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Hydraulic behaviour of particles in a drinking water distribution system

Design and first operation of a test rig

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Behaviour of Particles in a Network; Test Rig Results

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Test Rig Results

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Preface and Acknowledgments

This work is my Master of Science thesis, in which the description is given of the work done and the results achieved since December 2004 at the Department of Water Management, Faculty of Civil Engineering, Delft University of Technology. During my Master's courses, I decided to do my graduation project on drinking water. After doing an internship in Australia about the quality of drinking water and the behaviour of particles in drinking water transport mains, my attention was attracted. Back in the Netherlands I could join the Q21 project, about water quality in the 21st century. With great pleasure I worked on my project about sedimentation and resuspension of particles in drinking water mains. Before continuing with my thesis, I would like to thank some people who have been of great help during my Master's project.

I would like to thank my graduation committee. Prof. van Dijk had useful comments and remarks during the meetings of Q21 and the drinking water colloquia. Furthermore, it was always possible to discuss results or measurement methods with him. Jan Vreeburg supported me on a daily base and answered my questions in no time. Jasper Verberk had critical remarks, which made me consider the matter from all sides. With Wim Uijttewaal I had useful discussions about the hydraulics of the new test rig.
Building a test rig never ends. After each experiment, it became necessary to build a new pipe, valve or measuring point. Tonny Schuit and Cees Boeter were always there to help me optimising the test rig. Especially Cees Boeter worked days on the test rig. Without him my test rig would be a big mess of tubes, hoses and tie raps.
KIWA Water Research supported me financial and provided me with a particle counter and a turbidity meter.
My last (but not least) thanks go to my parents, who are always very in-

terested in what I am doing. Renze always has time to help me with the English language, making this report in LaTeX, opening valves that were too tightly for me. Beside that, he was able to motivate me when things were not going as I wanted.

Anke Grefte

Delft, The Netherlands
August 2005

Summary

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chapter

1

Introduction

On every website of a water supply company, you find a topic about brown water. Brown water is considered not to be harmful, but it causes inconvenience to customers. Brown water looks filthy and can cause stains in the laundry. On a yearly basis, between 3,000 and 6,000 complaints are made to the water supply companies in The Netherlands about brown water [14]. Discoloured water, another name for brown water, is caused by particles that are not removed during treatment [5] and particles that are formed in the drinking water distribution system [8]. These particles will settle on the bottom of the drinking water transport mains. Discoloured water can occur when the velocity in a drinking water main is increased due to maintenance or fire fighting. Even under normal circumstances, brown water can occur when the water demand is high. Due to the higher velocity, particles are resuspended, which causes the brown colour of the water. Although the water is of good quality after treatment, the customer can temporary get water of less quality. One of the problems in a modern water distribution system is the quality loss of the drinking water during transport. This problem is investigated in a program called "Q21" [8]. The research being done in this program aims to provide high quality water at the tap at all times. The Q21 research program is divided into six themes. The research described in this thesis is part of the "Distribution" theme. The topic will be the behaviour of particles in the drinking water distribution system.
Due to some processes, like corrosion, biofilm formation and coagulation, particles are produced [8]. As a result, a sediment layer has been formed in distribution networks, over the past years. In Figure 1.1, this is schematically shown. The storage of particles in a drinking water distribution system is determined by comparing the incoming mass to the outgoing and mass. The hydraulic conditions in the drinking water distribution system and the type and weight of particles are affecting the storage and transport of particles in the distribution system. In this thesis the sedimentation and

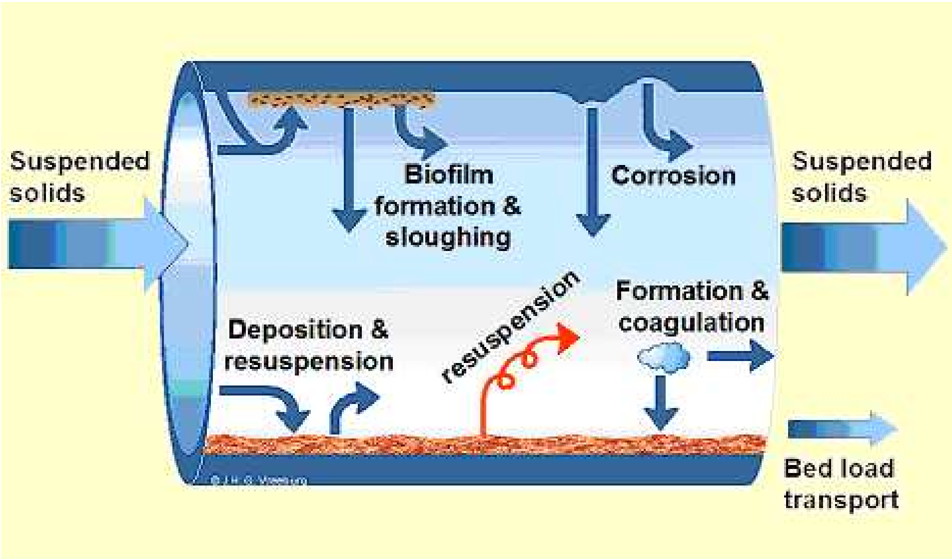

Figure 1.1 Mass balance for a distribution system (by J.H.G. Vreeburg).

