zowel het opgeloste als het in deeltjesvorm aanwezige ijzer en mangaan betreft. Het opgeloste deel kan uitvlokken tot deeltjes die bijdragen aan de deeltjes belasting aan of in het leidingnet. In theorie wordt met een TILVS bepaling onderscheid gemaakt tussen het discrete gedeelte van het ijzer en het opgeloste deel en deze informatie kan gedeeltelijk de uitvlokking-potentie van het water aangeven. Er is echter nog een hoeveelheid onderzoek en validatie noodzakelijk om geschikte streefwaarden te bepalen.
Met de Surf-90% en de Surf+90% kan worden aangetoond dat een groot gedeelte van de deeltjesbelasting in een relatief korte tijd geschiedt. Soms wordt in minder dan 10% van de tijd, meer dan 50% van het deeltjesvolume aan het leidingnet geleverd (Surf_90% > 50%). De deeltjesbelasting aan het leidingnet kan in dat geval sterk worden verbeterd zonder nieuwe zuiveringsstappen toe te voegen als er bij de bedrijfsvoering van de bestaande zuivering meer wordt geconcentreerd op het verminderen van de pieken. Dit resulteert in een lagere Surf+90% en een hogere Surf-90%. Voorlopige streefwaarden voor de Surf+90% zou minder dan 15% kunnen zijn en voor de Surf-90% dat vervolgens meer dan 85%. Illustratief hiervoor zijn de deeltjes tellingen van Fig 2-7 en de troebelheid van Fig 3-4. Deze grafieken laten zien dat de pieken meestal te maken hebben met het terugspoelprogramma van de filters. Deze pieken nemen af in het leidingnet wat betekent dat deeltjes verloren gaan in de leiding en bijdragen aan het bruin water risico. Een eerste stap om dit zuiveringsproces is het verminderen van deze korte periodes van belasting die de hoge Surf+90% waarden doen ontstaan. In paragraaf 3.4 wordt het effect van de pieken verderop in het leidingnet getoond (de start van het Referentie gebied), maar het is nog steeds duidelijk dat de pieken een belangrijke bijdrage hebben aan de belasting van het leidingnet.

De rol van het zuivering bij het belasten van het leidingnet met deeltjes versus de rol van het roesten van onbeschermd gietijzer wordt bevestigd door de case studie over de samenstelling en het gedrag van sediment (hoofdstuk 6). Onder vergelijkbare hydraulische omstandigheden was de hoeveelheid sediment in een gietijzeren leiding, gevoed door water met een lage deeltjesbelasting, lager dan de hoeveelheid sediment in een niet-gietijzeren leiding gevoed door water met een hogere deeltjesbelasting. Hoewel de lage deeltjesbelasting af zuivering (de op kunstmatig grondwater gebaseerde zuivering die al eerder is genoemd) behoorlijk toenam in het transportleidingnet waren de absolute waarden voor het discrete ijzer laag vergeleken met de belasting in het Referentiegebied, zoals dat is besproken in hoofdstuk 3.

In dit proefschrift is voornamelijk aandacht voor het gedrag van deeltjes en met de biologische implicaties wordt geen rekening gehouden. Onderzoek naar de samenhang tussen troebelheid, deeltjes tellingen en *Cryptosporidium* laat zien dat deze niet uniform is (Huck et al., 2002). Vooral tijdens suboptimale omstandigheden is er een relatie tussen de deeltje tellingen en de verwijdering van *Cryptosporidium*. Omdat de troebelheid in mindere mate was gerelateerd aan de verwijdering van *Cryptosporidium*, was dit geen geschikte parameter om dit te monitoren. Dit duidt erop dat het verbeteren van het zuiveringsproces voor wat betreft de deeltjes tellingen waarschijnlijk ook een effect zal hebben op de bioloische stabiliteit van het water.

Het wordt sterk aanbevolen om nader onderzoek te doen naar het effect van deeltjes in het water af pompstation en hun lot in het leidingnet. Dit onderzoek zal een beter inzicht opleveren in hoe de belasting van het leidingnet kan worden voorkomen en mogelijkerwijs in hoe de biologische stabiliteit daarvan zal verbeteren.

8.4 Hoge snelheid/zelfreinigende leidingnetten

Het beheersen van de accumulatie van deeltjes door het sturen van de snelheid in de leidingen is volgend het model van deeltjes gerelateerde processen één van de manieren om het bruin water risico te beperken. De snelheid in de leidingen moet zodanig worden gestuurd dat de deeltjes die gedurende de perioden van lage watervraag op een dag bezinken, tijdens perioden van hogere watervraag van die dag weer worden opgewerveld. Dat zou betekenen dat er op dagbasis geen accumulatie van sediment zou optreden.
Hoewel dit een logische vervolgstap is in het model van de deeltje gerelateerd processen, was het erg moeilijk om waterleidingbedrijven zover te krijgen dat de nieuwe ontwerpregels voor de hoge snelheid leidingnetten werden geaccepteerd. Belangrijkste reden voor de terughoudendheid is dat de consequentie van de nieuwe ontwerpregels is dat leidingnetten vertakt zijn in plaats van vermaasd. Dit is contra-intuïtief ten opzichte van het algemeen aanvaarde concept dat voorschrijft dat leidingnetten vermaasd moeten zijn vanwege de leveringszekerheid en dat dode einden verboden zijn vanwege de waterkwaliteit. De basla misvatting hierin is dat er een verschil is tussen een transport leidingnet en een distributie leidingnet. Als het leveringszekerheid concept voor transportleidingen (Vreeburg et al., 1994; Vreeburg et al., 1998) wordt vertaald naar distributieleidingnetten, levert dat geen betrouwbaardere leidingnetten op.
Voor de leveringszekerheid van transportleidingnetten moeten de leidingen vermaasd worden aangelegd om een dubbelzijdige voeding te garanderen voor het geval dat een leiding buiten gebruik is. Het doel is om hierdoor de continuïteit van de levering te vergroten of in andere woorden: “Als het water niet van de ene kant kan komen, dan komt het van de andere kant.” Voor een transportleidingnet wordt hiermee daadwerkelijk de leveringscontinuïteit vergroot omdat er gewoonlijk geen directe aansluitingen zijn op de transport leidingen. Transportleidingen moeten zo zijn ontworpen dat in geval een leiding buiten gebruik is er voldoende transport capaciteit overblijft om het water in zwaartepunten van verbruik te kunnen leveren (Vreeburg et al., 1994).
In een distributieleidingnet worden de aansluitingen echter wel direct gemaakt op de leidingen. Het uitvallen van een leiding heeft dus tot gevolg dat de levering naar die aansluitingen wordt wegvalt en het vermazen van de leidingen vergroot dus niet de leveringscontinuïteit. De leveringszekerheid van een distributieleidingnet moet dus worden gezocht in het vergroten van de leveringscontinuïteit. Als die omschakeling eenmaal is gemaakt, dan is ook duidelijke dat de tijd dat een leveringsonderbreking duurt bepalend is en niet de vermazing. Een nevenopbrengst van deze benadering is dat een kritische beschouwing van de plaatsing van afsluiters in een bestaand leidingnet, kan leiden tot een hogere leveringszekerheid c.q. leveringscontinuïteit, met minder afsluiters (Trietsch and Vreeburg, 2006).

De vermeende negatieve effecten van dode einden op de waterkwaliteit zijn voornamelijk gebaseerd op de zogenaamde klassieke dode einden die een verafgelegen brandkraan aansluiten op het leidingnet. Voor de levering van bluswater is een relatief grote diameter nodig hetgeen dan resulteert in een lange verblijftijd omdat er slechts een geringe drinkwatervraag is. De lange verblijftijd heeft een negatief effect op de waterkwaliteit. De dode einden in de nieuwe leidingnetten zijn in feite ‘stromende einden’. De leidingen zijn zodanig gedimensioneerd dat er dagelijks een snelheid optreedt van 0,4 m/s, waarmee het geaccumuleerde sediment wordt verwijderd en automatisch de verblijftijden beperkt.

