The application of Sediment Transport Models to predict discoloured water events

M.Sc. Report

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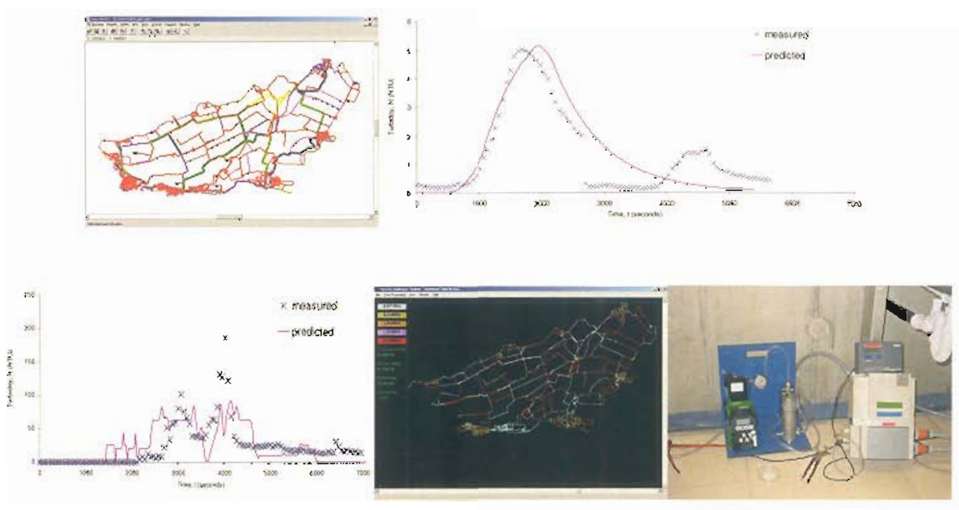

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The application of Sediment Transport Models to predict discolored water events

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Preface

This report is the result of my M.Sc. study in Civil Engineering at TU Delft and marks the end of a long academic period. The study was mainly performed at Kiwa Water Research in Nieuwegein in the period of July 2004 until March 2005 and a short period at the University of Sheffield in December 2004.

I would like to thank the people at Kiwa that helped me with this research: especially my mentor Jan Vreeburg for his patient help and Peter Schaap for his insight in the practical value of research. But also my temporary colleagues at Kiwa WR/ Waterinfrastructure and of course my roommate Nellie Slaats who was always kind enough to help me with any kind of problem.
Wilfred Burger, Bart Schulte and Arie Haasnoot of Hydron ZH and Hydron A&D were so kind to let me use the model of the supply area of De Laak. Many thanks especially to Wilfred for his struggle to get me the right data on my trip in England.

I have had a lot of help from the people from the Q21 research project. I think that the cooperation of the students involved in this project was inspiring to everyone and hope they have liked it very much, as have I. And of course the inspiration of my professor Hans van Dijk who during our monthly meeting knew how to keep everyone sharp.

Summary

This report will treat the use of models to predict discoloured water events. Discoloured water events happen when sediment in drinking water pipes is resuspended. This can lead to (brown coloured) sediment coming from the consumer's tap. As this is not desirable, models have been developed to predict these occurrences.

This report will start in Chapter 2 with the theory of water quality modelling and hydraulics of pressurized flow of water through pipes. In the same chapter the different processes inside pipes like deposition and resuspension of sediment and other processes are extensively described. These processes are based on the mass-balance that is used as a framework.

Chapter 3 shows a short list of commercial models that are designed to predict distribution of sediment in networks and it is shortly described why these models cannot be used for the prediction of sediment in networks.

The non-commercial models for the prediction of discoloured water events are described in Chapter 4. The theory and background of the PODDS model, developed by University of Sheffield (UK), is first described. PODDS uses the approach of a cohesive layer build up, that entrains fouling material. By the erosion of this layer a prediction of the turbidity pattern can be made. In the further paragraphs the PSM model, developed by the CRC (Cooperative Research Centre, Australia) is dealt with. PSM assumes that sediment settles and/ or resuspends at certain velocities, the characteristics of sediments are used to predict the distribution of sediment in a distribution network.

To say something of the applicability and validity PODSS, it is used on a two cases in the supply area of pumping station 'De Laak' in Chapter 5.

Chapter 6 deals with the cases performed at 'De Laak' with the help of the PSM model. To determine the amount of sediment coming from this treatment plant, a study for the application of filter systems has been performed. The results of these filter tests are used in the PSM model.

Chapter 7 shows the conclusions and recommendations of this report.

Chapter 8 shows the literature list of this report.

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1 Introduction

Drinking water in the Netherlands meets very high standards, the average citizen relies on the impeccable quality of tap water. To maintain this high quality the Dutch drinking water companies have started the Q21 research program in cooperation with Kiwa Water Research and TU Delft. This program focuses on three elements: introducing new water treatment techniques, avoid deterioration in the distribution network and avoid regrowth of Legionella. This report will discuss the distribution of sediment in drinking water networks.

Sediment settling in drinking water networks is not wanted as it can lead to deteriorated water and so-called discolored water or 'brown-water' complaints of customers. These brown-water events can be caused by hydraulic disturbances in the network, cleaning of pipes or a fire event for example. Sediment has different origins: the contribution of the treatment plant is one of the main origins but other processes contribute to sediment as well. These processes regarding the formation, transportation and distribution of sediment in networks are not very well understood and are closely studied (Vreeburg et. al, 2004). In Figure 1-1 the mass balance of a drinking water network can be seen, showing the in- and outgoing load and the different processes inside of the pipe.

The ingoing load consists of water containing Suspended Solids, Particles, Color etc. Inside of the pipe processes like biofilm formation and sloughing, corrosion, formation and coagulation of particles and deposition and resuspension occur; these processes influence the outgoing load. Corrosion, resuspension and biofilm sloughing can lead to an increase of sediment, whereas a decrease of sediment can occur when sediment becomes trapped inside biofilm, is deposited or is suspended by formation and coagulation.

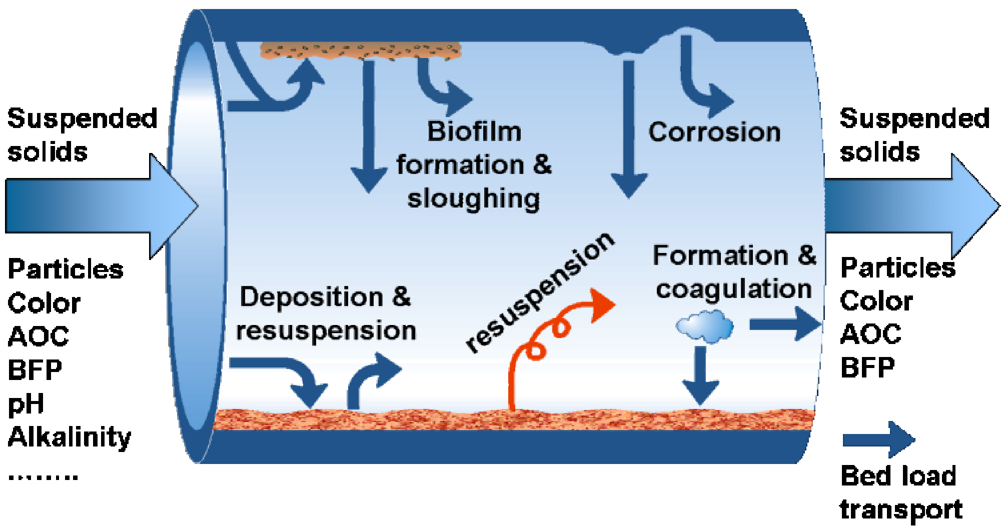


Figure 1-1 Mass balance of a drinking water network

When new treatment techniques are introduced, such as membrane filtration, the sediment load of the treatment plant is influenced because by membrane filtration less particles are introduced to the network. The decrease of this load leads to questions about the processes inside the network, this report however will focus on the behavior of the particles in the network.

The processes inside the pipe mentioned above are not further dealt with in this report, the main interest is the behavior of the particles coming from the treatment plant. Deposition and resuspension of particles from the plant will occur in networks. The most-used theory of the behavior of these particles is described by Stokes (deposition) and Shields (resuspension). Stokes' law assumes that particles with a larger density will settle faster than particles with a smaller density. However, the application of Stokes' law to predict settling of particles in drinking water is questionable. The same goes for Shields theory, this theory describes the starting of movement of particles caused by flowing water. Particles entering the network from the treatment plant tend to have a wide range of grain sizes and densities (Gaultier et. al. ;1996, 198, 2001). This difference in particles shapes and sizes can also be seen in Chapter 6, it shows the analysis of drinking water from pumping station 'De Laak' ; the sizes of the particles range between 1 and 100 µm. Because of the great variety of the particle sizes and shapes, the theories of Stokes and Shields are difficult to use.

As the behavior of particles is difficult to describe with these known formulas, models have been developed to make predictions of sediment build-up and distribution. Commercial models are available but these are usually based on the same formulas that are not directly applicable to predict sediment behavior. The goal of this report is:

1. Describe and test two (non-commercial) model:
   - PODDS (**P**rediction and Control **O**f **D**iscolouration in **D**istribution **S**ystems) model, developed by the University of Sheffield (UK)
   - PSM (**P**article **S**ediment **M**odel) model, developed by CRC (Cooperative Research Centre; Australia)

2. Test a method to determine the sediment load of a treatment plant , to be used with PSM model

The two models mentioned are tested on their use and applicability to predict discolored water events. The PODDS model is used to predict the reaction on a hydraulic disturbance, thus leading to the degree of fouling of a single pipe. The fouling is based on the average hydraulic conditions of that pipe, assuming that the pipe is conditioned by these hydraulic circumstances and the pipe is always 'saturated' with sediment. PSM calculates the distribution of sediment over a network with the source of sediment placed at the treatment plant and characterizations of that sediment. A complete picture of the distribution of sediment in the network can be given.

The sediment load of pumping station ‘De Laak’ is determined with the help of large volume sampling. By sampling the flow out of the plant and filtering the sampled water, the exact concentration of suspended solids is measured. This concentration is used in the PSM model to predict the distribution of sediment. The characteristics of the water of ‘De Laak’ are determined with the help of a research conducted at TU Delft (Lut, M.Sc. report; 2005). This contained the hydraulic behavior of particles in a test-rig. The results of settling and resuspension of different types of sediment (Kaolinite, $FeCl_3$ and flushing sediment) were used to determine the behavior of the sediment in the water of ‘De Laak’.

Both models are tested on a case: the network of the supply area of pumping station ‘De Laak’. PODDS is used to simulate two flushing acts, PSM is used to make a prediction of the distribution of sediment.

2 Theory

2.1 Introduction to modelling

A model is a system that is used to understand the behavior of another system that it tries to reproduce, generally by making it much more simple than the system it represents. Models are generally used to predict:

- System behavior at a location where it is difficult or costly to measure directly
- System behavior at another time under (a) changed environments(s)
- The likely impact of altering the structure of the system

In case of modeling of water quality processes, researchers often focus on the individual processes rather than fully modeling water quality throughout distribution networks (Powell et. al; 2004).. These water quality process models simulate the behavior of an isolated vessel of water and ignore the complexities caused by flow patterns and variation in the properties of pipes and other components which occur in real distribution networks. To apply water quality models to distribution networks the models are not (yet) completely extended to take into account:

- Variations in pipe properties (e.g. pipe material, condition and diameter)
- Variations in flow, velocity and water age with time and space
- Mixing between water that has traveled along different flow paths

Because a model is a simplified version of the reality a lot of assumptions are made. In hydraulic calculations some issues can be distinguished that influence the quality of a model, such as:

- Model development, skeletonization (simplifying model) and use of auto-calibration
- Quality and quantity of database
- Methods and accuracy of spatial and temporal demand allocation
- Quality of calibration and validation
- Etc.

With regard to water quality calculations some extra issues are also important:
- Research is performed on individual water quality parameters, combining of al these individual parameters will make a model very complex.
- Temperature changes have high impacts on various processes inside the network (processes like corrosion, biofilm formation etc.). When this is not implemented the model will not match the real situation.

The issues mentioned above describe the uncertainties that are involved in water quality models. In this chapter a description is given of the background of water quality models: the different processes involved in the origin of sediment in networks with the help of the Mass Balance and the hydraulic calculations and formulas.

Hydraulic modelling packages are used to provide more insight in the behaviour of drinking water networks. They are used to calculate flow, flow velocities and head loss for each pipe in the network. The sediment models that are described further in this report rely on these calculations. A description is given of the background of these programs and the important aspects in relation to sediment transport modelling.

2.2 Hydraulics

2.2.1 Headloss

Based on 'CT5500 - Water transport' (Vreeburg; 2003).

Drinking water networks are pressurized, this means that the pressure at the start is higher than atmospheric. Flow through pipes will cause an energy loss due to friction and local losses caused by the release of flow lines (entrance and deceleration losses).
The flow of pipes is described by two equations:

1. Continuity equation (Mass balance)
2. Motion equation (Momentum balance)

The continuity equation for a round pipe is:

$$\frac{\partial A}{\partial t}+\frac{\partial Q}{\partial x}=0$$ *Equation 1*

A = cross section of pipe [m^2]
Q = Volume flow [m^3/s]
t = Time [s]
x = Length [m]

The momentum equation is:

$$A\frac{\partial u}{\partial t}+u\frac{\partial A}{\partial t}+2Au\frac{\partial u}{\partial x}+u^2\frac{\partial A}{\partial x}+gA\frac{\partial p}{\partial x}+\frac{g}{C^2}\frac{A}{R}|u|u=0$$ *Equation 2*

Water transport through pipes is characterized with slow changing boundary conditions. When the so-called rigid column simplification is applied the following presumptions are made:

- Uniform and stationary flow
- Prismatic pipe, this means that the cross section of the pipe does not change over the length of the pipe ($\frac{\partial A}{\partial x}=0$)
- Water is incompressible
- Elasticity of the pipe is negligible ($\frac{\partial A}{\partial t}=0$)
- Viscosity of fluid is constant and only dependant on the temperature (Newton's criteria)

Combining Equation 1 and Equation 2 and considering a piece of pipe with length L gives:

$$p_2 - p_1 = -\frac{1}{gA}\frac{\partial Q}{\partial t}L - \frac{Q|Q|}{C^2A^2R}L \qquad \textit{Equation 3}$$

p_2-p_1 = Pressure difference between point 1 and 2 [mwc]
g = Gravitational constant [m/s^2]
A = Cross section of pipe [m^2]
Q = Volume flow [m^3/s]
L = Length of pipe [m]
C = Chézy coefficient [$m^{1/2}/s$]
R = Radius [m]
In stationary flows the term $\frac{\partial Q}{\partial t}$ becomes zero or negligible because flow will only change slowly over time. The term C (Chézy coefficient) is often replaced by the Darcy Weisbach coefficient λ.

$$\lambda = \frac{8g}{C^2} \qquad \textit{Equation 4}$$

When this Darcy-Weisbach constant is inserted in Equation 3 it leads to:

$$p_1 - p_2 = \lambda\frac{8L}{\pi^2 g}\frac{Q|Q|}{D^5} \qquad \textit{Equation 5}$$

This formula is usually written as:

$$\Delta H = \lambda\frac{L}{D}\frac{u^2}{2g} \qquad \textit{Equation 6}$$

The factor λ is determined by several researchers, the most used are those of Manning, Chézy and White Colebrook.

The most used formula is the formula of White Colebrook, this formula can only be iteratively solved. On network calculation programs the λ-value is calculated automatically as a function of the hydraulic circumstances expressed in the Reynolds number and the relative roughness of the pipe expressed in k/D.

$$\frac{1}{\sqrt{\lambda}} = -2\log(0{,}27\frac{k_N}{D} + \frac{2{,}5}{\mathrm{Re}\sqrt{\lambda}} \qquad \textit{Equation 7}$$

k_N = Nikuradse roughness coefficient [m]
D = Diameter of pipe [m]
Re = Reynolds number [-]

For convenience diagrams are developed, the most commonly used diagram is the Moody diagram to quickly determine the value of λ.

2.2.2 *Shear stress*

Parts of this paragraph are taken from 'Hydraulic behaviour of particles in a distribution system' (M.Sc. report, Lut;2005) and 'Zelfreinigend vermogen – invloed van de dynamiek van afnamepatronen', Van den Boomen en Van Mazijk; 2002).

A formula to calculate the shear stress in a pipe (Equation 8) can be derived from the general equation of motion.

$$\tau = -\rho \cdot R \cdot \left(\frac{\partial u}{\partial t} + g \cdot \frac{\partial \varphi}{\partial x} \right) \qquad \textit{Equation 8}$$

τ	= shear stress	[N/m²]
ρ	= density of the water	[kg/m³]
sediment mixture		[m]
R	= hydraulic radius	[m/s²]
g	= gravitational constant	[m/s]
u	= average flow velocity	[m]
φ	= piezometric level	[m/s²]
∂u/∂t	= acceleration term	[m/m]
∂φ/∂x	= velocity term	

Equation 8 shows that the shear stress depends on the pipe diameter, velocity term and the acceleration term. Because this study focuses on a stable flow, the acceleration term can be discarded ($\partial u/\partial t = 0$). Equation 8 reduces to:

$$\tau = -\rho \cdot R \cdot g \cdot \frac{\partial \varphi}{\partial x} \qquad \textit{equation 9}$$

For a turbulent, uniform and stable flow, the formula of Chézy can be used:

$$u = C\sqrt{R \cdot \left| \frac{\partial \varphi}{\partial x} \right|} \qquad \textit{equation 10}$$

With

C = Chézy coefficient [$m^{½}/s$]

The flow in a distribution system will almost always be turbulent as is more extensively described in paragraph 2.2.3. Therefore equation 9 and equation 10 can be combined to:

$$\tau = -\rho \cdot g \cdot \frac{u^2}{C^2}$$ *equation 11*

Appendix C shows the calculation for a pipe of 100 mm, a flow velocity of 0.06 m.s and a temperature of 28 ˚C. These calculations were taken from Lut (2005).

Table 1 shows some calculations of different shear stresses.

Table 1: Shear stress and Reynolds numbers for different flow velocities

Flow velocity [m/s]	Shear stress [N/m²]	Reynolds number
0.06	0.015	7142
0.14	0.066	16667
0.25	0.184	29761

2.2.3 *Turbulence*

Turbulence is a phenomenon that describes the state of the flow of water. Flow that is flowing smoothly past an obstacle is called laminar; turbulent flow is much more wild. If the flow is laminar, water flows through the pipe in nice even layers: molecules of water stay in the same layer of water as they continue on their path. When the flow is turbulent the nice laminar layers of fluid get mixed up and the molecules of the fluid no longer flow along nicely but rather bounce all over throughout the flow. In pipe flow the state of the flow is important as will be shown in the rest of this report.

Turbulence is represented by the Reynolds number. Water flowing through a pipe is called laminar when the Reynolds number is below 2000 and turbulent when the Reynolds number is larger than 4000. In between there is a transitional flow regime (2000 < Re < 4000).

$$\mathrm{Re} = \frac{u * D}{\upsilon}$$ *Equation 12*

u = Flow velocity [m/s]
Re = Reynolds number [-]
D = Diameter [m]
υ = Kinematic viscosity (1.31 $*10^{-6}$ at 10 ˚C) [m^2/s]

As can be seen, the Reynolds number is depended on the flow velocity, the diameter and the kinematic viscosity. The viscosity is temperature depended, the Reynolds number will increase when the temperature increases. The difference of the Reynolds number at 10 ˚C and at 30 ˚C is 62%: this is a difference that cannot be neglected, because the temperature changes in a distribution network can vary between 0 and 22 ˚C. Paragraph 2.5.4 shows the differences in viscosity for different temperatures.

Water flowing in drinking water pipes is usually turbulent as can be seen in the following calculations.

Table 2: Examples of turbulence calculations

D [mm]	u [m/s]	υ [m2/s]	Re [-]	υ [m2/s]	Re [-]
		T= 0˚C	T= 0˚C	T= 22˚C	T= 22˚C
63	0,05	1,79E-06	1760	9,59E-07	3285
63	0,1	1,79E-06	3520	9,59E-07	6569
100	0,05	1,79E-06	2793	9,59E-07	5214
100	0,1	1,79E-06	5587	9,59E-07	10428
200	0,05	1,79E-06	5587	9,59E-07	10428
200	0,1	1,79E-06	11173	9,59E-07	20855
400	0,05	1,79E-06	11173	9,59E-07	20855
400	0,1	1,79E-06	22346	9,59E-07	41710

Turbulence is lower close to the pipe wall, so particles could become trapped in a laminar sub layer, as can be seen in Figure 2.1. Very small particles that are driven to the pipe wall by turbulent forces are caught inside the laminar sub layer and can be held there because of electrostatic forces. The friction of the wall is influenced by the Reynolds number and the relative roughness of the wall (k/D). When the flow is laminar (Re<2000) particles that were previously suspended will not resuspend as they need turbulence to resuspend.
Particles will resuspend when the flow is turbulent because the profile of the flow velocity in a pipe is different for laminar or turbulent flow. The velocity in turbulent flows is changing little over pipe diameter compared to laminar flow. The turbulence is causing lateral mixing of the fluid, leading to flows that reach the pipe wall.

When fluid is flowing through a pipe the pressure in the flow in the centre of the pipe is constant. Close to the pipe wall a relative small zone, a so called boundary layer is formed where the velocity varies between u = 0 near the wall to u = U in the outer flow. Because of the wall resistance the velocity of the fluid particles close to the wall decreases in the flow direction. These particles influence the velocity of the fluid at a larger distance.

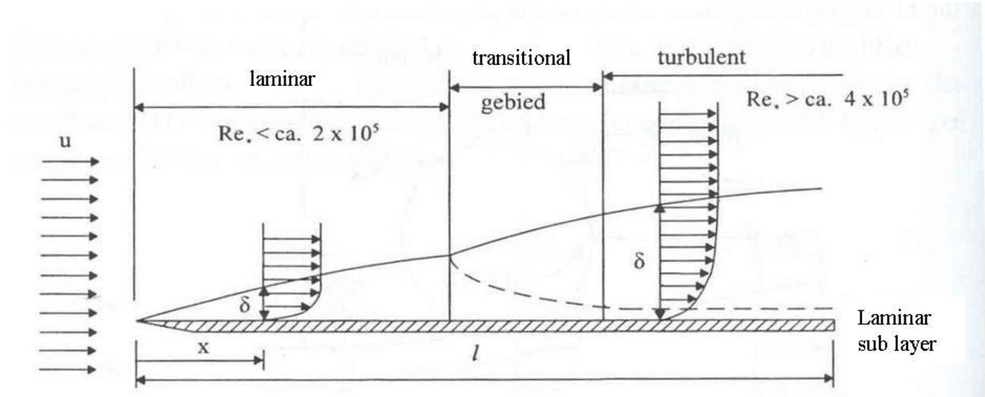


Figure 2-1: Boundary layer close to the pipe wall

As we have seen water flowing in drinking water networks is usually turbulent.

When water is flowing past a sphere the definition is slightly different. The Reynolds number are then expressed as Re_p numbers. The difference is that the velocity is the relative velocity, the velocity difference between the fall velocity and the velocity of the water flowing past the particle.

$Re_p<0.6$	laminar flow regime
$0.6<Re_p<600$	transitional flow regime
$Re_p>600$	turbulent flow regime

$$Re_p = \frac{v_{rel} * D}{\upsilon}$$ *Equation 13*

Re_p = Reynolds number for particle [-]
v_{rel} = relative velocity = $v_{setlle} - v_{water\ past\ particle}$ [m/s]
υ = kinematic viscosity [m²/s]

2.3 Water quality modeling

The main problem with modelling water quality is the conversion of a steady state model to the time series used for water quality calculation. The water quality simulators are based on the Langrangian time-based approach to track the fate of discrete parcels of water (based on EPAnet manual; Rossman, 2000) as they move along pipes and mix together at junctions between fixed-length time steps (Liou and Kroon, 1987). These water quality time steps are typically much shorter than the hydraulic time step (e.g., minutes rather than hours) to accommodate the short times of travel that can occur within pipes. As time progresses, the size of the most upstream segment in a pipe increases as water enters the pipe while an equal loss in size of the most downstream segment occurs as water leaves the link. The size of the segments in between these remains unchanged. (See Figure 2-2).

The following steps occur at the end of each such time step:

1. The water quality in each segment is updated to reflect any reaction that may have occurred over the time step.

2. The water from the segments of pipes before with flow into each junction is blended together to compute a new water quality value at the junction. The volume contributed from each segment equals the product of its pipe's flow rate and the time step. If this volume exceeds that of the segment then the segment is destroyed and the next one in line behind it begins to contribute its volume.

3. Contributions from outside sources are added to the quality values at the junctions. The quality in storage tanks is updated depending on the method used to model mixing in the tank (see below).

4. New segments are created in pipes with flow out of each junction, reservoir, and tank. The segment volume equals the product of the pipe flow and the time step. The segment's water quality equals the new quality value computed for the node.

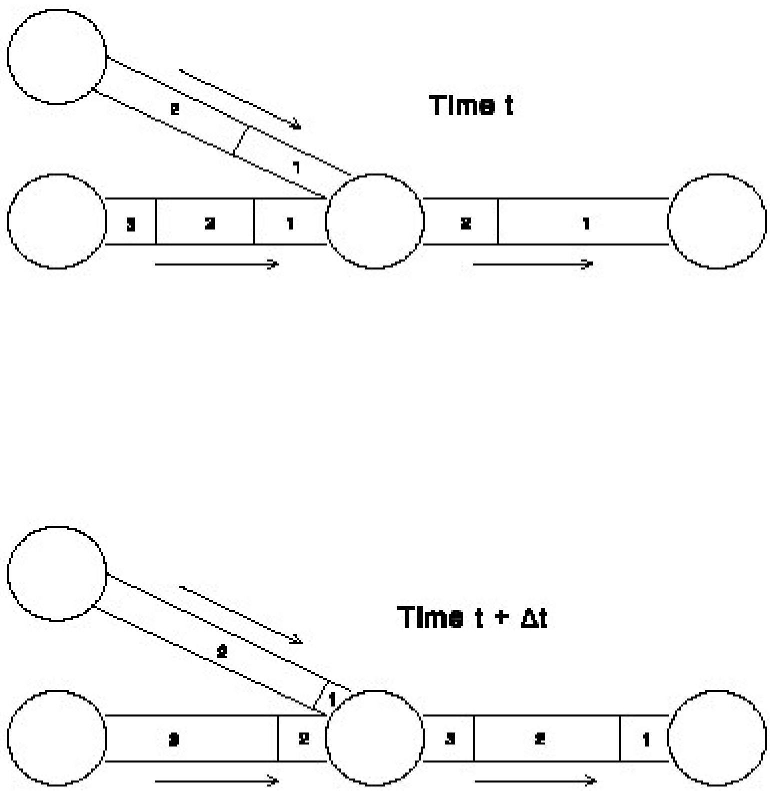


Figure 2-2: Behaviour of Segments in the Lagrangian Solution Method

To cut down on the number of segments, Step 4 is only carried out if the new node quality differs by a user-specified tolerance from that of the last segment in the outflow pipe. If the difference in quality is below the tolerance then the size of the current last segment in the outflow pipe is simply increased by the volume flowing into the pipe over the time step. This process is then repeated for the next water-quality time step. At the start of the next hydraulic time step the order of segments in any links that experience a flow reversal is switched. Initially each pipe in the network consists of a single segment whose quality equals the initial quality assigned to the upstream node.

2.4 Mass Balance theory

Many processes are involved in the formation of sediment in drinking water pipes, as shown in the introduction the framework of this research is the mass balance, see figure 2-3. The processes are:

- Deposition and resuspension
- Particles and substances from treatment plant
- Formation and coagulation
- Biofilm
- Corrosion

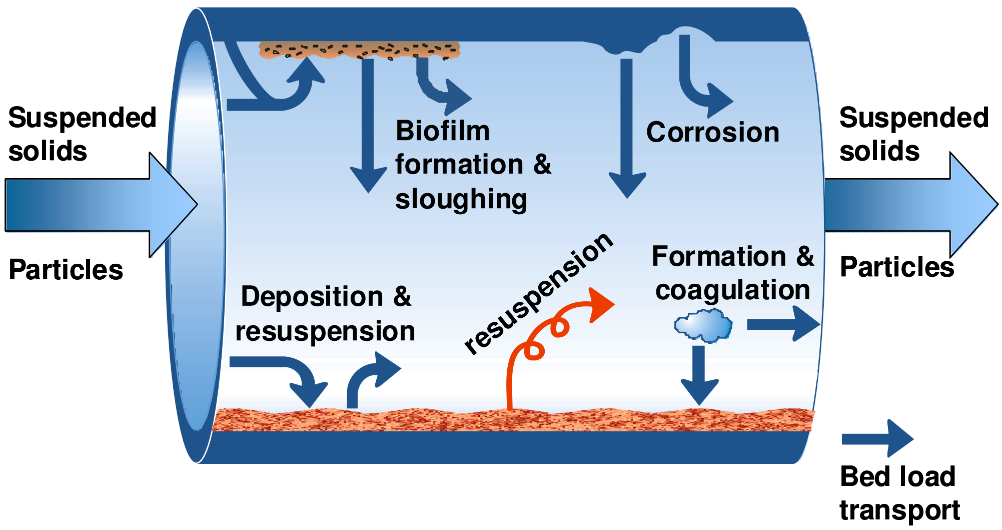


Figure 2-3: Mass balance of pipe in a drinking water network

The mass balance theory is based on the law of conservation of mass. This law means that whatever particles enter the system or are formed inside the system will either leave or stay inside the system.

$$\Delta \text{Mass} = \text{Mass}_{\text{in}} - \text{Mass}_{\text{out}} + \text{Mass}_{\text{production}} \qquad \textit{Equation 14}$$

$Mass_{in}$ = Particles from treatment plant, processes upstream, corrosion. biofilm, mixing and coagulation
$Mass_{out}$ = Supplied to customers or removed during cleaning/ flushing of pipes.
$Mass_{production}$ = Particles produced inside system

2.5 Deposition

2.5.1 *Introduction*

Deposition is the phenomenon that particles settle under influence of the gravity force. In water, the rate at which a particle settles is a function of both grain and fluid properties (i.e. grain size, density and shape for the particle; density, viscosity, flow rate and turbulence for the water). In water that is not flowing fast, the settling rate of fine-grained sediment is approximated by Stokes' Law. This equation is based on gravity settling and assumes that particles with a larger density will settle faster than particles with a smaller density. For different flow regime Stokes' law is not suitable, as we will demonstrate in the following chapters.

2.5.2 *Settling according to Stokes' law*

Assume that the particles present in water are spheres, with diameter D and specific weight ρ_s. If the sphere falls a distance *x* it displaces a cylindrical volume of water, the kinetic energy required to displace this volume of water is given by:

$$E = \frac{1}{2} m v^2 \qquad \textit{Equation 15}$$

E = kinetic energy of particle [kg*m/s²]
m = mass of particle [kg]
v = fall velocity of particle [m/s]

The mass of water that is displaced is given by:

$$m = \frac{\pi D^2}{4} * \rho * x \qquad \textit{Equation 16}$$

D = diameter of particle [m]
ρ = specific weight of water [kg/m³]
x = distance that particle is displaced [m]

Substituting Equation 15 into 16:

$$E = \frac{1}{2}\frac{\pi D^2}{4} * \rho * v^2 = \frac{\pi D^2}{8} * \rho * v^2$$ *Equation 17*

Energy can be defined in terms of work: *E = F*x;* Energy = drag force * distance, therefore the drag force can be written as:

$$\frac{\pi D^2}{8} * \rho * v^2 * x = F_d * x \Rightarrow F_d = \frac{\pi D^2 \rho v^2}{8}$$ *Equation 18*

F_d = drag force of particle [N]
D = diameter of particle [m]
ρ = specific weight of water [kg/m^3]
v = velocity [m/s]

F_d is the so-called theoretical drag force, the viscosity of the fluid or the dynamics of the flow around the sphere are not taken into account in this equation. Therefore the drag coefficient C_d is added.

$$F_d = C_d \frac{\pi D^2}{8} * \rho * v^2$$ *Equation 19*

C_d= drag coefficient [-]
F_d = drag force of particle [N]
D = diameter of particle [m]
ρ = specific weight of water [kg/m^3]

C_d has been found to be a function of the Reynolds Number of flow past rigid obstacles. The following diagram illustrates the correlation between Reynold's Number and drag coefficient C_d for perfectly spherical bodies, other types of particle have their own special charts .

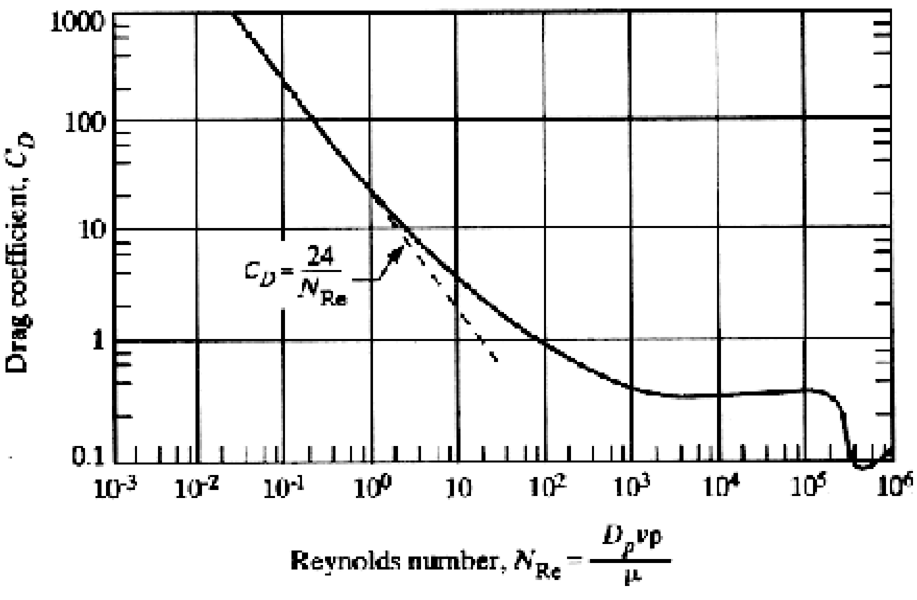


Figure 2-4: Relation Reynolds number Re and drag coefficient C_d

(source:http://www.rpi.edu/dept/chem-eng/Biotech-Environ/SEDIMENT/sedsettle.html)

In the diagram three flow areas can be defined:

$Re_p<0.6$	laminar flow regime, $C_D=24/Re$
$0.6<Re_p<600$	transitional flow regime, $C_D= 18{,}5/Re^{0.6}$
$Re_p>600$	turbulent flow regime, $C_D = 0{,}44$

These Reynolds numbers represent the state of flow when it is flowing past a rigid obstacle. These values differ from the Reynolds numbers described in paragraph 2.3. The Re_p number is found with the help of Equation 20 .

$$Re_p = \frac{v_{rel} * D}{\upsilon} \qquad \textit{Equation 20}$$

Re_p = Reynolds number for particle [-]
v_{rel} = relative velocity = $v_{setlle} - v_{water\ past\ particle}$ [m/s]
υ = kinematic viscosity [m^2/s]

Chapter 6.3 shows an analysis of the water from treatment plant 'De Laak': the particle sizes of sediment found in this water are representative for normal drinking water and are ranging from 1 to 100 µm. Table 3 shows that the flow conditions for these small sediments are usually laminar.

Table 3: Re_p numbers for different particle sizes, at 10 ˚C

Particle size		**Relative velocity**		**Kinematic viscosity**		**Re_p**
			at:	10	˚ C	
µm		m/s		m²/s		-
1		0.01		1.30652E-06		0.007654
2		0.01		1.30652E-06		0.015308
5		0.01		1.30652E-06		0.038269
10		0.01		1.30652E-06		0.076539
20		0.01		1.30652E-06		0.153078
25		0.01		1.30652E-06		0.191347
50		0.01		1.30652E-06		0.382695
100		0.01		1.30652E-06		0.76539

Figure 2-5 shows the forces that are working on the particle.