resuspension process of the particles in the drinking water are subject of research. In the simplest approach sedimentation is described by Stokes [10]. Shields [2] describes the resuspension process and Berlamont [13] used this theory to make a model for resuspension of particles in pipes. The first objective of this thesis is:

- verifying of the applicability of particle dynamics.

With the knowledge of the particle dynamics it will be possible to quantify the mass balance. For a known incoming load, the amount of sediment leaving a transport main can be calculated. To reach the objective a test rig will be used.In a distribution network the most common inner diameter of pipes is 100 mm. Lut [6] has built a test rig, which consists of one pipe with an inner diameter of 100 mm. Lut has proven that the test rig can be used to investigate the sedimentation process in a distribution main. With the 100 mm pipe test rig it is not known how the sediment is formed in time. To study the actual build up on different time scales an installation should be built with multiple pipes in parallel, see Figure 1.2. Multiple pipes are necessary for not disturbing the build up of the layers by taking out some of the sediment. With multiple pipes it is possible to take out pipes at different moments, while the build up continues in the other pipes, see Figure 1.3.

When using multiple 100 mm pipes, a bigger volume of water is necessary and the test rig will get too big. Scaling down the test rig is the most obvious solution. In this thesis the test rig built by Lut is used and is extended with two pipes with an inner diameter of 32 mm. Scaling down the normal pipe diameter to a smaller diameter can not be done without

Figure 1.2 An overview of a test rig with multiple pipes.

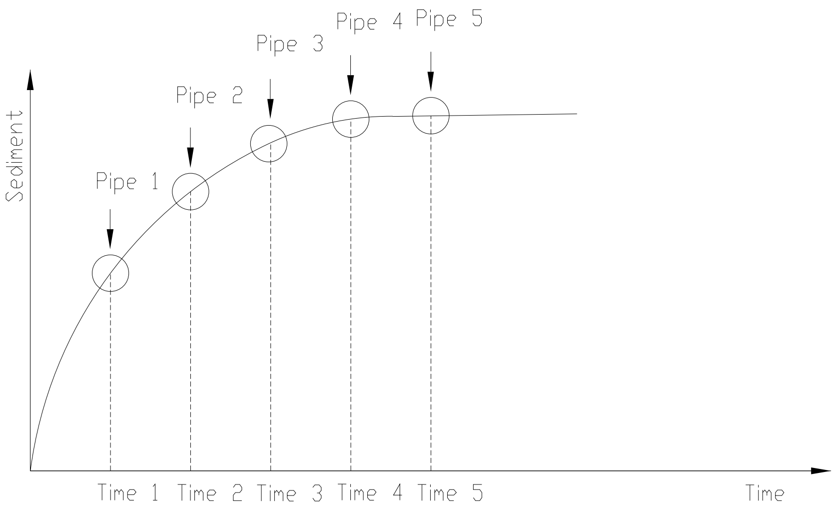

Figure 1.3 The build up of sediment in time in a test rig with multiple pipes.

looking into hydraulics. The second objective of this thesis is:

- investigating of the feasibility of a parallel recirculation test rig.

The remainder of this thesis contains 6 chapters and its contents is as follows. In Chapter 2, the hydraulic conditions in the pipes are explained. In Chapter 3, the theory about sedimentation and resuspension is given. In Chapter 4, the test rig that is build and used for the experiments is described. In Chapter 5, the experiments with kaolinite are described and the results are discussed. Finally, conclusions are drawn from the results and are summarised in Chapter 6, together with some recommendations for future research.

chapter

2

Hydraulics

To investigate the build up of sediment in time, a test rig with some small pipes is necessary. A common diameter for drinking water mains is 100 mm. The test rig consists of pipes with a diameter of 32 mm. Scaling down gives some problems with the hydraulics. The most important models for resuspension are from Shields and Berlamont. The most important parameter in these models is the shear stress exercised on the pipe wall [9]. Therefore, the shear stress needs to be taken into account when scaling down the installation. Sedimentation is described by Stokes. One of the processes that is important in this model is the flow regime. The flow regime can be expressed by the Reynolds number. The flow could be either laminar or turbulent. Under normal conditions the flow in a drinking water distribution system is turbulent. It is possible that the flow in a big pipe is turbulent for low velocities, while it is laminar in a small pipe, for the same velocity. The two pipes are not comparable when that happens. In this chapter, the hydraulics are explained that need to be taken into account for scaling down a distribution main. In Section 2.1, the different flow regimes are described. In Section 2.2, the formulas for wall friction used for laminar flow are explained and in section 2.3, are the formulas for turbulent flow explained. The effect of the temperature on the density, on the absolute viscosity, on the Reynolds number, and on the shear stress is explained in Section 2.4.