De drinkwatervraag in een leidingnet varieert sterk over 24 uur. Om de distributieleidingen goed te ontwerpen is het noodzakelijk om een gedetailleerde inzicht te hebben in het verbruikspatroon van een individuele aansluiting. Dit inzicht ontbrak op het moment dat de nieuwe ontwerprichtlijnen werden geïntroduceerd. Een praktische oplossing om deze ontbrekende kennis te overbruggen was om de ontwerpregels voor binneninstallaties te gebruiken: de zogenaamde q√n-methode. Zoals beschreven in hoofdstuk 4 levert deze benadering een leidingnet op dat zelfreinigend was gedurende de onderzoeksperiode. De verdere ontwikkeling van een stochastisch eind gebruik model laat zien dat het werkelijke verbruik wordt onderschat met de q√n methode (Blokker et al., 2006). Het leidingnet dat ontworpen was met de q√n methode was echter wel zelfreinigend, hetgeen tot de conclusie leidt dat de werkelijke snelheid waarbij zelfreiniging optreedt klaarblijkelijk lager is dan de geschatte 0,4 m/s.
De verder ontwikkeling van het SIMDEUM model (Blokker et al., 2006) zal de kennis omtrent de werkelijke verbruiken en snelheden vergroten zonder dat uitgebreide metingen noodzakelijk zijn, die in de praktijk overigens nagenoeg onmogelijk zijn. Dan is echter ook een betere bepaling van de zelfreinigende snelheid noodzakelijk, zodat het ontwerp verder kan worden geoptimaliseerd. De conclusie blijft echter staan dat de combinatie van de q√n methode en de 0,4 m/s een zelfreinigend leidingnet oplevert en als zodanig ook worden toegepast.

Voor een brandweerman is de brandkraan in het leidingnet een “natuurlijke bron” voor bluswater. In historisch perspectief is dit begrijpelijk omdat vroeger de drinkwatervoorziening en de brandweer tot dezelfde gemeentelijke dienst Openbare Werken behoorden. Deze situatie heeft lange tijd de noodzaak voorkomen om deze bron voor bluswater kritisch te beschouwen. Een internationale enquête over richtwaarden en wetgeving voor bluswaterlevering (Snyder and Deb, 2003) laat zien dat er nauwelijks formele overeenkomsten zijn, maar dat er ook nauwelijks inzicht is in de werkelijke bluswaterbehoefte. Tijdens discussies met brandweerlieden in Nederland werd langzaam duidelijk dat zelfs 30 m^3/uur nog twee keer zoveel is als daadwerkelijk wordt gebruikt tijdens een eerste aanval.
Door de nieuwe ontwerpregels werden waterleidingbedrijven en brandweerlieden gedwongen om na te denken over de noodzakelijke randvoorwaarden voor de bescherming van de bevolking tegen brand. Tijdens deze discussies werd de meerwaarde van woning sprinkler installaties aangegeven voor het beschermen van normale woonhuizen. Er is een zeer overtuigende toepassing van woningsprinklers in een test case in Scottsdale, Arizona (Ford, 1997). De mogelijkheden voor woningsprinklers worden echter nog steeds onderschat en zijn zeker niet voldoende onderzocht.

Het oorspronkelijke doel van de nieuwe ontwerpregels is het verbeteren van de water kwaliteit en het verlagen van het bruin water risico. Dit oorspronkelijke doel is bijna geheel ondergesneeuwd door de economische effecten van de nieuwe regels en de soms heftige en emotionele discussie met de brandweer. De vergelijkende metingen zoals gepresenteerd in paragraaf 4.5 laten zien dat het veronderstelde zelfreinigende effect daadwerkelijk kan worden vastgesteld. En hoewel dit in genoemde heftige en emotionele discussies bijna tot een ondergeschikt punt is geworden, zou dit toch de belangrijkste drijfveer moeten zijn om het ontwerp van distributieleidingnetten te verbeteren.

De nieuwe methode voor het ontwerpen van distributieleidingnetten laat ook zien dat de conventionele methode nauwelijks een systematische procedure kan worden genoemd. In feite kun je zeggen dat het ontwerp van een distributieleidingnet niet wezenlijk is veranderd sinds het begin van de openbare drinkwatervoorziening. Hoewel het oorspronkelijke doel van het nieuwe ontwerp was om de waterkwaliteit te verbeteren, blijkt het ook een start te zijn geweest om de ontwerp principes voor drinkwater distributie leidingnetten kritisch te bekijken. Een bredere toepassing van de nieuwe ontwerpregels op bijvoorbeeld het rehabiliteren van oude leidingnetten zal zeker aanpassingen vergen, maar potentieel kan ook hier 10 tot 20% bespaard worden op de investeringskosten.
Toegepast op de UN Millennium Doelstelling om het aantal mensen dat geen toegang heeft tot veilig drinkwater te halveren, zou dit betekenen dat dit met minder geld kan worden gerealiseerd of dat voor hetzelfde geld nog meer mensen toegang tot veilig drinkwater kan worden verschaft.

8.5 Schoonmaak methoden

Naast het vervangen van onbeklede gietijzeren leidingen was het conventioneel spuien van leidingen van oudsher de belangrijkste beheersmaatregel als reactie op lokale bruin water incidenten. Het schoonmaken van leidingnetten was chronologisch gezien het eerste onderdeel van het deeltjes gerelateerde model dat was onderzocht. Met de introductie van het continu monitoren van de troebelheid en de Opwerveling Potentie Meting (OPM) kwamen er methoden beschikbaar om de effecten van schoonmaken te meten (Vreeburg, 1996).
Het negatieve effect van het spuien met te lage snelheden op de mobiliteit van het sediment verklaarde waarom het conventioneel spuien niet effectief was net zoals het agressief schoonmaken van onbekleed gietijzeren leidingen een negatief effect heeft. Het ontrafelen en demystificeren van de effecten van schoonmaken en het ontwikkelen van heldere regels voor effectief schoonmaken met water spuien brachten imago van het schoonmaakwerk op een hoger plan. Meerdere waterleidingbedrijven investeerden meerder mensjaren aan de ontwikkeling van spuiplannen. Deze plannen waren gebaseerd op zogenaamde all-pipe-models die relatief eenvoudig konden worden afgeleid van Leiding Informatie Systemen (LIS). Het voordeel van deze toepassing van leidingnetberekeningen is dat het niet noodzakelijk is om een precieze modellering van de normalen watervraag te hebben. Het spui volume is zoveel hoger dan de normale watervraag dat deze laatste kan worden verwaarloosd in de berekening.

Oorspronkelijk was de minimum spuisnelheid van 1,5 m/s voornamelijk noodzakelijk om een praktische standaard te hebben. Belangrijk argument was dat de snelheid praktisch haalbaar was zonder dat grote investeringen noodzakelijk waren samen met serieuze aanwijzingen dat de snelheid voldoende zou zijn. (Fig 5-2). Samen met de randvoorwaarde van een schoonwaterfront bleek dit goed te werken voor de praktisch ingestelde distributie medewerkers. Anekdotisch in dit verband is het voorbeeld van een spui actie op een Ø400 mm PVC leiding met een lengte van 5 kilometer. Kort na middernacht werd de spui geopend en binnen tien minuten was het spuiwater zo troebel dat de fitters sceptisch zeiden dat het wel tot zonsopgang zou duren voordat dit weer een beetje fatsoenlijk drinkwater zou zijn. Na het verversen van precies één leidinginhoud, na ongeveer een uur, daalde de troebelheid sterk en na twee uur (twee leidingverversingen) was het water volkomen helder. Dit soort resultaten werden heel snel bekend in informele circuits en de randvoorwaarden voor effectief spuien werden heel snel verspreid en toegepast

Eén van de traditionele bezwaren tegen water spuien is het schijnbare verspillen van water. Dit is echter sterk verbonden met het conventionele spuien waarbij een brandkraan min of meer willekeurig wordt geopend en blijft spuien zolang het water nog niet helder is. Met alle effecten van lage snelheden op de troebelheid kan dit behoorlijk lang duren en dat geeft inderdaad een verspilling van water. Met een goed voorbereid spuiplan kan een 300 meter lange leiding binnen tien minuten drie maal worden ververst of twee maal binnen 6,5 minuten. Het waterverbruik is beperkt tot maximaal drie maal de inhoud van de leiding. De werkelijk tijd dat er bruin water in de leiding voorkomt tijdens het spuien is ook 6 tot 10 minuten.
Een goed spuiplan verbruikt maximaal drie maal de inhoud van het schoon te maken leidingnet. In de Nederlandse situatie betekent dit het waterverbruik voor een compleet spuiplan voor het gehele leidingnet 0,53 tot 0,94% van de jaarlijkse watervraag bedraagt. Aangezien het Nederlandse leidingnet niet wezenlijk verschilt van leidingnetten in andere landen voor wat betreft relatieve lengte en inhoud, kunnen deze cijfers worden toegepast op vele andere leidingnetten.