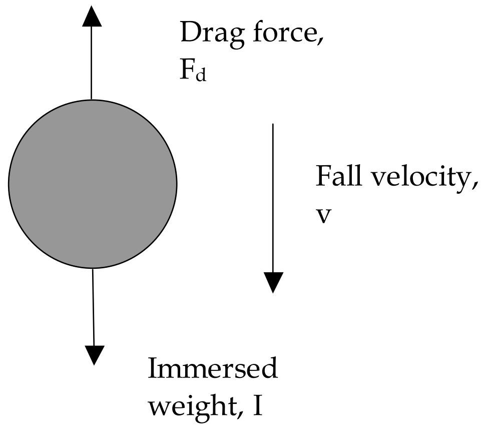


Figure 2-5: Forces on a sphere falling in water

$$I = W - U$$ *Equation 21*

I = immersed weight [N]
W = weight of the particle [N]
U = fluid upthrust according to Archimedes [N]

$$W = m * \rho_s * g = \frac{4}{3}\pi\frac{D^3}{8} * \rho_s * g$$ *Equation 22*

W = weight of the particle [N]
m = mass of particle [kg]
ρ_s = specific weight of particle [kg/m³]
g = gravity constant = 9,81 [m/s²]

The upthrust (buoyant force) is equal to the weight of water that the particle displaces:

$$U = \frac{4}{3}\pi\frac{D^3}{8} * \rho * g \qquad \textit{Equation 23}$$

U = upthrust force [N]
D = diameter of particle [m]
ρ = specific weight of water [kg/m³]
g = gravitational constant [m/s²]

Combining of Equations 21, 22 and 23 gives the immersed weight of the particle:

$$I = \frac{4}{3}\pi\frac{D^3}{8} g(\rho_s - \rho) \qquad \textit{Equation 24}$$

Stokes' law has been derived for laminar flow conditions. The relation has been empirically determined and can be seen in Figure 2.5. The relation between the drag coefficient and Reynolds number at laminar flow conditions is:

$$C_d = \frac{24}{\mathrm{Re}} \qquad \textit{Equation 25}$$

The Reynolds number is rewritten as:

$$R_e = \frac{v * D}{\upsilon}$$

with

$$\upsilon = \frac{\mu}{\rho}$$

$$R_e = \frac{\rho * v * D}{\mu}$$

Substituting Equation 25 into Equation 18 gives the drag force F_d:

$$F_d = \frac{3\pi D^2 \rho v^2}{\mathrm{Re}} = 3\pi\mu D v \qquad \textit{Equation 26}$$

F_d = drag force [N]
μ = dynamic fluid viscosity [Ns/m²]

D = particle diameter [m]

Within the regime where the flow is laminar the fall velocity of the particle is found by combining Equation 24 and 26:

$$F = I \Rightarrow \frac{4}{3}\pi\frac{D^3}{8}g(\rho_s - \rho) = 3\pi\mu D v \Rightarrow v = \frac{D^2 g(\rho_s - \rho)}{18\mu}$$ *Equation 27*

When $D = 2*r$ is substituted this results in Stokes' equation as it is usually written.

$$v = \frac{2r^2(\rho_s - \rho)g}{9\mu}$$ *Equation 28*

v = settling velocity [m/s]
r = radius of particle [m]
ρ_s = density particle [kg/m^3]
ρ = density water[kg/m^3]
g = gravitational constant [m/s^2]
μ = dynamic fluid viscosity [Ns/m^2]

Table 4: Stokes'velocities for different diameters

particle diameter	**v Stokes**
[µm]	[m/s]
1	1,16E-07
5	2,91E-06
10	1,16E-05
15	2,62E-05
20	4,66E-05
25	7,28E-05
30	1,05E-04
35	1,43E-04
40	1,86E-04
45	2,36E-04
50	2,91E-04
55	3,52E-04
60	4,19E-04
65	4,92E-04
70	5,71E-04
75	6,55E-04
80	7,46E-04
85	8,42E-04
90	9,44E-04
95	1,05E-03
100	1,16E-03

2.5.3 *Settling at turbulent flow*

When water is flowing at turbulent Re_p numbers the drag coefficient is calculated with: $C_d = 0.44$. To reach these high Re_p numbers the relative velocity has to increase, as is shown in Table 5. These high velocities are not likely to occur, this description is to show what relative velocities will be needed to reach turbulent flow past the particle.

Table 5: Turbulent velocities

Particle size		**Relative velocity**		**Kinematic viscosity**		**Re_p**
			at:	10	˚C	
µm		m/s		m^2/s		-
1		783.91		1.30652E-06	10	600
2		391.96		1.30652E-06	10	600
5		156.78		1.30652E-06	10	600
10		78.39		1.30652E-06	10	600
20		39.20		1.30652E-06	10	600
25		31.36		1.30652E-06	10	600
50		15.68		1.30652E-06	10	600
100		7.84		1.30652E-06	10	600

2.5.4 *Viscosity and temperature*

The viscosity of a fluid is a number that describes the state of the fluid: the lower the viscosity the more viscous the fluid is. The dynamic viscosity relates to the kinematic viscosity according to the following equation:

$$\upsilon = \frac{\mu}{\rho}$$ *Equation 29*

υ = kinematic viscosity [m^2/s]
μ = dynamic viscosity [Ns/m^2]
ρ = specific weight [kg/m^3]

Because the kinematic viscosity is temperature dependant there is an influence on the settling velocity of particles (according to Stokes' Law). The dependency of the kinematic viscosity and the temperature is given in Equation 30.

$$\upsilon = \frac{497 * 10^{-6}}{(T + 42.5)^{1.5}}$$ *Equation 30*

υ = kinematic viscosity [m^2/s]

T = temperature [K]

Figure 2-6 shows the change of settling velocities of particles with a given particle size and density in water with a fixed density for different temperatures (and changing dynamic viscosities).
Data:

- radius particle: 10 µm
- density particle: 1280 kg/m^3
- density water: 1000 kg/m^3

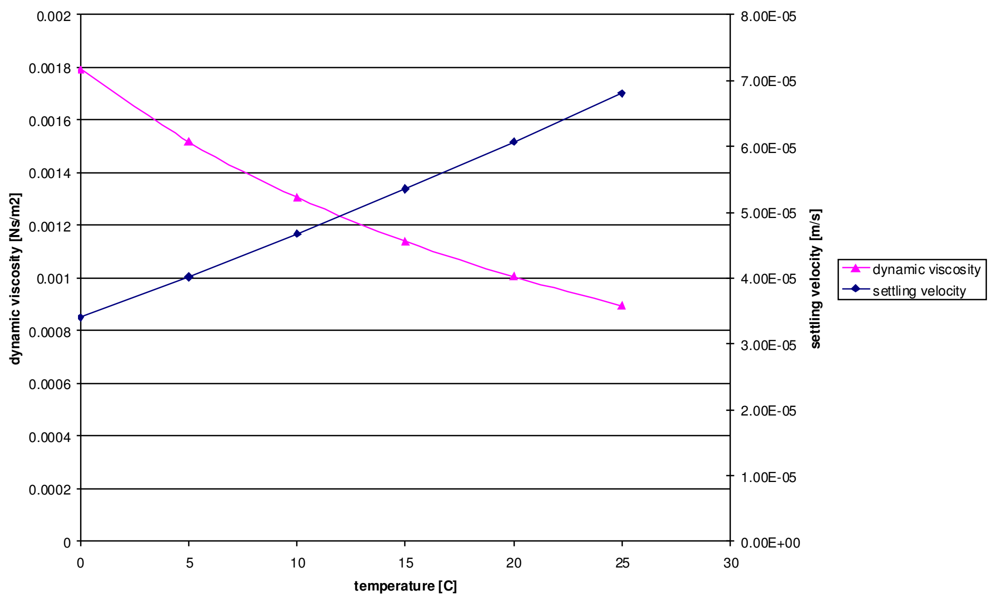


Figure 2-6: Settling velocities and dynamic viscosities at different temperatures

The settling velocities at 0 ˚C and at 25 ˚C are respectively $1.94*10^{-4}$ and $3.89*10^{-4}$ m/s, this is a difference in settling velocity of 200%. This shows that the influence of the temperature on settling in laminar conditions (Stokes) is very large.

2.5.5 *Other settling formulas*

Several other formulas besides the theory of Stokes are used in some commercial sediment transport models (shortly described in chapter 3), these are usually derived for use in open channel flow and are derived for sandy sediments. Because of this, these formulas are not directly applicable to be used for the type of sediment in drinking water networks.

Ackers-White formulas

The Ackers-White formulas are used to determine the transport of sediment and are usually used for dredging purposes. At each computational point along each pipe, a non-dimensional carrying capacity, C_v is calculated that represents the maximum concentration of a given sediment fraction that can be held within the flow. The equation used to calculate C_v is:

$$C_v = J\left(\frac{W_e R}{A}\right)^{\alpha}\left(\frac{d_{50}}{R}\right)^{\beta}\lambda_c^{\gamma}\left\{\frac{|u|}{\sqrt{g(s-1)R}} - K\lambda_c^{\gamma}\left(\frac{d_{50}}{R}\right)^{\varepsilon}\right\}^{m}$$

λ_c = composite friction factor which is calculated using the Colebrook-White formula (as described in Ackers J.C. et al (1994))
R = hydraulic radius (=A/Po) [m]
P_o = wetted perimeter [m]
W_e = effective bed width [m]
A = cross sectional area of the flow [m^2]
U = local flow velocity [m/s]
J = parameter
K = parameter
α, β, γ, E = coefficients

The remaining parameters are all functions of the dimensionless grain size:

$$D_{gr} = d_{50}\left(\frac{g(s-1)}{\nu^2}\right)^{1/3}$$

υ = kinematic viscosity of water [m^2/s]
g = acceleration due to gravity [m/s2]
s = specific gravity of the sediment fraction [-]
d_{50} = average sediment particle size [m]

The non-dimensional carrying capacity number is converted to a maximum concentration by:

$$C_{max} = C_v \rho s$$

If the actual concentration is greater than C_{max} then the excess sediment is deposited. If the actual concentration is less than C_{max} the bed is eroded until either C_{max} = C_{actual} or all the bed has been eroded. Erosion is assumed to occur instantaneously while the rate of deposition is a function of the sediment settling velocity.All flow concentrations and bed masses are updated before the sediment is advected at the next time step.

This formula is derived for transport of sand and/or gravel like sediment in open water flows. The size of sediment on which the formula is based are particles in the range of 45 to 200 µm and a density of 200 to 400 kg/m^3. The sizes of particles coming from a treatment plant and in distribution networks are usually ranging from 1 to 100 µm and a density that is smaller than 2000 kg/m^3. This is why the use of Ackers-White formulae is not very representative for settling of sediment in this case.

2.5.6 Conclusion deposition

The theory of settling of Stokes is difficult to apply to use for settling of sediment in drinking water. The settling takes place at laminar flow conditions of the particle (Re_p<0.6). The water layers of the flow of water through a pipe are perpendicular to the fall direction of the particle in an ideal situation, a schematic drawing can be seen in Figure 2-7 .

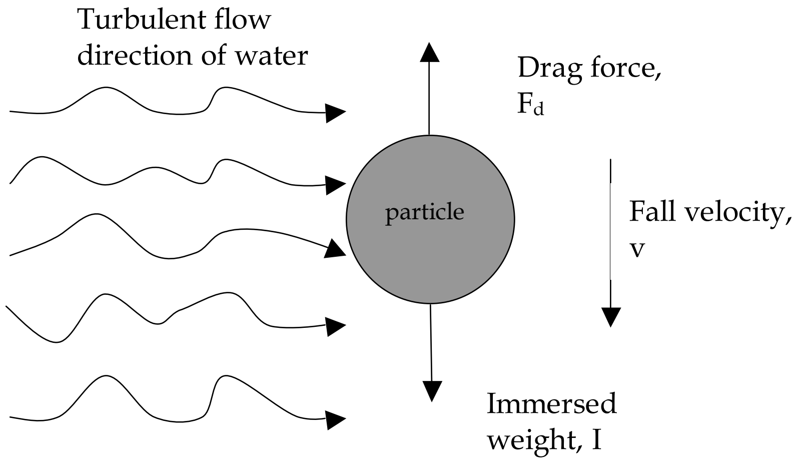


Figure 2-7: Flow direction of water and fall direction of particle

In this situation the drag force F_d is influenced by the flow of the water, creating lift on the particle. This lift can lead to particles settling faster of slower because of the practiced drag force of the flow.

Also the temperatures that can occur inside networks range from 0 to 22 ˚C (Vreeburg; 2005), this means that the viscosity of the fluid is influenced. This viscosity directly influences the settling velocity at laminar flow conditions, a difference of settling speed of 200 % between 0 and 25 ˚C is possible. If these temperature differences are not considered the settling of sediment can be totally different.

2.6 Resuspension

2.6.1 Introduction

Resuspension is the phenomenon that particles collected in drinking water pipes are resuspended due to hydraulic changes. Resuspension will occur when forces caused by the flow of a fluid are larger than the forces of the own weight of a particle captured inside a sediment bed under water (Vrijling et. al, 2001). Grains forming the boundary between a fluid and a sediment possess a finite weight and finite coefficient of friction. When the applied shear stress is low they are not brought into motion. As applied shear stress is increased, a critical shear stress is reached at which grains will begin to move. The value of the critical stress will depend primarily on the size and density of the particles and secondarily on their shape and packing and the cohesive forces acting between particles.

Once the critical stress is exceeded, particles will advance in the direction of flow due to irregular jumps or less commonly rolls. This mode of transport is termed the bedload, that is the behavior of a particle once in motion is dominated by the gravity force. As the stress is further increased, particles will also begin to be suspended in solution and subject to turbulent forces; this mode of transport is termed the suspended load. Due to these two modes of transport there will be a flux of material across a plane perpendicular to the flow.

2.6.2 Resuspension formulas

For the calculation of resuspension of particles not so many theories are available, most of the studies on the start of sediment transport are performed on open channel flow and large sized particles with, compared to drinking water sediment, high densities. Hjulström (1935; from Graf et al.; 1970)) has performed an extensive analysis on mono-disperse material on a bed of loose material of the same size of particles. He concludes that the average flow velocity cannot be used as the relation to the bottom velocity is seldom available, that fine sands are easiest to erode and that the great resistance to erosion in the smallest particle range must depend on the cohesion and adhesion forces. Where applicable the Hjulström diagram (figure 2-8) might serve as a useful concept. For the determination of v (critical velocity) a lot of (experimental) theories are described, these are not further mentioned here.

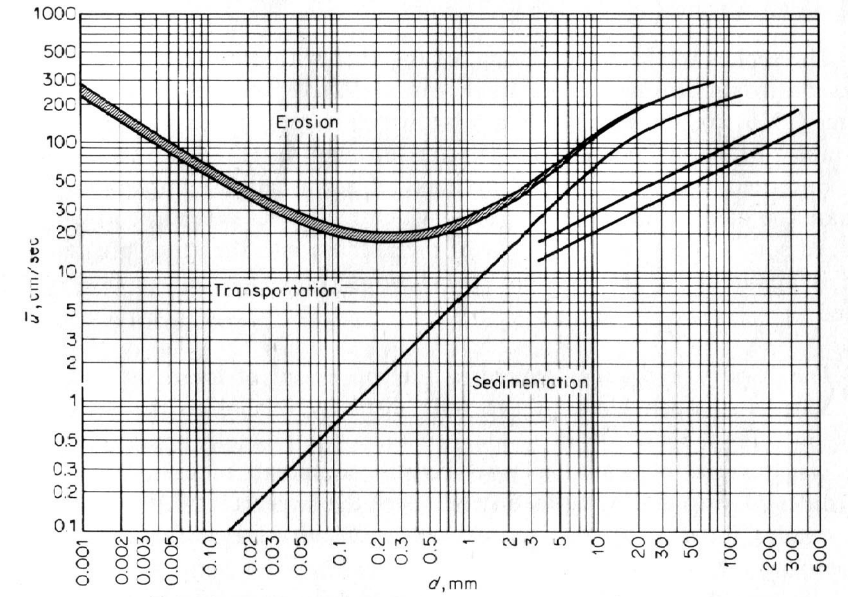


Figure 2-8: Hjulström diagram

Other authors that have tried to construct theories and equations to describe when particles are in a state of sedimentation or resuspension are: Shields, Berlamont and Migniot (Mazijk (2000)). These authors have conducted experiments under different circumstances: Shields and Migniot equations were constructed for open waters and Berlamont for conduits. Shields and Berlamont used non-cohesive material, while Migniot used freshly deposited colloidal cohesive material. For drinking water practice the pipes are obviously closed. Whether sediment is cohesive or non-cohesive is unknown since this has not been subject of any study.

The Shields theory is a theory that is extensively described in literature and is used for the calculation of resuspension of particles. The theory is not really applicable to drinking water sediment, because is it derived for sandy to gravel like sediment and may not be used for fine sediments. Because the different forces on the sediment bed are difficult to identify individually, these forces are combined into one variable: the hydraulic shear stress. Shields calculations are performed for a given particle diameter and density; the critical hydraulic shear stress is calculated with an iterative calculation.

Shields:

$$u_{cr} = C\sqrt{\alpha_{shields} \cdot \Delta \cdot d} \text{ , with :}$$

$$\alpha_{shields} = \frac{{u_{*cr}}^2}{\Delta \cdot g \cdot d} \qquad \textit{Equations 31}$$

$$u_{*cr} = \frac{u_{cr}\sqrt{g}}{C} = \sqrt{\frac{\tau}{\rho_w}}$$

(at a $\alpha_{Shields}$ value of 0,02 - 0,03 sand is just in motion)

In these the formulas the parameters are different for each theory.

Migniot:

$$u_{*cr} = 0{,}018\tau_b^{\ 0,25} \qquad \tau_\mathrm{b} < 1{,}5 N/m^2$$
$$u_{*cr} = 0{,}016\tau_b^{\ 0,25} \qquad \tau_\mathrm{b} > 1{,}5 N/m^2$$

Equation 32

Berlamont

$$u_{*cr} = \sqrt{\alpha_{berlamont} \cdot g \cdot \Delta \cdot d}$$
$$\alpha_{berlamont} = 0{,}8 \text{ to resuspend particles}$$
$$\alpha_{berlamont} = 0{,}04 \text{ to keep particles in motion}$$

Equations 33

u_{cr} = critical velocity [m/s]
u_{*cr} = shear stress velocity [m/s]
g = gravitational force [m/s2]
C = Nikuradse factor [-]
α = dimensionless constant [-]
Δ = specific weight factor [-]
d = diameter of the sediment [m]
τ_b = bottom shear stress [N/m^2]

The application of these resuspension formulas to small particles is difficult. Guo (2002) presented additional equations to be used in combination with the Shields equations for particles with a dimensionless sediment diameter. Julien (1998) described additional equations that are related to the angle of repose, because of inaccuracies found using the equations of Guo for a dimensionless sediment diameter less than 1; the angle of repose is the maximum slope on which an object will not slide.

These additional equations can be applied to sand particles smaller than 50 µm, flocks, and organic matter with an approximate specific gravity of 1300kg/m^3 with a diameter smaller than 80-90 µm. The determining of the angle of repose of the sediment from a drinking water main is difficult.

Taking into account the circumstances of the drinking water practice, Berlamont (conduit, non-cohesive) is the closest candidates for the application in modeling particle behavior and Migniot could be possible of help in case sediment show more cohesive characteristics (for example in case the organic fraction of the sediment at a certain location is high).

A certain difference exists between the experiments of Migniot and Berlamont. One would expect that Migniot, while using the same parameters, would require a higher velocity for particles to resuspend relative to Berlamont because of the cohesivity of the particles. Calculations however show the opposite. This could be explained by the fact that Migniot used material with a low specific mass, while Berlamont used material with a higher specific mass.

2.6.3 *Conclusion resuspension*

As shown there are several theories for the description of the start of movement of sediment. The basis of the calculations is the Shields theory, which was derived for larger and heavier (sand like) particles. The extra equations and parameters that are mentioned (Migniot, Berlamont, Guo and Julien) are to make the Shields theory applicable to finer materials. Sediment in drinking water is different than the sandy or gravel like sediments that are used for the Shields theory. Particles tend to have a wide range in grain sizes and densities, so these extra equations look very promising for the use of resuspension of finer sediments, but because of the large property differences of the sediment the Shields theory and extra equations are difficult to predict the amount of particles that will resuspend . Also a sediment bed is difficult to define with fine sediments and the cohesive forces of the bed will have influence on the resuspension of particles. This means that the particles are closely packed and are not easily resuspended. This phenomenon has been noticed in a study of particles in a test-rig (M.Sc. report; Lut, 2005), in this study settlement of sediment was studied. After settling of the sediment it was tried to resuspend the particles that were previously settled. Figure 2-9 shows the sediment bed in the bottom of the pipe at different time intervals during which the flow was increased to clean the pipe. It can be seen that the bed itself is very slowly eroded by the increased flow velocity. Possible cohesive forces keep the particles closely packed.

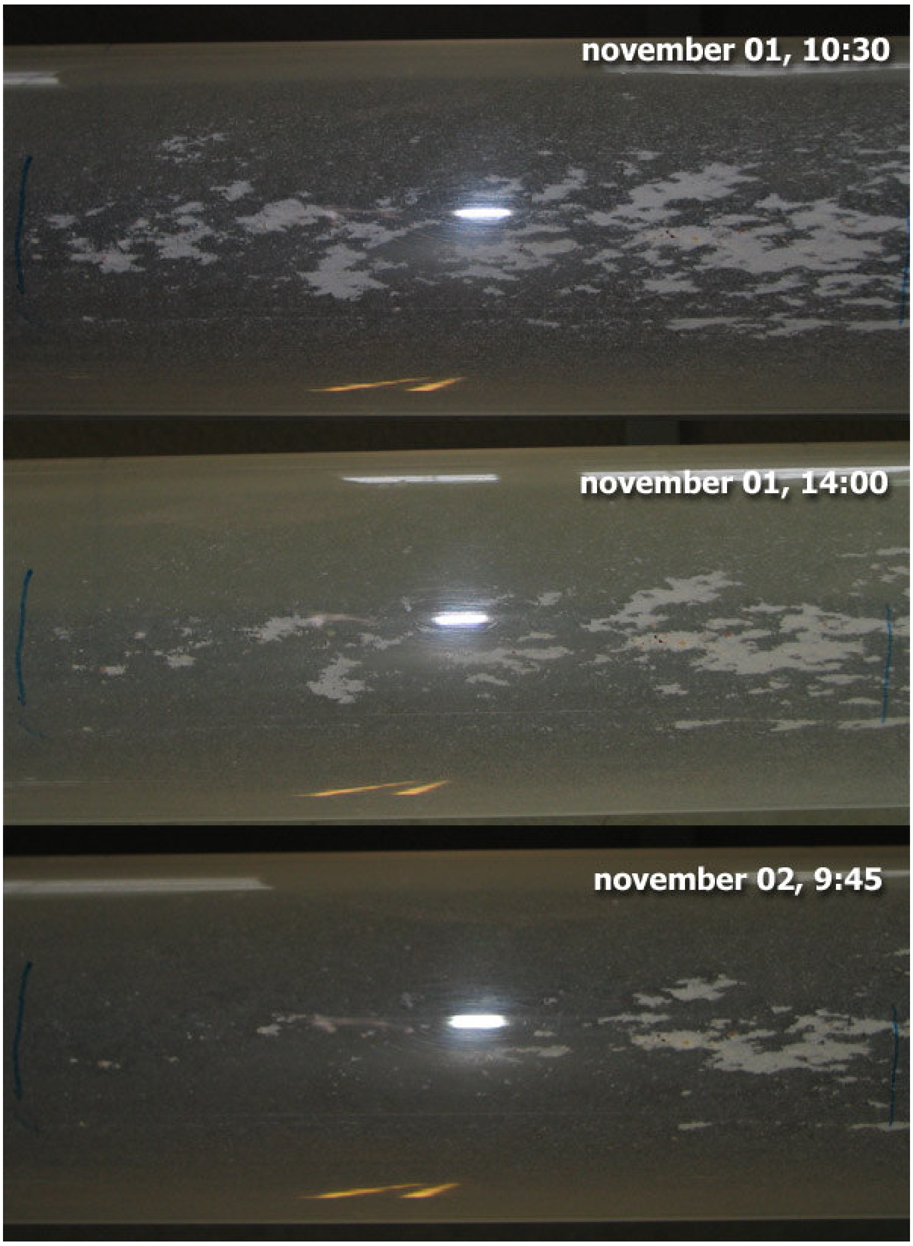


Figure 2-9: Remobilisation of sediment in test rig

2.7 Other processes

In chapter 2.2 the different processes, besides deposition and resuspension of particles, have shortly been named; in this chapter these processes are more extensively described.

2.7.1 Formation and coagulation

Coagulation can occur in drinking water due to chemical processes originating from the treatment plant that are not in equilibrium yet. Different types of water (coming from different pumping station) can also be a cause of coagulation, because the characteristics of these water are different.

2.7.2 Biofilm

A lot of research has been done to the growth and origin of biofilm (Boe Hansen et al; 2003 and Van de Kooi et al; 1995). Van der Kooij for instance suggests that iron and manganese are entrained in the biofilm, leading to a smaller iron and manganese concentration in the water. Biofilm is a small layer of organic material that is formed in drinking water pipes. Biofilm grows on the pipe walls, nutrients that are present in the water are converted into biomass.. One of the uncertainties of biomass growth is that the detachment of biomass to suspended matter is not equally to each other because a part of the biofilm can be lost during respiration.

The amount of detached biofilm can be monitored with the help of the Biomass Production Potential (BPP) developed by Van der Kooij et.al. The BPP can determine the concentration of active biomass on pipe walls. Combined with another research performed by Boe Hansen (2002), which was about the detachment rate of biomass, a conclusion can be drawn on the influence of detached biomass to the total mass load. To be able to measure the biofilm growth rate Van der Kooij et al. have developed a standard test, the Biofilm Formation Potential (BFP). The BFP value indicates the average value of the sum of the concentration of attached biomass after 8, 12 and 16 weeks incubation at 25 ˚C. Typical BPP values are in the range of 40 to 7000 pg ATP/ cm^2. Boe-Hansen et al. have studied the colonization and detachment rate at a circular pilot plant fed with normal drinking water which was considered biological stable according to the criterion suggested by Van der Kooij (1995), 10 pg ATP/ cm^2.

2.7.3 Corrosion

Corrosion is another internal growth process that contributes to the total mass of sediment in drinking water networks. Corrosion is the release of pipe wall material caused by chemical reactions. Two problems are related with corrosion of pipes. Firstly the loss of pipe material due to corrosion leads to a loss of strength of pipes. Secondly the release of iron corrosion by-products leads to a contribution to sediment and possible discolored water events during resuspension of sediment.
There are two parameters that influence the corrosion rate:

1. Water quality parameters such as pH, alkalinity, Saturation Index (SI) buffer capacity and the concentration of certain elements.
2. Network characteristics, like retention time, pipe material and state and flow velocity.

A number of methods are available to determine the corrosion potential of water.

- Langelier Saturation Index (LSI)

  The Langelier or Saturation Index is a very common used index based on the calculation the water quality parameters pH and HCO_3^-. This method gives an indication of the saturation of water with respect to Calcium Carbonate ($CaCo_3$). An LSI number of 0 means that the water is in equilibrium and $CaCo_3$ will not dissolve or lead to scaling of the pipe wall. A number of LSI larger than 0 (positive) means that scaling of $CaCo_3$ will occur and a number smaller than 0 (negative) means that the water will dissolve Calcium Carbonate. Because of the buffer capacity of drinking water a certain concentration of particles of HCO_3^- is necessary. The norm in the Netherlands for HCO_3^- is a concentration of 2.0 mmol/l and an LSI number of 0.
- Corrosion Potential

  A guideline for the use of the Corrosion Potential is based on the work of Larson and Skold and also published in a Kiwa report (Van den Hoven, Van Eekeren; 1988). The Langelier index is calculated by the water quality parameters SO_4^{2-}, Cl^- an HCO_3^-. A Langelier index number larger than 1 means that the water is aggressive and that it has a certain corrosion potential. When the index is smaller than 1, the risk of corrosion can be neglected. The calculation of the index is done with the help of Equation 34.

$$\frac{[Cl^-] + 2[SO_4^{2-}}{[HCO_3^-]} < 1$$

Equation 34

Figure 2-10 shows a picture of continued measuring of the turbidity in a distribution network. A pattern can be seen, thus identifying the extra contribution to sediment caused by corrosion. At night when stagnant water occurs, the (cast-iron) pipes corrode leading to a deterioration of the water quality.

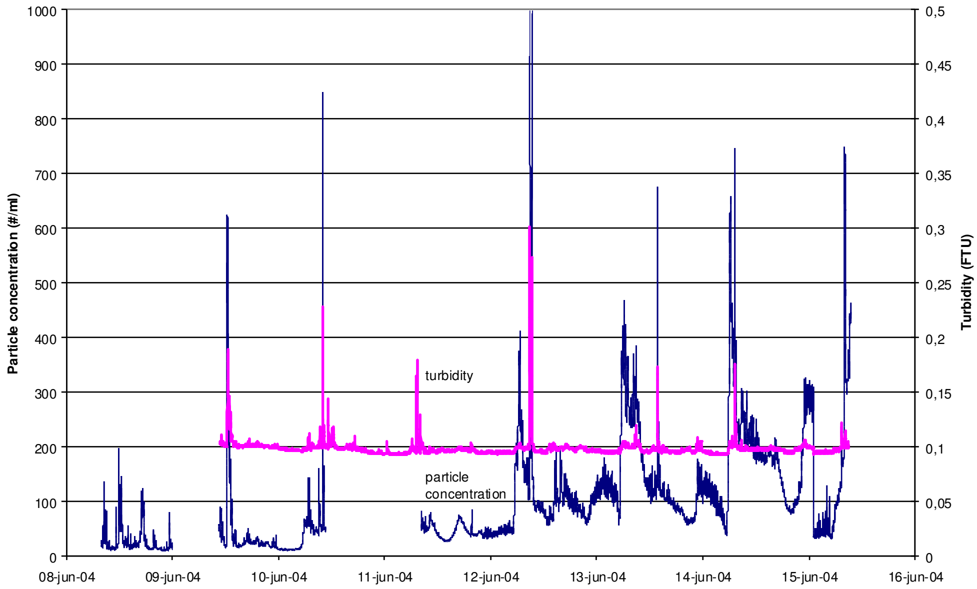


Figure 2-10: Turbidity and particle measurements of distribution network in The Hague

2.7.4 *Particles from treatment plant*

A large contribution to the total sediment load originates from the treatment plant. Elements like Fe, Mn and Al are present in treated drinking water. These elements are mainly present as oxides: Fe as $Fe(OH)_3$, Mn as MnO_2 and Al as $Al(OH)_3$. This assumption does not necessarily hold stand, because Al could also come from leaching of Asbestos-cement pipes and Fe from corroding cast-iron pipes.
The contribution to the sediment load can be calculated using the following formulae:

$$c_{oxide} = \frac{c_{element} * MW_{oxide}}{MW_{element}}$$

$$c_{oxide} = \alpha * c_{element} \qquad \text{Equations 35}$$

$$\alpha = \frac{MW_{oxide}}{MW_{element}}$$

c_{oxide} = concentration oxide [μg/l]
$c_{element}$ = concentration element [μg/l]
MW = molecular weight element/oxide [mol]
α = conversion factor of the oxide

The conversion factors for Fe, Mn and Al are respectively 1.91, 1.58, and 2,89. In Appendix D a list is shown of all the pumping stations in the Netherlands and its contribution to the sediment load of Fe, Mn and Al based on the REWAB figures of 2002. REWAB figures are figures that track water quality by measuring certain substances. The values of the FeMnAl oxides contribution range between 16- 332 µg/l.

These calculations must be critically observed, because these data were collected by samples taken at the treatment plant.

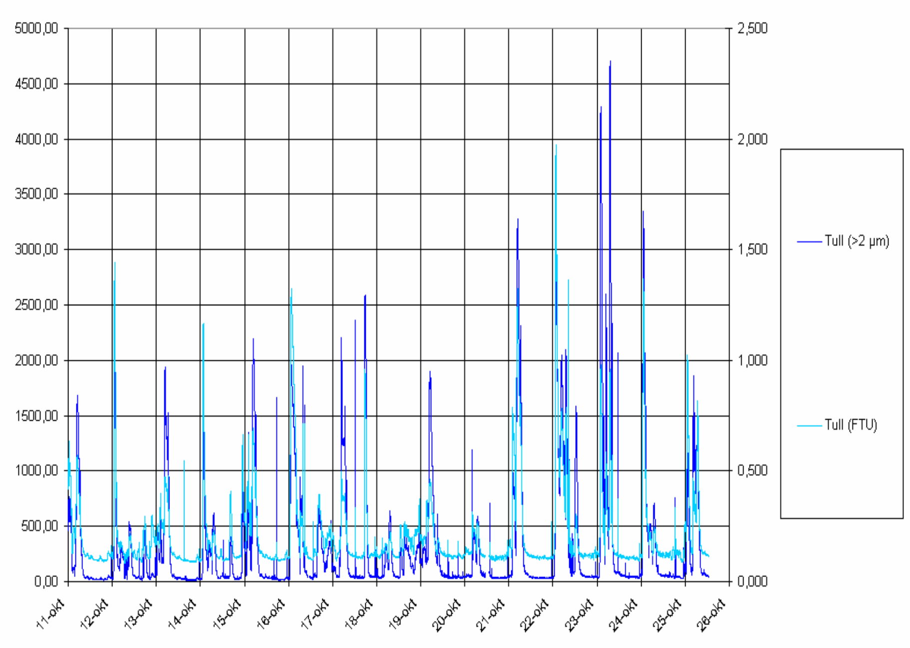


Figure 2-11: Turbidity pattern of PS 'Tull en 't Waal'

The figure shows the turbidity over a few days of a treatment plant, turbidity measurements are used to monitor water quality. It can clearly be seen that there is a significant variation of water quality on a time scale of one day. Peak events or variation occurring in cycles at the treatment plant can be seen. These peaks can be distinguished at almost the same time each day. If samples are taken at the same time every day and they are taken during one of the peaks or at a 'normal' time, the differences in water quality will be very different when the assumption is right that turbidity is a good measuring parameter for water quality.

3 Commercial Sediment Transport Models

3.1 General

To design drinking water networks (computer) models are developed for the analysis of flows and pressures in pressurized networks or open channel networks. The calculation of flows in pipes is relatively simple and can be done with a manual calculation. In a network these calculations are more complex because of the magnitude of boundary conditions and the interconnection of the pipes. With the help of computer models the analysis of the hydraulic behaviour of a network can be done within a reasonable period of time.
The use of models has some advantages: models are cheap, they help understand or explain the system, they are a tool for prediction and simplify the complex system. A lot of commercial models are available all with their own specialisms , these mainly used for hydraulic calculations.
In this section a short overview is given of commercial models that predict sediment transport.

3.2 Commercial Sedimentation Transport Models

Two models are shortly described in this paragraph, at the end of the chapter a list is given with some information and website links to the manufacturers.

Infoworks / Watsed

Infoworks is a hydraulic modeling software package developed by Wallingford Software Ltd. (UK). The package consists of three different modeling parts:

A. Infoworks RS; River Simulation, used to model rivers and estuaries.
B. Infoworks CS; Collection Systems, used for modeling of sewer systems
C. Infoworks WS; Water Supply, used for hydraulic modeling of closed water pipe systems.

Within Infoworks WS a Sediment Module called Watsed has been implemented to predict sedimentation in drinking water networks. This sediment module is based on the distribution of sediment according to the Ackers-White formulae (see paragraph 2.6), this formula can only be used for sand or gravel. Because sediment in drinking water networks is (mostly) not of this origin, this formula is not suitable in this case. The sediment that can be entered in the model has to lie in the range of 45 to 200 µm. Normal sediment found in dutch drinking water networks is much smaller, lying between 1 and 100 µm (See Chapter 6.3). The specific weight of the particles can be entered between 2000 and 4000 kg/m3. This is also quite large because Dutch sediment tends to be lighter than this, as can be seen in the case of De Laak in Chapter 6. This case concluded that the sizes of particles are in the range of 1 to 100 μm with a density of 1280 kg/m^3.

Aquis

Aquis is a hydraulic modeling package developed by Seven Technologies in Denmark. Aquis is specialized in the calculation of age of water in drinking water networks, age is the time that water remains in the pipes until it is consumed.
Sediment types in Aquis can be determined by the size and the rate at which it suspends and resuspends. Aquis deposition and/or resuspension is based on the May's equations, these equations are developed from experimental data and describe the relationship between volumetric sediment concentrations and the flow velocity at the limit of deposition and is mainly used for the maximum bedload transport. The size of sediment that can be entered can be chosen sufficiently low, 1 μm is possible. The same counts for the specific gravity, this is the comparison of the density of the particle to the density of water. The fall velocity of the sediment in Aquis is determined with the help of Equation 36 .

$$v = \frac{\sqrt{9\upsilon^2 + d^2 g * 10^{-9}(s-1)(0.03869 + 0.0248d)} - 3\upsilon}{(0.11607 + 0.074405d) * 10^{-3}} \qquad \textit{Equation 36}$$

v = fall velocity [m/s]
υ = kinematic viscosity [m^2/s]
d = particle grain size [μm]
g = gravitational acceleration [m/s^2]
s = specific gravity [-]

It is not very clear where this equation is coming from, the origin of this equation cannot be found in literature. A comparison of this equation with Stokes' settling is shown in Figure 3-1. It shows that according to the equation used in Aquis the settling velocities are much larger than the velocity according to Stokes.

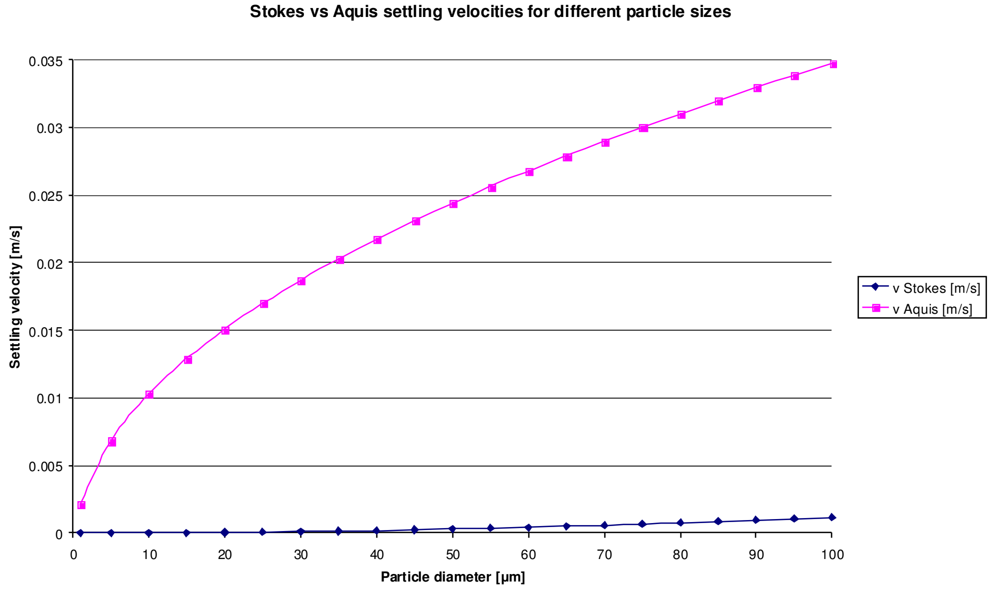


Figure 3-1: Stokes and Aquis settling velocities

The settling equation that is used by Aquis shows a faster settling of partcles than according to the Stokes equation.
It can be concluded that Aquis is promising in the field of the ability of entering small particle diameters and densities, but the settling formula cannot be directly explained.

3.3 Overview and links

Name	Manufacturer	Short description	Link	Applicability	Reason
Infoworks/ Watsed	Wallingford software	Hydraulic modeling package, sediment add-on .	www.wallingfordsoftware.com /products/infoworks	+/- -	Size and density of sediment to large; settling formula not applicable
Aquis	7 Technologies	Hydraulic modeling package, designed for age calculation of water	www.7t.dk/aquis	+/-	Settling formula gives high cettling velocities; size and density of particle can be chosen in the right range
PICCOLO	Safege	Hydraulic modeling package, water quality options	www.safege.fr/english /dom/ logiciel/reseaux/piccolo/present.htm	?	No information
EPAnet	U.S. Environmental Protection Agency	Hydraulic modeling package, with water quality options	www.epa.gov/ORD/NRMRL/ wswrd/epanet.html	+/-	Open source program, water quality modelling can be extended with own programming code
OTHER MODELS AVAILABLE ARE NOT FURTHER DEALT WITH					

4 Non-commercial models

4.1 Introduction

In chapter 2 the known formulas for deposition and resuspension are found to be difficult to apply to drinking water sediments. That is why (non-commercial) models are developed to predict the fouling of drinking water pipes. The aim of the models described is to cope with the problem of the behaviour of sediment by making some assumptions. Predictions are being made based on simplified equations and validated and calibrated with the help of field tests. By improving these models the problems with known formulas can be dealt with.

Discolouration in drinking water networks is mainly caused by the presence of colloidal and/ or resuspended material in drinking water (Boxall, Saul et al. 2003; Gaultier, Rosin et al.1996; Ryan and Jaraytne, 2003).

During treatment of water it is unavoidable that particles are introduced to the drinking water network. The mains and distribution pipes will collect these deposits, because low velocities will occur in these segments. Sediment in pipes can lead to customer complaints because the resuspended sediment causes discoloured or 'brown' water during hydraulic events. To avoid these kind of problems it is necessary to regularly clean the drinking water network. Determination of the degree of fouling can be done by the Resuspension Potential Method (RPM) developed by Kiwa WR (Vreeburg; 2004), this method is developed to predict discoloured water events based on a standard testing method. Another method would be to use a model to predict discoloured water events instead of using measuring in the field, then it becomes a management tool to make a prediction of the quantity of fouling and the decision to clean pipes. In this chapter two different models are tested and described.

The two models that are described in this chapter have a different approach of determining the degree of fouling in networks, the first model is the PODDS model and the second the PSM model.
There are roughly two mechanisms on which these model approaches are based. The first mechanism is gravity settling on which the PSM software is based, the second is wall attraction that the PODDS model uses.

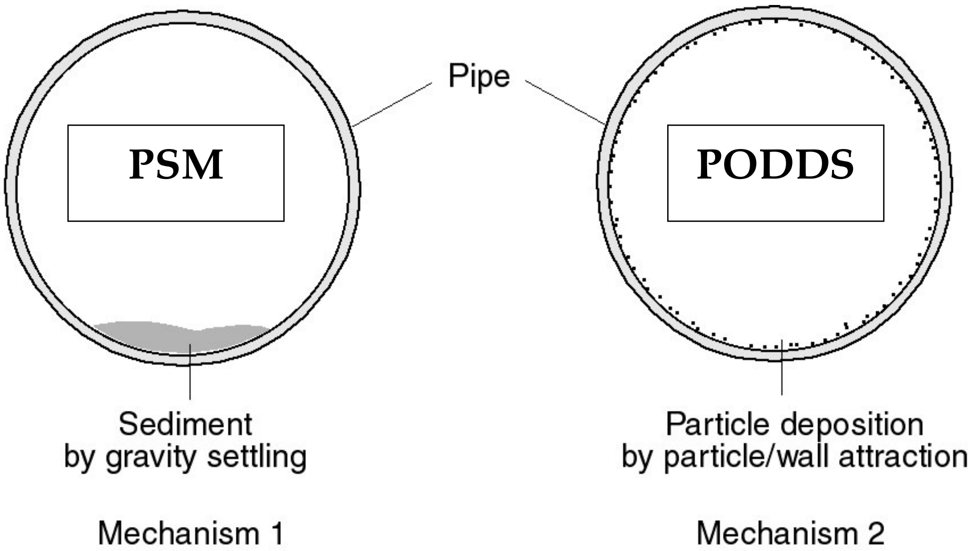


Figure 4-1: Mechanisms of sedimentation in drinking water pipes

Gravity settling only deals with the settling of the sediment in drinking water on the influence of gravity. In the model approach of PSM different velocities have been determined in laboratories for settling, resuspension and deposition of sediment found in Australian networks. These velocities are used in a model to determine the distribution of sediment in a network.
The PODDS model is based on the second mechanism, wall attraction, and determines how much sediment is kept in layers near the wall. This layer near the wall is assumed to entrain all the fouling material, if this layer is eroded it will cause brown water.

4.2 University of Sheffield: PODDS model

4.2.1 Introduction PODDS

The PODDS (**P**rediction an control **O**f **D**iscolouration in **D**istribution **S**ystems) model is a model that does not explicitly deal with gravitational settling or wall attraction, because particle size analysis of discoloured water samples in the UK show that the gravitational settling forces of the material are significantly less than the hydraulic forces generated by the lowest flows within distribution systems. Hence once mobile the material will be maintained as a permanent suspension (Boxall et al; 2001). Together with the size range of the particles found and the factors affecting accumulation (flocculation, biofilm interaction, zeta potential etc.) this suggests that cohesive forces prohibit the continual movement of these particles. In all the field tests in the UK (iron/ manganese driven) and Australia (clay driven) performed so far the cohesive nature of material layers has shown the potential to cause discolouration.

PODDS can calculate the build up of a cohesive layer containing fouling material based on the normal daily hydraulic situation in (a) pipe(s). With the known parameters of a hydraulic disturbance a prediction of the rate at which the layer erodes can be made. The erosion of the layer is visually shown with a turbidity graph on a time scale. The model is build in EPAnet, with an extra plug-in for turbidity measurements.

The goal of the calibrating of the model is the development of a database of parameters for each type of pipe. Through this well calibrated database the application of the model to each individual situation will be possible. Because of the commercial value of these parameters the numbers will not be named in this report, as they are confidential.

4.2.2 PODDS background

The PODDS model is a model developed by the Pennine Water Group at the University of Sheffield (UK) funded by the UK Engineering and Physical Science Research Council. The approach is based on Mechanism 2: wall attraction, but the PODDS model does not specifically deal with one of the two mechanisms but combines both mechanisms (deposition and wall attraction) into one mechanism called *Cohesion Theory*.

During research in the UK it was found that for discolouration of drinking water caused by hydraulic events a force larger than particle self-weight resists the initiation of particle movement. This theorem supposes that this is caused by cohesive forces (Parchure and Mehta 1985; Skipworth, Tait et al. 1999). On site observations of distribution network pipes and excavations showed loose layers formed around the entire pipe circumference. While monitoring flushing operations it was seen that a relatively smooth exponential turbidity trace was produced with progressive accumulation in turbidity along the pipe length (Husband, Saul, Boxall; June 2004). An example of such a flushing operation can be seen in figure 4-2, figure 4-3 shows a further analysis of the content of the water.

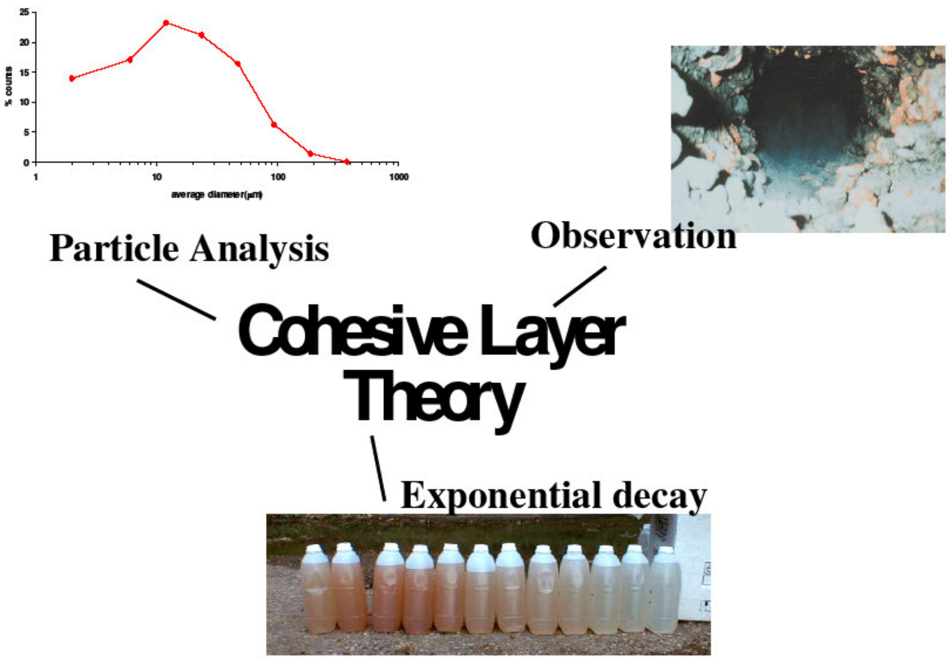


Figure 4-2: On site measurements and proof of cohesion theory

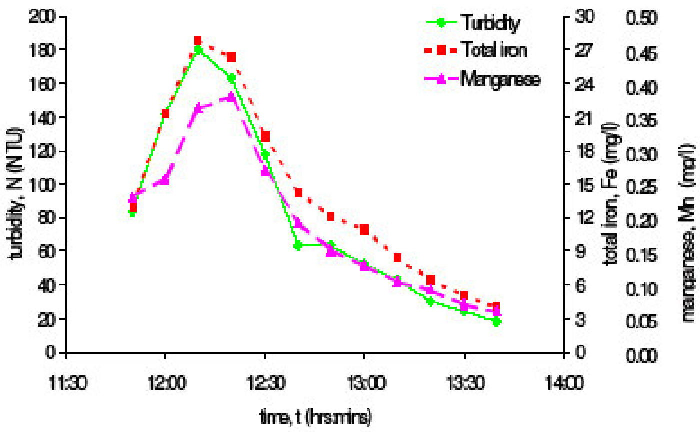


Figure 4-3: Sample analysis of water during PODDS fieldwork showing iron and manganese correlation to turbidity

If sediments would be responsible for the discolouration of the water, changes in the elevation and characteristics of the pipe network would create fluctuations in the measured turbidity. This means that height fluctuations in the network lead to dead-ends ('valleys' in networks) where sediment is accumulated. Because this is not the case with this data: there are no differences between high or low measuring points, two conclusions can be drawn from these measured data. The first conclusion is that sediment itself is not responsible for the discolouration risk of the water, the second is that the material must be uniformly distributed along the entire pipe length. As mentioned before it was observed that a force beyond particle self-weight must be present to enable particle to adhere to pipe walls. These forces are not clearly dealt with in the PODDS model approach, but it is known and assumed that these forces are a combination of physio-chemical and biological forces. These forces interact with each other to produce the overall cohesive force responsible for the development of the layers that create a discolouration risk. The material accumulation, characteristics of the layers within the pipes and the subsequent triggering of the release of material leading to discoloured water are complex and are dependent on many system and material specific factors. These are (next to the physio-chemical and biological forces mentioned before): particle size, density and concentration, material, corrosion processes and electrochemical interactions with the pipe wall.

In the PODDS model approach the cohesive forces are responsible for the layer build up during low flow conditions, build up takes place because of the capturing of discolouration material in a layer close to the pipe wall. Entrainment of material takes place when flow conditions differ from the normal daily hydraulic regime. The PODDS model does not consider any of the discolouration processes implicitly, but fits the model to predict the discolouration by calibrating it with field measurements. This means that the model can simulate a range of material layer types, characteristics and processes (Husband et al. ; 2004). This is because there is no distinction made between different processes but they are combined into one.

The PODDS model describes the turbidity potential of attached layers at the pipe wall, the strengths of these layers are conditioned by the daily hydraulic regime of each pipe. Discolouration occurs when the (maximum) daily hydraulic regime is exceeded, thus leading to erosion of the layers of cohesive material.

PODDS has the potential to provide more information to predict discolouration events, it can be used as a tool to facilitate pro-active management and risk assessment of discolouration. By modelling a flushing action or other hydraulic event a prediction can be made of the degree of fouling of a pipe (turbidity).

4.2.3 PODDS theory and equations

The PODDS model is based on balancing the strength of cohesive layers at the pipe wall with the shear stress forces generated by the system hydraulics. The strength of the layers is based on the hydraulic shear stress, the

maximum shear stress during the day is responsible for the strength and the amount of fouling material that is eroded during a hydraulic event.
The hydraulic shear stress is calculated with Equation 37 .

$$\tau = -\rho \cdot g \cdot \frac{u^2}{C^2}$$ *Equation 37*

$$\lambda = \frac{8g}{C^2}$$ *Equation 38*

$$\Delta H = \lambda \frac{L}{D} \frac{u^2}{2g}$$ *Equation 39*

Equation 37, 38 and 39 combined:

$$\tau = \rho g \frac{D}{4} \Delta H$$ *Equation 40*

τ = Boundary shear stress [N/m²/m]
ρ = Density of water [kg/m³]
g = Gravity acceleration [m/s²]
D = Diameter of pipe [m]
ΔH = Hydraulic gradient [m/km]

To run the PODDS model the maximum shear stress has to be calculated in order to determine the initial layer strength. This initial shear stress is inserted into the model to determine how much discolouration material can be 'stored' into the cohesive layers: the lower the maximum shear stress the more discolouration material is stored inside the layers and the discolouration risk is bigger.

Other (empirical) equations that are used in the model describe the relationship between layer strength and turbidity potential of the cohesive layers. Equation 41 and figure 4-4 shows the maximum turbidity potential (C_{max}) of the layers within the pipes. τ_s' is the value of the shear stress during the highest velocity in the pipes during the day.

$$\tau_s' = \frac{C^b - C_{max}}{k}$$ *Equation 41*

τ_s' = Current layer strength [N/m²]
k = gradient of the potential as a function of the layer strength [m⁵]
b = extra parameter to allow non-linear forms of the relationship [-]
C = turbidity potential [NTUm³]

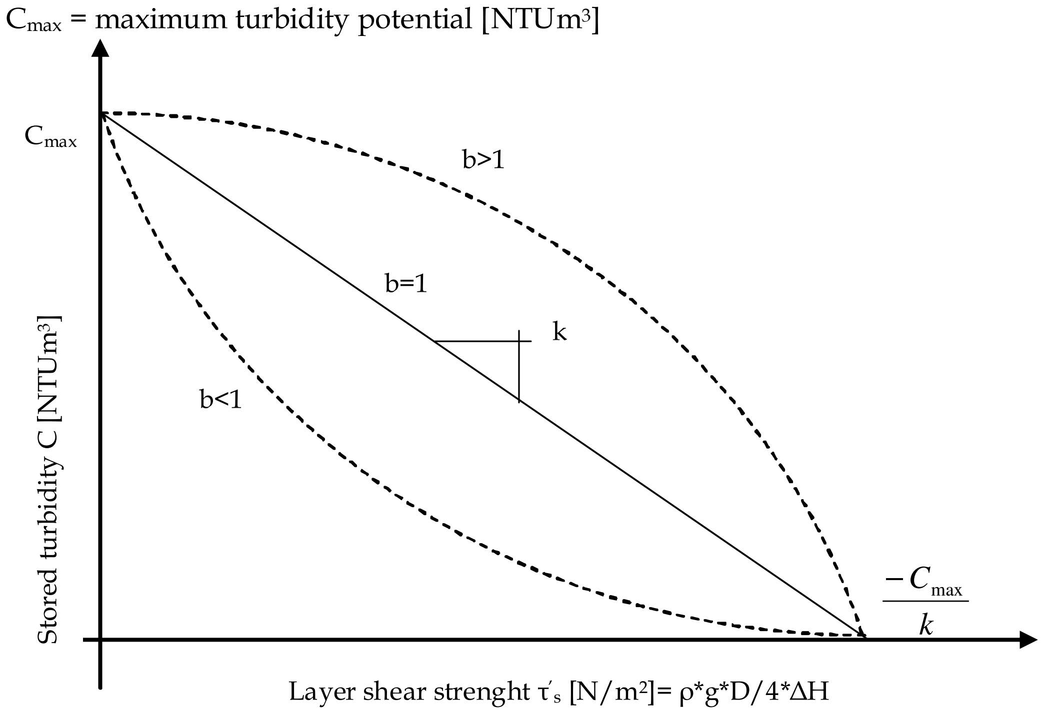


Figure 4-4: Layer strength versus stored turbidity volume

Figure 4-4 shows that the layers have a higher stored turbidity volume when the layer strength decreases. This means that in pipes that have low flow rate (i.e. dead ends, over dimensioned pipes) and also low shear stresses the turbidity potential is high, leading to more brown water complaints. It also means that pipes with a daily hydraulic regime that incorporates higher velocities will have a much smaller discolouration potential (because higher shear stresses occur).
This discolouration potential of the pipes must be related to a mobilizing force before discolouration response can be simulated, such as a hydraulic disturbance like a fire event, pipe-burst, flushing etc.

Equation 42 describes the mobilization of material from the cohesive layers when exposed to a mobilizing or disturbing hydraulic force.

$$R = P(\tau_a - \tau_s^{'})^n \qquad \textit{Equation 42}$$

R = Rate of supply [NTU/m^2]
P = Gradient term [NTU/N]
n = Power term [-]
τ_a = Applied shear stress [N/m^2]
τ_s' = Current layer strength [N/m^2]

This means that (τ_a - τ_s') is the excess shear stress, the difference between the conditioned layer strength and the increased shear stress generated by the increased flow velocity.

The in- or decrease of the turbidity caused by the erosion of the cohesive layer can be evaluated through multiplication of R by the pipe surface area (A_s).

$$\Delta N = RA_s \qquad \text{Equation 43}$$

ΔN = In/ decrease of turbidity [NTU]
R = Rate of supply [NTU/m²]
A_s = Pipe surface area [m²]

The change in turbidity potential and the change in the strength of the layer is calculated using Equation 44.

$$\Delta C_e = R\Delta t \qquad \text{Equation 44}$$

ΔC_e = Change in turbidity potential [NTU*m²/s]
R = Rate of supply [NTU/m²]
Δt = Change of time [s]

In the PODDS model the processes of material regeneration and/ or accumulation are not implicitly investigated or quantified, the model does include these functionalities to facilitate this. The source of the material is not considered explicitly within the model, but is derived by calibration of the empirical parameters that describe the layer strength characteristics and mobilization mechanisms. By doing this a range of processes and materials can be simulated.

An extra equation is added to the model, but does not function yet. This equation is described down here. This equation is to simulate material regeneration through incremental changes in layer turbidity potential and layer strength. This is the influence of the change of Rate of Supply in time. It means that the material regeneration is influenced by temperature in time, this influences the amount of turbidity that is given off by the wall.

$$\Delta C_r = P'\Delta t T^1 \tau_s'^m \qquad \text{Equation 45}$$

ΔC_r = Change of layer turbidity potential [NTU]
P′ = Emperical time based constant [NTU*m²/K*s]
Δt = Change of time [s]
T = Temperature, may be raised to the power (here 1) to allow temperature dependence of material accumulation [K]
τ_s' = Current layer strength [N/m²]
m = Power term allowing regeneration to be a function of current layer strength [-]

4.2.4 Requirements to run PODDS

To be able to run PODDS the following information is needed:

For the build up of the layer strengths:

- Sound (calibrated) hydraulic model, including demands, flows, velocities, pipe lengths, diameters and roughness.
- Working EPAnet program with PODDS plug-in of turbidity

As mentioned in the introduction, the influence of hydraulics is very important. The layer build-up is based on the shear stress which, in turn, relates directly to the flow velocities, pipe diameters and roughnesses of the pipes. No good prediction can be made with a malicious model.

For the actual event that is simulated:

- Accurate data of hydraulic disturbance: flow, build up of flow, time. To calibrate the model, the data of the flushing act have to well known; for use with other hydraulic disturbances, the right data are needed as well.

The PODDS model facilitates the simulation response of pipe elements within complex distribution systems due to change in hydraulic conditions. It uses the hydraulic modeling package EPAnet version 2.0 for Windows that can be freely downloaded at
http://www.epa.gov/ORD/NRMRL/wswrd/epanet.html.
EPAnet is used to edit network input data and viewing the results in a variety of formats. The integrated PODDS mode maintains the full graphical user interface (GUI) abilities of EPAnet and incorporates turbidity (NTU) as a modeled quality parameter. The PODDS model parameters (see paragraph 4.2.3 for equations and explanation of parameter) are incorporated as pipe level user definable variables.

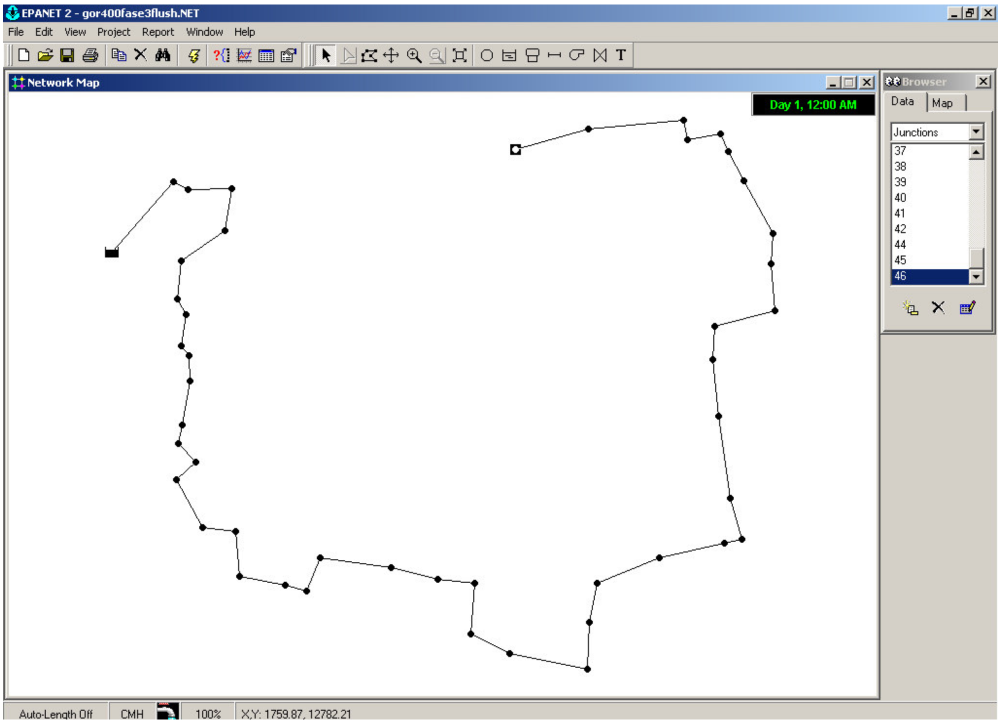


Figure 4-5: Screenshot EPAnet

The following parameters can be changed to 'curve-fit' the predicted turbidity with the measured turbidity (same parameters as in the equations in paragraph 4.2.2.).

C_{max}	Maximum turbidity potential	[NTUm^3]
τ_s'	Current layer strength	[N/m^2]
k	Gradient	[m^5]
b	Power term of turbidity	[-]
P	Gradient term of rate of supply	[NTU/N]
n	Power term of rate of supply	[-]
P'	Regeneration coefficient	[NTU*m^2/K*s]
l	Regeneration order	[-]
m	Power term of regeneration order	[-]

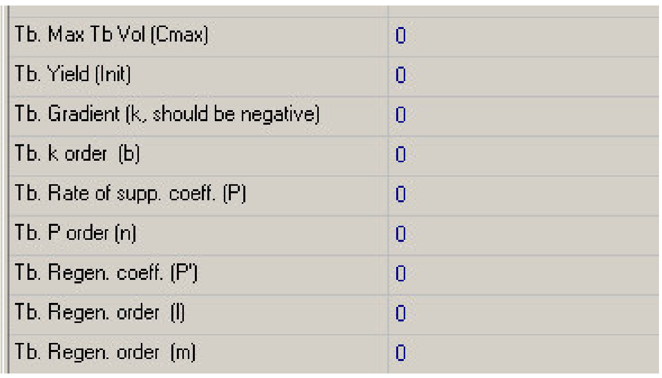


Figure 4-6: PODDS mode parameters

The model parameters are shown as zero for confidentially reasons. The values found to date are covered under confidentiality agreement to protect the commitment and investments of the project partners.

As mentioned before there are parameters that have been implemented in the program to be able to model types of regeneration, but these parameters are not used yet. The parameters that can be used are:

τ_s'	Current layer strength	[N/m²]	Calculated in advance of simulation
k	Gradient	[m⁵]	Determined after simulation
b	Power term of turbidity	[-]	Determined after simulation
P	Gradient term of rate of supply	[NTU/N]	Determined after simulation
n	Power term of rate of supply	[-]	Determined after simulation

Appendix E show a short manual to run PODDS. In chapter 5 two cases (flushing acts) are simulated with the help of PODDS.

4.2.5 Conclusion PODDS

PODDS approach is totally different than other model approaches. The main idea is to capture deposition and resuspension and other processes like biofilm detachment, corrosion, formation and coagulation into one process called *Cohesive Theory*. The hypothized build up of this cohesive layer is tried to describe with logical but not really transparent mathematical equations. Values for the different parameters are subject of further investigation, as they will be used for the building of a database. For some of the parameters (like C_{max}) it is not really clear what the importance to the rest of the calculations is. A case study in chapter 5 will show how the models performs.

4.3 Cooperatieve Research Centre: Particles Sediment Model (PSM)

4.3.1 Introduction PSM

PSM is a model that is based on the ideas mechanism 1: gravity settling. It assumes that all particles entering the network come from the treatment plant and that no other processes occur inside the network. These particles will settle under influence of gravity and/ or resuspend when the flow velocity is above a certain level. The sediment will slowly be distributed over a network, the model calculates how much sediment settles and where in the network. In the approach of the model bed-load transport is not implemented. Bed load transport is the (slow) movement of sediment at the bottom of the pipe.
To be able to use the program some water quality parameters have to be determined: the concentration of particles from the treatment plant and the different characterization velocities for the sediment. These velocities are the settling velocity of the sediment and velocities for which the sediment will settle or resuspend.
The result of PSM is a graphical visualization of the network with colored pipes, meaning pipes with different amounts of sediment deposited or suspended inside of that pipe.

4.3.2 PSM Background

The PSM (Particle Sediment Model) is developed by the Cooperative Research Centre (CRC) in Australia. The model is currently based on mechanism 1: gravity settling but will be expanded with mechanism 2: wall attraction in the future.

The aim of the model was to develop a Particle Sediment Model (PSM) in the form of software to add to existing hydraulic software packages for the purpose of tracking the transport, settling and resuspension of (cohesive) particles in drinking water distribution systems. This model could then be used by water companies as a guide for pipe cleaning (Grainger et.al; 2003).

The PSM model uses the characteristics of sediment as input, this means that velocities at which the sediment suspends, resuspends and/ or settles have to be determined. This has been performed by obtaining samples of particulates from water distribution systems of Melbourne, Adelaide, Sidney and Brisbane. The samples were analyzed using a pipe test-loop and a water tunnel at CMIT (CSIRO Manufacturing & Infrastructure Technology). The rig consisted of a pipe with a diameter of 100 mm, a picture and schematic drawing of the test pipe are shown in Figure 4-7 and 4-8. The test rig was used to determine the flow velocity of the water when sediment starts to settle and the sediment velocity with which the sediment settles.

Figure 4-7: Pipe test-loop

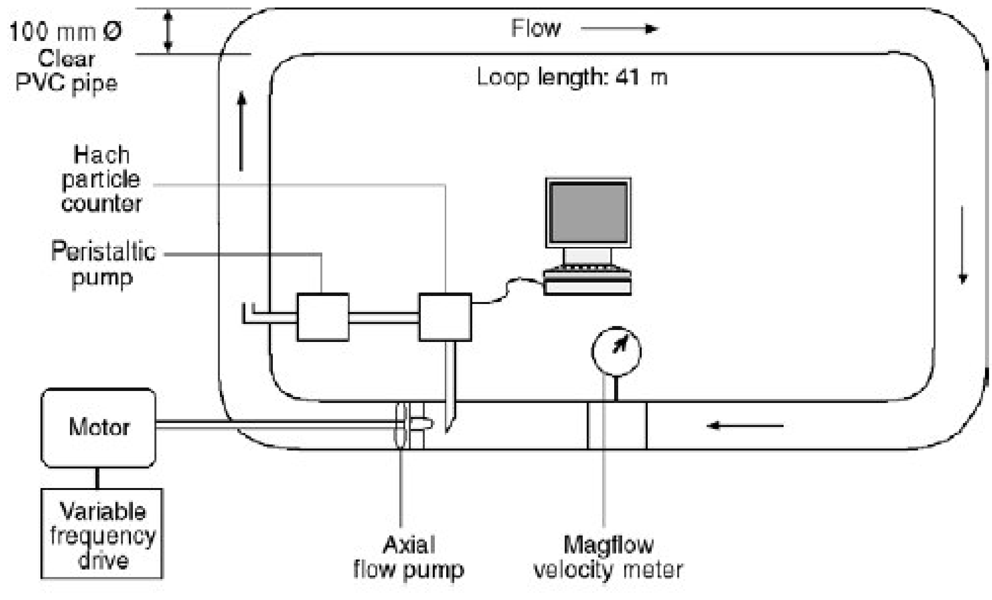


Figure 4-8: Schematic drawing of pipe test loop

The aim of the analysis of the sediments has been to determine three velocities:

u_{rs} = *flow velocity of the water at which the sediment resuspends*
u_d = *flow velocity of the water at which the sediment deposits*
u_s = *settling velocity of the sediment*

By determining the velocities for typical sediment found in Australian networks the problems with the theory of settling (Stokes) and resuspension (Shields) are short-cut. This simplification was made to characterize the sediment characteristics and use them in the PSM (computer) model. The sediment was collected from existing distribution networks by flushing and from storage tanks in the network. The sediment was boiled and dried before it was used for further investigation. This means that the sediments used in the test-pipe are not exactly the same as in the network.

The u_{rs} has been determined by placing particle samples in trays which where placed in the water tunnel. Water flowing through the channel flowed over the particle samples in the trays and particles were eroded and re-suspended by the flowing water.
The u_d has been determined with the help of the pipe loop. Sediment was inserted in the test pipe and the water was monitored with a turbidity meter. Different velocities were tested to determine he velocity at which the sediment would settle.
u_s was determined with the help of the water tunnel, individual suspension test with different samples w ere performed. The degree of suspension was determined by visual observation of the cloud of particles carried downstream from the sample.
These velocities are used to perform tests with the software to predict hydraulic behavior in distribution networks.

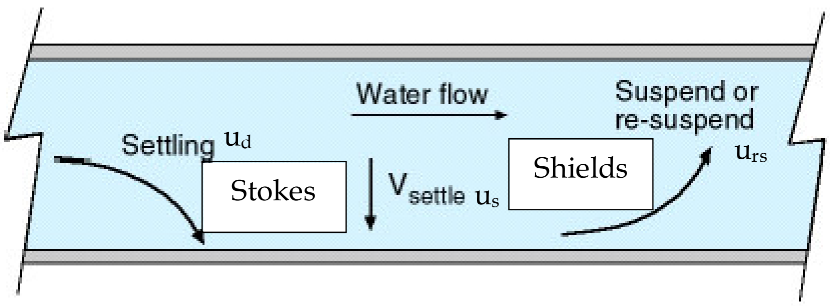


Figure 4-9: Cross section of pipe illustrating suspension, resuspension and settling

Figure 4-9 shows the possible behaviour of the sediment and the corresponding velocities.

To be able to use the program in other countries (i.e. Netherlands) the different velocities for (re)suspension and settling of sediment in drinking water networks have to be determined. Some types of sediment have been investigated at Delft University (Lut et al; 2005), these sediments were

Kaolinite, $FeCl_3$ and sediment from a flushing operation in the Netherlands. In chapter 6.3 these three sediment types are used to determine the behaviour of sediment from a treatment plant.

4.3.3 PSM theory and equations

The PSM software has been developed to easily calculate the distribution of sediment in a network. Data required to be able to run the model are:

1. Model of a network
   - Hydraulic data of network
   - x, y and z coordinates of all nodes
   - Length, diameter and roughness of all pipes
2. Sediment characterization (see further)
   - u_{rs}= velocity at which the sediment resuspends
   - u_s = velocity with which the sediment deposits
   - u_d =velocity at which the sediment will start to deposit

The two mechanisms that are perceived in modelling sediment in drinking water networks (Ryan et al. 2003) are: gravity settling and wall attraction. Only the first of these mechanisms is incorporated in the PSM program so far.

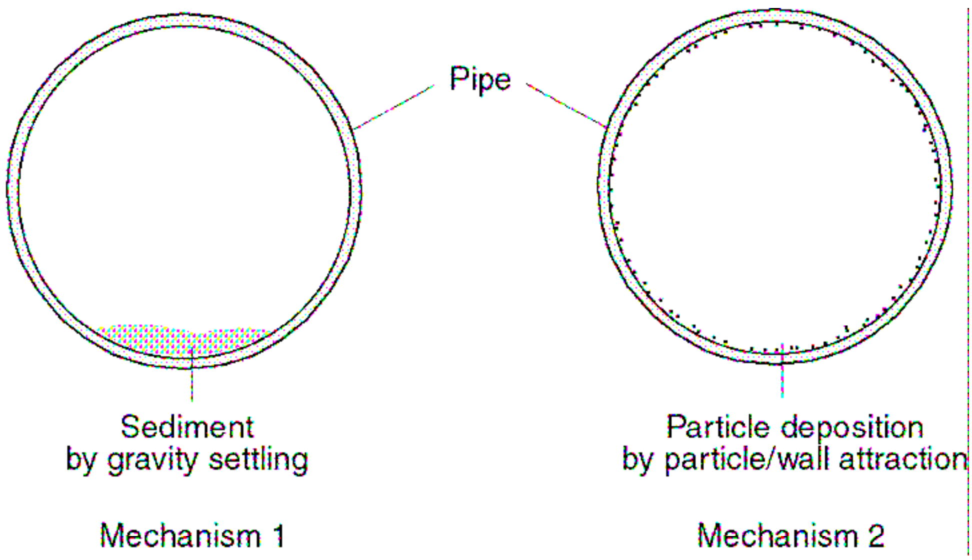


Figure 4-10: Mechanisms of sedimentation in drinking water pipes

Mechanism 1: Gravity settling is the settling of particles under gravity, this mechanism is shown in a simplified model in Figure 4-11. The velocity at which the water flows is u, the velocity at which it resuspends is called u_{rs} and the velocity at which all particles will suspend is called u_d.