Met de OPM kan ook worden vastgesteld dat de effectiviteit van een schoonmaakactie niet zozeer wordt bepaald door de hoeveelheid sediment uit een leiding wordt verwijderd, maar door de hoeveelheid sediment die achter blijft. Proppen wordt vaak intuïtief beschouwd als een effectieve methode omdat het water dat vlak voor de prop uit de leiding komt extreem vuil is. Als de hoeveelheid sediment echter wordt verdeeld over de gehele leidinglengte dan is het resultaat niet meer zo spectaculair. Het onderzoek heeft laten zien dat de meeste alternatieve methoden voor water spuien meer water in plaats van minder water gebruiken.

Het schoonmaken van leidingen kan ook een effect hebben op de biofilm in die leidingen, hoewel dat aspect niet is meegenomen in het beschreven onderzoek. Het spuien met water is voornamelijk gericht op het verwijderen van los sediment en zal waarschijnlijk de biofilm niet aantasten omdat die aan de wand vast zit. Mechanische methoden zoals proppen zulle de biofilm waarschijnlijk wel aantasten, hoewel het moeilijk zal zijn om de kleverige film echt te verwijderen. Aan de andere kant zal het verwijderen van de biofilm geen langdurig effect hebben als er niets veranderd aan de Biofilm vorming potentie van het water. Waarschijnlijk is de biofilm snel weer hersteld. In een biofilm monitor is komt de ontwikkeling van een biofilm relatief snel tot een evenwicht, binnen een aantal maanden (van der Kooij et al., 2003).

8.6 Conclusies

Het doel van het beschreven onderzoek is om de deeltjes-gerelateerde processen te analyseren die betrokken zijn bij het ontstaan van bruin water problemen in het drinkwaterleidingnet. Hiervoor zijn nieuwe meetmethoden ontwikkeld zoals het continu meten van de troebelheid en deeltjes tellingen, de Opwerveling Potentie Meting (OPM) en de Time Integrated Large Volume Sampling (TILVS). Met deze methode kon het bruin water probleem worden gerelateerd aan het losse sediment in het leidingnet. De incidentele opwerveling van de geaccumuleerde deeltjes is de belangrijkste oorzaak van bruin water incidenten in het leidingnet. De bron van de deeltjes is voornamelijk het drinkwater zelf, gevolgd door processen in het leidingnet zoals na-coagulatie, roesten, uitlogen en biologische groei en nagroei.
Onafhankelijk van de bron van de deeltjes kan het accumuleren van de deeltjes tot een laagje van los sediment de oorzaak zijn van waterkwaliteitsproblemen. Het beheersen en sturen van die accumulatie is mogelijk door de snelheden in de leidingen te sturen en door de losse deeltjes te verwijderen door effectief schoon te maken.

Om het bruin water risico in het leidingnet te beheersen zijn drie niveaus van maatregelen bepaald:

- Voorkom dat deeltjes in het leidingnet komen of in het leidingnet gevormd worden.
  De belangrijkste bron voor deeltjes in het leidingnet zijn de deeltjes in het drinkwater zelf en in het bijzonder de deeltjes die als gevolg van variaties in het zuiveringsproces vrijkomen en in het leidingnet worden gebracht. Er zijn verschillende manieren om de deeltjes belasting van het leidingnet te beperken, afhankelijk van de dominante oorzaak:
    - Verbeter het zuiveringsproces met betrekking tot de verwijdering van deeltjes door bijvoorbeeld een extra polishing of filtratie stap.
    - Verbeter het bestaande zuiveringsproces zodanig dat pieken worden voorkomen hetgeen kan leiden tot een substantiële vermindering van de deeltjesbelasting naar het leidingnet.
    - Pas het behandelingsproces aan zodat de corrosiviteit of de na-coagulatie wordt verminderd.

    De deeltjesbelasting vanaf het pompstation kan worden bepaald met de parameters Surf+90% en Surf-90% samen met het gemiddelde berekende deeltjesvolume. De ervaring met deze parameters tot nu toe laat zien dat met een Surf-90% van 80 tot 85% of meer er sprake is van een zeer stabiel zuiveringsproces. Een streefwaarde voor alleen het gemiddelde deeltjesvolume is niet voldoende, maar zou gecombineerd moeten worden met een parameter die de na-coagulatie potentie karakteriseert, de biofilm vorming potentie (van der Kooij et al., 2003) en de corrosie-index (van den Hoven and van Eekeren, 1988). Het totaal ijzer en mangaan gehalte zou gecombineerd moeten worden met een TILVS-analyse om het aandeel van discreet ijzer te bepalen, nadat de TILVS voldoende is gevalideerd voor dit doel.
- Voorkom accumulatie van deeltjes
  Deeltjes accumuleren specifiek in het distributie leidingnet als gevolg van de lage snelheden. Conventionele distributie leidingnetten zijn ontworpen op een hoge vraag naar bluswater. Verlaging van de bluswatervraag, samen met een nieuwe aanpak van het leidingnet ontwerp met vertakte systemen met een afnemende diameter waarin minimaal eens per dag een snelheid van 0,4 m/s optreedt en gebaseerd op een drinkwatervraag die bepaald is met de q√n methode leidt tot een distributieleidingnet dat zelfreinigend is en waarin geen sediment zal accumuleren.
- Verwijder geaccumuleerd sediment
  Door regelmatig schoonmaken van het leidingnet zal het sediment niet accumuleren tot onaanvaardbare niveau's. Het kritisch niveau van sediment accumulatie kan worden gemeten met behulp van de Opwerveling Potentie Meting die ook gebruikt kan worden om de effectiviteit van de toegepaste schoonmaak methode te bepalen. Door he spuien met water onder de randvoorwaarden van 1,5 m/s, twee tot drie maal verversen van de leiding en een schoonwaterfront wordt het losse sediment dat het bruin water risico bepaald effectief verwijderd

8.7 Aanbevelingen voor verdere ontwikkeling en onderzoek

Het in dit proefschrift beschreven onderzoek verklaart enkele van de deeltjes gerelateerde processen in een leidingnet en laat zien dat een onconventionele benadering van het distributieleidingnet leidt tot nieuwe uitgangspunten voor de bedrijfsvoering. Deze nieuwe uitgangspunten gaan in tegen de tot nu toe algemeen aanvaarde overtuigingen:

- Bruin water wordt voornamelijk veroorzaakt door deeltjes uit het drinkwater zelf tegenover het algemeen aanvaarde uitgangspunt dat onbekleed gietijzer de belangrijkste bron is voor bruin water problemen.

- Schoonmaken door middel van waters spuien is effectief mits uitgevoerd onder strikte randvoorwaarden versus de indruk dat water spuien op de conventionele manier niet werkt.
- Distributieleidingnetten moeten vertakt worden aangelegd met een afnemende diameter en hoge snelheden versus het uitgangspunt van vermaasde leidingnetten met grote diameters.

Toepassing van de nieuwe uitgangspunten vraagt veel van de flexibiliteit van alle partijen die betrokken zijn bij het ontwerp, de bouw en het beheren van de leidingnetten. Het vraagt echter ook om meer onderbouwing en bewijs om de daadwerkelijke implementatie eenvoudiger te maken. De volgende aanbevelingen hebben betrekking op de belangrijkste aspecten voor nieuw en/of verder onderzoek:

- Een verdere analyse van het effect van verbeterde zuivering door ofwel toevoegen van nieuwe behandelingsstappen of een betere bedrijfsvoering van bestaande zuiveringsstappen:
  Zoals in deze studie aannemelijk is gemaakt, kunnen problemen in het distributienet voorkomen worden of op zijn minst beperkt als het drinkwater af pompstation aan bepaalde richtlijnen voldoet. Er zal een nieuwe definitie moeten worden gemaakt voor de distributie-stabiliteit analoog aan de biofilmvormingspotentie (van der Kooij et al., 2003) of de chemische stabiliteit (van den Hoven and van Eekeren, 1988), waarin inbegrepen een on-line en mobiele meettechniek die zowel op het pompstation als in het leidingnet kan worden ingezet.
- De ontwikkeling van een waterkwaliteitsmodel voor deeltjes
  Op dit moment wordt de toepassing van waterkwaliteitsmodellen beperkt door de nauwkeurigheid van de berekening van de hydraulische beweging. De ontwikkeling van het model SIMDEUM zoals voorgesteld door Blokker et al (2005)zal de waterkwaliteitsmodellering een stap verder brengen in de richting van een echt water kwaliteitsmodel.
  Met deze nieuwe basis voor waterkwaliteitsmodellering kan ook het aantal modelparameters worden uitgebreid verder dan het tot nu toe gebruikelijke restchloorgehalte. Uiteraard zullen deeltjes deel uitmaken van de nieuwe parameters , maar ook het modelleren van parameters als DOC, AOC ijzer en mangaan zullen de mogelijkheden voor leidingnetonderhoud uitbreiden.
- Het ontwikkelen van een proefinstallatie om de micro-processen die de bezinking en opwerveling bepalen nader te bestuderen. De uitdaging bij de ontwikkeling van de proefinstallatie is het vinden van mogelijkheden om ofwel de processen te versnellen ofwel te concentreren om de proeven binnen een redelijke termijn te laten verlopen.
- Verdere ontwikkeling en validatie van meetmethoden als de TILVS en de Hemoflow.
  de concentratiemethoden zijn van cruciaal belang om meer kennis te vergaren over de samenstelling van het deeltjes materiaal in het water en de effecten daarvan op de opbouw van sedimentlagen. De methodologie moet goed worden gestandaardiseerd en beschreven. Op termijn kan een database worden opgebouwd met gegevens over de samenstelling van discreet materiaal in verschillende watertypen en het effect daarvan in het leidingnet. De combinatie van de TILVS analyse samen met het totaal ijzer gehalte van het water zou een goede parameter kunnen zijn om de nacoagulatie-potentie van het water te bepalen.
- Ontwikkelen van alternatieven voor de levering van bluswater via brandkranen
  Brandkranen zijn nog steeds dominant bij het ontwerp van leidingnetten, hoewel hun daadwerkelijke bijdrage aan de brandveiligheid relatief gering is. De discussie met de brandweer hebben de belangstelling gewekt voor brandpreventie en pro-actie, waarin water duidelijk een belangrijke rol speelt. Voor water-specialisten is er een interessant

nieuw gebied om te ontdekken waarin de rol van de openbare drinkwatervoorziening kan worden geoptimaliseerd binnen het totale beeld van preventie en pro-actie. De woningsprinkler is bijvoorbeeld een bewezen toepassing die de brand veiligheid sterk bevorderd, maar die een zeer slecht imago hebben bij de waterleidingbedrijven, voornamelijk vanwege de eisen die industriële toepassingen van sprinklers stellen aan leidingnetten. De belangrijkste ontwikkeling voor woningsprinklers zou erop gericht moeten zijn om sprinklerkoppen te ontwikkelen die minder volumestroom nodig hebben dan de huidige zodat ze op grote(re) schaal kunnen worden toegepast. Potentieel zou dit de totale schadekosten van brand met 90% kunnen beperken en 40 tot 50 levens per jaar kunnen sparen om niet te spreken van vele gewonden.

- Een gedurfdere en flexibeler bedrijfsvoering van leidingnetten
  Door het snelheidscriterium mee te nemen in het ontwerp van distributieleidingnetten zijn nieuwe manieren voor het beheren van leidingnetten geïntroduceerd. Het heeft de rol van afsluiters expliciet gemaakt binnen het concept van de continuïteit van de levering (Trietsch and Vreeburg, 2004; Trietsch and Vreeburg, 2006). De mogelijkheden voor de introductie van het snelheidscriterium in het transportleidingnet alsmede de strategische plaatsing van afsluiters zou verder moeten worden ontwikkeld samen met de mogelijkheden voor een meer dynamische netwerksturing met bijvoorbeeld op afstand bediende afsluiters en pompen om het water ‘in beweging te houden’.

9 References

Alere, I. and Hanæus, J. (1997). *"Particle dynamics in the drinking water distribution network of Luleå."* Vatten Vol: 53 (4): 381-390.

Antoun, E.N., Dyksen, J.E. and Hiltebrand, D. (1999). *"Unidirectional flushing: A powerful tool."* Journal American Water Works Association Vol: 91 (7): 62-71.

Antoun, E.N., Tyson, T. and Hiltebrand, D. (1997). *"Unidirectional flushing: a remedy to water quality problems such as biologically mediated corrosion"*. AWWA Annual Conference 1997, Denver, CO.

APHA (1998). *"Standard Methods for the Examination of Water and Wastewater"*, APHA, AWWA, WEF.

Barbeau, B., Gauthier, V., Julienne, K. and Carriere, A. (2005). *"Dead-end flushing of a distribution system: Short and long-term effects on water quality."* Journal of Water Supply: Research and Technology - Aqua Vol: 54 (6): 371-383.

Berlamont, J. and van Goethem, J. (1984) *"Avoiding mud accumulation in harbours and their entrances (in Dutch)"*, (Eds), Hydrualic laboratory KU, Leuven, Belgium

Beuken, R.H.S. (2001) *"Validation new design rules for DWDS: lab experiments on standard sediment' (In Dutch)"*, (Eds), Kiwa WR NV, Nieuwegein, The Netherlands

Blokker, E.J.M. and Vreeburg, J.H.G. (2005). *"Monte Carlo Simulation of Residential Water Demand: A Stochastic End-Use Model"*. World Water Congress, Reston, VA, USA, American Society of Civil Engineers.

Blokker, E.J.M., Vreeburg, J.H.G. and Vogelaar, A.J. (2006). *"Combining the probabilistic demand model SIMDEUM with a network model"*. 8th annual water distribution system analysis symposium, Cincinnati, Ohio, USA.

Boxall, J.B., Skipworth, P.J. and Saul, A.J. (2001). *"A novel approach to modelling sediment movement in distribution mains based on particle characteristics"*. Proceedings of the Computing and Control in the Water Industry Conference, Water Software Systems: Theory and Applications, De Monfort University, UK.

Boxall, J.B., Skipworth, P.J. and Saul, A.J. (2003). *"Aggressive flushing for discolouration event mitigation in water distribution networks."* Water Science and Technology: Water Supply Vol: 3 (1-2): 179-186.

Brashear, K. (1998). *"Distribution water quality problems created by upgraded water treatment plants"*. 1998 AWWA WQTC, San Diego, USA.

Camp, T.R. and Stein, P.C. (1943). *"Velocity gradient and internal work in fluid motion."* J. Boston Soc. Of Civ. Engrs Vol: 30 (4): 219-237.

Caporaloni, M., Tampieri, F., Trombetti, F. and Vittori, O. (1975). *"Transfer of particles in nonisotropic air turbulence."* Journal Atmospheric Science Vol: 32: 565-568.

Carriere, A., Gauthier, V., Desjardins, R. and Barbeau, B. (2005). *"Evaluation of loose deposits in distribution systems through unidirectional flushing."* J Am Water Works Assoc Vol: 97 (9): 82 - 92.

Ceronio, A.D. and Haarhoff, J. (2002). *"Dealing with large particle counting data sets."* Water Science and Technology: Water Supply Vol: 2 (5-6): 35-40.

Ceronio, A.D. and Haarhoff, J. (2005). *"An improvement on the power law for the description of particle size distributions in potable water treatment."* Water Research Vol: 39 (2-3): 305-313.

Cheng, C.Y., Atkinson, J.F. and Bursik, M.I. (1997). *"Direct measurement of turbulence structures in mixing jar."* Journal of Environmental Engineering Vol: 123 (2): 115-125.

Clark, R., Lykins, B., Block, J., Wymer, L. and Reasoner, D. (1994). *"Water quality changes in a simulated distribution system."* Aqua Vol: 43 (6): 263-77.

Clark, R.M., Grayman, W.M., Males, R.M. and Hess, A.F. (1993). *"Modelling Contamination Propagation in Drinking Water Distribution Systems."* Journal of Environmental Engineering, ASCE Vol: 119 (2): 349-354.

Clement, J.A., Hayes, M., Kriven, W.M., Sarin, P., Bebee, J., Jim, K., Beckett, M., Snoeyink, V.L., Kirmeyer, G.J. and Pierson, G. (2002) *"Development of red water control strategies"*, (Eds), American Water Works Association, Denver

de Moel, P.J., Verberk, J.Q.J.C. and van Dijk, J.C. (2006). *"Water consumption"*. Drinking water Principles and practices(Eds). Singapore, World scientific publishing Co. Pte. Ltd.

DWI (2007) *"Information Leaflet discoloured water"*, D.W. Inspectorate (Eds), http://www.dwi.gov.uk/consumer/faq/discolour.htm

Ellison, D. (2003) *"Investigation of Pipe Cleaning Methods"*, AWWARF (Eds), Rep. No. 90938, Denver, USA.