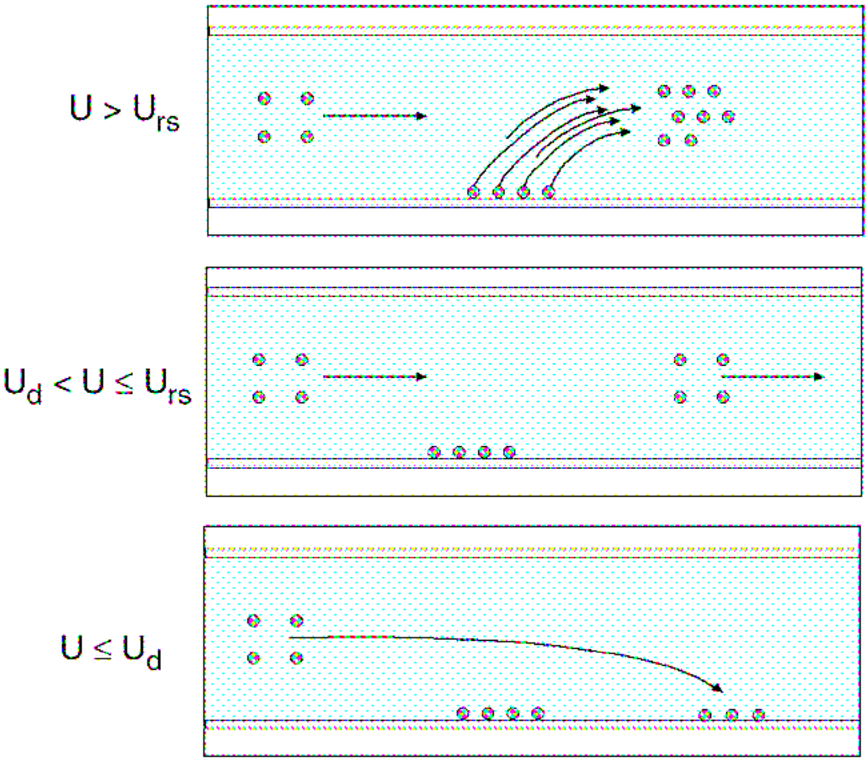


Figure 4-11: Model gravitational settling

There are three situations that can occur, depending on the flow velocity u:

A. $u > u_{rs}$:
The flow velocity is more than the resuspension velocity so resuspension of all sediments occurs. u_{rs} is the critical velocity beyond which particles are resuspended, u_{rs} is a function of particle diameter, density and packing of sediment.
B. $u_d \leq u \leq u_{rs}$.
The particle mass is transported through the pipe with no settling/ resuspension, because the flow velocity u is between the velocity at which the sediment suspends (u_{rs}) and the velocity at which it settles (u_d).
C. $u < u_d$:
All particles will settle, because the velocity of the water is so low that all sediment will suspend.

To be able to characterize the settling statuses of particles, a vertical particle cloud height H_s is defined (see below). From this cloud height H_s a dimensionless particles cloud height factor is derived.

$$s = \frac{H_s}{D}$$

s = dimensionless cloud height factor [-]
H_s = Particle cloud height [m]
D = pipe diameter [m]

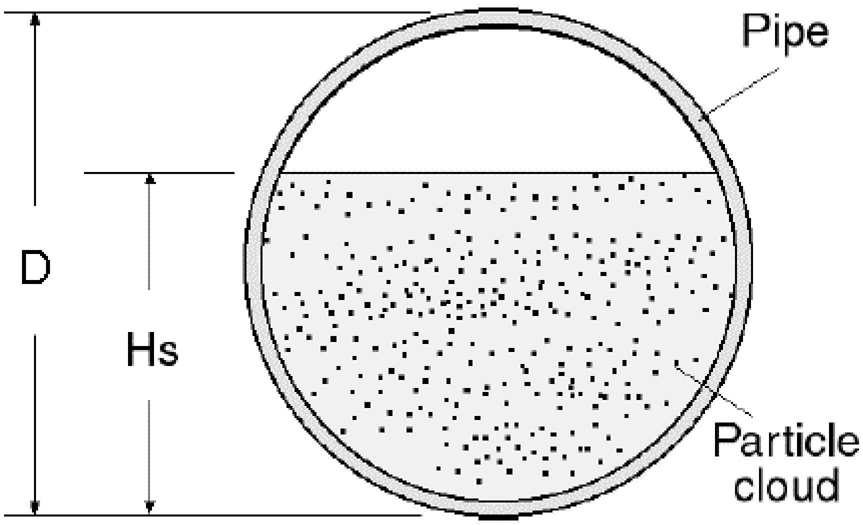


Figure 4-12: Particle cloud height defined by H_s

The settling of the particles is characterized by the following equation. s will become zero in the end, representing complete settling of the sediment.

$$s(t + \Delta t) = s(t) - \frac{u_s \Delta t}{d}\text{; for } u<u_d$$

It can be concluded that particles:
- fully suspend when s=1
- fully settle when s=0
- partially settle/ resuspends when 0<s<1

The program calculates for each pipe individually the following parameters:

- dM_{in}, particle mass transport into the pipe during time step Δt
- dM_o, particle mass transport out of the pipe during time step Δt
- M_x, particle mass per unit length distribution along the pipe at time t
- M_s, particle mass settled at time t
- M_w, particle mass deposited at the wall at time t *due to wall attraction* (TO BE IMPLEMENTED IN THE FUTURE)

The particle mass deposited at the wall (Mw) is not yet implemented, but could be simulated within the gravity settling. This could be done by adding a percentage to the settled sediments.

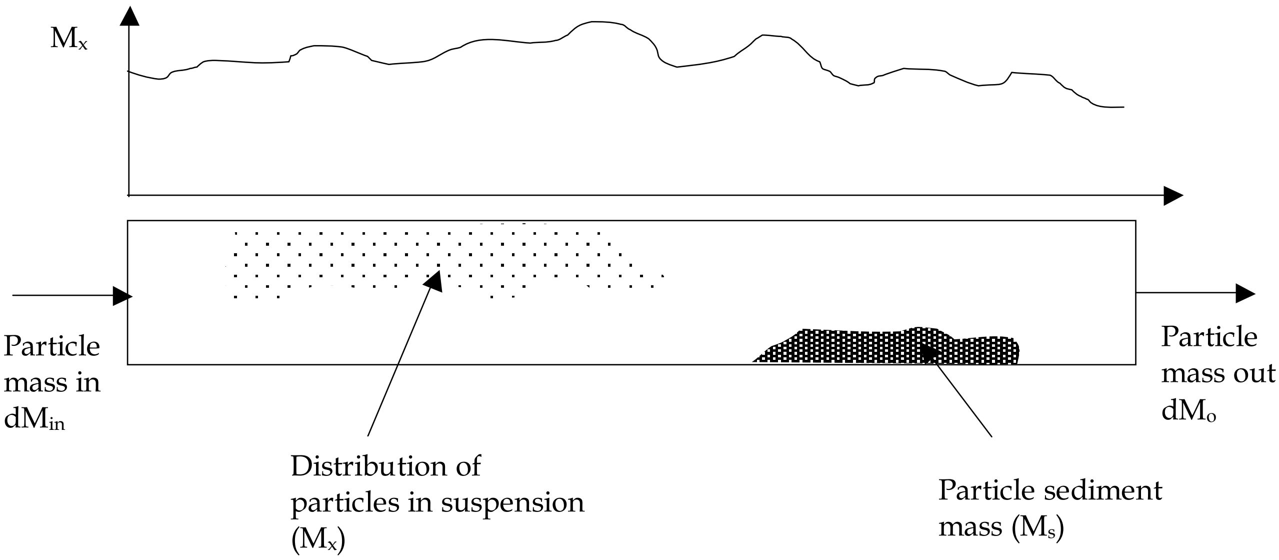


Figure 4-13: Calculated particle mass in pipe by PSM at each time step

Appendix A shows detailed formulas and mathematics used to calculate the particle mass parameters, based on mechanism 1: gravity settling.

The following steps are taken by the PSM program:

1. The particle output dM_o from each pipe at time t is used to calculate the mass input into the next pipe at the next time step $t+\Delta t$.

$$dM_{in}(t+\Delta t) = \frac{Q}{\Sigma Q_k} \Sigma dM_o(t)$$

Q = flow rate in pipe [m^3/s]
Q_k = flow rate from k^{th} pipe into current pipe [m^3/s]
Σ = flow of all pipes connected to current pipe [-]
dM_o = particle mass output [kg]

2. Calculate particle mass parameters (dM_{in}, M_x, M_s, dM_o) as described in figure 4-13, given input mass dM_{in}.
3. Calculate particle settling status parameter s (ratio of particle cloud height over pipe diameter).
4. Repeat calculations for all pipes

5. Calculate the next time step from 1.

In this calculation the assumption is made that at each node complete mixing of all the sediment transported for each of the pipes is occurring.

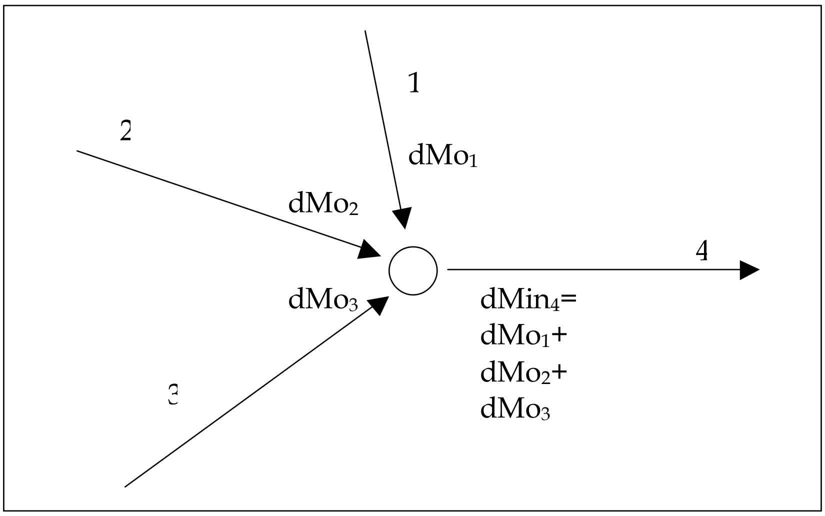


Figure 4-14: Schematic drawing of mixing at node

During the mixing at the node the value of *s* is set to 1 again. This means that the settling from a node the calculations for the individual pipe start all over again. Extra sediment input caused by other processes can be modelled as well, by injecting sediment at a pipe extra processes can be simulated. This injection has to be done with each pipe individually.

4.3.4 *Implementation of Mechanism 2*

Although the PSM model uses gravity settling as mechanism, wall attraction will be incorporated in the future. In this paragraph a short description of the theory behind this mechanism is given.
According to test carried out (Ryan et al, 2003) using a test pipe loop, particles disappeared from the suspension in a wide range of velocities up to 0,3 m/s or more. Particles resuspended again from the pipe bottom in the range of 0,15- 0,25 m/s. This is also found on tests that were performed at Delft University (Lut et al, 2004).
A hypothesis could be that the particles deposit onto the wall surface because of the Vanderwaals force (Ryan et al, 2003). The Vanderwaals force can be described by Fick's law, two coefficient *a* and *β* are introduced.

$$\frac{\delta C}{\delta t} = -\alpha(C_0 - C_\infty)$$

$$C_w = \beta C_\infty$$

C = concentration of particles in suspension [mg/l, ppm]
C_0 = initial concentration of particles [mg/l, ppm]
C_w = mass of particles on the wall, per unit volume of water [mg/l, ppm]
C_∞ = final steady state concentration of particles [mg/l, ppm]

α = decay coefficient [-]
β = wall mass coefficient [-]

The solution of the first differential equation is simple, namely:

$$\frac{\partial C}{\partial t} = -const \Rightarrow C = -(C_0 - C_\omega) * t$$

So the concentration is assumed to be in a linear form, the only distinction is made with the help of coefficient α and β.

From the above equations the following things can be seen:

- C increases in time if $C < C_\infty = C_w/\beta$, so stripping of particles from the pipe wall into suspension occurs
- C decreases in time if $C > C_\infty = C_w/\beta$, so deposition of particles onto the pipe wall from the water
- If $C = C_\infty = C_w/\beta$, a steady state condition is reached. C will stay constant in time.

An extensive laboratory research was performed (Ryan et al, 2003) to determine the quantity of parameter β. When parameter β is known the particle mass sticking at the wall can be calculated using the equations mentioned above, in the steady state. It was found that for a particle mass concentration of 0,5 ppm the value of β was between 0.72 and 13.28, which are large differences. The experimental values differ so much that it is not very easy to determine an exact number for sediment and have to be defined for each type of sediment individually, this means that a general value can not be given and to make a good prediction labour intense work has to be done. Different values were found for different water companies, leading to a mass of the sediment found of 0.012 to 0.052 g/m for a 100 mm pipe.

4.3.5 Conclusion PSM

In the development of the PSM program a lot of laboratory research has been performed to identify the type of sediment and the different velocities that determine the sediment behavior. The determination of these velocities is very labor intensive to determine, for good modelling results the determination of these sediment characteristics has to be done. However, some starting values can be used to run the program and obtain a prediction. The empirical approach shortcuts the use of formulas for sedimentation and resuspension. By determining different velocities, large scale calculations are avoided.
The theory of the program looks good, for each pipe is individually determined how much sediment is entering the pipe, how much settles and how much is transported (in suspension) into the next adjoining pipe(s). The force of the program is that it calculates all of these parameters for a complete network and gives a total result. The limitations are that there can be large differences in behavior of sediment, leading to complete different results in total sediment distribution over the network.

5 Case analysis: Supply Area 'de Laak' with PODDS

5.1 General

To test the different models the supply area of PS 'De Laak' is used. First the PODDS program is tested , then the PSM program. Because PODDS uses turbidity as test variable and PSM uses mass of sediment settled in the network, the comparison of the two programs is not possible. The two models use two different processes, PODDS predicts the amount of resuspended material and PSM the amount of settled material. They are being tested and judged on their practical value, instead of comparing the two results.

PODDS will be tested on two cases, each trying to make a prediction of a flushing action performed on a part of the network. The aim is to see if and how a flushing simulation can be modelled and to determine the parameters to reach a good simulation. These parameters give extra information for the database that the developers are building. By modelling these flushing acts the model is calibrated and validated.

PSM is also be tested on the same case network as PODDS. It will analyze the distribution of sediment over the complete network. The goal is to see how much and where the sediment is deposited in the network. Different scenarios will be described to see how much the distribution of the sediment will differ if different parameters (sediment characterization velocities) are used. The model is not calibrated or validated, more information of different flushing acts or samples in the network is necessary to validate the model.

5.1.1 Limitations PODDS

PODDS has some limitations:

- Hydraulic situation has to be well known

The layer is supposed to be fully filled with material at each hydraulic event. This means that despite the history of the pipe, a prediction is given that is based on the assumption of an undisturbed pipe. This could be true but usually the pipes are not undisturbed.

- Prediction made on pipe level

The prediction can only be made for a (series of) pipe(s) and not for a complete network. The model can be used to manage the problem, not to improve a network and see what the effects are.

5.1.2 *Benefits PODDS*

The benefits of PODDS are:

- Prediction of rate of fouling

If the different parameters and the history of the pipe are known a prediction of the rate of fouling can be given. If the same conditions are applied that were previously found for a good prediction, the rate of fouling can be predicted.

5.2 Case using PODDS

This case was performed on a part of the supply area of pumping station 'de Laak'. Two flushing operations in the city of Gorinchem were simulated with the use of PODDS. Gorinchem is located in the south-eastern part of the supply area, see figure 5-1 and 5-2.

The first case has a pipe length of approximately 1500 m, the second was much longer: 5500 m. From the first case the PODDS parameters for a good simulation are determined, these values will be used in the second case to see if this leads to a good second simulation.

Table 5 shows the parameters that will be determined for this first case.

Table 6: parameters PODDS

τ_s'	Current layer strength	[N/m^2]	Calculated in advance of simulation
k	Gradient	[m^5]	Determined after simulation
b	Power term of turbidity	[-]	Determined after simulation
P	Gradient term of rate of supply	[NTU/N]	Determined after simulation
n	Power term of rate of supply	[-]	Determined after simulation

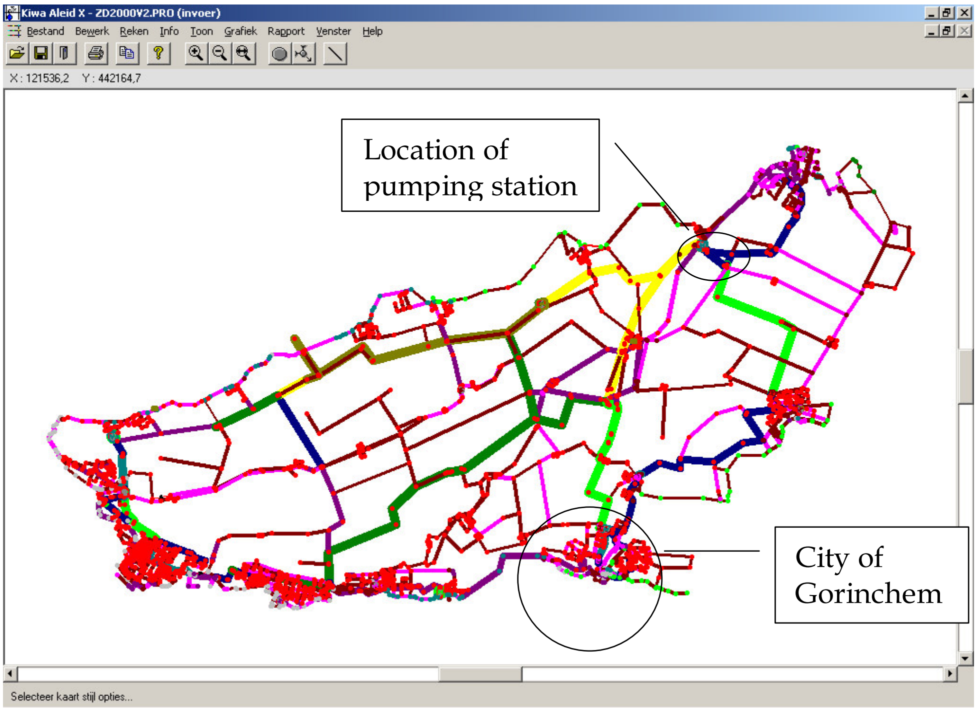


Figure 5-1: ALEID model of 'de Laak'

The complete hydraulic model of the supply area was available, this model was the maintenance model of Hydron ZH. The model is made in ALEID, a hydraulic software package developed by Kiwa WR and originally based on EPAnet. The two flushing acts were performed on two transport pipes that form a circle around the city of Gorinchem.

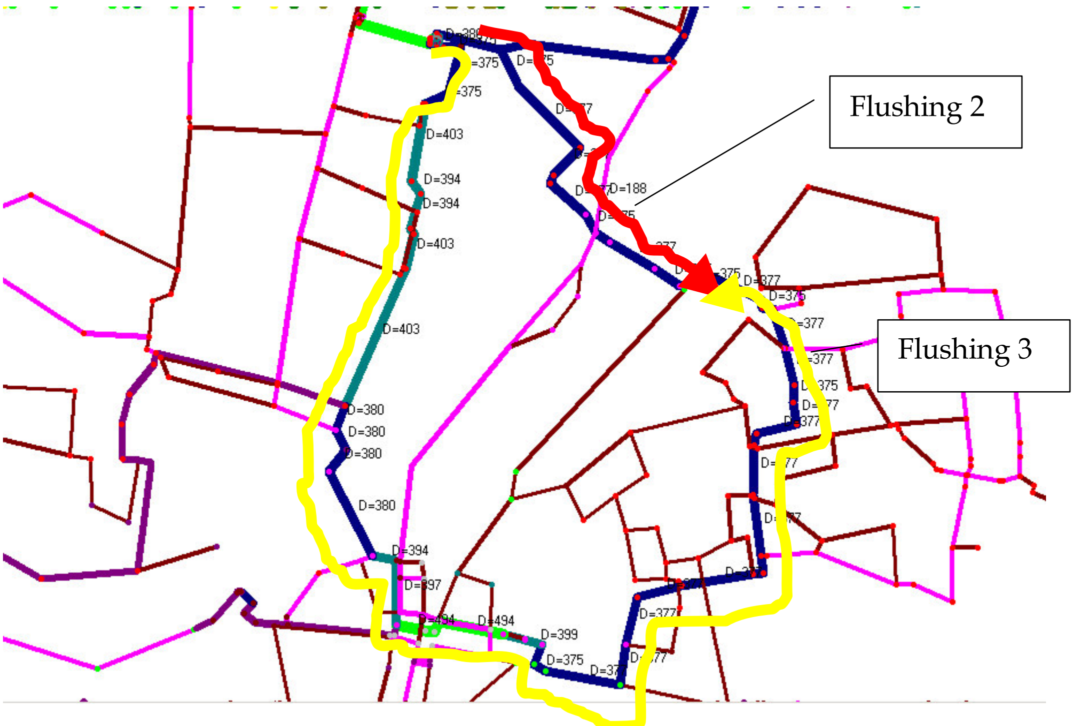


Figure 5-2: Flushings in Gorinchem

The transport pipes are roughly in the same range: internal diameter varying between 375 and 403 mm. A part of the pipe with a length of 400 m had an internal diameter of 494 mm internal (in the south of the supply area).

5.2.1 *Gorinchem Case 1*

The first flushing operation was performed at the northern part of the transport ring, in figure 5-2 called Flushing 2. The part consists of 10 pipes with the same diameter of 375 mm with a total length of 1500 m. The flushing was performed with a flow of 542 m^3/h, resulting in a (calculated) velocity of 1,36 m/s. The hydraulic data was taken from the ALEID model and together with the pipe data put into EPAnet/ PODDS.

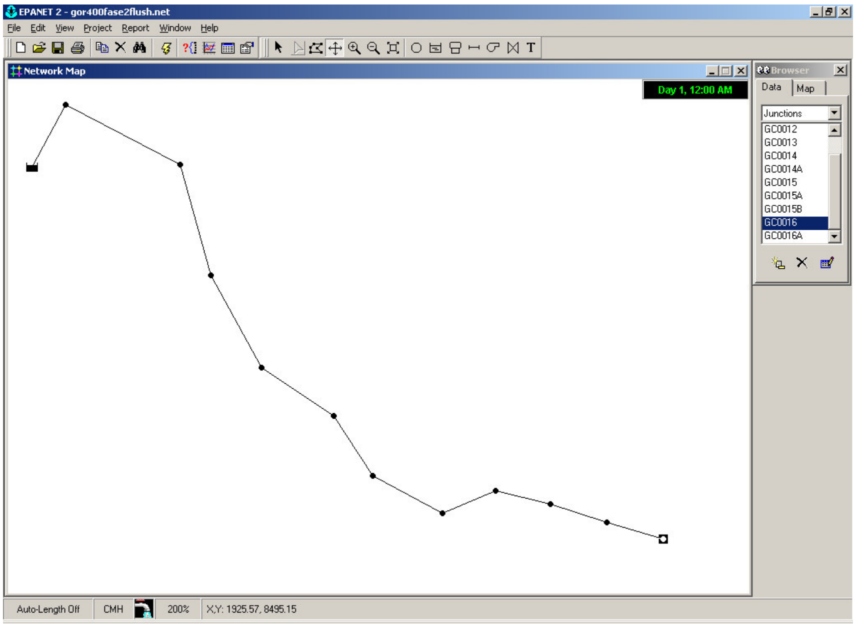


Figure 5-3: Model of Flushing 2 in EPAnet/ PODDS

The different τ values (hydraulic shear stress) were computed for each of the 10 pipes. These values were inserted in the program and are used to determine the initial layer strength. A larger value means stronger layers and and smaller values the opposite.

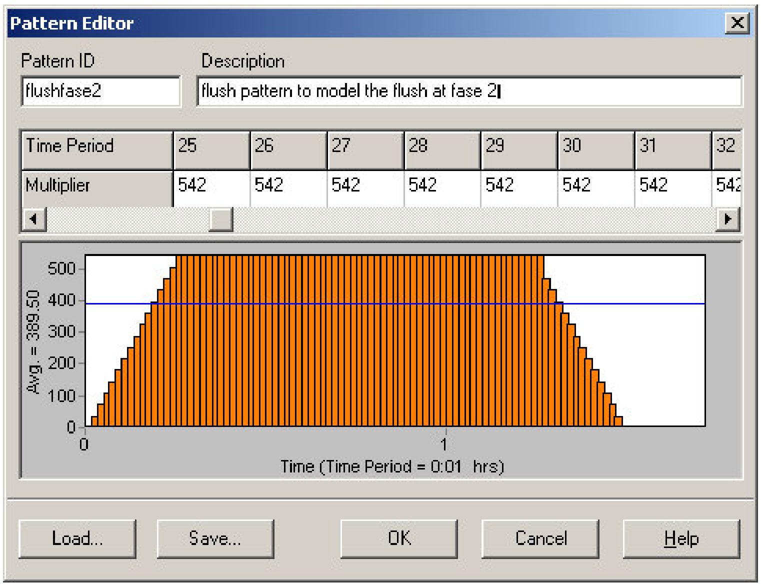


Figure 5-4: Pattern of Flushing 2 at end node (node GC0016)

The run with the initial values set for τ_s', k, b, P and n gave the following result. These values were 'guessed' as no initial values were available. The dotted line is the measured value of the turbidity during flushing and the other line is the predicted value. There is not a good resemblance between the predicted and the measured values.

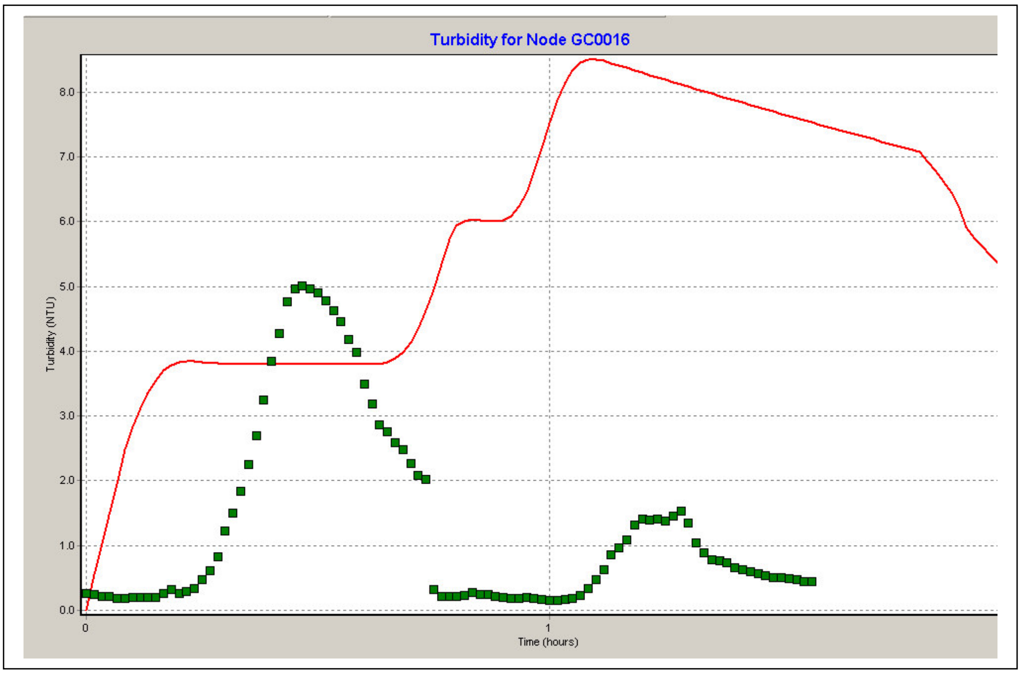
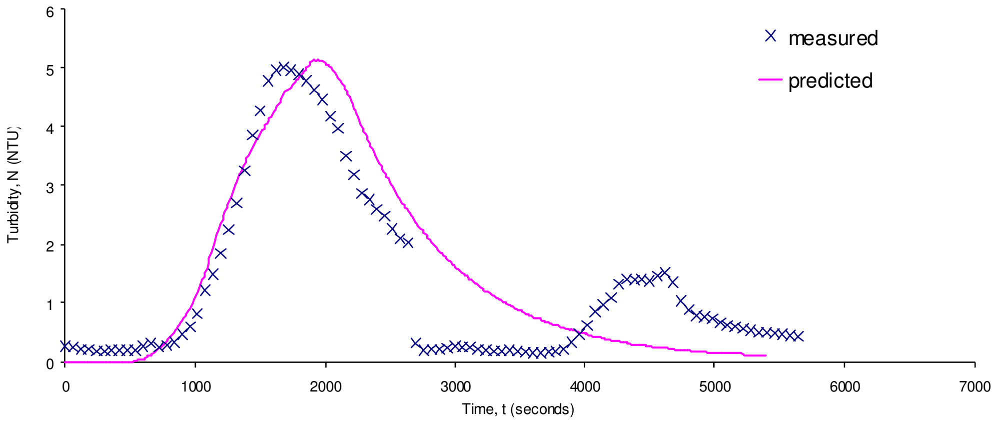


Figure 5-5:First try turbidity prediction for flushing point(endnode GC0016)

After changing of the parameters a fairly good resemblance between the predicted and measured data was found, see figure below. The values of the parameters were found after extensive trying of different numbers.



Conclusion Case 1:
A reasonably good prediction can be made of a simulation of a unidirectional flushing act of a simple part of a transport network. The largest peak is quite well predicted, the pattern of the curve is following the measured values quite nicely. The second peak is not predicted with PODDS, but can be explained by a leaking valve upstream of the flushing point. This part upstream of the flushing point was flushed before the flushing operation that was modeled. If this has not been performed well enough possible sediment could have come into the rest of the transport pipe. The second peak comes after approximately 3000 s (50 minutes). With a maximum flow velocity in the pipe of 1,4 m/s this leads to a run time of approximately 4200 m. As this is much longer than the whole flushing pipe, this peak has to have an origin further north on the map.

The parameters found are used as starting data for the second case to see if these parameters give a good curve-fit. The second case is a bit different than the first case, the internal diameters are also around 400 mm with a part with slightly bigger diameters (500mm).

5.2.2 *Gorinchem Case 2*

The second case which was simulated with PODDS was the second part of the transport ring around Gorinchem, called Flushing 3 in Figure 5-2. This pipe system was much longer than the first flushing pipe length, approximately 5500 m length in total. The diameters of the pipes of this transport ring differ more than of the first flushing operation, between 375 and 494 mm. There is also a 'tidal point' where the city centre is connected to the transport pipes leading to stagnant water and resulting in very low flow conditions. This makes the conditions a lot weaker than during the first case.

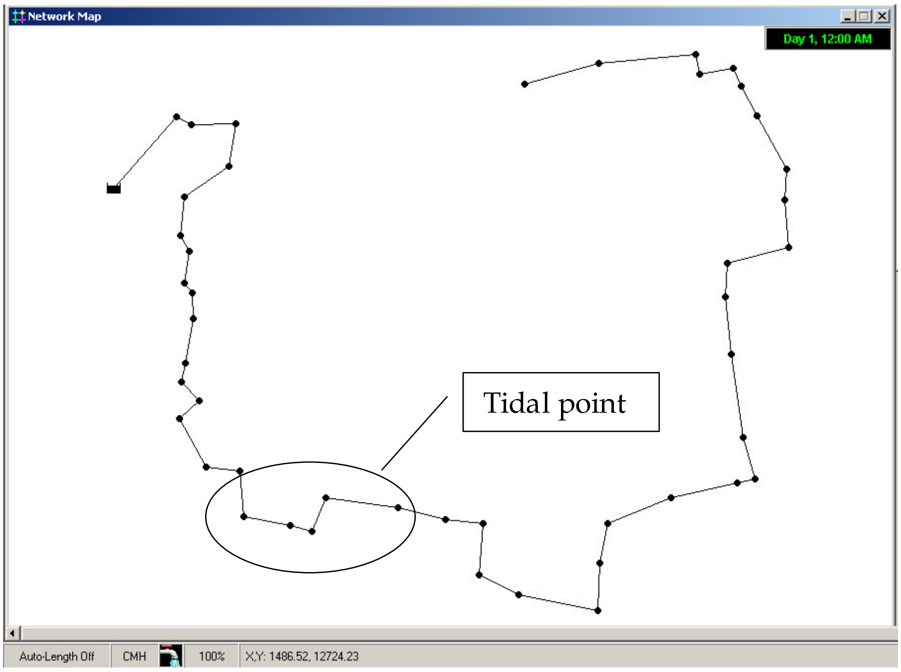


Figure 5-6: Map of Flushing 3 including tidal point

A flushing pattern as can be seen in Figure 5-7 was put at the end node represented by a large demand, this end node was the flushing point during the flushing act. The flow was slowly increased to the maximum flow of 542 m^3/h following the given pattern in Figure 5-7, the velocity in the pipes was then between 0,79 and 1,36 m/s.

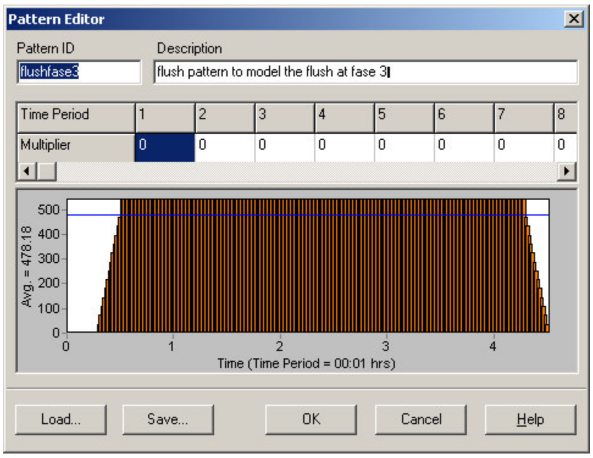


Figure 5-7: Flush pattern Case 2

The values of τ_s', k, b, P and n found during the first case where used to see if they were useable for the simulation of the second case, this resulted in the graph that can be seen in Figure 5-8.

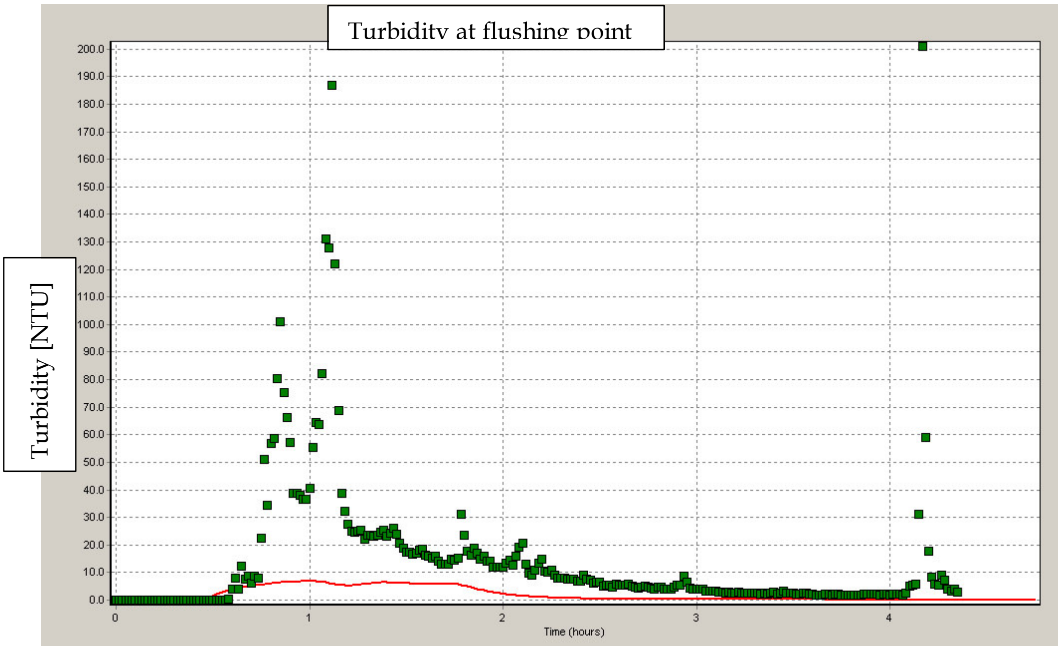


Figure 5-8: First try simulation Case 2 using parameters found at Case 1

The prediction was not looking really good, so changes to the parameters had to be taken. The parameters were then changed to give the best possible match, resulting in the following graph.

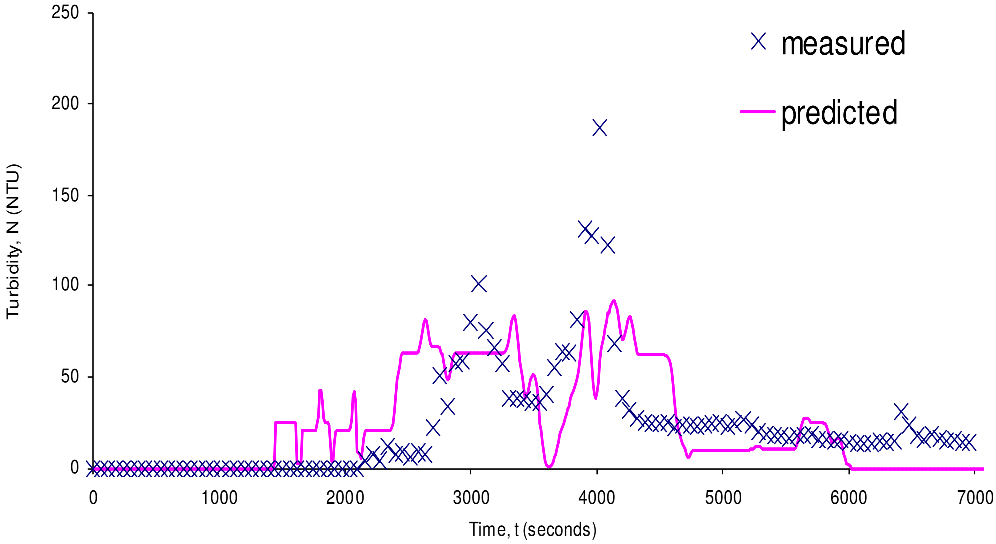


Figure 5-9: Second (best) curve fit Case 2

Conclusion Case 2:

The prediction is not as good as the first case: the parameters of the first case were used in the second one, but these parameters give not a good result. A comparison of the parameters is given in Table 6. Because the number of the parameters are not given, the difference in percentages between the different parameters is given. The better fit was found while trying different parameters, so this was 'curve fitting' to match the predicted values with the measured values.
It shows that the pattern looks slightly the same, with two maximums in the predicted and measured values occurring at the expected time. The maximums are not as high as predicted however, the values not exceeding 90 NTU.

Table 7: Comparison parameters Case 1 and Case 2

PODDS Parameter	*Case 1*	*Case 2*
k	-	+2000%
b	-	0%
P	-	40000%
n	-	0%

There are a few things that could be of influence to the result of the simulation:

- Pipe length is very long (nearly 5500 m)
- Because of the long pipe length, hydraulic events like fire events or leakage could have happened on parts of the pipe. This would mean that the cohesive layer has been (partly) eroded, leading to a different type of emitting of material from the cohesive layers.
- On a large part of the pipe there is a 'tidal point', a point where the water can flow in two ways. The flow velocities in this tidal point are also very low (between 0.01 and 0.05 m/s). As these velocities are very low, they influence the conditions set for the layer build up by τ'_s. This layer build up is responsible for the magnitude of erosion.

5.3 Conclusion PODDS

PODDS is not very easy to set up, a lot of knowledge of hydraulic models is necessary to be able to set up a simulation. Up till now it is not user-friendly, but in the future this will perhaps be improved.
When looked at the two results that were simulated with the help of PODDS it can be concluded that a good simulation based on the calibrated values of the parameters found in Case 1 is not good. The first case looked quite good, but this could have been a coincidence, because the second case was not very good at all with the use of the same parameters. The parameters that were found in the second case were completely different compared to the first case.

The developers are trying to build a database of the parameters of all types of pipes (diameter, material, roughness etc.) that can be directly used to make a prediction of a flushing act. The results of the two cases here are not really useful for this database. This was the first time that the PODDS model was tested on a flushing act on a very large diameter pipe: maybe the model is better suited to be used in small distribution systems. Perhaps the resulting parameters of the first case were good and the parameters of the second case not, or vice-versa. This is possible because the second case was a very long pipe, with diameters ranging from 375 to 494 mm and a 'tidal point' in a large part of the simulated pipe. The differences in the pipe parameters and hydraulic changes could be the explanation of the poorly simulated second case.
For now the application of PODDS is not recommended, because a lot of things are unclear about the theory of the model and the results of the testing are not satisfactory. The mathematical background leads to more questions about the different processes involved in discoloured water. Based on the two cases the conclusion can be drawn that the curve-fitting of the model can also lead to coincidental numbers of the parameters.

6 Case analysis: Supply Area 'de Laak' with PSM

6.1 Introduction

The PSM program is tested with the help of the supply area of 'De Laak' . The difference with the test performed with PODSS is that not individual pipes are examined: the complete network is analyzed with the help of PSM, with the maximum number of pipes and nodes possible. The distribution of sediment over the network is predicted with PSM.

A condition to simulate with PSM is the concentration of sediment from the treatment plant. This concentration is determined in paragraph 6.3. Another conditions for the running of PSM is the determination of sediment characteristics as they are previously described. From the results of the test rig at TU Delft (Lut, 2005) the sediment characteristics have been derived. These settings have been used to simulate an initial sediment distribution, after that different parameters are used to see if the distribution was affected.

6.2 Setting up PSM

The network has been converted from a hydraulic program (ALEID in this case) to the PSM program. As input of PSM the normal daily hydraulic regime of all the pipe is inserted, as for the data of all the nodes and pipes. This is a limitation to the use of the program because the hydraulic situation of a network is changing over the week and over the year. To be able to use it in practice different scenarios have to be calculated and this is still a time consuming process because it requires to set up the entire network for each scenario. Different hydraulic conditions can occur because of changing demands by users, but also because of changing parameters like pipe diameters, roughness of pipes etc. This means that to make a good prediction the data of the model of the network must be well known. In drinking water networks reservoirs are usually placed in the network to be able to store water, this is not dealt with separately in the model, it assumes that sediment coming from the treatment plant is not stored inside of these reservoirs but transported (or suspended) in the pipes. The supply area of 'De Laak' is equipped with such a reservoir, a so-called 'Suppletie Pompstation' (SPS). This reservoir is filled during the night and water transported from this SPS is pumped into the networks with pumps. When these pumps are started an acceleration of the water will occur, leading to resuspension of particles present in the pipes adjacent to the pumps. Until now PSM is limited to only 3000 pipes and nodes that can be entered in the model. This is why the model of the supply area of 'De Laak' is simplified. Figure 6-1 shows the network with pipes left out for the western part of the supply area. These pipes do not influence the transportation of the sediment to the city of Gorinchem, where this test focuses on. The demands of all the nodes were kept the same as with the complete model, so the same hydraulic conditions are used as they would have been in the complete model.

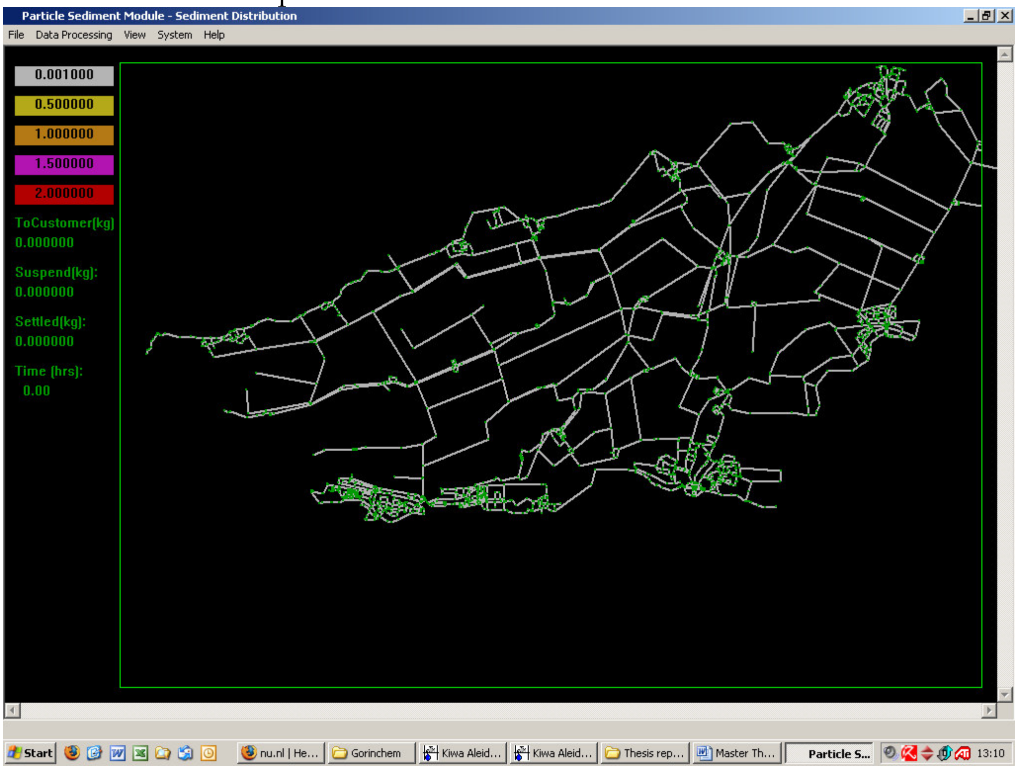


Figure 6-1: Simplified model with maximum number of pipes, including pipes near Gorinchem

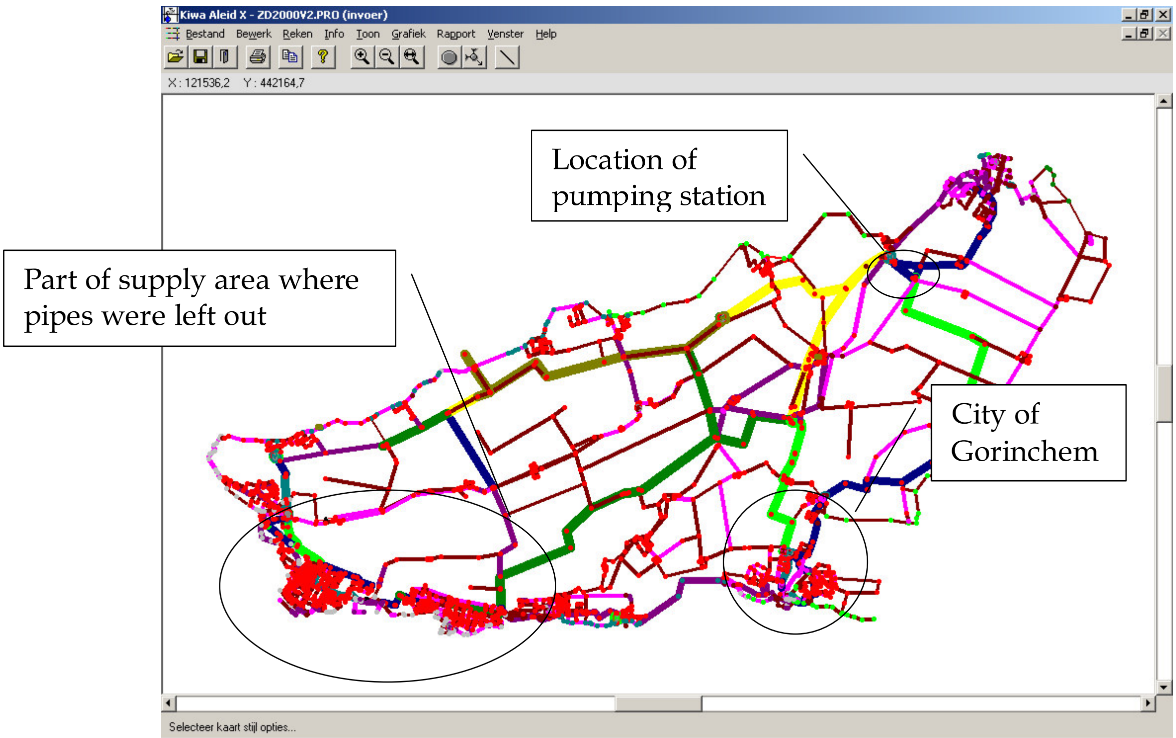


Figure 6-2: ALEID model of supply area of De Laak

The supply area is not completely distributed with water from 'De Laak'. A small part in the south is fed by pumping station " 't Kromme Gat" in Hardinxveld-Giessendam. This can be seen in the (colour) pictures in the Appendix, where the pipes in the city of Hardinxveld-Giessendam are completely not fouling, this also shows that the model is correct and that the origin of the water is PS 'De Laak'.
PSM needs the concentration of the particles entering the network at the treatment plant. Is also needs the sediment characteristics: velocities at which the sediment suspends/ deposits and the settling velocity of the sediment itself. The amount of sediment is determined with the help of a filtration experiment performed at the treatment plant, the sediment characteristics are determined with the help of experimental data collected at test rig experiments performed at TU Delft.

A short manual to run PSM is given in Appendix G.

6.2.1 Limitations PSM

There are some limitations to the PSM program:

- Hydraulic model is limited.
The model can only calculate with one set of hydraulic data at a time, if different day patterns are used the program has to be started all over with the new data. But the suspended and deposited sediment from the first run can be transferred to the next run(s), so this makes different scenarios possible but time-consuming.

- Only one type of sediment can be entered.
Although the sediment can be entered at different locations in the network (besides at the treatment plant) different types of sediment cannot be entered simultaneously. The characteristics of the sediment are based on the velocities: these velocities have to be changed each time a new type of sediment is entered. The problem is that there is no distinction made in the mass of sediment that settles, there is no 'memory' of what type of sediment had deposited. This makes it very difficult to combine different sediment types.

- Deposited sediment is shown as mass.
This may look like an advantage, but it is more important to know the amount of deposited sediment per meter pipe length. The deposited sediment is shown as an absolute number of sediment present in the pipe.

6.2.2 Benefits PSM

There are also some advantages to the PSM program:

- Provides a quick overview of deposited sediment
Based on the average sediment concentration and characteristics an overview can be given of the distribution of sediment in the network.

- Different fouling and re-fouling scenarios can be calculated

The processes in the pipe are stored in a file is accessible and can be changed using a text editor or Excel. This gives the opportunity to calculate re-fouling of (a) pipe(s). The initial fouling can be calculated, some pipes can be cleaned (by changing the deposited sediment in the pipe(s) to 0) and a second simulation can be performed to predict the re-fouling of the pipe.

6.3 Sediment load treatment plant 'De Laak'

6.3.1 Introduction

Treatment plant 'De Laak' is used to test the determination of the sediment load to a network. The information of the sediment load of this treatment plant is used in this chapter to model the distribution of sediment over the network with the help of the PSM model. The pumping station supplies water to approximately 300.000 connections in a large part of the province of 'Zuid-Holland'. It is located in the middle of the Netherlands and is owned by the water company Hydron ZH.

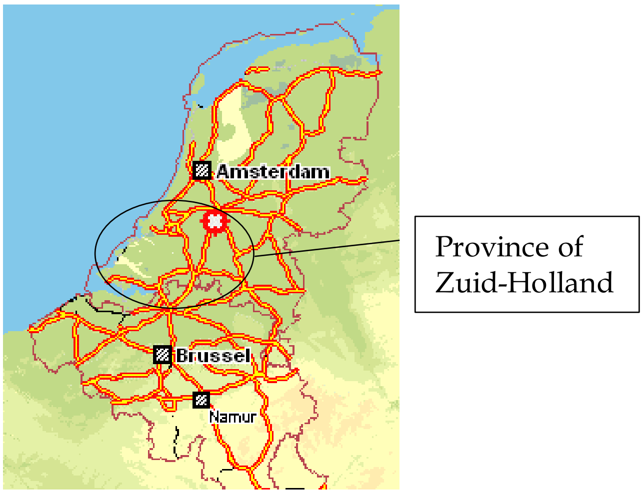


Figure 6-3: Map of the Netherlands

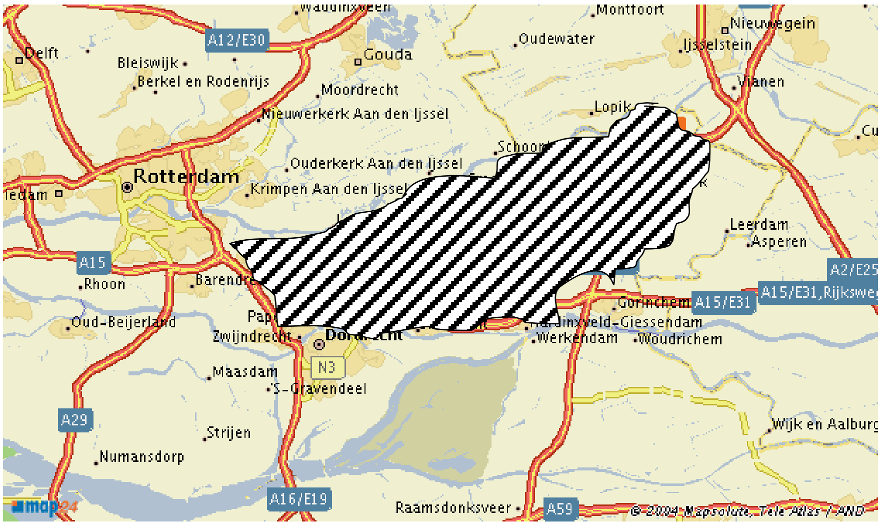


Figure 6-4: Supply area PS 'de Laak'

PS 'de Laak' is a pumping station that uses borehole water as source water. The treatment process is shown below in picture 6-5.

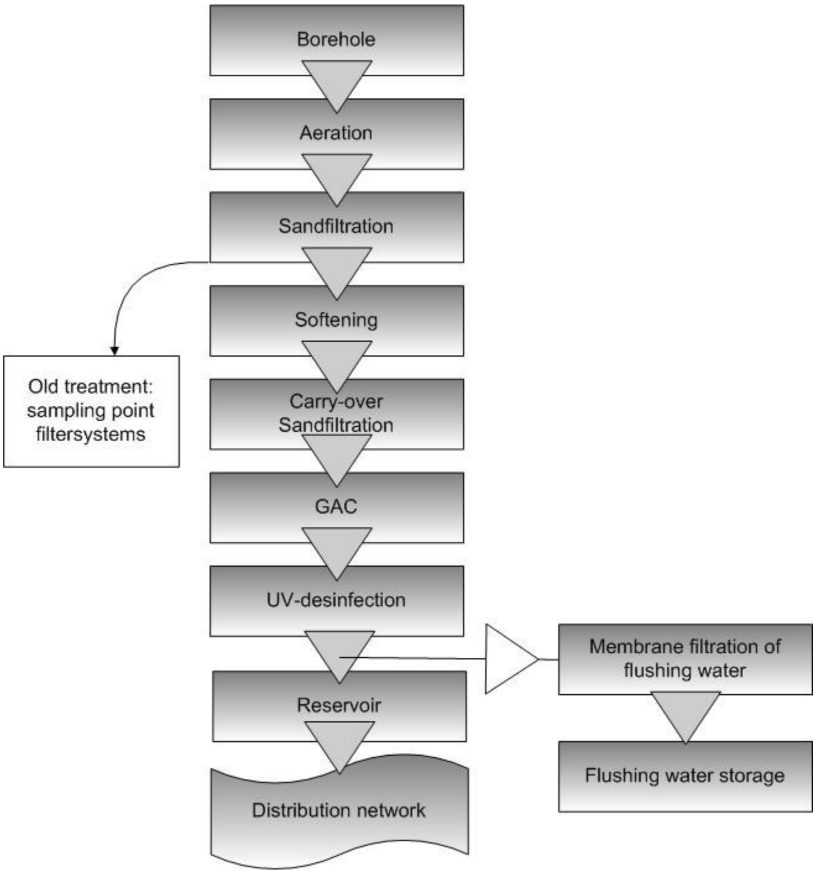


Figure 6-5: Treatment process PS 'De Laak'

As mentioned in paragraph 2-2 the plant is one source of sediment introduced to the network. During this research the treatment plant is

considered to be the onbly source of sediment . The measuring of the sediment load has been investigated by Gauthier (2002) , his conclusion is that a large part of the sediment is of biological origin. The determination of particles of this treatment plant is not divided into different categories, it is assumed that all of the filtered particles are non-cohesive particles.
The determination of the sediment load of PS 'De Laak' has been done using the following equipment:

- TrueDos pump, constant flow, variable pressure
- Stainless steel filter holders (Ø 47 mm)
- Membrane filters, pore size 0.2 µm
- Pressure meter
- Overflow device, to keep constant pressure

Figure 6-6: Filter system PS 'De Laak'

The TrueDos pumps are used because they can be adjusted to keep a constant flow through the filter. Because of fouling of the filter the flow through the filter is reduced. This is automatically adjusted by the pump by increasing the pressure with a maximum pressure of 14 Bar. The exact amount of filtered water can be calculated by multiplying the time with the established flow. The filter system was placed in line with a Dr. Lange turbidity meter and connected to the outgoing water flow of the first treatment. Until 2002 the treatment of 'De Laak' only consisted of aeration and sand filtration (see Figure 6-5). To adapt the data of these filter experiments to PSM, only the load of the old treatment process is measured. This is because the main contribution to the sediment accumulated in the network is caused by this 'old' sediment. A prediction of the distribution of the sediment in the network can be given. The turbidity meter was used to determine if strange peaks of turbidity appeared during the filter tests.

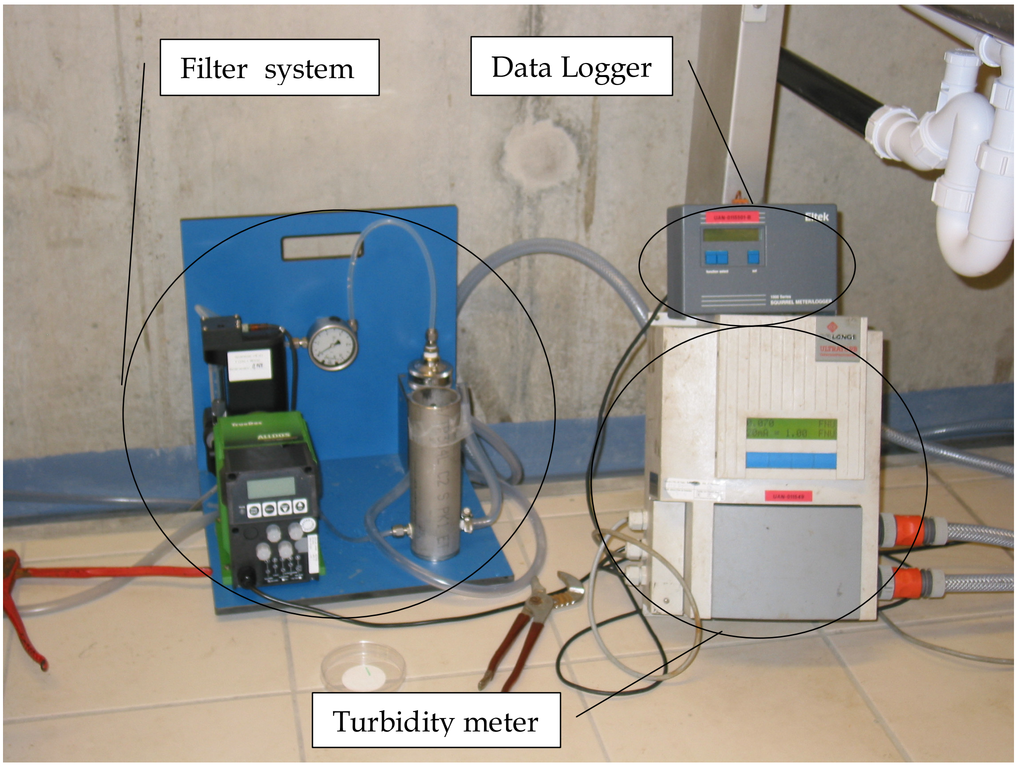


Figure 6-7: Measuring setup PS 'De Laak'

To measure the sediment load the filters were weighed and dried before filtering and afterwards dried and weighed again. The total weight of the sediment is measured and calculated to the concentration of suspended solids in the water.

6.3.2 *Detection limit*

The filter systems are designed by Kiwa Water Research, in order to be able to interpret the results the detection limit of the systems is determined. The detection limit is needed to know what types of low concentration can be measured: values below this limit are not reliable.

The complete calculation of this limit is done in Appendix B , the smallest value was found to be *24 mg/m³*. Some results were rejected because visible dirt was detected on the filters, this dirt may have been present inside of the pumps and/or overflow. The detection limit was found by filtering approximately 10 litres of so-called milli-Q water (water with a very low amount of particles) with the help of the filter systems. This detection limit can be lowered by using even larger volumes of filtered water, although the values of the filtered water at 'De Laak' where above this detection limit. When these filter systems are placed at a membrane filtration treatment plant the concentration of the particles will be lower, probably close to the detection limit. In this case the detection limit has to be determined again with a larger volume of filtered water.

6.3.3 *REWAB figures De Laak*

The REWAB figures (water quality measurements for different substances) for De Laak are analyzed to see what the contribution to the sediment load is of the oxides of Fe, Mn and Al.

	Pumping station	Outgoing 10 ^6 m^3	Fe µg/l	Fe as SS production µg/l	Mn µg/l	Mn as SS production µg/l	Al µg/l	Al as SS production µg/l	Total load SS mg/m^3
1999	Zuiveringsstation De Laak	9.5	40	0.076	20	0.032	1	0.003	0.111
2000	Zuiveringsstation De Laak	9.966	20	0.038	20	0.032	3	0.009	0.079
2001	Zuiveringsstation De Laak	10.042	26.3	0.050	30	0.047	1	0.003	0.101
2002	Zuiveringsstation De Laak	8.4	18	0.03439	11	0.01740	2	0.00578	0.058
2003	Zuiveringsstation De Laak	8.3	21	0.040	13.1	0.021	2	0.006	0.067

The concentrations of Fe, Mn and Al are converted into their respective oxides as shown in paragraph 2.7.4. The concentration of SS caused by the FeMnAl oxides ranges between 0.08 and 0.11 mg/m³. This result can only be seen as an indication because a large variation can occur in water quality. As the figures of 1999 until 2003 are shown, a change in water quality can be distinguished. Until 2002 the treatment was really simple, from that year the treatment has been extended with softening, extra sand filtration, GAC and UV disinfection . This can be seen in the figures: the total load has decreased since 2002.

6.3.4 *Results*

The concentration of the outgoing water has been measured two times during at least 24 hours. Because the type of filter that was used was very small (0.2 µm) the limitation was the clogging of the filters. Therefore the maximum possible volume of water that could be sampled was around 50 litres. To determine the fluctuation in water quality an online (Hach Lange) turbidity meter was placed at the treatment plant to determine the turbidity pattern of the water coming from the pumping station. During filtering the turbidity meter was placed in line with the filter system to detect possible strange peaks. The first measurement showed a quite stable turbidity pattern, while the second measurement showed a large peak. This peak did not lead to a much higher concentration of the sediment load.

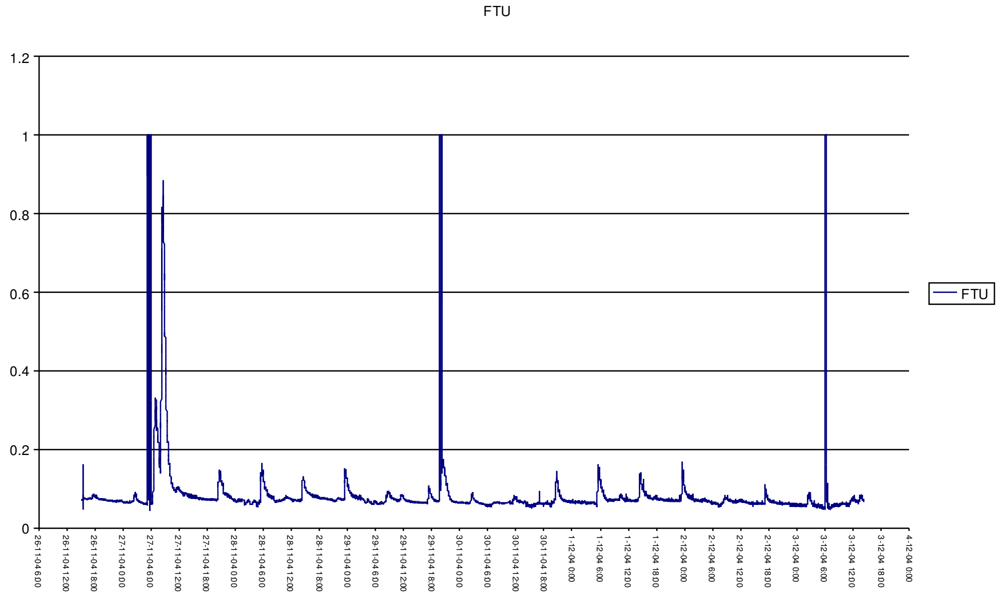


Figure 6-8: Turbidity measurment PS 'De Laak' during one week

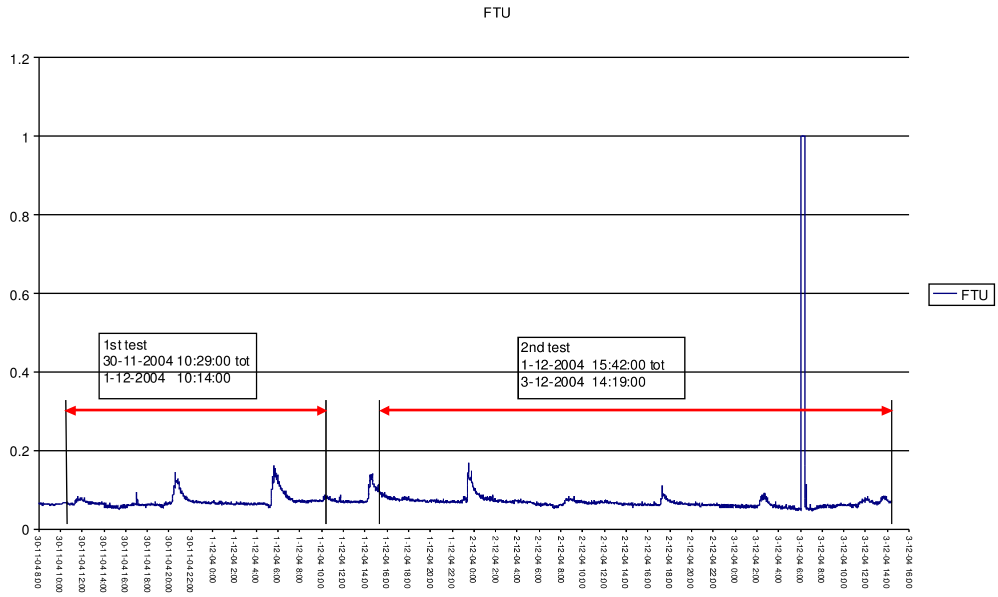


Figure 6-9: Turbidity measurements during filtering PS 'De Laak'

Test 1 was performed with a fixed filter flow of 2 liters per hour, during almost 24 hours, resulting in a total amount of filtered water of 47,50 liters of water and the total of filtered suspended solids 7.27 mg. The averaged concentration for 24 hours of the water for test 1 at PS 'De Laak' resulted in ***153 mg/m³.***
Test 2 was performed at a lower flow (1.5 liters per hour instead of 2 liters per hour). This lower flow was to be able to filter as much water as possible. The filtration experiment was conducted for a little over 46 hours (almost two days). This resulted in a total volume of 70.3 liters and a weight of 9.85 mg of filtered suspended solids. The concentration of SS found at experiment 2 was ***141 g/m³.***

During the filtering of the water a sample was taken to see what kind of particles were present in the water. This sample was taken at 13:00 on December 3^{d}, at a normal level of turbidity. Although the water was not examined on all possible substances like ATP, Fe/Mn/Al, the water was examined with a particle counter to see what size of particles were present in the water. In Figure 6-10 the distribution of the particles can be seen. The smallest particles are the largest fraction in the water, but when these amounts are calculated into their volumes their volume is much smaller than the volume of the larger particles present in the water, Figure 6-11 shows this.

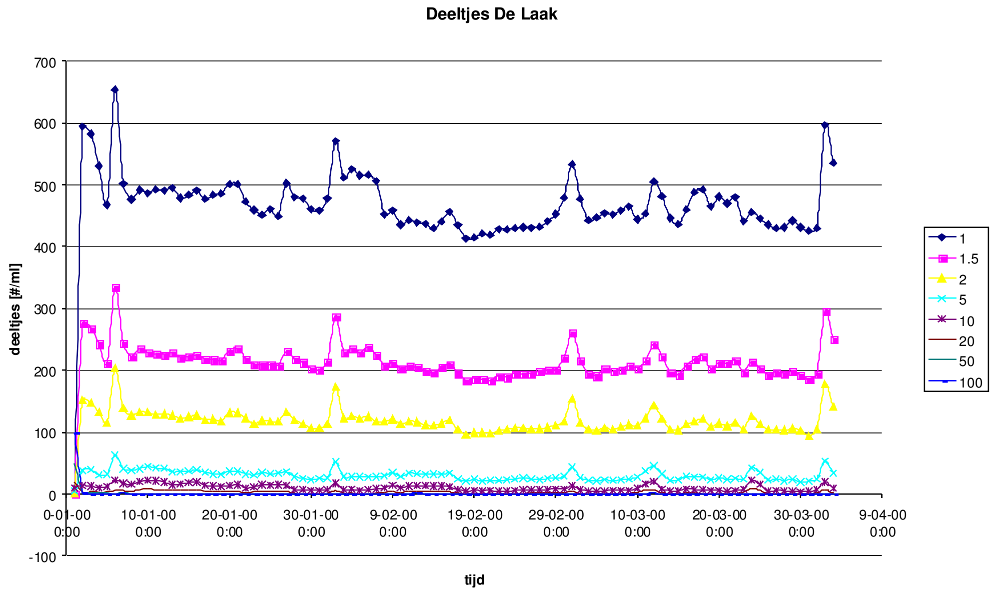


Figure 6-10: Particle size distribution water PS 'De Laak'

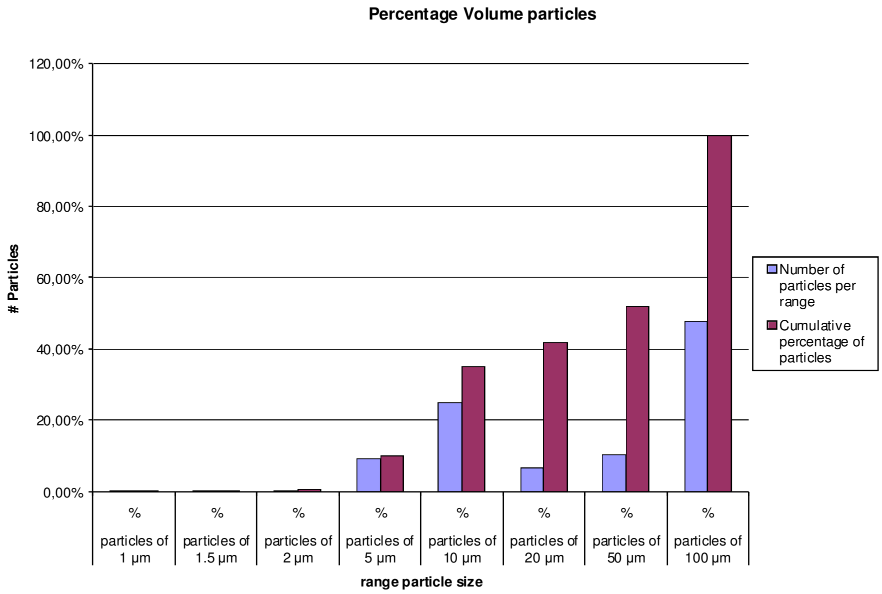


Figure 6-11: Volumes of particles PS 'De Laak'

The average volume of the particles can be calculated assuming that the particles are spheres and their volume is described with the help of the formula Volume = 32/3 *π*D³ . When the volumes of all ranges per m³ are calculated the total average volume per m³ can be calculated by summing up all the individual volumes. As the total mass of the particles per m³ is known, the average specific weight of the particles can be calculated with the help of Equation 46.

$$M = \rho * V$$

$$\left[\frac{mg}{m^3 water}\right] = \left[\frac{kg}{m^3 particles}\right] * \left[\frac{m^3 particles}{ml, water}\right] \qquad \textit{Equation 46}$$

The total average volume of the particles is 1.175 $*10^{-13}$ m^3 per ml of water and the average SS concentration found at the filtration experiments is 150 mg/m^3 water. The volume of the particles in 1 m^3 of water is 7.52$*10^{-6}$ m^3. This leads to a specific weight of the particles of :

$$\rho = \frac{150 * 10^{-6} kg}{1.175 * 10^{-7} m^3} = 1277 kg / m^3$$

This is not completely in the line of expectation, usually the density of the flocks are around 1030 kg/m^3. This is only a rough calculation because not all of the particles are measured with the particle counter, they are categorized into ranges. The assumption that all particles are spheres is also probably not correct, the calculation of the volume of the particles will be different if these are not round.

6.3.5 *Conclusion sediment load De Laak*

As a conclusion can be drawn that it is possible to determine the sediment load of a treatment plant to a drinking water network. An average value of the concentration of approximately 150 mg/m3 was found for the old treatment process of pumping station 'De Laak'. By measuring the quantity of particles in the water sample, the average specific weight is determined, these values will be used in the case with the PSM program. During a M.Sc. research (Lut; 2005) on the behaviour of particles in drinking water networks tests were performed on a test rig using three types of sediment. These three types were $FeCl_3$, Kaolinite and sediment collected at a fire hose during a flushing operation. $FeCl_3$ has a specific weight of approximately 1200 kg/m^3 and Kaolinite around 2600 kg/m^3 (Lut, 2005) , the specific weight of the flushing material was not exactly known. The suspended solids present in the water of pumping station 'De Laak' are quite similar to $FeCl_3$ as used in the experiments of Lut. Further data of these experiments will be used for calculations with the PSM program.

6.4 Determining of sediment behavior

As mentioned in chapter 4 the sediment characteristics (u_s, u_{rs} and u_d) of the sediment are used to make a prediction with the help of PSM. These characteristics have to be determined for the type of sediment in the water of PS De Laak. During the research performed in Australia, these sediment characteristics were analyzed with the help of a test rig and a water tunnel. Almost identical tests were performed at Delft University (M.Sc. report, Lut; 2005) to determine the hydraulic behavior of particles in a test rig. In this report different types of sediments were tested, leading to more insight of the settling speed, amount of resuspension and speed at which the sediment would start to deposit. As we have mentioned before the Iron Chloride ($FeCl_3$) particles used in the test rig in Delft were quite similar to the particles in the water of PS De Laak when it concerns the specific weight: 1280 kg/m^3. The test rig was modelled in PSM to determine the velocity (u_s) at which the particles settle in the pipe.