Esveld, A.L. (1996). *"Model ordinance for fire safety and emergency aid (in Dutch)"*. The Hague, Association of Dutch Municipalities 90 322 1133 1: 90 322 1133 1.

Ford, J. (1997) *"Automatic sprinklers, a ten year study"*, (Eds), Home fire sprinkler coalition, Scottsdale, Arizona, USA

Friedman, M., Kirmeyer, G.J. and Antoun, E.N. (2002). *"Developing and implementing a distribution system flushing program."* Journal AWWA Vol: 94 (7(july 2002)): 48-56.

Friedman, M.J., Martle, K., Hill, A., Holt, D., Smith, S., Ta, T., Sherwin, C., Hiltebrand, D., Pommenrenk, P., Hinedi, Z. and Camper, A. (2003) *"Establishing site-specific flushing velocities"*, AwwaRF&KiwaWR (Eds), 6666 West Quincy Avenue, Denver, CO

Gauthier, V., Barbeau, B., Milette, R., Block, J.-C. and Prevost, M. (2001). *"Suspended particles in the drinking water of two distribution systems."* Water Science and Technology: Water Supply Vol: 1 (4): 237-245.

Gauthier, V., Barbeau, B., Tremblay, G., Milette, R. and Nernier, A.-M. (2003). *"Impact of raw water turbidity fluctuations on drinking water quality in a distribution system."* Journal of Environmental Engineering and Science Vol: 2 (4): 281-291.

Gauthier, V., Gérard, B., Portal, J.-M., Block, J.C. and Gatel, D. (1999). *"Organic matter as loose deposits in a drinking water distribution system."* Water Research Vol: 33 (4): 1014-1026.

Gauthier, V., Portal, J.-M., Rosin, C., Block, J.-C., Cavard, J. and Gatel, D. (1997). *"How good are water distribution systems for transport of particulate matter?"* Proceedings Water Quality Technology Conference of American Water Works Association, Denver, Colorado, USA.

Gauthier, V., Rosin, C., Mathieu, L., Prortal, J.M., Block, J.C., Chaix, P. and Gatel, D. (1996). *"Characterization of the loose deposits in drinking water distribution systems"*. Water Quality Technology Conference of the American Water Works Association, Boston, MA, USA.

Geudens, P.J.J.G. (2006) *"Water Supply Statistics 2005"*, VEWIN (Eds), Rijswijk, The Netherlands

Hamilton, P.D., Standen, G. and Parsons, S.A. (2003). *"An analysis of particel monitor sensitivity in potable water treatment."* Journal of Water Supply: reserach and Thechnology AQUA Vol: 52 (1): 1-11.

Huck, P.M., Coffey, W.B., Anderson, M.B., Maurizio, D.D., Slawson, R.M., Douglas, I.P., Jasim, S.Y. and O'Melia, C.R. (2002). *"Using turbidity and particle counts to monitor*

Cryptosporidium removals by filters." Water Science and Technology: Water Supply Vol: 2 (3): 65-71.

Huck, P.M. and Gagnon, G.A. (2004). *"Understanding the distribution system as a bioreactor: a framework for managing heterotrophic plate count levels."* International Journal of Food Microbiology Vol: 92: 347-353.

Kirmeyer, G.J., Friedman, M., Clement, J., Sandvig, A., Noran, P.F., Martel, K.D., Smith, D., LeChevallier, M., Volk, C., Antoun, E., Hiltebrand, D., Dyksen, J. and Cushing, R. (2000) *"Guidance manual for maintaining distribution system water quality"*, (Eds), AWWA Research Foundation and American Water Works Association, Denver, USA.

KNMI (2006). Temperature data. Vol.

Le Chevallier, M.W., Babcock, T.M. and Lee, R.J. (1987). *"Examination and characterization of distribution system biofilms."* Applied and Environmental Microbiology Vol: 53 (12): 2714-2724.

Lin, J. and Coller, B.A. (1997). *"Aluminum in a water supply, Part 3: Domestic Tap Waters."* Water, Journal of the Australian Water Association Vol: 24: 11-13.

Lut, M. (2005) *"Hydraulic behaviour of particles in a drinking water distribution system (Msc Thesis report)"*, (Eds), University of Technology Delft, Delft, The Netherlands

McNeill, L.S. and Edwards, M. (2001). *"Iron pipe corrosion in distribution systems."* J Am Water Works Assoc Vol: 93 (7): 88-100.

Meches, M. (2001) *"Chapter 13 - Biofilms in drinking water distribution systems"*, R.M. Clark and B.K. Boutin (Eds), Controlling disinfection by-products and microbial contamination in drinking water, National Risk Management Research Laboratory, Office of Research and Development, US Environmental Protection Agency, Cincinnati, Ohio, USA

Nguyen, B.V., Wu, J., Grainger, C., Ryan, G., Jayaratne, A. and Mathes, P. (2002) *"Particle concentration measurement in water distribution systems"*, (Eds), Cooperative Research Centre for Water Quality and Treatment, Adelaide, Australia

Oberoi, K. (1994). *"Distribution flushing program: Benefits and results."* AWWA Annual Conference, New York, AwwaRF.

Pothof, I.W.M. (1998) *"Pipe flushing with water/air mixtures (In Dutch)"*, W.D. Hydraulics (Eds), Delft, The Netherlands

Prince, R., Goulter, I. and Ryan, G. (2001). *"Relationship Between Velocity Profiles And Turbidity Problems In Distribution Systems"*. World Water and Environmental Resources Congress, Orlando, Florida.

Prince, R.A., Goulter, I. and Ryan, G. (2003). *"What causes customer complaints about discoloured drinking water? Correlating customer complaints with online monitoring of flow rate and turbidity."* Water Vol: 30 (2): 62-67.

REWAB (2003) *"Registration system yearly analysis results of Dutch water companies, available through ministry of VROM (Netherlands Ministry of Housing, Spatial Planning and the Environment"*, (Eds), Den Haag, The Netherlands

REWAB (2004) *"Registration system yearly analysis results of Dutch water companies, available through ministry of VROM (Netherlands Ministry of Housing, Spatial Planning and the Environment"*, (Eds), Den Haag, The Netherlands

Rosenthal, L.P.M., Mesman, G.A.M., de Koning, M. and Vreeburg, J.H.G. (2001) *"Key criteria for valve operation and maintenance"*, (Eds), Kiwa Water Research, Nieuwegein, the Netherlands

Russell, S. (1993) *"Water Industry Instrument Handbook – Book 4 Turbidity"*, W. plc (Eds), Wiltshire, UK.

Ruta, G. (1999). *"Assessing Water Quality at Customers' Internal Taps"*. 18th Federal Convention Proceedings Aust. Wat. Wastewat. Assoc. .

Sarin, P., Snoeyink, V.L., Lyte, D.A. and Kriven, W.M. (2004). *"Iron corrosion scales: models for scale growth, iron release and colored water formation."* Journal of Environmental Engineering ASCE Vol: 130 (4): 364-373.

Schaap, P.J. and Vreeburg, J.H.G. (1999) *"Cleaning of networks: Manual for set up, performance and monitoring of cleaning programs (in Dutch)"*, (Eds), Kiwa Water Research, Nieuwegein, the Netherlands

Sippola, M.R. and Nazaroff, W.W. (2002). Particle deposition from turbulent flow: review of published research and its applicability to ventilation ducts in commercial buildings. Berkely, CA, USA, Lawrence Berkely National Laboratory. Vol.

Slaats, N., Rosenthal, L.P.M., Siegers, W.G., Boomen, M.v.d., Beuken, R.H.S. and Vreeburg, J.H.G. (2002) *"Processes involved in generation of discolored water"*, (Eds), American Water Works Association Research Foundation / Kiwa, The Netherlands.,

Sly, L.I., Hodgkinson, M.C. and Arunpairojana, V. (1989). *"The importance of high aesthetic quality potable water in tourist and recreational areas."* Wat. Sci. Tech. Vol: 21 (2): 183-187.

Sly, L.I., Hodgkinson, M.C. and Arunpairojana, V. (1990). *"Deposition of Manganese in Drinking Water Distribution System."* Applied and Environmental Microbiology Vol: 56 (3): 628-639.

Smith, S.E., Bisset, A., Colbourne, J.S., Holt, D.M. and Lloyd, B.J. (1997). *"The occurrence and significance of particles and deposits in a drinking water distribution system."* Journal of the New England Water Works Association Vol: 111 (2): 135-150.