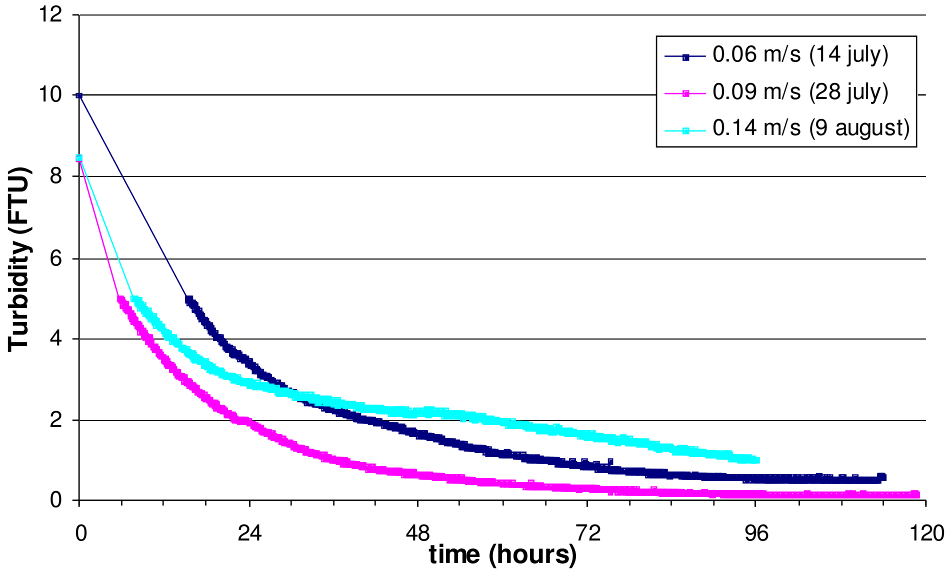


Figure 6-12: Turbidity during experiments with $FeCl_3$

In the Delft test pipe experiment the turbidity of the outgoing flow was measured. The dosing of $FeCl_3$ took place at the start of the experiment and led to a turbidity of approximately 9 FTU (at flow velocity, u = 0.09 and u = 0.14 m/s) and 10 FTU (u = 0.06 m/s). The test pipe consisted of a pipe with a length of approximately 4,5 m and a diameter of 100 mm, the water was constantly pumped through this pipe and transported through much smaller pipes (50 to 63 mm outer diameter) to avoid settling of particles.

The test rig is modelled with the help of the PSM program, the large 100 mm 4,5 m test pipe and the smaller transport pipes are inserted into the model.

By reproducing the experiments the settling velocities are determined for the three flow velocities from Figure 6-12. In these experiments the only velocity that is determined is the settling velocity (u_s), the others (u_d and u_{rs}) are not dealt with and are determined from the results of the MSc report of Lut.

As an indication of the settling velocity a small calculation is made to start with:

$x = v*t$
$x = 100*10^{-3}$ m
$t = 72$ hours $= 259200$ s
$v = 100*10^{-3} / 259200 = 3.8*10^{-7}$ m/s

The test rig has been built in the PSM program and different settling velocities are tried to find the matching velocity for the experiments.

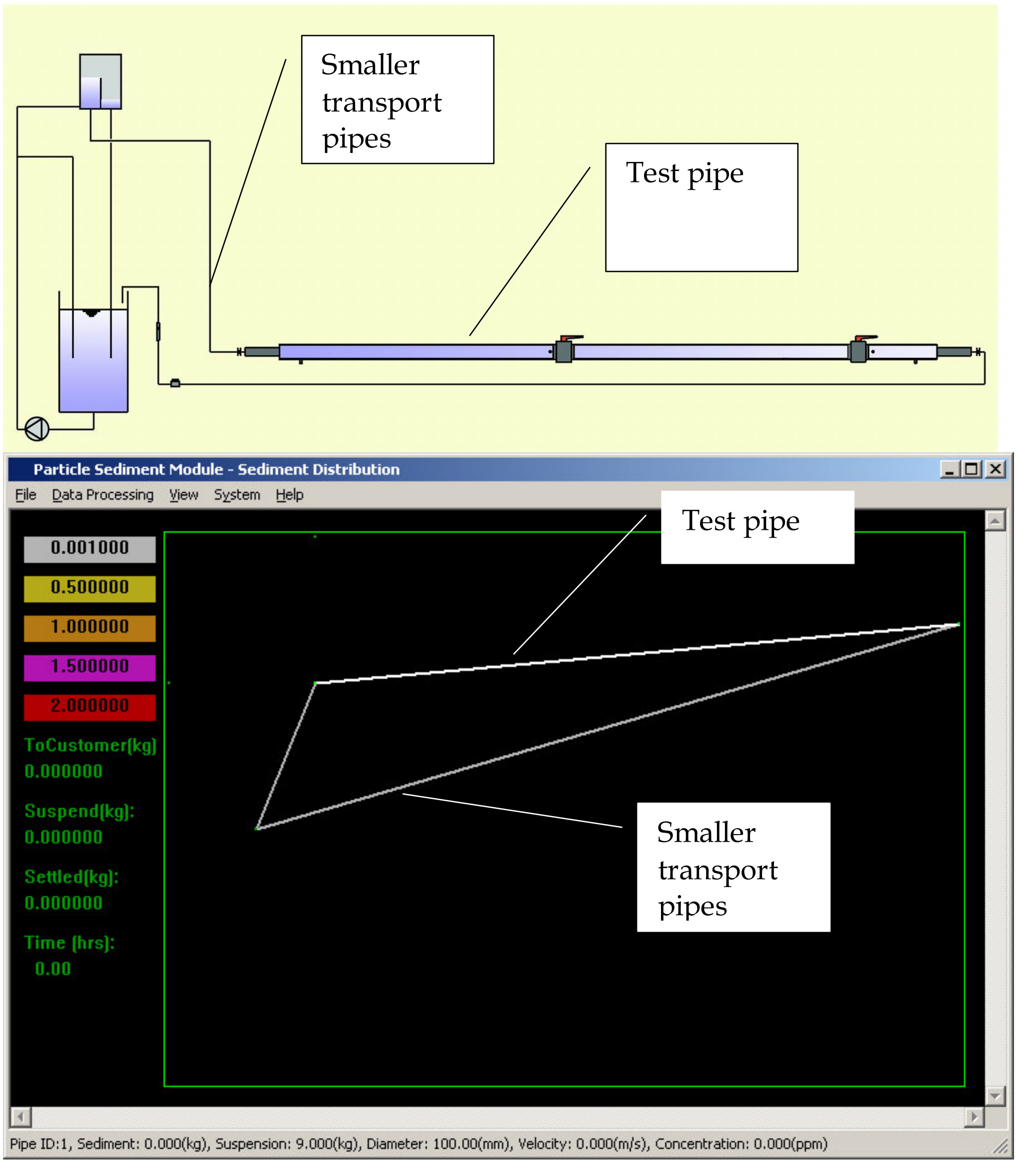


Figure 6-13: Original test rig and modelled test rig in PSM

The suspension velocity was determined for the three experiments by choosing the amount of particles (in kg sediment) the same as the turbidity (FTU). The results of the different experiments can be found in Table 8.

Table 8: Reproducing experiments test rig

u [m/s]	Turbidity at 0 hours [FTU]	Turbidity after 72 hours [FTU]	Sediment in suspension at 0 hours [kg]	Sediment left in suspension after 72 hours [kg]	u_s [m/s]
0.06		+/- 0.3	+/-0.30	0.315	$5.5*10^{-6}$
0.09		+/- 0.15	+/-0.15	0.147	$5.5*10^{-6}$
0.14		+/- 0.6	+/-0.60	0.597	$2*10^{-6}$

From the experiments of Lut the velocity at which particles start to deposit (u_d) was found to be at least 0.15 m/s. The velocity at which the particles start to resuspend (u_{rs}) was found to be 0.25 m/s. These values are used to make a prediction for the whole network of the supply area of PS De Laak.

6.5 Case analysis of De Laak

To make a prediction of the distribution of sediment in the network, all available parameters are known. The amount of particles was taken lower than the concentration that was found during the filter experiments. This is because of an initial wrong calculation that led to this lower concentration. The results are only slightly influenced by this lower value: the differences between the fouled and less fouled regions are not changed, only the total sediment inserted into the network is lower.

- Amount of particles from treatment plant: ***106 ppm***
- Deposit velocity of sediment: $u_s = 2\text{-}5.5*10^{-6}$ ***m/s***
- Velocity of water at which sediment will start to deposit: $u_d = 0.15$ ***m/s***
- Velocity of water at which the sediment will start to resuspend: $u_{rs} = 0.25$ ***m/s***

According to the Aleid model of Hydron of an average day, the velocities will sometimes exceed 0.15 m/s, but most pipes reach velocities between 0.05 and 0.15 m/s. This is why the parameters of the velocities are chosen:

- Dosing concentration at PS: ***106 ppm***
- $u_s = 5.5*10^{-6}$ ***m/s***
- $u_d = 0.15$ ***m/s***
- $u_{rs} = 0.25$ ***m/s***

The constant concentration is inserted at the pumping station, see Figure 6-14. From this point the sediment is distributed over the network.

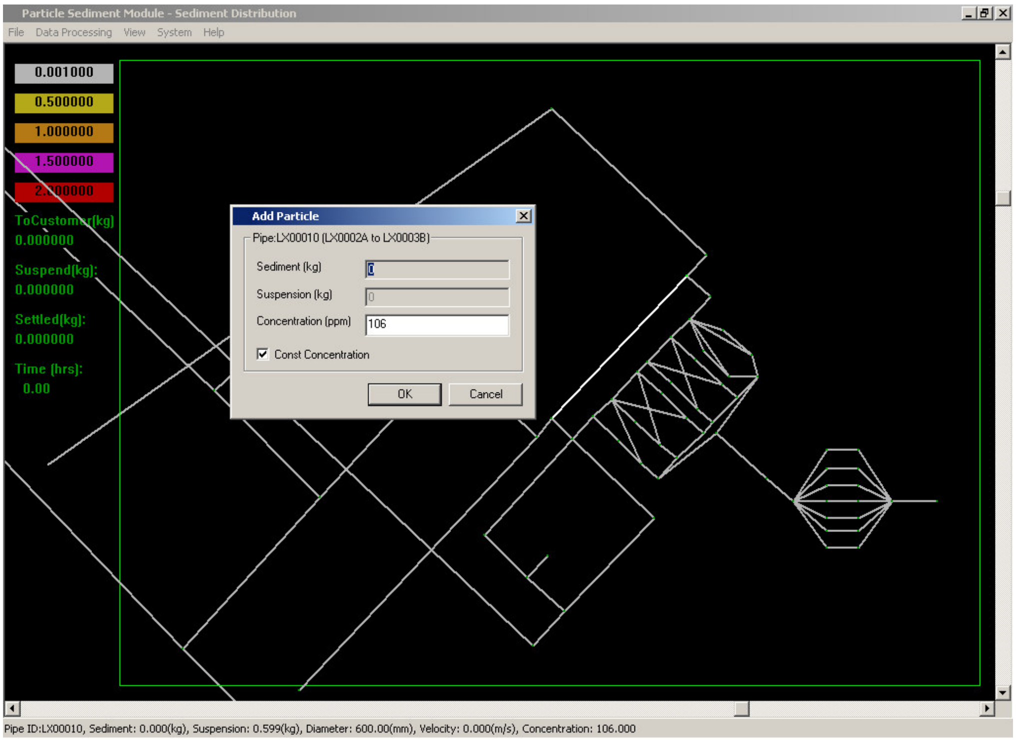


Figure 6-14: Constant concentration at pumping station

The results of 1 week of simulation with the given parameters can be seen in Figure 6-15. To be able to see differences in sedimentation in pipes, only a small part (near the city of Gorinchem) is shown. The total mass of deposited and suspended sediment is shown in Table 9.

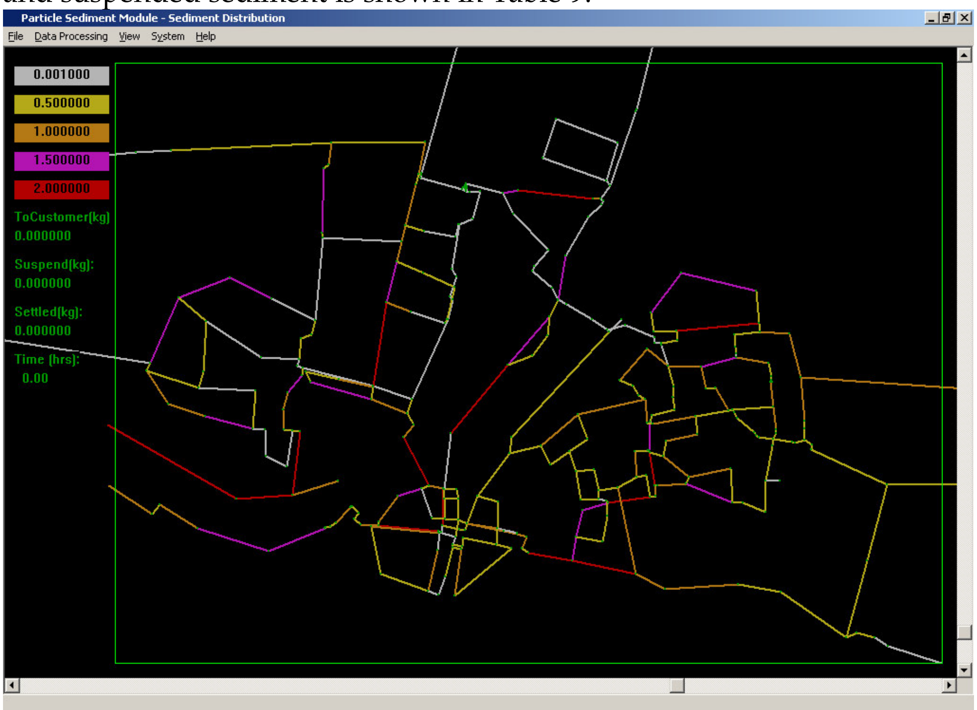


Figure 6-15: Distribution of sediment, 1 week simulation, , detail of city of Gorinchem

Table 9: Total mass settled and suspended 1 week initial settings

PIPEID	MassSettled(kg)	MassSuspended(kg)
kg	1138.729088	344.929146

The average demand of the supply area is 2434.61 m^3/h, the total supplied quantity of sediment and the sediment per meter pipe are shown in Table 10.

Table 10: Total sediment load PS De Laak

Total average demand	***SS concentration***	***Total sediment inserted in network***			
m3/h	mg/m3	kg per day	kg per week	kg per month	kg per year
2434	106	6192.10	43344.67	185762.88	2260115.04
Total length of network	***Sediment per meter pipe (theory)***				
m	kg/m				
795 435.60	2.84				

The simulation has been extended to a total simulation time of 4 weeks. Because the hydraulic data of the model are not changed during the simulations the behaviour of the sediment is the same, this means that the amount of sediment that settles in four weeks in pipe X is roughly the same as four times the sedimentation of 1 week. It was found that there were small differences in the calculation factor, not exactly 1:4. The conversion from four weeks to one year has been calculated by multiplying four weeks with thirteen to make 52 weeks. Figure 6-16 shows the distribution of sediment over the network after one year, according to the calculations based on the four weeks distribution calculations.

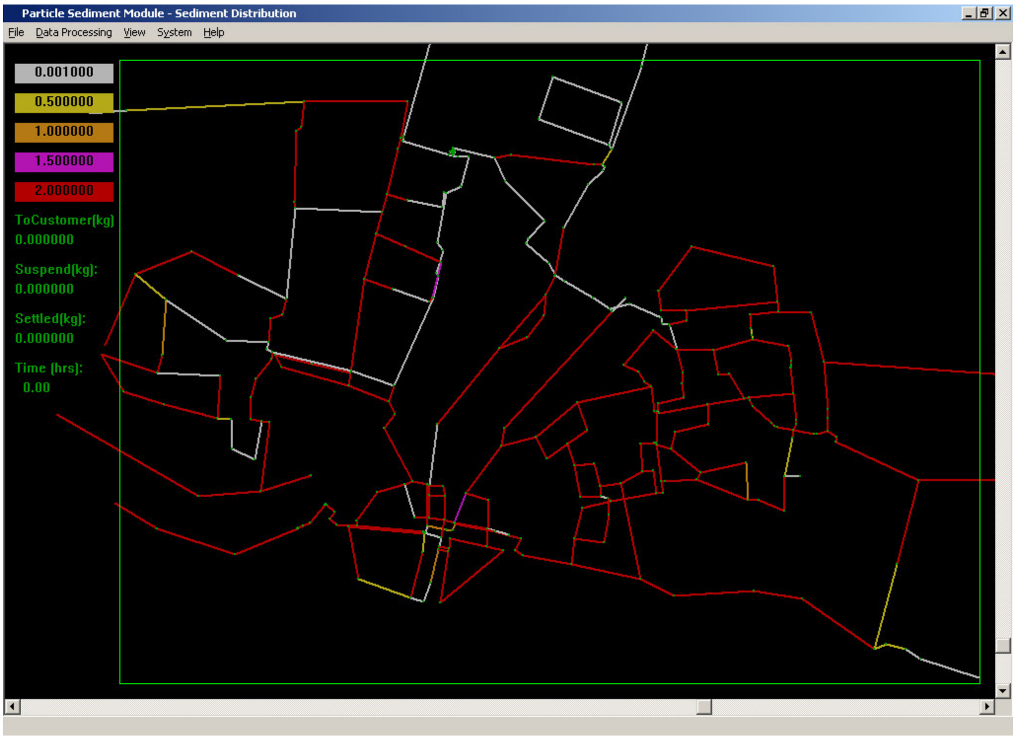


Figure 6-16: Distribution of sediment, 52 weeks simulation, detail of city of Gorinchem

Table 11: Total mass settled and suspended 52 weeks initial settings

PIPEID	MassSettled(kg)	MassSuspended(kg)
kg	62316.9	344.9291

Appendix G shows the calculations of the conversion factors and the simulation results of 1, 2, 3 and 4 weeks. The distribution of sediment over the complete network can be seen in Figure 6-17.

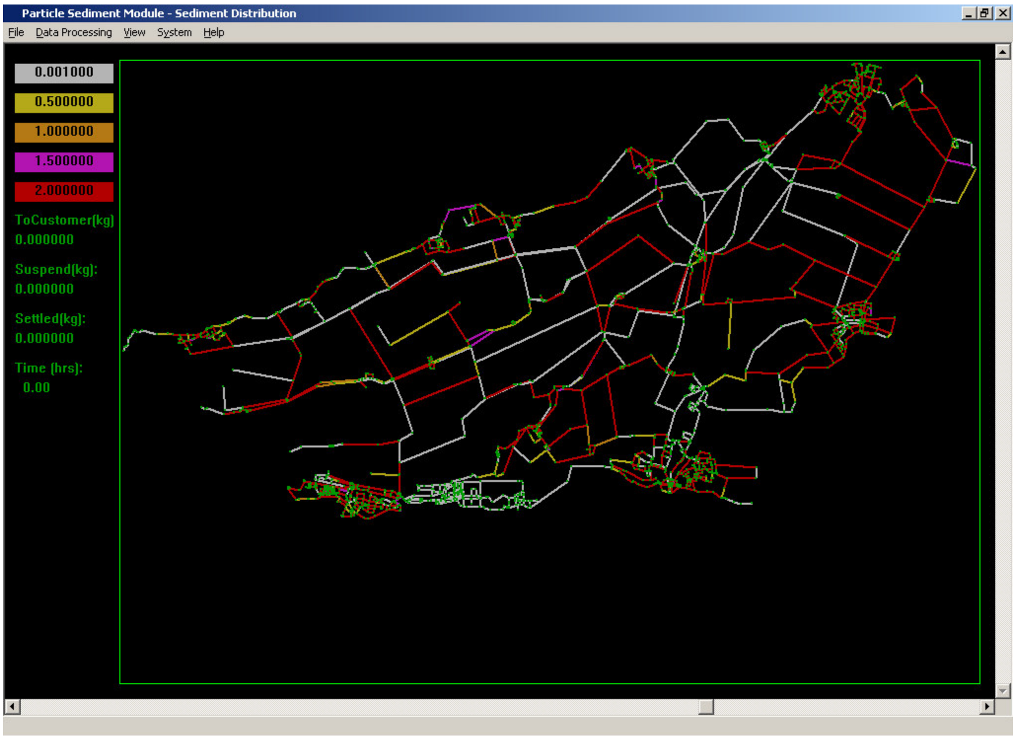


Figure 6-17: Distribution of sediment after 1 year, complete supply area of PS De Laak

6.5.1 Legend of PSM

PSM calculated the total of suspended sediment and the sediment in suspension (not yet suspended). In this report the calculations are only performed to determine the mass of *suspended sediment.*

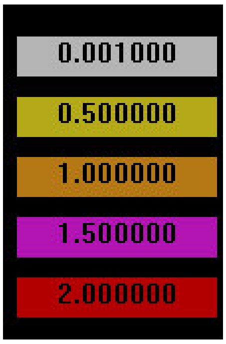


Figure 6-18 shows the colour legend of the masses of suspended material. This is the total mass of suspended sediment in the pipe. This legend is not scaleable, so there is no (visual) distinction between a pipe containing 2 kg or 400 kg suspended sediment.

6.6 Case analysis with different parameters

In section 6.4 the distribution of sediment from the treatment plant is calculated with parameters (u_s, u_{rs}, u_d) for the behavior of sediment that were found during experiments with the test rig. Now will be tested if the distribution of sediment will change a lot if these parameters (u_s, u_{rs}, u_d) are slightly changed. The distribution of sediment of one year is simulated. Because it is difficult to compare the complete network two small sections are looked at more closely: the first near Gorinchem (area that was also used in the PODDS cases) and the second closer to the pumping station (in the North-east of the supply area, city of Vianen) . The pictures of the distribution of sediment over the complete network can be found in Appendix G. Two different pipes at the city of Vianen and at Gorinchem have been chosen to quantify the amount of settled sediment :

- pipe VI100195 (Vianen)
- pipe GC0025 (Gorinchem)

6.6.1 *Scenario 1: change of u_s*

u_s is the velocity with which the sediment deposits. If u_s is lowered this will lead to a slower settling of the particles and a different kind of distribution of sediment over the network. As sediment will settle much more slow, the sediment will settle further away in the network. If u_s is raised this will lead to quicker settling of sediment and more sediment close to the pumping station.

HYPOTHESIS	
$u_s \downarrow$	$u_s \uparrow$
Slower settling of sediment	Quicker settling of sediment
Sediment further from PS	Sediment closer to PS
Less sediment deposited in network	More sediment deposited in network

Change u_s from $5.5*10^{-6}$ to $1*10^{-7}$ m/s

SETTINGS	
u_s	$1*10^{-7}$ m/s
u_d	0.15 m/s
u_{rs}	0.25 m/s

Lowering us to $u_s = 1*10^{-7}$ m/s leads to a different distribution of sediment after one year, as can be seen in the figures in Appendix G.
Near Vianen the deposited sediment is almost the same as in the initial situation, some parts are even cleaner than they were in the initial situation.

But near Gorinchem the differences are larger: here it can clearly be seen that there is much less sediment deposited. The hypothesis looks right: sediment is settling slower, but the sediment is not deposited further from the PS.

Table 12: Settled and suspended mass compared to initial situation, u_s ↓

INITIAL	MassSettled(kg)	Deposited per meter pipe (kg/m)	MassSuspended(kg)	In suspension per meter pipe (kg/m)
	62316.9	0.078343112	344.9291	0.000433635
u_s ↓	MassSettled(kg)		MassSuspended(kg)	
	1810.415	0.002276004	421.1971	0.000529518

Change u_s from $5.5*10^{-6}$ to $1*10^{-5}$ m/s

SETTINGS	
u_s	$1*10^{-5}$ m/s
u_d	0.15 m/s
u_{rs}	0.25 m/s

The hypothesis says that the sediment will settle faster, this can be seen in the figure: near Vianen and Gorinchem there is more sediment that has been deposited. Sediment closer to the PS is the other part of the hypothesis, this can be seen in the figures and in table 13. The total sediment that is deposited

Table 13: Settled and suspended mass compared to initial situation, u_s↑

has also increased, this is according to the hypothesis.

INITIAL	MassSettled(kg)	Deposited per meter pipe (kg/m)	MassSuspended(kg)	In suspension per meter pipe (kg/m)
kg	62316.9	0.078343112	344.9291	0.000433635
us ↑	MassSettled(kg)		MassSuspended(kg)	
kg	93805.81	0.117930113	401.5568	0.000504826

Table 14: Deposited sediment for change of u_s

	VI100195 (close to PS)	GC0025 (further from PS)
	[kg]	[kg]
us = $5.5*10^{-6}$ m/s	0	0
us = $1*10^{-7}$ m/s	0.231	0
us = $1*10^{-5}$ m/s	23.14	0

6.6.2 *Scenario 2: change of u_d*

If the parameter u_d is changed, it means that the flow velocity of the water at which the sediment starts to deposit is changed. When this velocity is lowered it will lead to more sediment that suspends. This would lead to a different kind of distribution of sediment over the network, because during this case the only source of sediment is the pumping station. When the sediment from the PS settles sooner than it did in the first case, less suspended solids will 'reach' the pipes far off of the pumping station.
If the velocity u_d is increased this will lead to less material settling, with less material settling near the treatment plant and more further away in the network.

HYPOTHESIS	
u_d ↓	u_d ↑
More sediment in network	Less sediment in network
Sediment concentration 'closer' to PS	Sediment concentration 'further' from PS

Change u_d from 0.15 to 0.10 m/s

SETTINGS	
u_s	5.5e-6 m/s
u_d	0.10 m/s
u_{rs}	0.25 m/s

The figures in Appendix G show the distribution of sediment for the system with changed parameter u_d. The hypothesis is proven that there is more sediment in the network: it shows that almost all pipes are red It could also be concluded that there is more sediment closer to the PS, because near Gorinchem there are a few pipes that are fairly 'clean'; near Vianen (close to PS) this is not the case.

Table 15: Settled and suspended mass compared to initial situation, ud ↓

INITIAL	MassSettled(kg)	Deposited per meter pipe (kg/m)	MassSuspended(kg)	In suspension per meter pipe (kg/m)
kg	62316.90	0.078343112	344.93	0.000433635
ud ↓	MassSettled(kg)		MassSuspended(kg)	
kg	52780.39	0.066354065	361.24	0.000454136

Change u_d from 0.15 to 0.25 m/s

SETTINGS	
u_s	5.5e-6 m/s
u_d	0.25 m/s
u_{rs}	0.25 m/s

It can be seen that the sediment distribution near Vianen: it looks that there is not less sediment in the network. In the second area it does look that there is less sediment in the network: this is according to the hypothesis. The sediment concentration further away from the PS can clearly be distinguished.

Table 16: Settled and suspended mass compared to initial situation, ud ↑

INITIAL	MassSettled(kg)	Deposited per meter pipe (kg/m)	MassSuspended(kg)	In suspension per meter pipe (kg/m)
kg	62316.90	0.078343112	344.93	0.000433635
ud ↑	MassSettled(kg)		MassSuspended(kg)	
kg	85026.50	0.106892997	312.93	0.00039341

Table 17: Deposited sediment for change of ud

	VI100195 (close to PS)	GC0025 (further from PS)
	[kg]	[kg]
u_d = 0,15 m/s	0	0
u_d = 0,10 m/s	2,66	0
u_d = 0,25 m/s	43,45	3,51

6.6.3 *Scenario 3: change of u_{rs}*

If the velocity at which the sediment resuspends is lowered, sediment that was previously suspended will resuspend. This is because during the day the flow velocity in the pipes will increase because of larger water demand. This flow velocity then exceeds the resuspension velocity of the sediment.
This means that compared to the first test the clean pipes will stay clean and the pipes that were already dirty start to foul even more.
On the opposite, when u_{rs} is raised little change will occur compared to the first test. This is because in the first case the resuspension velocity was already chosen quite high, which lead to a small quantity of resuspended material.

HYPOTHESIS	
u_{rs} ↓	u_{rs} ↑
More fouling of dirty pipes	Dirty pipes will foul a little bit more
Clean pipes become cleaner	Clean pipes will foul a little bit more
More sediment in suspension	Less sediment in suspension
More sediment in network	Less sediment in network

Change u_{rs} from 0.25 to 0.10 m/s

SETTINGS	
u_s	5.5e-6 m/s
u_d	0.15 m/s
u_{rs}	0.10 m/s

The dirty and clean pipes did not really change, Table 18 shows that the pipes have not been fouled more compared to the initial situation, this is not according to the expectations.

Table 18: Settled and suspended mass compared to initial situation, urs ↓

INITIAL	MassSettled(kg)	Deposited per meter pipe (kg/m)	MassSuspended(kg)	In suspension per meter pipe (kg/m)
kg	62316.90	0.078343112	344.93	0.000433635
urs ↓	MassSettled(kg)		MassSuspended(kg)	
kg	1654300271.49	2079.741303	49527.14	0.062264173

Change u_{rs} from 0.25 to 0.50 m/s

SETTINGS	
u_s	5.5e-6 m/s
u_d	0.15 m/s
u_{rs}	0.50 m/s

In the Appendix it shows that there is less sediment compared to the initial situation in the system. The hypothesis that the dirty pipes will foul more cannot be clearly seen, as with the clean pipes: it looks that there is more sediment in the system, this is not according to the hypothesis. The hypothesis that clean pipes are becoming cleaner is not true either.

Table 19: Settled and suspended mass compared to initial situation, urs ↑

INITIAL	MassSettled(kg)	Deposited per meter pipe (kg/m)	MassSuspended(kg)	In suspension per meter pipe (kg/m)
kg	62316.90	0.078343112	344.93	0.000433635
urs ↑	MassSettled(kg)		MassSuspended(kg)	
kg	76383.80	0.096027634	336.52	0.000423061

Table 20: Deposited sediment for change of u_{rs}

	VI100195 (close to PS)	GC0025 (further from PS)
	[kg]	[kg]
u_{rs} = 0,2 m/s	0	0
u_{rs} = 0,10 m/s	0	0
u_{rs} 0,50/s	181,1	16,36

6.6.4 Conclusion scenarios

It is clear that with a little parameter changing the distribution of sediment over the network is influenced. The largest changes were seen with the change of the velocity with which the sediment deposits (u_s).

The parameter us is probably the parameter that can be measured most easily with the help of a settlement tank or cylinder. Figure 6-18 shows a possible setup that can be easily installed at a treatment plant.

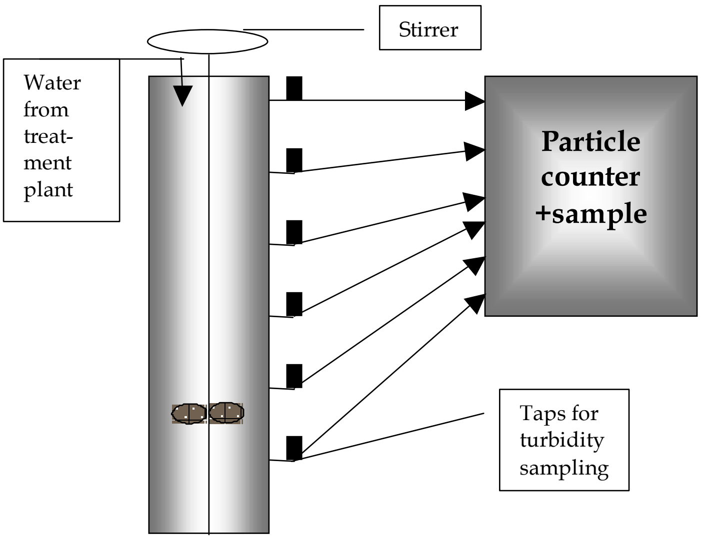


Figure 6-18: Possible set-up for determination of u_s

Fill the cylinder with water from the treatment plant and possibly (shortly, to avoid coagulation) stir the water sample. Remove the stirrer and close the cylinder and leave the sample quiet. Take samples at regular intervals and measure the amount of particles at each tap. With the help of these measurements the settling velocity of the sediment present in the water can be calculated.

It is necessary to use a cylinder that is large enough for the samples to be taken and can still easily be transported. Each sample needs at least 500 ml to analyse with a particle counter. This means 3 litres of sampling volume each time. When in total 10 samples are taken, 30 litres of water are used. This volume must not influence the further settling of the sediment, so the total volume inside of the cylinder needs to be sufficient. A rough estimation is that 10 % of the total volume can be used for sampling, this means that the cylinder must contain 300 litres of water. When a cylinder of 1 meter high is used the radius must be:

$$V = 300l = 0,3m^2 = \pi * R^2 * h = \pi * R^2 * 1$$
$$\Rightarrow R = 0.31m$$

The samples have to be taken without disturbing the column of water.

The other parameters will be more difficult to determine, as it is not easy to determine these velocities; this requires the use of a flow through test-rig with monitoring facilities. The most important parameter is the velocity at which the sediment resuspends (u_{rs}).

6.7 Conclusion PSM

PSM can be used to predict the distribution of sediment over a network. Although the program itself is not really user-friendly, an experienced user can setup a new model from a general hydraulic package to PSM within a reasonable amount of time.

The power of the program is the ability to calculate the settling, suspension and resuspension of sediment in one pipe and meanwhile combine the results of all the pipes and calculate the combined sediment that settles in the whole network. Basically all the calculations of each pipe could be done by hand, but the ability to combine all the pipes and work out an entire network with one mouse-click is very useful. In the model it looks as if the larger pipes contain more sediment than the smaller pipes, this is explained by lower velocities in the larger pipes and higher velocities in the smaller pipes.
The key aspects of this model are the hydraulic data used for the model. When these data are not accurate, an accurate prediction cannot be expected. If hydraulics in the networks are completely different to the real situation the distribution of sediment will be very different. Different hydraulic data leads to different distributions of sediment. If the demands is raised the flow velocities in the smaller pipes will be higher and theoretically the sediment deposited will be lower, however this is not further investigated in this report.

The amount of sediment from a treatment plant is not very easy to measure, not every water company wants to do twenty-four hour sampling to determine the sediment load. The quantity of suspended solids is not the most important factor, because the only difference is that more material is inserted into the network: the form of distribution is not influenced by the quantity.

The characteristics of the sediment behavior are quite important. In most cases it is not possible to take large samples of the treatment plant and/ or use a test rig at the plant to determine the different velocities (u_s, u_{rs}, u_d). As seen in paragraph 6-6 changes of the distribution and quantity of the sediment over the network are shown, so determination of these velocities is important. A tool could be the test cylinder to determine the settling velocity of the sediment in the water

Overall it can be concluded that PSM can be a useful tool to predict the distribution of sediment over the network. If all the required data are not very well the prediction will not match the reality. If the hydraulic data are good and the hydraulic behavior of the sediment is well known, the qualitative distribution will be realistic. The quantity of the sediment settled in the network is then not very important, the model can than be used to determine the 'hot spots' in the network. If the determination of the amount of suspended solids is very well determined as well, the prediction of the amount and the distribution of the sediment over the network will be realistic. Because usually the hydraulic data of the network are quite well modeled and the hydraulic parameters (u_s, u_{rs}, u_d) of the sediment can realistically be chosen, the quantitative distribution of the sediment over the network can well be predicted by PSM.

7 Conclusions and recommendations

7.1 PODDS

The PODDS model has been tested on two flushing acts performed at Gorinchem on a 400 mm pipe. It has been possible to validate the data of the testing because the results of the flushing act (turbidity pattern) was available. The first flushing act has been predicted quite well, the second flushing act was not well predicted.

Based on these two test cases it can be concluded that PODDS cannot be directly used to predict discoloured water events. The model makes a prediction that has to be curve-fit to match the measured values of the hydraulic disturbance. In this report it has not been found possible to make solid predictions.

The parameters that were found at the simulation of the first case were not the same as the parameters of the second case, this is why the recommendations are:

- More testing with PODDS to calibrate and validate the model.

With extra test results it is possible to say if the calculations in this report were wrong or that is indeed difficult to make a good simulation.

- Test on smaller diameter pipes.

The flushing of 400 mm pipes had not been modelled in the PODDDS research program. Perhaps the prediction of smaller pipes is better than for large pipes.

- Eventually test on larger pipes.

7.2 Test method sediment load

The test method to determine the sediment load has been tested and it has been proven to work. To further improve the test method the recommendations are:

- Use larger pore membrane filters.

The tests were performed with 0.2 µm membrane filters. Because of the very small size of the pores these pores clog very fast, hereby limiting the quantity of water that is able to pass the filter. When larger pores are used the clogging will be much slower and more water can be filtered. The detection limit can be lowered by filtering more water, by this the technique can be used at treatment plants that produce water with very little particles (membrane filtration). Because the smallest particle contribute less to the total volume (and mass) of the particles, the largest part of the mass is still filtered.

- Use turbidity meter.

The use of a turbidity meter is recommended because this gives more insight in strange peaks occurring in the treatment. Possible (temporary) concentrations can be explained by this insight.

7.3 PSM

The PSM program has been used to make a prediction of the distribution of sediment in the network of PS De Laak. By using the hydraulic data of an average day this distribution has been predicted. The sediment behaviour has been determined with the help of data from a test rig in Delft and the sediment concentration has been determined with the help of the filter system. By constantly dosing sediment at the pumping station the distribution of sediment has been simulated.

The distribution of sediment in the network of PS 'De Laak' has been calculated with initial conditions of sediment characteristics. Also simulations with different parameters have been performed, this led to more insight in the differences of distribution of sediment.

To further improve the PSM program the following recommendations are given:

- Improve determination of sediment characteristics.

Because the sediment characteristics strongly influence the distribution of sediment, the determination of the characteristics has to be improved. For the use of PSM in the Netherlands a database can be started with all available data of sediment known so far, with the density of the particles as input. The use of tools, as mentioned in the previous paragraph, to determine the characteristics is recommended.

- Use different hydraulic situations.

The calculations in this report are based on one hydraulic situation. It is recommended that the calculations are repeated with different flow types. The use of completely different demand pattern is one of the recommendations. For example: multiplying the demand at each node by 2 will lead to completely different flow velocities. The distribution of sediment will be very different.

- Validation of results.

In this report the validation of the predicted deposition of the sediment has not been performed. To validate these data with the real situation a lot of flushing acts and RPM data is necessary. The data can be compared with the fouling in the pipe predicted by PSM.

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Appendix A: PSM equations

Parameters:
d = pipe diameter [m]
L = pipe length [m]
dM_{in} = particle mass transport in pipe during time step Δt [kg]
dM_o = particle mass transport out of the pipe during time step Δt [kg]
M(x) = particle mass per unit length distribution along the pipe at time t [kg/m]
M_s = particle mass settled at time t [kg]
s = particle cloud height ratio [-]
t = time [s]
u = average velocity in pipe [m/s]
Δt = time step [s]

A. **$u>u_{rs}$: particles are assumed to be resuspended instantly**

1. $u\Delta t \le L$

$$M(x) = M(x) + \frac{M_s}{L}; x \in 0, L$$
$$M_s = 0$$
$$\Delta M_o = \int_{L-u\Delta t}^{L} M(x)dx$$
$$M(x) = M(x - u\Delta t); x \in u\Delta t, L$$
$$M(x) = \frac{\Delta M_i}{u\Delta t}; x \in 0, u\Delta t$$
$$s = 1$$

2. $u\Delta t > L$

$$\Delta M_o = \Delta M_i * \frac{\Delta t - \frac{L}{u}}{\Delta t}$$
$$M(x) = M(x) + \frac{M_s}{L}; x \in 0, L$$
$$M_s = 0$$
$$s = 1$$
$$\Delta M_o = \Delta M_o + \int_0^L M(x)dx$$
$$M(x) = \frac{\Delta M_i}{u\Delta t}; x \in 0, L$$

B. $u_d<u\le u_{rs}$: Particles move through the pipe without any change of mass

1. $u\Delta t \le L$

$$\Delta M_o = \int_{L-u\Delta t}^{L} M(x)dx$$

$$M(x) = M(x-u\Delta t); x\in u\Delta t, L$$

$$M(x) = \frac{dM_i}{u\Delta t}; x\in 0, u\Delta t$$

2. $u\Delta t \le L$

$$\Delta M_o = \Delta M_i * \frac{\Delta t - \frac{L}{u}}{\Delta t}$$

$$\Delta M_o = \Delta M_o + \int_0^L M(x)dx$$

$$M(x) = \frac{\Delta M_i}{u\Delta t}; x\in 0, L$$

C. $0<u\le u_d$: Particles settle out with velocity u_s, settling position described by s

1. $u\Delta t \le L$

If $\frac{u_s\Delta t}{d} \ge s$:

$$M_s = M_s + \Delta M_i$$

$$t_s = \frac{sd}{u_s}$$

$$\Delta x = ut_s$$

$$M_s M_s + \int_0^{L-\Delta x} M(x)dx + 0.5\int_{L-\Delta x}^{L} M(x)dx \, \mathrm{s}$$

$$\Delta M_o = 0.5\int_{L-\Delta x}^{L} M(x)dx$$

$$M(x) = 0; x\in 0, L$$

$$s = 0$$

If $\dfrac{u_s \Delta t}{d} \geq s$:

$$M_{s1} = 0.5 \frac{u_s \Delta t}{sd} \int_{L-u\Delta t}^{L} M(x)dx$$

$$\Delta M_o = \int_{L-u\Delta t}^{L} M(x)dx - \Delta M_{s1}$$

$$M(x) = M(x - u\Delta t); x \in u\Delta t, L$$

$$M(x) = \frac{\Delta M_i}{u\Delta t}; x \in 0, u\Delta t$$

$$\Delta M_s = \frac{u_s \Delta t}{sd} \int_0^L M(x)dx + \Delta M_{s1}$$

$$M_s = M_s + \Delta M_s$$

$$M(x) = M(x) - M(x)\frac{u_s \Delta t}{sd}; x \in 0, L$$

$$s = s - \frac{u_s \Delta t}{d}$$

2. $u\Delta t \leq L$

If $\dfrac{Lu_s}{du} \geq s$

$$M_s = M_s + \Delta M_i$$

$$\Delta x = \frac{sd}{u_s} u$$

$$\Delta M_s = \int_0^{L-\frac{\Delta x}{2}} M(x)dx$$

$$M_s = M_s + \Delta M_s$$

$$\Delta M_o = \int_{L-\frac{\Delta x}{2}}^{L} M(x)dx$$

$$M(x) = 0; x \in 0, L$$

If $\frac{Lu_s}{du} \leq s$

$$\Delta y = \frac{u_s L}{u}$$

$$\Delta M_s = \frac{\Delta y}{2sd} \int_0^L M(x) dx$$

$$M_s = M_s + \Delta M_s$$

$$\Delta M_o = \int_0^L M(x) dx - \Delta M_s$$

$$\Delta M_{s1} = \frac{Lu_s}{usd} \Delta M_i$$

$$M_s = M_s + \Delta M_{s1}$$

$$\Delta M_o = \Delta M_o + \Delta M_i (1 - \frac{Lu_s}{usd})(1 - \frac{L}{u\Delta t})$$

$$M(x) = \frac{\Delta M_i}{u\Delta t}(1 - \frac{Lu_s}{usd}); x \in 0, L$$

D. u=0: Particles settling out, zero mass in and out of the pipe

If $\frac{u_s \Delta t}{d} \geq s$

$$M_s = M_s + \int_0^L M(x) dx$$

$$M(x) = 0; x \in 0, L$$

$$\Delta M_o = 0$$

$$s = 0$$

If $\frac{u_s \Delta t}{d} < s$ and s>0

$$M_s = M_s + \frac{u_s \Delta t}{sd} \int_0^L M(x) dx$$

$$s = s - \frac{u_s t}{d}$$

$$M(x) = M(x) - M(x) \frac{u_s t}{sd}; x \in 0, L$$

$$\Delta M_o = 0$$

E. u_{rs} correction equation for pipe diameter

$$u_{rs} = u_{rs100}\sqrt{\frac{f_{100}}{f}}$$

$$f = (1.14 - 2\log_{10}(\frac{\varepsilon}{d} + \frac{21.25}{\mathrm{Re}^{0.9}}))^{-2}$$

f = pipe friction coefficient [mm]
ε = pipe roughness [mm]

$_{100}$ denotes the pipe diameter of d= 100 mm

The equations are used to calculate u_{rs} for any pipe diameter, based on the urs data at d=100mm. The conversion is based on the fact that it is the shear stress which controls the critical condition at which sediment re-suspends.

An example of u_{rs} conversion is listed in Table 20.

Table 21: variation u_{rs} with pipe diameter d

d [mm]	100	87	151	387	626
u_{rs} [m/s]	0.20	0.196	0.211	0.237	0.256

Appendix B: Filtration experiments

Protocol

- Flush filter with ultra pure water, dry for at least 2 hours at 105 °C
- Weigh dried filter
- Filter approximately 10 liters of water
- Dry filter in oven at least 2 hours at 105 °C
- Cool down in exsiccator for at least 30 minutes
- Weigh filter and filtered Suspend Solids
- Calculate amount of Suspend Solids on filters, convert to concentration [mg/m^3]

Determining of detection level

'Lowest possible concentration of a component in a sample of which the presence can be determined with a certain certainty'

According to the Dutch water regulations (Drinkwaterbesluit 2001 [1])

[1] Drinkwaterbesluit 2001 -

Detection Level determination filters

Filter +/-10 litres of ultrapure water over a 0,2 µm filter.

	wet weight													
	filter 1	filter 2	filter 3	filter 4	filter 5	filter 6	filter 7	filter 8	filter 9	filter 10	filter 11	filter 12	filter 13	filter 14
metingen	0.1254	0.1208	0.1139	0.1087	0.1146	0.1163	0.1116	0.1105	0.1096	0.1202	0.1125	0.1141	0.1160	0.1203
	0.1253	0.1207	0.1141	0.1086	0.1146	0.1163	0.1117	0.1105	0.1093	0.1199	0.1124	0.1140	0.1162	0.1203
	0.1254	0.1206	0.1142	0.1088	0.1144	0.1160	0.1120	0.1106	0.1093	0.1200	0.1121	0.1140	0.1161	0.1204
	0.1252	0.1207	0.1139	0.1090	0.1142	0.1162	0.1118	0.1103	0.1093	0.1201	0.1123	0.1142	0.1162	0.1203
	0.1254	0.1207	0.1140	0.1089	0.1146	0.1159	0.1119	0.1102	0.1093	0.1202	0.1122	0.1142	0.1162	0.1202
	0.1252				0.1147	0.1163		0.1104						
					0.1145	0.1164								
gem	0.1253	0.1207	0.1140	0.1088	0.1145	0.1162	0.1118	0.1104	0.1094	0.1201	0.1123	0.1141	0.1161	0.1203

	brand	diam	pore			
	Pall Ultipor	47 mm	0,2 µm			
	setting pu	filter time	filtered wat	dry weigh	wet weig	concentration
filter:	l/h	s	l	gram	gram	mg/m3
1	6.35	5520.00	9.7366667	0.1252	0.12532	11.9822
2	6.00	5700.00	9.5	0.1203	0.1207	42.1053
3	4.78	5460.00	7.2420833	0.1137	0.1140	44.1862
4	6.00	5880.00	9.8	0.1082	0.1088	61.2245
5	5.00	6420.00	8.9166667	0.1142	0.1145	35.247
6	6.00	5760.00	9.6	0.1160	0.1162	20.8333
7	5.00	6300.00	8.75	0.1116	0.1118	22.8571
8	6.00	5700.00	9.5	0.1102	0.1104	22.807
9	5.00	10800.00	15	0.1091	0.1094	17.3333
10	6.00	9420.00	15.7	0.1198	0.1201	17.8344
11	5.00	7500.00	10.416667	0.1121	0.1123	19.2
12	6.00	7500.00	12.5	0.1138	0.1141	24
13	5.00	6420.00	8.9166667	0.1158	0.1161	38.1308
14	6.00	5520.00	9.2	0.1201	0.1203	21.7391

n-1	1	2	3	4	5	6	7	8	9	10

$$s = \sqrt{\Sigma \frac{(x_i - x_{gem})^2}{n-1}}$$

99% betrouwbaarheidsinterval

xi = gemeten waarde nummer i

xgem = gemiddelde gemten waarde

n = aantal meetwaardes

1-10 metingen

meting i	x_i	$(x_i - x_{gem})$	$(x_i - x_{gem})^2$
	mg/m³	mg/m³	mg/m³
x_{gem}	29.6410		
1	11.9822	-17.6588	311.8345
2	42.1053	12.4642	155.3570
3	44.1862	14.5451	211.5613
4	61.2245	31.5835	997.5146
5	35.2470	5.6060	31.4268
6	20.8333	-8.8077	77.5756
7	22.8571	-6.7839	46.0212
8	22.8070	-6.8340	46.7038
9	17.3333	-12.3077	151.4795
10	17.8344	-11.8066	139.3967
Σ	296.4103	-3.91E-14	2168.8710
aantal #	10		
s =	15.5237		
aantoonbaarheidsfa		43.7769	mg/m³
aantoonbaarheidsgre		73.4179	mg/m³

2-10 metingen

meting i	x_i	$(x_i - x_{gem})$	$(x_i - x_{gem})^2$
	mg/m³	mg/m³	mg/m³
x_{gem}	31.6031		
2	42.1053	10.5021	110.2948
3	44.1862	12.5831	158.3332
4	61.2245	29.6214	877.4251
5	35.2470	3.6439	13.2778
6	20.8333	-10.7698	115.9885
7	22.8571	-8.7460	76.4923
8	22.8070	-8.7961	77.3716
9	17.3333	-14.2698	203.6270
10	17.8344	-13.7687	189.5780
Σ	284.4282	-5.68E-14	1822.3882
aantal #	9		
s =	15.093		
aantoonbaarheidsfa		43.7697	mg/m³
aantoonbaarheidsgr		75.3728	mg/m³

3-10 metingen

meting i	x_i	$(x_i - x_{gem})$	$(x_i - x_{gem})^2$
	mg/m³	mg/m³	mg/m³
x_{gem}	30.2904		
3	44.1862	13.8958	193.0938
4	61.2245	30.9341	956.9203
5	35.2470	4.9566	24.5682
6	20.8333	-9.4570	89.4354
7	22.8571	-7.4332	55.2527
8	22.8070	-7.4833	56.0004
9	17.3333	-12.9570	167.8846
10	17.8344	-12.4560	155.1511
Σ	242.3229	1.78E-14	1698.3065
aantal #	8		
s =	15.5761		
aantoonbaarheidsfa		46.7283	mg/m³
aantoonbaarheidsgre		77.0187	mg/m³

2-12 metingen			
meting i	x_i	$(x_i - x_{gem})$	$(x_i - x_{gem})^2$
	mg/m³	mg/m³	mg/m³
x_{gem}	29.7844		
2	42.1053	12.4642	155.3570
3	44.1862	14.5451	211.5613
4	61.2245	31.5835	997.5146
5	35.2470	5.6060	31.4268
6	20.8333	-8.8077	77.5756
7	22.8571	-6.7839	46.0212
8	22.8070	-6.8340	46.7038
9	17.3333	-12.3077	151.4795
10	17.8344	-11.9500	142.8021
11	19.2000	-10.5844	112.0290
12	24.0000	-5.7844	33.4590
Σ	327.6282	1.1467	2005.9299
aantal #	11		
s =	14.1631		
aantoonbaarheidsfa	39.0901	mg/m³	
aantoonbaarheidsgre	68.8745	mg/m³	

3-12 metingen			
meting i	x_i	$(x_i - x_{gem})$	$(x_i - x_{gem})^2$
	mg/m³	mg/m³	mg/m³
x_{gem}	28.5523		
3	44.1862	14.5451	211.5613
4	61.2245	31.5835	997.5146
5	35.2470	5.6060	31.4268
6	20.8333	-8.8077	77.5756
7	22.8571	-6.7839	46.0212
8	22.8070	-6.8340	46.7038
9	17.3333	-12.3077	151.4795
10	17.8344	-11.9500	142.8021
11	19.2000	-10.5844	112.0290
12	24.0000	-5.7844	33.4590
Σ	285.5229	-11.3175	1850.5729
aantal #	10		
s =	14.3394		
aantoonbaarheidsfa	40.4372	mg/m³	
aantoonbaarheidsgre	68.9895	mg/m³	

4-12 metingen			
meting i	x_i	$(x_i - x_{gem})$	$(x_i - x_{gem})^2$
	mg/m³	mg/m³	mg/m³
x_{gem}	26.8152		
4	61.2245	31.5835	997.5146
5	35.2470	5.6060	31.4268
6	20.8333	-8.8077	77.5756
7	22.8571	-6.7839	46.0212
8	22.8070	-6.8340	46.7038
9	17.3333	-12.3077	151.4795
10	17.8344	-11.9500	142.8021
11	19.2000	-10.5844	112.0290
12	24.0000	-5.7844	33.4590
Σ	241.3367	-25.8626	1639.0117
aantal #	9		
s =	14.3135		
aantoonbaarheidsfa	41.5092	mg/m³	
aantoonbaarheidsgre	68.3244	mg/m³	

4-14 metingen			
meting i	x_i	$(x_i - x_{gem})$	$(x_i - x_{gem})^2$
	mg/m^3	mg/m^3	mg/m^3
x_{gem}	21.9397		
4	61.2245	31.5835	997.5146
5	35.2470	5.6060	31.4268
6	20.8333	-8.8077	77.5756
7	22.8571	-6.7839	46.0212
8	22.8070	-6.8340	46.7038
9	17.3333	-12.3077	151.4795
10	17.8344	-11.9500	142.8021
11	19.2000	-10.5844	112.0290
12	24.0000	-5.7844	33.4590
13	38.1308	8.3465	69.6635
14	21.7391	-8.0452	64.7260
Σ	241.3367	-25.8626	1639.0117
aantal #	11		
s =	12.8024		
aantoonbaarheidsfa		35.3346	mg/m^3
aantoonbaarheidsgr		57.2743	mg/m^3

5-14 metingen			
meting i	x_i	$(x_i - x_{gem})$	$(x_i - x_{gem})^2$
	mg/m^3	mg/m^3	mg/m^3
x_{gem}	18.0112		
5	35.2470	5.6060	31.4268
6	20.8333	-8.8077	77.5756
7	22.8571	-6.7839	46.0212
8	22.8070	-6.8340	46.7038
9	17.3333	-12.3077	151.4795
10	17.8344	-11.9500	142.8021
11	19.2000	-10.5844	112.0290
12	24.0000	-5.7844	33.4590
13	38.1308	8.3465	69.6635
14	21.7391	-8.0452	64.7260
Σ	180.1122	-57.4461	641.4971
aantal #	10		
s =	8.4426		
aantoonbaarheidsfa		23.8081	mg/m^3
aantoonbaarheidsgr		41.8193	mg/m^3

6-14 metingen			
meting i	x_i	$(x_i - x_{gem})$	$(x_i - x_{gem})^2$
	mg/m^3	mg/m^3	mg/m^3
x_{gem}	16.0961		
6	20.8333	-8.8077	77.5756
7	22.8571	-6.7839	46.0212
8	22.8070	-6.8340	46.7038
9	17.3333	-12.3077	151.4795
10	17.8344	-11.9500	142.8021
11	19.2000	-10.5844	112.0290
12	24.0000	-5.7844	33.4590
13	38.1308	8.3465	69.6635
14	21.7391	-8.0452	64.7260
Σ	144.8652	-63.0520	610.0703
aantal #	9		
s =	8.73263		
aantoonbaarheidsfa		25.3246	mg/m^3
aantoonbaarheidsgr		41.4208	mg/m^3

a. **Sediment load of PS de Laak**

FILTRATIE PS DE
LAAK

start	30-11-2004 10:29			1-12-2004 15:42	
eind	1-12-2004 10:14			1-12-2004 15:56	
filtertijd	23:45:00	uu:mm:ss		0:14:00	uu:mm:ss
	0.98958333	dag		0.00972222	dag
instelling	2	l/h		1.5	l/h
	48	l/d		36	l/d
	0.0480	m3/d		0.0360	m3/d
hoeveelheid water	0.0475	m3		0.0004	m3
	47.5000			1-12-2004 15:56	
				3-12-2004 14:19	
			1 day and	22:23:00	uu:mm:ss
				1.93263889	dag
				1.5	l/h
				36	l/d
				0.0360	m3/d
				0.0696	m3
			TOTAAL	0.0699	m3
2	0.1124		3	0.1175	
voor	0.1124		voor	0.1175	
	0.1122			0.1173	
gem	0.112333333	g	gem	0.117433333	
2	0.1195		3		
na	0.1197		na	0.1273	
	0.1195			0.1274	
	0.1195			0.1272	
	0.1198			0.1272	
gem	0.1196	g		0.1273	
delta massa	0.007266667	g	gem	0.12728	
	7.266666667	mg	delta massa	0.009846667	g
				9.846666667	mg
concentratie	**152.98**	**mg/m3**	**concentratie**	**140.82**	**mg/m3**

Appendix C: Calculation of the shear stress

D = 100 mm k = 0.05 mm T = 28 °C U = 0.06 m/s	R = ¼ D = 0.025 m $\upsilon = 497 \cdot 10^{-6} / (T + 42{,}5)^{1{,}5} = 8.40 \cdot 10^{-7}$
Hydraulic smooth	$\delta = \dfrac{12 \cdot \nu \cdot 18 \log\left(\dfrac{48R}{\delta}\right)}{\sqrt{g} \cdot u}$ $\delta = 9.65 \cdot 10^{-4} \log\left(\dfrac{1.2}{\delta}\right)$ $\delta = 2.57 \cdot 10^{-3}$
Technical rough	$\delta = \dfrac{12 \cdot \nu \cdot 18 \log\left(\dfrac{12R}{k + 1/4\delta}\right)}{\sqrt{g} \cdot u}$ $\delta = 9.65 \cdot 10^{-4} \log\left(\dfrac{0.3}{0.05 \cdot 10^{-3} + 1/4\delta}\right)$ $\delta = 2.55 \cdot 10^{-3}$
Hydraulic rough	$\delta = \dfrac{12 \cdot \nu \cdot 18 \log\left(\dfrac{12R}{k}\right)}{\sqrt{g} \cdot u}$ $\delta = 9.65 \cdot 10^{-4} \log\left(\dfrac{0.3}{0.05 \cdot 10^{-3}}\right)$ $\delta = 3.65 \cdot 10^{-3}$
Determine hydraulic condition: hydraulic smooth ($\delta/k > 4$) technical rough ($1/6 < \delta/k < 4$) hydraulic rough ($\delta/k < 1/6$)	$\delta / k = 2.57 \cdot 10^{-3} / 0.05 \cdot 10^{-3} = 51.4$ $\delta / k = 2.55 \cdot 10^{-3} / 0.05 \cdot 10^{-3} = 51$ $\delta / k = 3.65 \cdot 10^{-3} / 0.05 \cdot 10^{-3} = 73$
Conclusion: Condition is hydraulic smooth, with $\delta = 2.57 \cdot 10^{-3}$	
With δ and the hydraulic condition known, the Chézy coefficient can be calculated: $C = 18 \log\left(\dfrac{48R}{\delta}\right) = 48.05$ $m^{1/2}/s$	
$\tau = \rho g \left(\dfrac{u}{C}\right)^2 = 1000 \cdot 9.81 \cdot \left(\dfrac{0.06}{48.05}\right)^2 = 0.015$ N/m^2	

Appendix D: Calculation of contribution of Fe Mn Al oxides to total sediment load year 2002, based on REWAB figures

Pumping stations with large contribution

rating	Code Bedrijf	Bedrijf	G=gr water, O=opp vl water	Code pomp station	Pompstation	Uitgaand 10 ^6 m3	Fe µg/l	Fe als SS productie µg/l	Mn µg/l	Mn als SS productie µg/l	Al µg/l	Al als SS productie µg/l	Totaal belasting SS mg/l	Totaal belasting SS kg/jaar
1	029	N.V. Hydron Midden- Nederland	G	08	Montfoort	0,4	161	0,30763	12	0,01898	2	0,00578	0,332	132,95
2	029	N.V. Hydron Midden- Nederland	O	19	Loosdrecht	3,1	84	0,16050	2	0,00316	55,9	0,16149	0,325	1007,97
3	009	Waterleiding Maatschappij Overijssel	G	30	Pompstation Rodenmors	1,08	145	0,27705	10	0,01582	5	0,01444	0,307	331,90
4	003	Vitens Fryslan	G	01	VLIELAND	0,176	30	0,05732	2	0,00316	85	0,24556	0,306	53,86
5	017	Vitens WG Betuwe Veluwe	G	22	Boele	3,696	100	0,19107	10	0,01582	5	0,01444	0,221	818,05
6	027	N.V. Bronwaterleiding Doorn	O	27	Pompstation Doorn	0,907	10	0,01911	73	0,11547	24	0,06933	0,204	184,95
7	029	N.V. Hydron Midden- Nederland	G	14	Bilthoven	1,3	82	0,15668	5	0,00791	8,9	0,02571	0,190	247,39
8	015	Vitens WG Achterhoek	G	06	Lochem	1,73	80	0,15286	10	0,01582	5	0,01444	0,183	316,80
9	003	Vitens Fryslan	G	08	OLDEHOLTPADE	4,98	50	0,09554	2	0,00316	27	0,07800	0,177	879,96
10	029	N.V. Hydron Midden- Nederland	G	23	Hogeweg	6,3	61	0,11655	30	0,04745	2	0,00578	0,170	1069,65

Pumping stations wit small contribution

rating	Code Bedrijf	Bedrijf	G=gr water, O=oppvl	Code pompstation	Pompstation	Uitgaand 10^6 m3	Fe µg/l	Fe als SS productie µg/l	Mn µg/l	Mn als SS productie µg/l	Al µg/l	Al als SS productie µg/l	Totaal belasting SS mg/l	Totaal belasting SS kg/jaar
1	051	n.v. Hydron Zuid-Holland	G	05	Z'station Lekkerkerk	2,8	5	0,00955	2	0,00316	1,05	0,00303	0,016	44,10
2	029	N.V. Hydron Midden- Nederland	G	24	Berg	2,9	2	0,00382	5	0,00791	2	0,00578	0,018	50,77
3	029	N.V. Hydron Midden- Nederland	G	16	Soestduinen 3	4,4	5	0,00955	2	0,00316	2	0,00578	0,018	81,38
4	051	n.v. Hydron Zuid-Holland	G	03	Z'station C. Rodenhuis	13,95	5	0,00955	2	0,00316	2	0,00578	0,018	258,01
5	051	n.v. Hydron Zuid-Holland	G	11	Z'station Hendrik Ido Am	0,8	5	0,00955	2	0,00316	2	0,00578	0,018	14,80
6	051	n.v. Hydron Zuid-Holland	G	09	Z'station 't Kromme Gat	0,95	5	0,00955	2	0,00316	2	0,00578	0,018	17,57
7	029	N.V. Hydron Midden- Nederland	G	21	Laren 2e meetpunt	1,6	5	0,00955	2	0,00316	2,3	0,00664	0,019	30,98
8	029	N.V. Hydron Midden- Nederland	O	11	Driebergen	1,4	7	0,01338	2	0,00316	2,2	0,00636	0,023	32,05
9	029	N.V. Hydron Midden- Nederland	G	17	Soest	1	5	0,00955	2	0,00316	4,3	0,01242	0,025	25,14
10	029	N.V. Hydron Midden- Nederland	G	02	Veenendaal	2,9	11	0,02102	2	0,00316	2	0,00578	0,030	86,88

Appendix E: Steps to run PODDS

A. CALCULATE HYDRAULIC DATA

To be able to make a good simulation the hydraulic data has to be inserted into EPAnet. This includes data of diameter, length and roughness of pipes and the diurnal pattern of the flows.

B. ENABLE TURBIDITY OPTION

To be able to make predictions with PODDS the turbidity option has to be enabled, this is located in the Quality Options box.

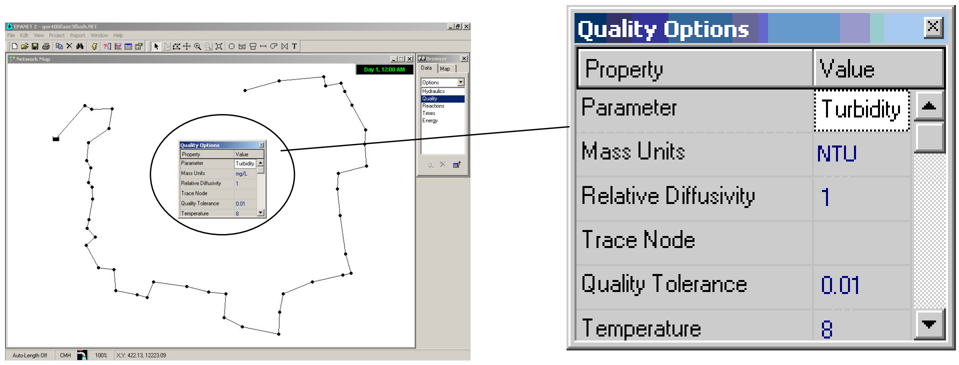


Figure 8-1: Enable turbidity parameter

C. SET VARIABLES

The variables that need to be set are suitable accuracy, tolerances, calculation and reporting time steps, (flow) units and head loss formulae.

D. CALCULATE INITIAL LAYER STRENGTH

The initial layer strength of each pipe of the network that is calculated with the use of Equation 50:

$$\tau = \rho g \frac{D}{4} S_0 \qquad \textit{Equation 47}$$

τ = Boundary shear stress [N/m^2]
ρ = Density of water [kg/m^3]
g = Gravity acceleration [m/s^2]
D = Diameter of pipe [m]
S_0 = Hydraulic gradient [m/km]

S_0 can be taken from the hydraulic simulation program, it is the maximum hydraulic gradient that is reached during the day. Usually this is calculated with the Darcy-Weisbach formula:

$$S_0 = f \frac{L}{D} \frac{u^2}{2g} \qquad \textit{Equation 48}$$

S_0 = Hydraulic gradient [m/km]
f = friction factor [m]
L = length of pipe [m]
D = diameter of pipe [m]
u = velocity [m/s]
g = gravity acceleration [m/s²]

E. IMPLEMENT HYDRAULIC EVENT

By implementing a hydraulic event a simulation of a flushing operation or pipe burst can be simulated. This can be done by adding a very large demand on a node where the hydraulic event takes place. In figure XX an example is shown of such an event (simulation of a slowly increasing flushing operation in this case).

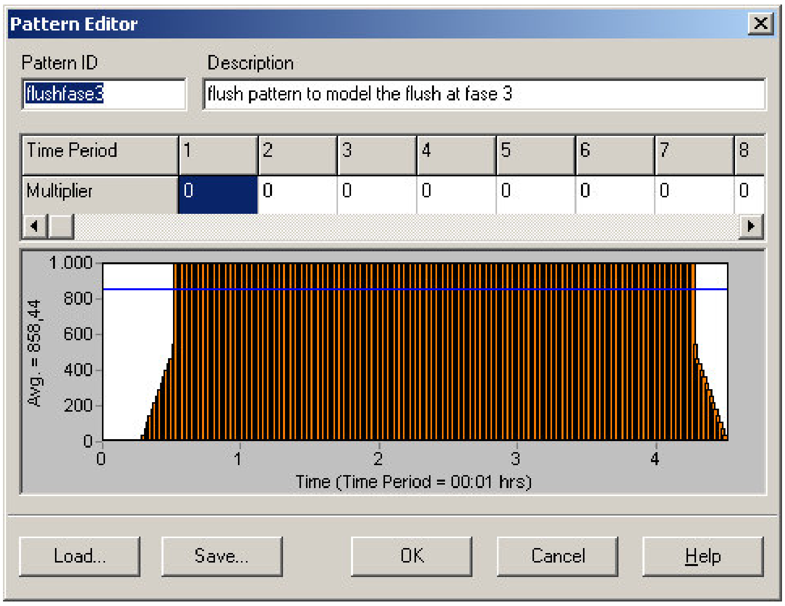


Figure 8-2: Example of simulation of flushing opration

F. INSERT CALIBRATION FILE

By inserting a calibration file both values of the measured data (calibration file) and predicted values can be plotted into one graphic. The calibration file can be inserted in the menu under Projects -> Calibration Data. The calibration file itself has to be a text file containing the measured data and has to be stored with the extension .dat.

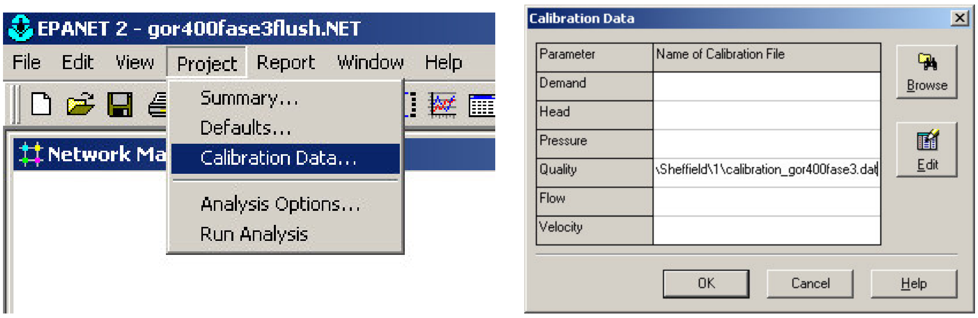


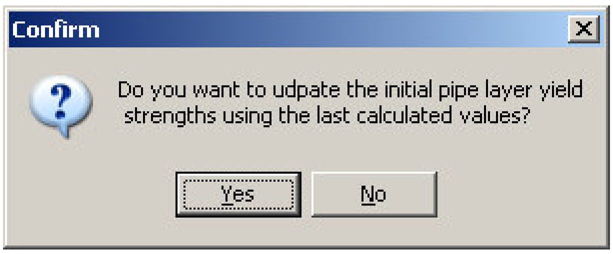
Figure 8-3: Inserting Calibration Data file

G. RUN MODEL

The last step is to run the turbidity model, in PODDS this is done by pressing the run button on the menu.

Figure 8-4: Run PODDS

After a successful EPAnet analysis using the PODDS model with quality parameter turbidity enabled a confirmation box will appear. In most cases it is necessary to press No, because the updating of the initial layer yield strengths is not wanted because further trials will then commence with the initial conditions equal to those calculated at the end of the previous run.



H. COMPARE RESULTS

By comparing the results of the simulation with the measured data it can be seen if the prediction is good. In case of a poor resemblance changes have to be made to the parameters of each pipe.

Appendix F: Particle count and volume of water De Laak

Particles water PS De Laak

1	volume 1	1.5	volume 1,5	2	volume 2	5	volume 5
# particles	1 particle m3	# particles	1 particle m3	# particles	1 particle m3	# particles	1 particle m3
471.1247312	5.23599E-19	215.5066667	1.76715E-18	118.5393548	4.18879E-18	30.47612903	6.54498E-17
	total m3		total m3		total m3		total m3
	2.4668E-16		3.80832E-16		4.96536E-16		1.99466E-15
10	volume 10	20	volume 20	50	volume 50	100	volume 100
# particles	1 particle m3	# particles	1 particle m3	# particles	1 particle m3	# particles	1 particle m3
10.35139785	5.23599E-16	2.781935484	4.18879E-15	0.265376344	6.54498E-14	0.152688172	5.23599E-13
	total m3		total m3		total m3		total m3
	5.41998E-15		1.16529E-14		1.73688E-14		7.99473E-14

TOTAL Volume	1.17508E-13	m3/ml	average # particles	106.1497849	#/ml = ppm
	1.17508E-07	m3 particles/ m3 water			

Appendix G: steps to run PSM

(All taken from short manual 'PSM Trial Version' 18 November 2003)

Data Preparation Steps

- Prepare hydraulic modelling flow data into the data format in "PSMInputDataTemplate.xls", in the CD;
- Convert the data in the spread sheet into the 3 txt files (ascii format)
  H2Odata.txt, (flow data from hydraulic modelling, 0.1hrs time step, and total 24hrs duration fixed.
  NodeData.txt
  MassDistribution.txt
  in the same form of the demonstration files in CD;
- Double click **PSM.exe** to run.
- You should install all these files in a user working directory. The directory name is defaulted to PSM. If you prefer to use a different name, run PSM, and use ***System/Data Source*** to change the file directory.

Demonstration Run

- Run PSM.exe;
- Check System/Data Source, to ensure that PSM read the input files at the correct directory. You can change the operating directory, if required.
- Select ***Data Processing/Clean*** manual to clean the pipes;
- Choose a pipe, double click, you will see a pipe dialog box;
  you have options of operating in two modes:
  1) default mode, which require input of sediment/suspension mass in (kg);
  2) constant concentration mode, you need click the check box (constant concentration), and then you can enter the concentration, at which the particle concentration will be maintained during the run (at this pipe)
- Right click mouse, you can select to show ***Suspension Weight, Concentration or Sediment***. You can change it during the simulation;
- Run from PSM engine from ***Data Processing/PSM***, a dialog box pop up;
- Change the value of ***Filer Flow*** (default to 0.02L/s) if necessary;
- Enter No. of time steps.
  Times of simulation=No of time steps × 0.1 (hours). Note the time step used in PSM is fixed to 360sec (0.1hrs).
- Click ***OK*** will start the run.
- On finishing, you will be prompted to update the flow data for continuing the simulation. Click ***Yes*** to update the flow data, and you will have to select the file name for the new flow data. Click ***No*** to exit.

- Before leaving **PSM**, click ***File/Save*** to save the data into MassDistribution.txt.

FAQ

- What is the time step used in the hydraulic input data?
  Time step=0.1hrs, at the moment the input data structure is fixed. The input data duration are fixed at 24 hours, with time step of 0.1 hrs step, starting at 0:00 am, and finishing at 24:00:00.
- Is the software ready for use?
  The software is intended as a demonstration software; the concept of predicting particles sediment in water distribution system is a trial. It is not a commercial software, therefore you may find many window control features need to be improved. Most noticeable is the flexibility of the input data structure, the post processing analysis.
- What is the accuracy of the software?
  To our best knowledge, it predicts particle concentration within 50% of the measurement. It is very difficult to collect field sediment data for comparison.
- What is the input field "Filer Flow" for?
  This is the flow released into filter meters, to collect particles for calibration. A list of total mass collected on the filers will be output to the MassDisstribution.txt, once the "File save" is clicked.
- Can we run PSM over multiple days?
  Yes, you have three options:
  1) run the same input flow data over as many days as required. You can simply enter the total number of time step. For example, if you want to use the same input flow data (24hrs flow profile) for 30 days, enter a total number of time steps to 7200 (=0.1x30x24). You can run this up to a maximum of 10 years (≈90,000 steps).
  2) you can update the flow input data, as many times as you wish. You need to prepare fresh input flow data in advance, and select them under prompt.
  3) you can mix the two methods, e.g. run a single input flow data (24hrs data) for 7 days, and select a different flow input data (24hrs data) and run another seven days, … etc.
  note: make sure you save the results before exiting PSM.
- What is the maximum number of pipes PSM can handle?
  At the moment, it is set at 3000. This figure can be upgraded, if a more powerful computer is used in the future. Note: this software is to demonstrate the concept of particle modelling. A commercial version will have to be developed to handle a larger pipe network system.
- Is there any discrepancy if running the same flow input data over 1x48hrs, as vs. 2x24hrs?
  No. The results will be identic. Check the included two files: MassDistribution2_24.txt, MassDistribution48.txt, they from 2x24hrs run, and 1x48 hrs run. The two results are identic.