Snyder, J.K. and Deb, A.K. (2003) *"Impacts of Fire Flow on Distribution System Water Quality, Design, and Operation"*, (Eds), AWWARF report 2554, AWWARF

South-East-Water (1998) *"Water Quality Report 97/98"*, S.E. Water (Eds), Melbourne, Australia

Stephenson, G. (1989) *"Removing loose deposits from water mains: Operational guidelines"*, W.R. Centre (Eds), Swindon, UK

Stumm, W. and Morgan, J.J. (1996). *"Aquatic Chemistry, Chemical Equilibria and Rates in natural Waters"*. New York, John Wiley & Sons, Inc.

Torvinen, E., Suomalainen, S., Lehtola, M., Miettinen, I., Zacheus, O., Paulin, L., Katila, M. and Martikainen, P. (2004). *"Mycobacteria in water and loose deposits of drinking water distribution systems in Finland."* Applied And Environmental Microbiology Vol: 70 (4): 1973-1981.

Trietsch, E.A. and Vreeburg, J.H.G. (2004). *"Reliability of valves and section isolation."* Water Science and Technology; Water Supply Vol: 5 (2): 47-51.

Trietsch, E.A. and Vreeburg, J.H.G. (2006). *"Effect of valve failures on network reliability"*. 8th annual water distribution system analysis symposium, Cincinnati, Ohio, USA, August 27-30, 2006.

van den Hoven, T.J.J. and van Eekeren, M.W.M. (1988) *"Optimal composition of drinking water"*, (Eds), Kiwa Water Research, Nieuwegein, The Netherlands

van den Hoven, T.J.J. and Vreeburg, J.H.G. (1992). *"Distribution system analysis by continuous monitoring and network calculation."* Water Supply Vol: 20 (1): 117-124.

van der Kooij, D. (1992). *"Assimible organic carbon as an indicator of bacterial regrowth."* Journal of AWWA Vol: 84 (2): 57-65.

van der Kooij, D. (2000). *"Biological stability: a multidimensional quality aspect of treated water."* Water, Air and Soil Pollution Vol: 123: 25-34.

van der Kooij, D. (2002) *"Managing regrowth in drinking water distribution systems"*, (Eds), Kiwa Water Research, Nieuwegein, the Netherlands van der Kooij, D., Vrouwenvelder, J.S. and Veenendaal, H.R. (2003). *"Elucidation and control of biofilm formation processes in water treatment and distribution using the unified biofilm approach."* Water Science and Technology Vol: 47 (5): 83-90.

van Dijk, J.C. and van der Kooij, D. (2005). *"Water Quality 21 research programme for water supplies in The Netherlands."* Water Science and Technology: Water Supply Vol: 4 (5-6): 181-188.

Verberk, J.Q.J.C. (2007). *"The Down Under water experience"*. Vakantiecursus, Delft, The Netherlands, TU Delft.

Verberk, J.Q.J.C., O'Halloran, K.J., Hamilton, L.A., Vreeburg, J.H.G. and Van Dijk, J.C. (2006). *"Measuring particles in drinking water transportation systems with particle counters."* submitted to Journal of Water Supply: Research and Technology - AQUA Vol.

Versteegh, J.F.M. and Dik, H.H.J. (2006). *"The drinking water quality in the Netherlands in 2004 (in Dutch)."* Report of Ministry of Housing, Spatial Planning and the Environment Vol: (http://www.vrom.nl/pagina.html?id=7769).

VEWIN (2004) *"Reflections on performance 2003"*, Accenture (Eds), Benchmark 2003, VEWIN, Rijswijk, The Netherlands, www.vewin.nl

Vreeburg, J.H.G. (1996) *"Brown water, cause and consequence. Efficiency of cleaning with flushing, water/air scouring and pigging (in Dutch)"*, (Eds), SWE 96.008, Kiwa Water Research, Nieuwegein, The Netherlands

Vreeburg, J.H.G. and Boxall, J.B. (2007). *"Discolouration in potable water distribution systems: A review."* Water Research Vol: 41: 519-529.

Vreeburg, J.H.G., Hoogsteen, K.J. and K., P. (1998). *"Risk management: Integral appraoch in the Netherlands."* Water Supply Vol: 16 (1-2): 472-474.

Vreeburg, J.H.G., Schaap, P. and van Dijk, J.C. (2004a). *"Measuring discolouration risk: Resuspension potential method"*. IWA Leading Edge Conference, Prague, IWA.

Vreeburg, J.H.G., Schaap, P.G. and van Dijk, J.C. (2004b). *"Particles in the drinking water system: from source to discolouration."* Water Science and Technology Vol: 4 (5-6): 431-438.

Vreeburg, J.H.G., van den Hoven, T.J.J. and Hoogsteen, K.J. (1994). *"A quantitative method to determine reliability of water supply systems."* Water Supply Vol: 12 (1-2): ss7.9-7.13.

Vreeburg, J.H.G., Wang, Q. and van Dijk, J.C. (2007). *"Composition and hydraulic behaviour of drinking water distribution systems sediments."* Journal of AWWA Vol: Submitted 2007.

Walski, T.M. (1991). *"Understanding Solids Transport in Water Distribution Systems"*. Water Quality Modelling In Distribution Systems, USA, pp 305-309.

Water-Act (1958). *"Water Act."* Vol: (150).

Young, J. and Leeming, A. (1997). *"A thory of particle deposition in turbulent pipe flow."* Journal of Fluid Mechanics, Cambridge University Press Vol: 340: 129-159.

Zacheus, O.M., Lehtola, M.J., Korhonen, L.K. and Martikainen, P.J. (2001). *"Soft deposits, the key site for microbial growth in drinking water distribution networks "*, Water Research Vol: 35 (7): 1757-1765.

Publications and presentations

International refereed publications

Vreeburg, J.H.G. and Boxall, J.B. (2007), *"Discolouration in potable water distribution systems: A review"*, Water Research, Vol: 41: 519-529.

Vreeburg, J.H.G., Wang, Q. and van Dijk, J.C. (2007), *"Composition and hydraulic behaviour of drinking water distribution systems sediments"*, Journal of AWWA, Vol: Submitted, (##): ##-##.

Vreeburg, J.H.G. (2006), *"Fate of particles in the distribution system"*, Interface science in drinking water treatment, G. Newcombe and D. Dixon (Eds), Melbourne, Australia, Elsevier: 279-298.

Vreeburg, J.H.G., Schaap, P. and van Dijk, J.C. (2004), *"Measuring discolouration risk: Resuspension potential method"*, IWA Leading Edge Conference proceedings, Prague, May 2005, IWA.

Vreeburg, J.H.G., Schaap, P.G. and van Dijk, J.C. (2004), *"Particles in the drinking water system: from source to discolouration"*, Water Science and Technology, Vol: 4, (5-6): 431-438.

Vreeburg, J.H.G. and van den Boomen, M. (2002), *"Dutch challenge to network design"*, Water 21, (december 2002): 43-45.

Vreeburg, J.H.G., Hoogsteen, K.J. and K., P. (1998), *"Risk management: Integral appraoch in the Netherlands"*, Water Supply, Vol: 16, (1-2): 472-474.

Vreeburg, J.H.G., van den Hoven, T.J.J. and Hoogsteen, K.J. (1994), *"A quantitative method to determine reliability of water supply systems"*, Water Supply, Vol: 12, (1-2): ss7.9-7.13.

Dane, P.A., Hoogsteen, K.J., van den Hoven, T.J.J. and Vreeburg, J.H.G. (1997), *"National report Planning and design for continuity and reliability in distribution systems"*, Water Supply, Vol: 14, (3-4): 98-99.

van den Hoven, T.J.J., van der Kooij, D., Vreeburg, J.H.G. and Brink, H. (1996), *"Simplified methods to monitor water quality changes in distribution systems."* Water Supply, Vol: 14, (3-4): 522-525.

Trietsch, E.A. and Vreeburg, J.H.G. (2004), *"Reliability of valves and section isolation"*, Water Science and Technology; Water Supply, Vol: 5, (2): 47-51.

van den Hoven, T.J.J., Vreeburg, J.H.G. and Hoogsteen, K.J. (1993), *"Sicherheit der Trinkwasserversorgung in den Niederlanden"*, GWA (Gas, Wasser, Abwasser), Vol: 73, (1): 28.

van den Hoven, T.J.J. and Vreeburg, J.H.G. (1992), *"Distribution system analysis by continuous monitoring and network calculation"*, Water Supply, Vol: 20, (1): 117-124.

Verberk, J.Q.J.C., O'Halloran, K.J., Hamilton, L.A., Vreeburg, J.H.G. and Van Dijk, J.C. (2007), *"Measuring particles in drinking water transportation systems with particle counters"*, submitted to Journal of Water Supply: Research and Technology - AQUA.

Verberk, J.Q.J.C., Hamilton, L.A., O'Halloran, K.J., van der Horst, W. and Vreeburg, J.H.G. (2006), *"Analysis of particle numbers, size and composition in drinking water transportation pipelines: result of online measurements"*, Water Science and Technology, Vol: 6, (4): 35-43.

International Conference presentations/proceedings

Vreeburg, J.H.G. (2005), *"Experiences with collaboration on research Asset Management in the Netherlands"*, GWRC Workshop Asset Management Research, Alexandria, VA, USA, May 4 – 7, 2005.

Vreeburg, J.H.G. and Schaap, P.G. (2004), *"High quality water and discolouration tools in Dutch drinking water systems"*, UKWIR symposium discolouration, London, 25 May 2004.

Vreeburg, J.H.G. (2003), *"Drinking water quality in the network: Developments in the Netherlands"*, IWW-Kolloquien, Muhlheim, 3 july 2003, IWW.

Vreeburg, J.H.G., Rosenthal, L.P.M., van den Boomen, M. and Beuken, R.H.S. (2002), *"Distribution system research in the Netherlands"*, Workshop Global Research on Distribution System Water Quality, Melbourne, Australia, April 10, 2002.

Vreeburg, J.H.G., Rosenthal, L.P.M., van den Boomen, M. and Beuken, R.H.S. (2002), *"Customer based performance criteria for asset management"*, IWA world Conference; Workshop Asset Management, Melbourne, April 10, 2002.

Vreeburg, J.H.G., Rosenthal, L.P.M. and van den Boomen, M. (2001), *"Discoloured Water in Perspective"*, Discolouration seminar, Coventry, UK, University of Warwick.

Vreeburg, J.H.G., Rosenthal, L.P.M. and van den Boomen, M. (2001), *"Red Water Control Strategies in the Netherlands"*, Distribution System Symposium, San Diego, USA, September 24, 2001.

Vreeburg, J.H.G. (2000), *"Mechanisms and solutions for red water"*, Awwa Annual congres; International Distribution Research Symposium, Denver, CO, USA, 10/11 June 2000.

Blokker, E.J.M., Vreeburg, J.H.G. and Vogelaar, A.J. (2006), *"Combining the probabilistic demand model SIMDEUM with a network model"*, 8th annual water distribution system analysis symposium, Cincinatti, Ohio, USA, August 27-30, 2006.

Blokker, E.J.M. and Vreeburg, J.H.G. (2005), *"Monte Carlo Simulation of Residential Water Demand: A Stochastic End-Use Model"*, World Water Congres, Restom, VA, USA, American Society of Civil Engineers.

Breen, J., Boersma, A., Slaats, P.G.G. and Vreeburg, J.H.G. (2004), *"Long term performance prediction of existing PVC water distribution systems"*, XII Plastic Pipes conference, Madrid, Spain.

Lavooy, C.S.W., Ent, W.I.v.d. and Vreeburg, J.H.G. (1993), *"Maintenance of network models - a demand allocation method"*, Integrated computer applications in Water supply, Taunton, UK., Research study press.

Rosenthal, L.P.M., Mesman, G.A.M., de Koning, M. and Vreeburg, J.H.G. (2001), *"Key criteria for valve operation and maintenance"*, BTO 2001.155, Kiwa Water Research, Nieuwegein, the Netherlands.

Trietsch, E.A. and Vreeburg, J.H.G. (2006), *"Effect of valve failures on network reliability"*, 8th annual water distribution system analysis symposium, Cincinatti, Ohio, USA, August 27-30, 2006.

National publications

Vreeburg, J.H.G. (2007), *"Grenzen verleggen in distributie"*, 59e Vakantiecursus in drinkwatervoorziening, Delft, The Netherlands.

Vreeburg, J.H.G., Blokker, E.J.M. and Beuken, R.H.S. (2006), *"Risicomethodiek ook bruikbaar voor waterleidingen?"* H2O, Vol: 39, (9): 28.

Vreeburg, J.H.G., Kivit, C., van Dijk, J.C. and Verberk, J.Q.J.C. (2005), *"Herkomst en gedrag van deeltjes in leidingnetten"*, H2O, Vol: 38, (7): 65-68.

Vreeburg, J.H.G. (2004), *"Leveringszekerheid drinkwater distributienetten"*, PAO Onzekerheden rioleringsbeheer, Delft, 13 mei 2004, PAO.

Vreeburg, J.H.G. (2003), *"Bluswaterregelingen internationaal weinig uniform: Ieder huis zijn eigen sprinkler?"* H2O, Vol: 35, (1): 4.

Vreeburg, J.H.G. (2003), *"Drinkwatertransport door de jaren heen"*, PAO cursus Distributie van Drinkwater, Nieuwegein, PAO.

Vreeburg, J.H.G. (2003), *"Ontwikkelingen HQ water en de impact op het leidingnet"*, PAO cursus distributie van drinkwater, Nieuwegein.

Vreeburg, J.H.G. (2003), *"Algemene inleiding"*, 55e Vakantiecursus in drinkwatervoorziening, Delft, The Netherlands.

Vreeburg, J.H.G. (2002), *"Van leiding naar leidingnetbeheer"*, Lijn in leidingbeheer, Amersfoort, 26 september 2002.

Vreeburg, J.H.G. (2002), *"Rol van verblijftijd op waterkwaliteit in het leidingnet"*, H2O, Vol: 34, (24): 24.

Vreeburg, J.H.G., Beuken, R.H.S. and Mul, P. (2002), *"Nieuwe ontwerprichtlijnen voor distributienetten: de stand van zaken"*, H2O, Vol: 35, (5): 28-30.

Vreeburg, J.H.G. (1996), *"Brown water, cause and consequence. Efficiency of cleaning with flushing, water/air scouring and pigging (in Dutch)"*, SWE 96.008, Kiwa Water Research, Nieuwegein, The Netherlands.

Vreeburg, J.H.G. (1989), *"A new fase in the development of the Kiwa network calculation program ALEID (in Dutch)"*, H2O, Vol: 22, (17): 535.

Beuken, R.H.S., Vreeburg, J.H.G. and Kerkhoven, W.-J. (2003), *"Telemeting bij huishoudelijke watermeters"*, H2O, Vol: 36, (21): 41.

Grefte, A., Vreeburg, J.H.G., verberk, J.Q.J.C. and van Dijk, J.C. (2006), *"Experimenteel onderzoek naar hydrualisch gedrag van sediment in het distributienet"*, H2O, Vol: 39, (13): 50-53.

Kivit, C., Schaap, P.G., Kreukniet, F. and Vreeburg, J.H.G. (2006), *"Sedimentverwijdering bij verschillende spuisnelheden"*, H2O, Vol: 39, (6): 65-67.

Schaap, P.J. and Vreeburg, J.H.G. (1999), *"Cleaning of networks: Manual for set up, performance and monitoring of cleaning programs (in Dutch)"*, SWE 99.009, Kiwa Water Research, Nieuwegein, the Netherlands.

Slaats, P.G.G., Vreeburg, J.H.G., Boersma, A. and Breen, J. (2003), *"PVC waterleidingen: hoe lang gaan ze mee"*, H2O, Vol: 36, (16): 25-28.

Urbanus, J.F.X., Vreeburg, J.H.G. and Mazijk, A.v. (1988), *"Calam: A calamity model for the Northern Delta. (in Dutch)"*, H2O, Vol: 21, (24): 717.

van den Hoven, T.J.J., Kramer, J.E., Hoefnagels, F.E.T. and Vreeburg, J.H.G. (1993), *"Environmental effects of pipe materials (in Dutch)"*, H2O, Vol: 26, (22): 651.

van den Hoven, T.J.J. and Vreeburg, J.H.G. (1992), *"Waterkwlaiteit in distributie netwerken: helderheid met monitor systemen"*, H2O, Vol: 25, (10): 255.
Verberk, J.Q.J.C., Vreeburg, J.H.G., Bosch, A. and van Dijk, J.C. (2006), *"Analyse van Deeltjes in het transportleidingnet van Amsterdam"*, H2O, Vol: 39, (1): 29-32.