Appendix H: Results PSM simulation De Laak

Ratio and calculation of distribution of settled sediment (in kg) in pipes after 1, 2, 3, 4 and 52 weeks.

PIPEID	MassSettled(kg)	MassSettled(kg)	MassSettled(kg)	MassSettled(kg)	MassSettled(kg)
	1 week	2 weeks	3 weeks	4 weeks	52 weeks
ratio	1	2.069870306	3.139740611	4.209611985	54.7249558
AM00005	0	0	0	0	0
AM00010	0	0	0	0	0
AM00015	0	0	0	0	0
AM00020	0.936664	1.938773	2.940882	3.942992	51.258896
AM00025	0.949121	1.963343	2.977565	3.991786	51.94060477
AM00030	0.406079	0.837281	1.268483	1.699685	22.22265533
AM00035	1.564448	3.223219	4.88199	6.540762	85.61434765
AM00040	0.264709	0.54769	0.830671	1.113652	14.48618833
AM00045	0.713656	1.476308	2.23896	3.001613	39.05479306
AM00050	0.221118	0.456387	0.691656	0.926925	12.10067278
AM00055	0.633681	1.315871	1.998061	2.680252	34.67816472
AM00060	0	0	0	0	0
AM00061	0.026048	0.026048	0.026048	0.026048	1.425475649
AM00065	1.64058	3.397171	5.153762	6.910352	89.78066799
AM00070	1.168831	2.418883	3.668935	4.918988	63.96422481
AM00075	0.226049	0.468364	0.710679	0.952994	12.37052153
AM00080	0.9135	1.889808	2.866116	3.842424	49.99124712

Initial distribution of sediment

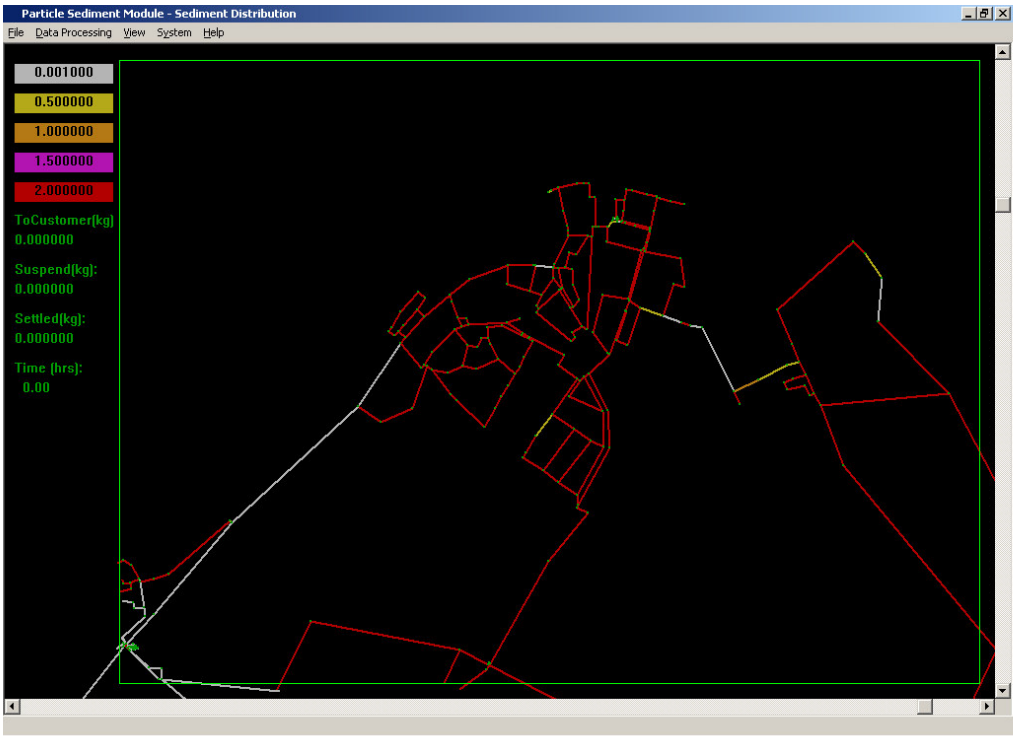


Figure 8-5: Initial sediment distribution after 1 year, parameters: u_s = 5.5e-6 m/s; u_d= 0.15 m/s; u_{rs} = 0.25 m/s near Vianen

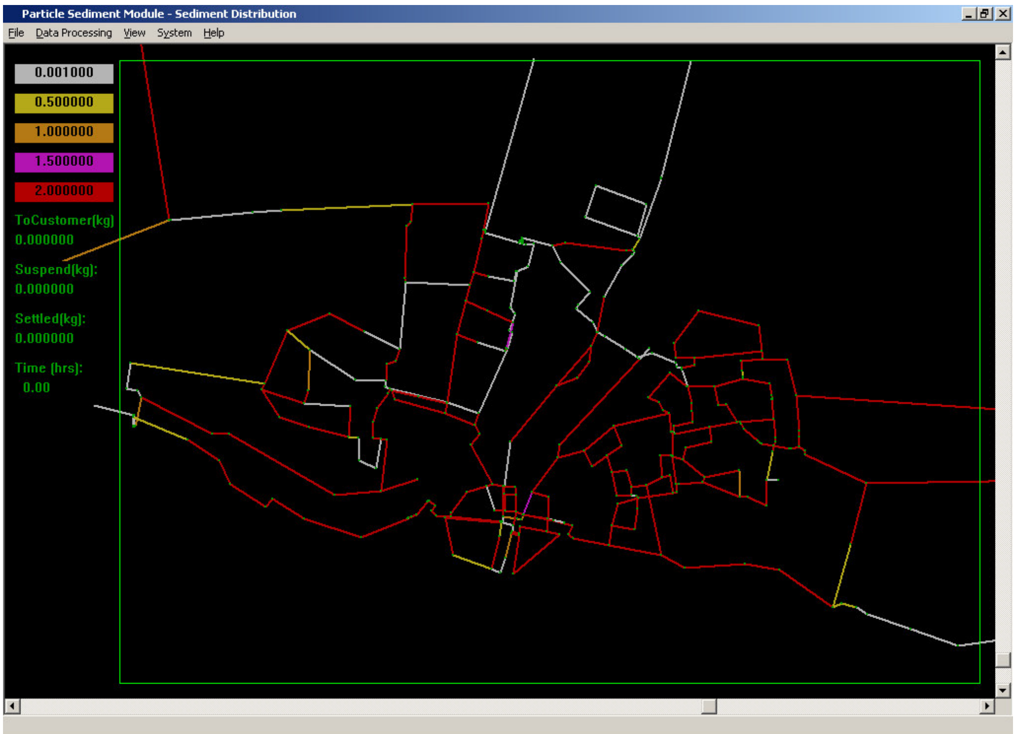


Figure 8-6: Initial sediment distribution after 1 year, parameters: u_s = 5.5e-6 m/s; u_d= 0.15 m/s; u_{rs} = 0.25 m/s, near Gorinchem

Scenario 1: $u_s \downarrow$

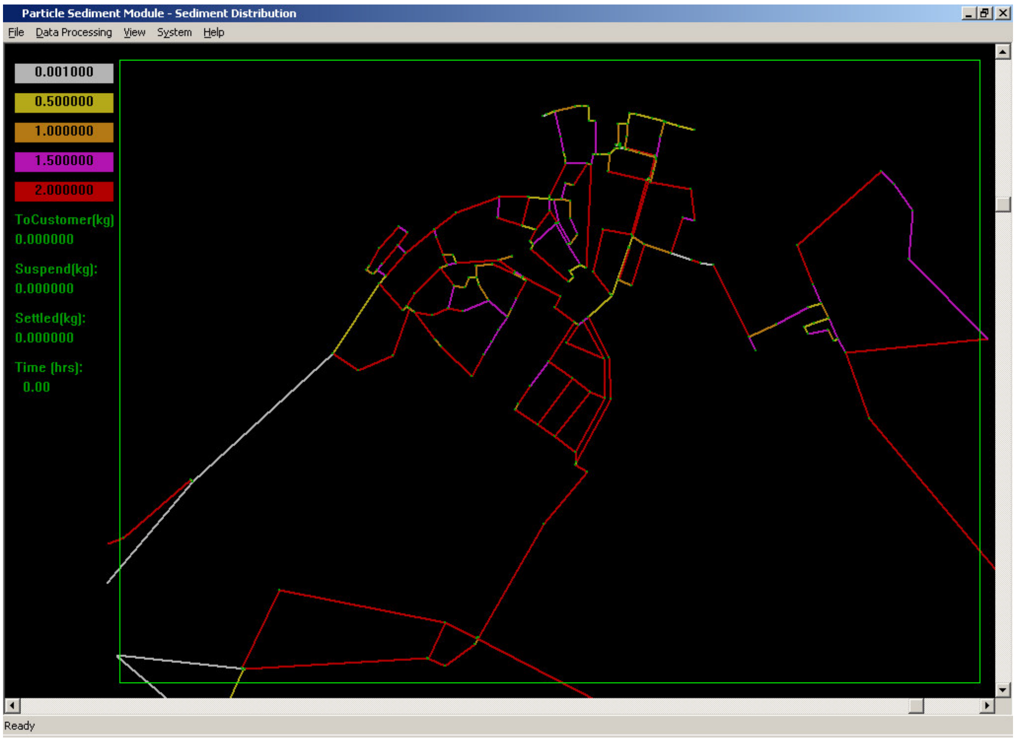


Figure 8-7: Sediment distribution after 1 year, parameters: ***u_s = 1e-7 m/s****; u_d= 0.15 m/s; u_{rs} = 0.25 m/s, near Vianen*

Scenario 1: $u_s \downarrow$

Figure 8-8: Sediment distribution after 1 year, parameters: ***u_s = 1e-7 m/s***; u_d= 0.15 m/s; u_{rs} = 0.25 m/s, near Gorinchem

Scenario 1: $u_s \uparrow$

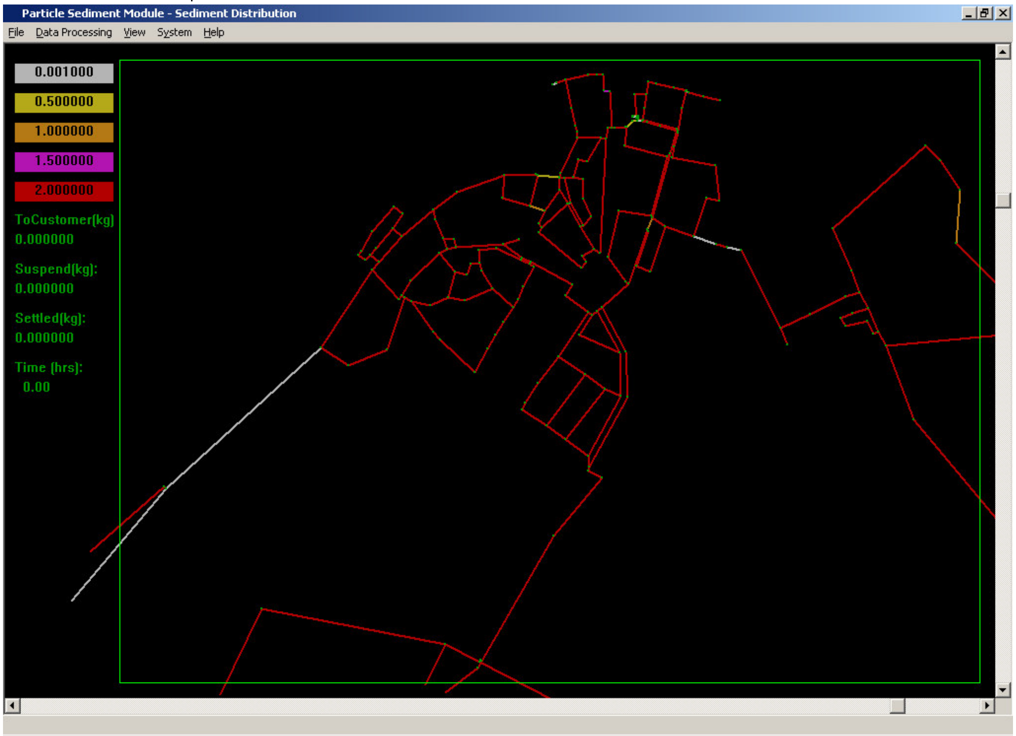


Figure 8-9: Sediment distribution after 1 year, parameters: ***<u>us = 1e-6 m/s</u>****; ud= 0.15 m/s; urs = 0.25 m/s near Vianen*

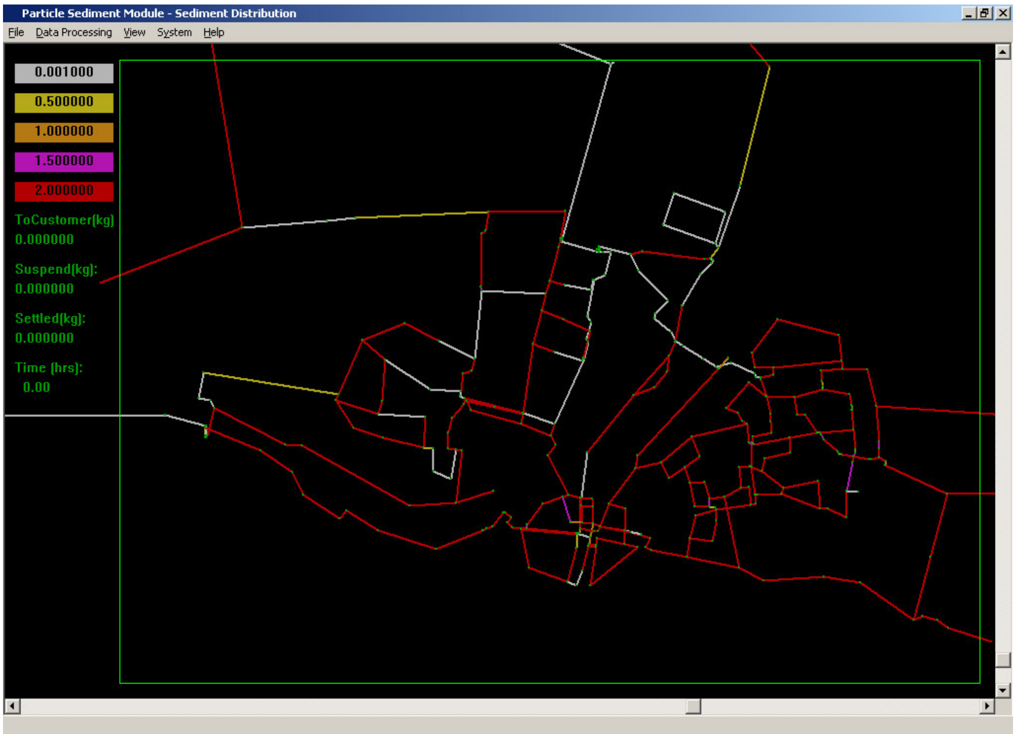


Figure 8-10:Sediment distribution after 1 year, parameters: ***<u>us = 1e-6 m/s</u>****; ud= 0.15 m/s; urs = 0.25 m/s near Gorinchem*

Scenario 2: $u_d \downarrow$

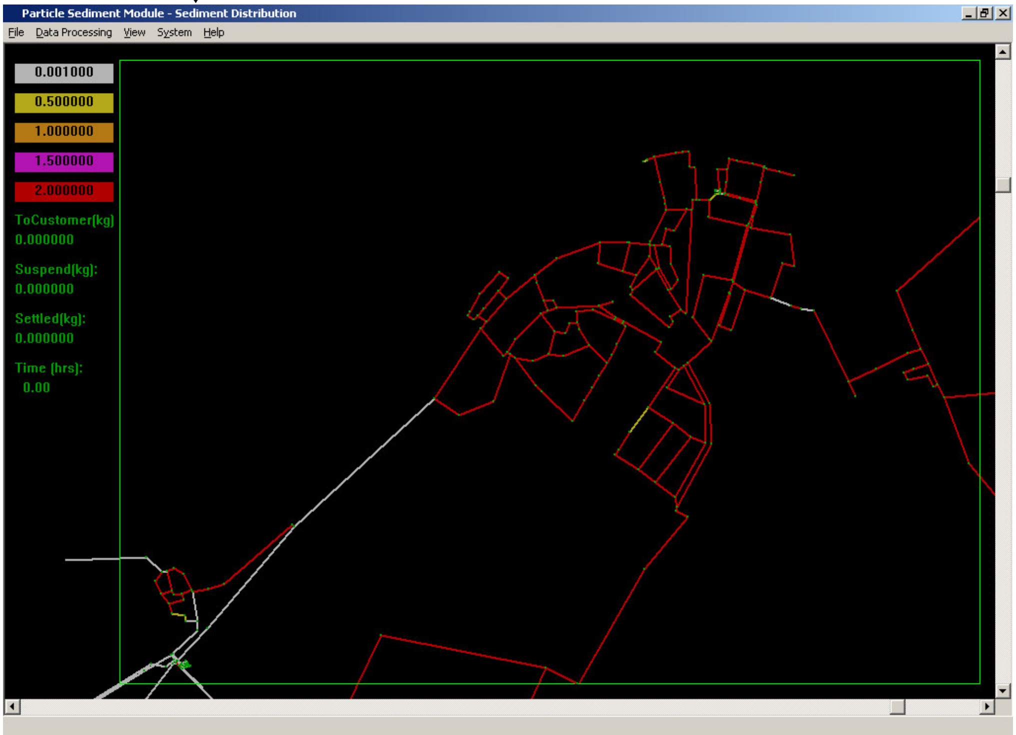


Figure 8-11: Sediment distribution after 1 year, parameters: u_s = 5.5e-6 m/s; ***u_d= 0.10 m/s****; u_{rs} = 0.25 m/s near Vianen:*

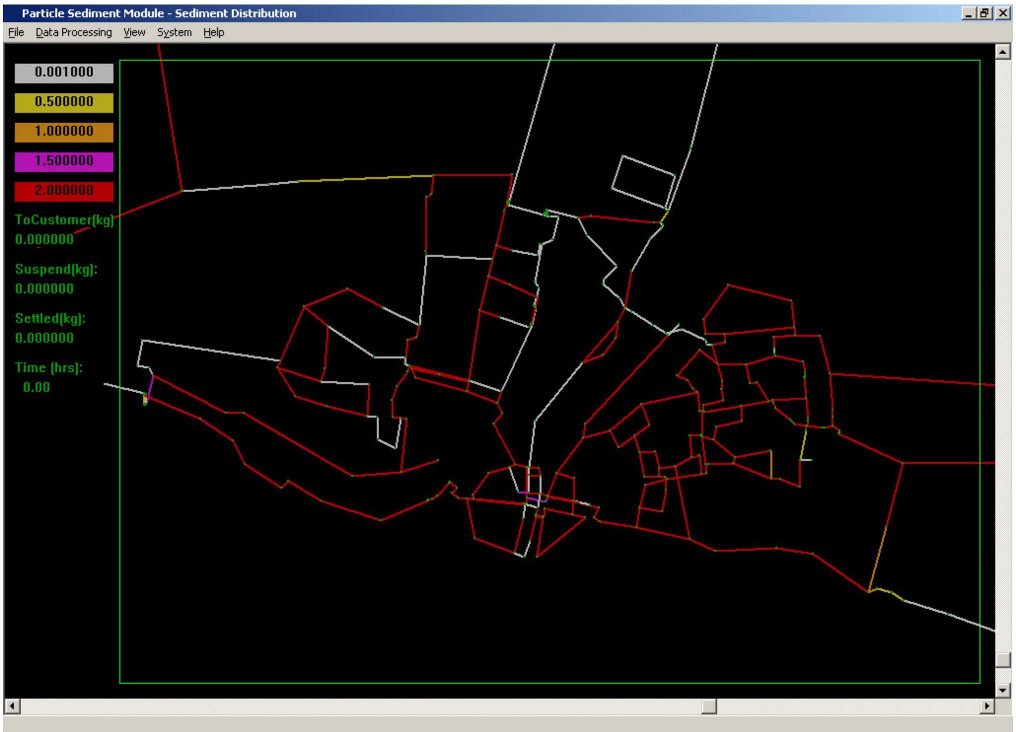


Figure 8-12: Sediment distribution after 1 year, parameters: u_s = 5.5e-6 m/s; ***u_d= 0.10 m/s****; u_{rs} = 0.25 m/s near Gorinchem*

Scenario 2: $u_d \uparrow$

Figure 8-13: Sediment distribution after 1 year, parameters: u_s = 5.5e-6 m/s; ***$\underline{u_d}$= 0.25 m/s****; u_{rs} = 0.25 m/s, near Vianen*

Figure 8-14: Sediment distribution after 1 year, parameters: us = 5.5e-6 m/s; ud= 0.25 m/s; urs = 0.25 m/s, near Gorinchem

Scenario 3: $u_{rs}\downarrow$

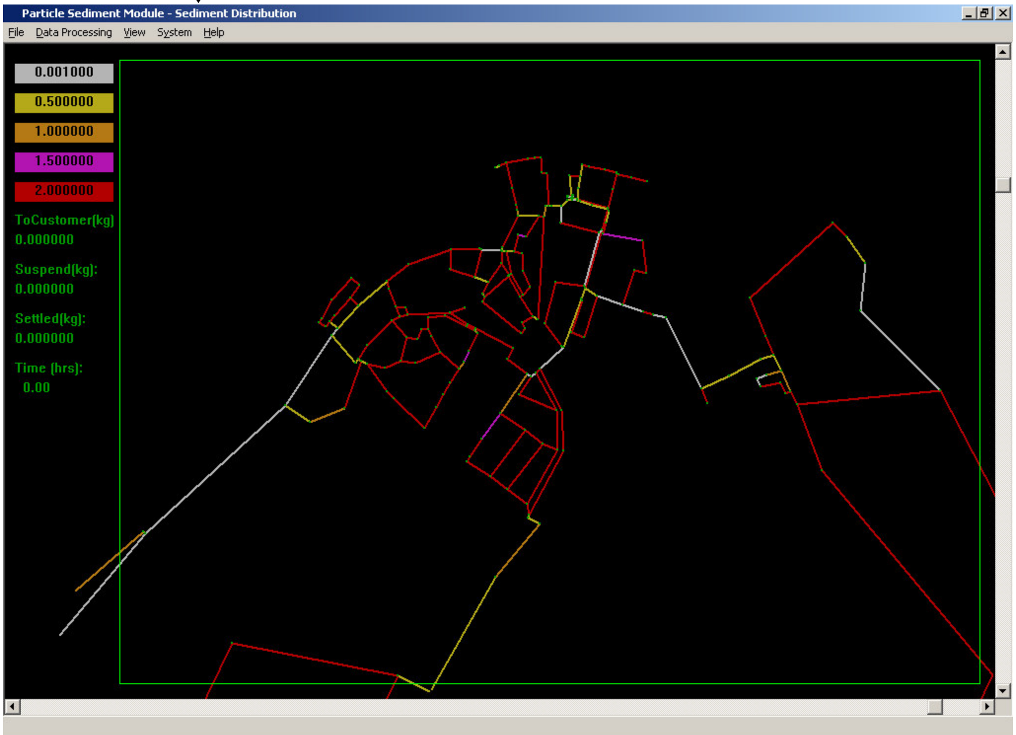


Figure 8-15: Sediment distribution after 1 year, parameters: u_s = 5.5e-6 m/s; u_d= 0.15 m/s; ***<u>u_{rs} = 0.10 m/s</u>****, near Vianen*

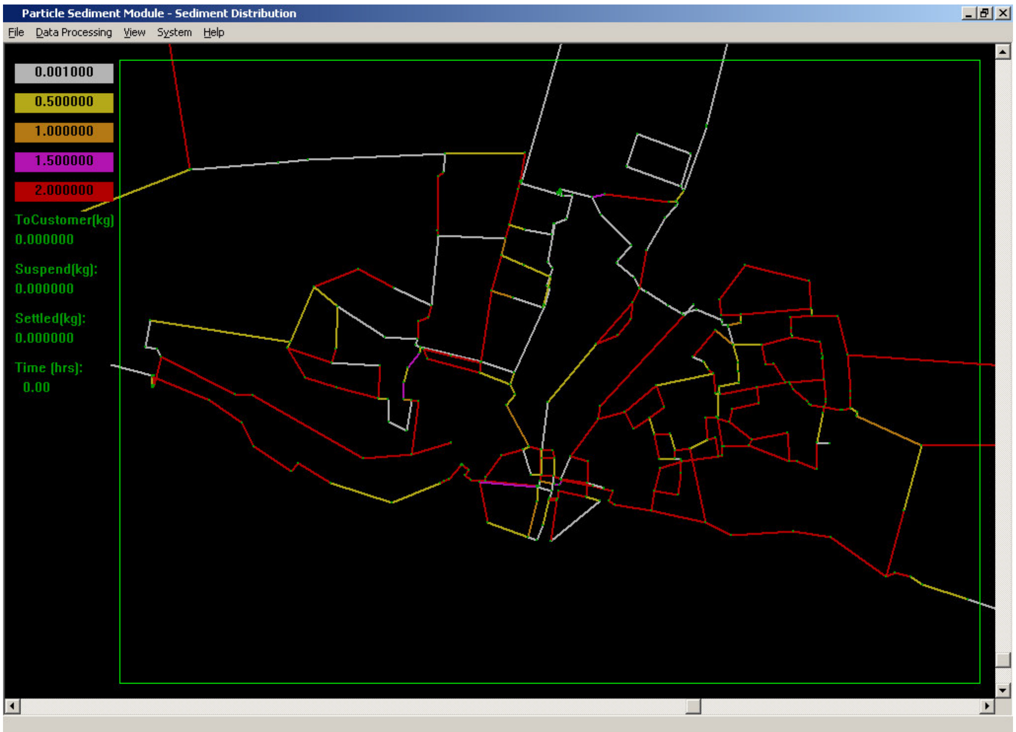


Figure 8-16: Sediment distribution after 1 year, parameters: u_s = 5.5e-6 m/s; u_d= 0.15 m/s; ***<u>u_{rs} = 0.10 m/s</u>****, near Gorinchem*

Scenario 3: u_{rs} ↑

Figure 8-17: Sediment distribution after 1 year, parameters: u_s = 5.5e-6 m/s; u_d= 0.15 m/s; ***<u>u_{rs} = 0.50 m/s</u>****, near Vianen*

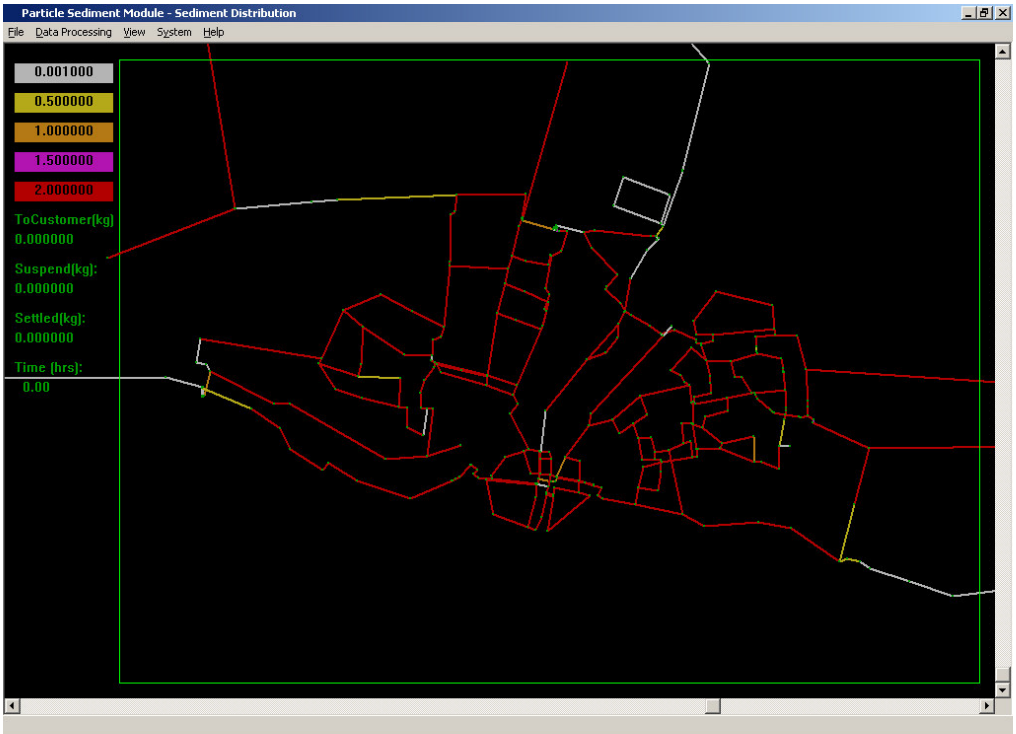
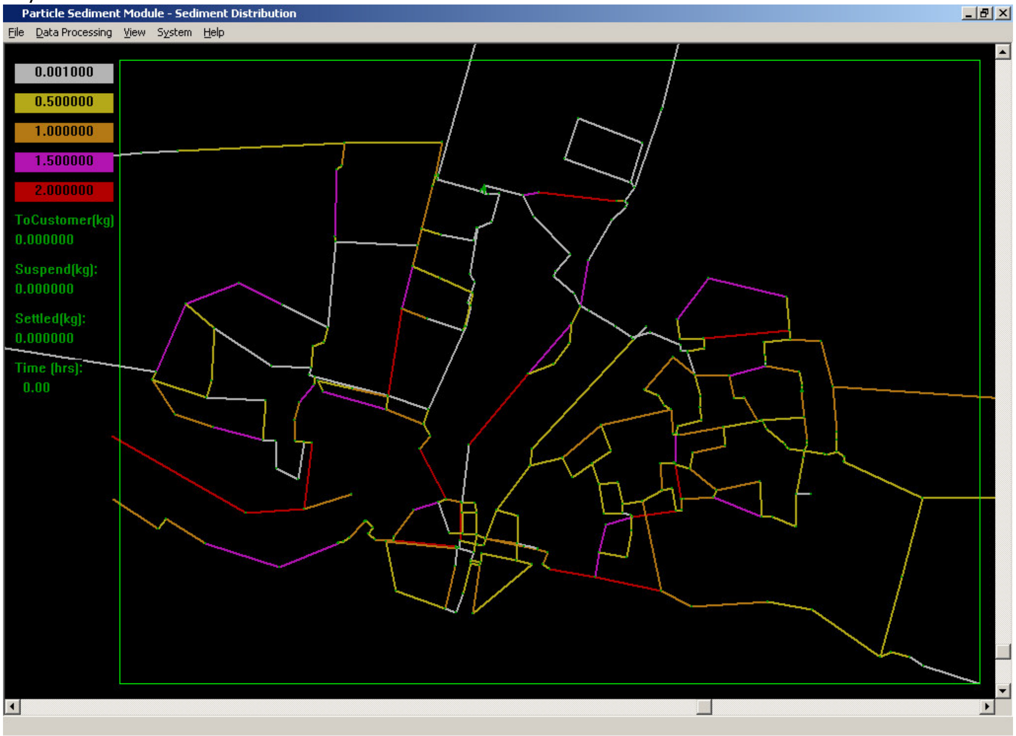
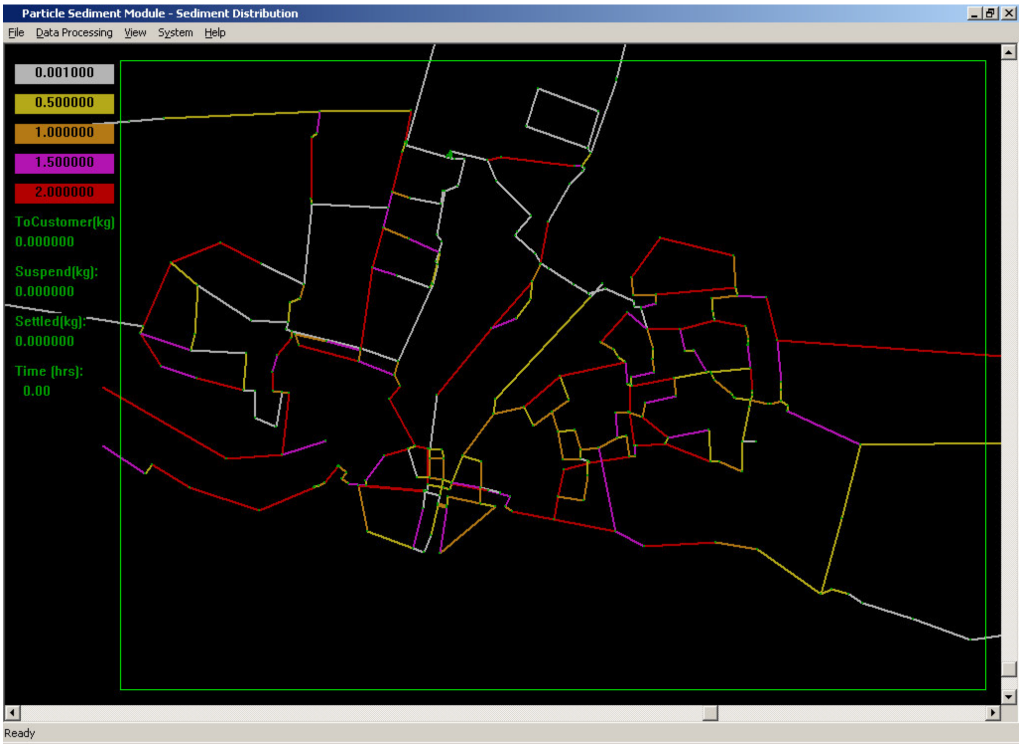


Figure 8-18: Sediment distribution after 1 year, parameters: u_s = 5.5e-6 m/s; u_d= 0.15 m/s; ***<u>u_{rs} = 0.50 m/s</u>****, near Gorinchem*

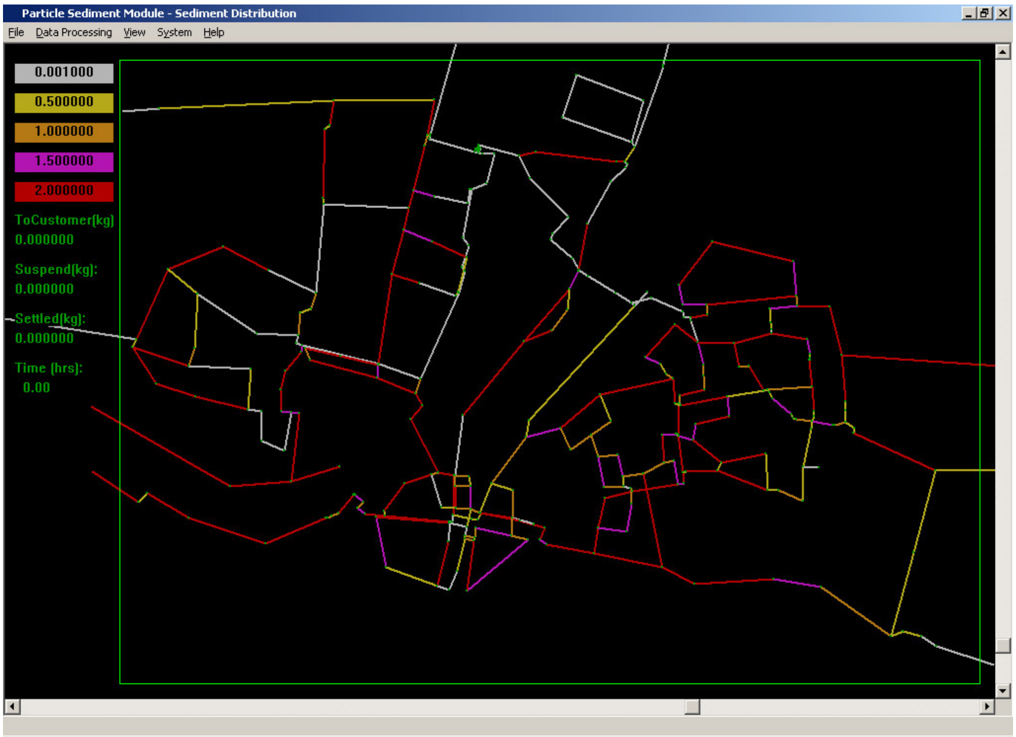
1 week simulation; initial parameters; 5.5e-6 m/s; u_d= 0.15 m/s; u_{rs} = 0.25 m/s



2 weeks simulation



3 weeks simulation



4 weeks simulation