List of abbreviations

AC	:	Asbestos cement
AOC	:	Assimible Organic Carbon
BTO	:	BedrijfsTakOnderzoek van de Nederlandse Waterleidingbedrijven Joint research Program of the Dutch water companies
CI	:	Cast Iron
CSR	:	Critical Stirring Rate
DS	:	Dissolved Solids
DWDS	:	Drinking Water Distribution System
FTU	:	Formazine Turbidity Unit
lpppd	:	Liter per person per day
NCI	:	Nodular Cast Iron
NTU	:	Nephelometric Turbidity Unit
PE	:	Poly Ethelyn
PVC	:	Poly Vinyl Chloride
RPM	:	Resuspension Potential Method
RSF	:	Rapid Sand Filtration
TSS	:	Total Suspended Solids
TU	:	Tapping Units
VSS	:	Volatile Suspended Solids

Dankwoord

Hoewel het schrijven van een proefschrift een individuele activiteit is, is het onderzoek dat daaraan ten grondslag ligt absoluut geen individuele bezigheid. In mijn geval is dat zeker zo, omdat het onderzoek dat in dit boekje is beschreven alleen maar tot stand heeft kunnen komen doordat velen zich ermee hebben bemoeid. Aan het einde van het boekje is het dan ook eigenlijk de leukste taak om al die mensen te bedanken die een bijdrage hebben geleverd. Het frustrerende van die taak is echter dat het onmogelijk is om iedereen te noemen omdat het er vreselijk veel zijn geweest, maar ook, hoe vreemd dit wellicht ook mag klinken, omdat ik lang niet iedereen bij naam ken die metingen hebben verricht en de resultaten hebben toegepast.

Hans van Dijk en Ron van Megen zijn ontzettend belangrijk geweest voor het totstandkomen van dit boekje. Hans is de inspirerende mentor die met slechts kleine opmerkingen in staat is geweest mij te motiveren om het nu eindelijk allemaal eens op te schrijven, maar vooral ook de verdiepingsslag te maken en om met name het belang van de bronterm in de zuivering te benadrukken. De tijd is nu rijp om op een andere manier naar de zuivering te kijken in relatie tot de waterkwaliteit in het leidingnet en ik kijk ernaar uit om de discussies die we daarover hebben gehad om te zetten in daden, onderzoek én onderwijs. Ron heeft goed gezien dat het noodzakelijk is voor zowel Kiwa als de TU om nauwe banden met elkaar te hebben en heeft voor mij de mogelijkheden geschapen om dit daadwerkelijk invulling te geven. Daarmee heeft hij een mes aan drie kanten weten te laten snijden: Kiwa en de TU zijn er meetbaar beter van geworden en ik heb het beste van twee banen gekregen.

Een cruciale factor in het onderzoek van de afgelopen jaren zijn de mensen geweest die de gelegenheid hebben geboden om te “spelen in het netwerk”. Hoewel ze het vast niet zo bedoeld hebben, heb ik het praktijk onderzoek altijd zo ervaren: lekker buiten spelen met je vriendjes en dan nog leuke dingen doen ook. In de lange rij van mensen die dit hebben mogelijk gemaakt wil ik hier met namen noemen Cees Maasakkers die in 1989 de moed had om een jonge knaap met een vlotte babbel zijn gang te laten gaan en Gerard Engels die het als één van de eersten aandurfde om een netwerk volgens de nieuwe richtlijnen te ontwerpen en te bouwen. Theo van den Hoven is mijn mentor en klankbord geweest tijdens die eerste onderzoeken en hij heeft me geleerd om de essentie uit een brei aan gegevens te halen met de simpele opmerking: “Wat was de vraag ook al weer?”. In hen wil ik alle mensen danken die mij mijn gang lieten gaan en me vertrouwen hebben geschonken maar me daarbij bovendien het gevoel hebben gegeven met iets nuttigs bezig te zijn.

Uiteraard zijn al mijn collega’s van Kiwa en de TU uiterst belangrijk geweest. Met name natuurlijk mijn directe colleges uit de kennisgroep Water Infrastructuur bij Kiwa Water Research. Over de jaren hebben we als groep verschillende namen gehad, maar we zijn altijd de loyale en hardwerkende, maar vooral leuke club mensen geweest.
Heel bijzonder vind ik het dat Peter Schaap mijn paranimf wil zijn op de dag van mijn promotie. Peter is naast een aardige vent en goede college ook de persoon die me inhoudelijk scherp heeft gehouden. Zijn karakteristieke opmerking: “Je weet het allemaal

weer prachtig te zeggen, Vreeburg, maar klopt dat nu wel” is een voortdurende inspiratie om met beide voeten op de grond te blijven. Daarnaast heeft hij ervoor gezorgd dat veel van mijn wilde plannen voor projecten en metingen daadwerkelijk werden uitgevoerd. Zonder hem had dit proefschrift niet tot stand kunnen komen.
De andere kring van collega’s, bij de TU, bieden een heel eigen manier van ondersteuning. Mijn kamergenoten Bram van der Veer en Peter de Moel hebben zeer bijgedragen aan het onderzoek. Bram door me mogelijkheden te bieden om daadwerkelijk onderzoek te doen in een bijzonder leidingnet en Peter door me continu uit te dagen om scherp in one-liners te formuleren om hem een beetje onder de duim te houden. Mijn andere paranimf en TU-collega, Jasper Verberk, heeft op een heel bijzondere manier bijgedragen. Allereerst door ‘studenten te regelen’, maar vooral door zijn kritische en enthousiaste blik op het concept van mijn proefschrift, waarmee ook hij een meer dan wezenlijke bijdrage heeft geleverd aan het eindresultaat.

Zoals gezegd hebben veel mensen binnen de bedrijfstak van de drinkwatervoorziening bijgedragen aan het onderzoek en is het onmogelijk om ze allemaal te noemen. Ik prijs me gelukkig met zo veel mensen die zodanig geloven in wat ik doe dat ze daadwerkelijk mijn adviezen uitvoeren en me hebben verbaasd met de gegevens uit de praktijk waarmee ik nu kan aantonen dat het allemaal werkt zoals we dachten dat het zou werken.

Naast de mensen die aan het daadwerkelijke onderzoek hebben bijgedragen, heeft de grote groep familie en vrienden mij gemaakt tot wie ik ben, waardoor ik het onderzoek heb kunnen uitvoeren zoals het is gedaan.
Mijn broer heeft hierin een speciale plaats omdat hij, uiteraard onbewust, mij altijd heeft gestimuleerd en uitgedaagd om het beste uit mijzelf naar boven te brengen. Vanaf onze vroegste jeugd hebben wij eendrachtig samengewerkt en samen geleefd en ik hoop dat we dat nog vele jaren zullen volhouden samen met onze gezinnen.
Mijn ouders zijn er altijd voor mij en mijn gezin geweest op een vanzelfsprekende manier die zich niet laat omschrijven. Het is mij een bijzonder genoegen om dit proefschrift aan hen op te dragen als een kleine premie op de opvoedschuld.
Sabine en Linda maken dat ik met vreugde en liefde de oude schuld aan het terugbetalen ben. Jullie zijn voor mij het klankbord dat ik nodig heb en ik heb jullie een beetje misbruikt om te testen of waar ik mee bezig ben zinvol genoeg is: geen kritischer publiek dan twee tieners. Ik hoop dat ik jullie op dezelfde manier kan stimuleren het beste uit jezelf naar boven te halen op ieder relevant gebied dat nodig is om gelukkig en tevreden te zijn.

Voor Willemijn kom ik woorden tekort om te beschrijven wat ze voor mij betekent en hoe zij heeft bijgedragen aan mijn carrière. Het is zeker dat het zonder haar allemaal niet mogelijk zou zijn geweest.

Curriculum vitea

Jan Vreeburg was born in ’s-Gravenhage on October 13, 1960. He graduated from Gymnasium-β at the Sint Maartenscollege in Voorburg in 1979. In that year he started his study Civil Engineering at the University of Technology in Delft from which he graduated in 1987. During the last years of this study, from 1984 to 1987 he also worked at the department of Sanitary Engineering of the faculty of Civil Engineering. From 1987 to 1989 he worked with the Dune water company of ‘s-Gravenhage on the field of drinking water distribution. In 1989 he joined Kiwa Water Research where he is still employed as principal researcher. In 2001 he joined the University of Technology Delft for a part-time job to incorporate his findings on the field of drinking water distribution within the curriculum of the Department Sanitary Engineering.