2.1 Flow Regime

The flow regime can be characterised by the Reynolds number [3]. The Reynolds number can be thought of as the ratio between inertial and viscous forces in a fluid. For full flowing circular pipes the Reynolds number can be found by using equation 2.1. In this formula Re is Reynolds number, D is the pipeline diameter (m), v is the average fluid velocity (m/s), ρ_w is the fluid density (kg/m^3), μ is the viscosity ($kg/m/s$) and ν is the kinematic viscosity (m^2/s).

$$Re = \frac{vD\rho_w}{\mu} = \frac{vD}{\nu} \tag{2.1}$$

The kinematic viscosity ν is related to the viscosity by:

$$\nu = \frac{\mu}{\rho_w} \tag{2.2}$$

In Table 2.1, the different flow regimes and their accompanying Reynolds number are shown. Normally, the flow of water through drinking water mains is turbulent. This means that the flow is characterised by eddies that produce random variations in the velocity. When the velocities and length scales are small, the viscosity dominates the internal pipe roughness and the flow will therefore be laminar. In the laminar profile, the fluid particles travel in parallel layers. They produce very strong shear stresses between adjacent layers. Although the velocity profile of turbulent flow is more erratic than that of laminar flow, the mean velocity profile actually exhibits less variation across the pipe. The velocity profiles are shown in Figure 2.1.

Table 2.1 Reynolds numbers for various flow regimes.

Flow Regime	Reynolds Number
Laminar	< 2000
Transitional	2000-4000
Turbulent	> 4000

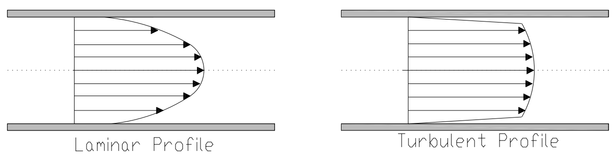

Figure 2.1 Velocity profiles for different flow regimes.

2.2 Wall friction - Poiseuille Equation

Information about pressure drop, head loss and flow rate can be derived from the knowledge of the velocity profile. In this section the equation for the velocity profile of a fully developed laminar flow is developed [16] [17]. In Figure 2.2, the stress balance is given for a circular cylinder of fluid of length L (m) and radius r (m) centered on the axis of a horizontal pipe of diameter D. As shown in Figure 2.1 the velocity is not uniform across the pipe. The cylinder of fluid moves along the pipe. Every movement causes a distortion, but if the flow is fully developed and steady, the distortion on each end of the fluid element is the same. The pipe can be divided in stream lines parallel to the pipe wall. Along these stream lines fluid flows with a constant velocity, although neighboring particles have slightly different velocities. Every stream line has its own velocity. This velocity gradient, combined with the fluid viscosity, produces the shear stress. Although the pressure varies along the pipe from one section to the next, the pressure is constant across any vertical cross section of the pipe, which is only valid if the gravitational effects are neglected. Thus, if the pressure (p (N/m^2)) at section 1 is $p = p_1$, it is $p_2 = p_1 - \Delta p$ at section 2. The pressure will decrease in the direction of flow, so $\Delta p \geq 0$. On the surface of the cylinder of fluid acts a shear stress, τ $(kg/m/s^2)$. This viscous stress is a function of the radius of the cylinder, $\tau = \tau(r)$. Fully developed horizontal pipe flow is a balance between pressure and viscous forces. This because the flow is moving, but not accelerating. Applying Newton's second law: $F_x = ma_x$ and $a_x = 0$ (F (N), m (kg), a (m/s^2)), in Figure 2.2, this is schematically shown. In an equation this gives:

$$(p_1)\pi r^2 - (p_1 - \Delta p)\pi r^2 - (\tau)2\pi r L = 0 \tag{2.3}$$

and simplified to give

$$\tau = \frac{\Delta p}{L}\frac{r}{2} \tag{2.4}$$

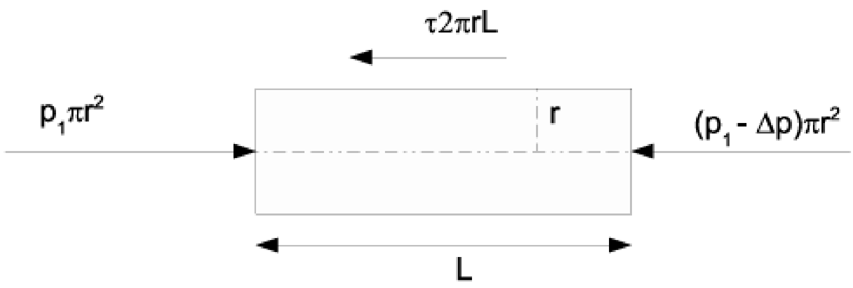

Figure 2.2 Diamgram of a cylinder of fluid

The shear stress is related to the velocity by Newton's equation of viscosity: $\tau = -\mu(\frac{dv}{dr})$. The negative sign being adopted because r is measured outwards. Substitution in Equation 2.4 is obtained:

$$dv = \frac{\Delta p}{L}\frac{r}{2\mu}dr \tag{2.5}$$

Integrating

$$v = -\frac{\Delta p}{L}\frac{r^2}{4\mu} + constant \tag{2.6}$$

There can be no slip at the boundary, so $v = 0$ when $r = \frac{D}{2}$. Using this relationship to evaluate the constant in the above equation it is obtained that

$$v = \frac{\Delta p}{l}\frac{L}{4\mu}(\frac{D^2}{4} - r^2) \tag{2.7}$$

which is the equation of a parabola, thus the velocity in the pipe is in the form of a paraboloid. At the center, $r = 0$, the maximum velocity occurs.

$$v_{max} = \frac{\Delta p}{L}\frac{D^2}{16\mu} \tag{2.8}$$

The mean velocity is

$$v = \frac{v_{max}}{2} = \frac{\Delta p}{L}\frac{D^2}{32\mu} \tag{2.9}$$

Substituting for $\frac{\Delta p}{L}$ in Equation 2.4,

$$\tau = \frac{16\mu vr}{D^2} \tag{2.10}$$

This is the equation of a straight line. The pipe center is where $\tau = 0$. From equation 2.9 the pressure head loss, which is $\frac{\Delta P}{w}$ and w is the specific weight, is equal to $\frac{32\mu vL}{\rho g D^2}$. The head loss ($h_L$ (m)) is:

$$h_L = \frac{32\mu vL}{\rho_w g D^2} = \frac{32\nu vL}{gD^2} \tag{2.11}$$

This is known as Poiseuille's equation. With a measured head loss, combining equation 2.10 and 2.11, the shear stress can be calculated by:

$$\tau = \frac{(h_L)\rho_w g r}{2L} \tag{2.12}$$

For the shear stress on the wall, $r = \frac{1}{2}D$ is obtained

$$\tau_w = \frac{(h_L)\rho_w g D}{4L} \tag{2.13}$$

Laminar flow is unaffected by the nature of the boundary surface, thus the wall roughness can be neglected and is not used in the analysis.

2.3 Wall Friction - Darcy-Weisbach Formula

For turbulent flow, Darcy-Weisbach developed a formula by using dimensional analysis [3] [16]. For the head loss in a pipe, Darcy-Weisbach found that:

$$h_L = 8\frac{fLQ^2}{gD^5\pi^2} = f\frac{Lv^2}{2Dg} \tag{2.14}$$

In these equations, f is the Darcy-Weisbach friction factor (-), g is the gravitational acceleration constant (m/s^2), and Q is the pipeline flow rate (m^3/s). For the Darcy-Weisbach friction factor, f, a functional relation is:

$$f = F(\frac{vD\rho_w}{\mu}, \frac{k}{D}) = F(Re, \frac{k}{D}) \tag{2.15}$$

In this equation k is the internal pipe roughness (m). Numerous formulas exist that relate the friction factor to the Reynolds number and relative roughness. Examples are the Colebrook-White equation (most popular formula), the Swamee-Jain formula (only accurate in specific ranges), the Hazen-Williams formula (particularly used in North America), and the Manning equation (for open channel flow) [3]. Also the Kutter formula (typically used for sewerage calculations) and the Bazin formula (used for irrigation works) are known [7]. The Colebrook-White equation is the most obvious equation to use. The Colebrook-White equation defines:

$$\frac{1}{\sqrt{f}} = -0.86ln\left(\frac{k}{3.7D} + \frac{2.51}{Re\sqrt{f}}\right) \tag{2.16}$$

This equation is an implicit function of the friction factor, iterating is necessary. The Moody diagram, shown in Figure 2.3, is a graphical solution for the Darcy-Weisbach friction factor, developed from the Colebrook-White equation. Next a consideration of the shear stress at the pipe boundary is given. The frictional resistance is equal on the reduction in pressure force or,

$$\tau_w \cdot \pi DL = \Delta p \cdot \frac{\pi D^2}{4} \tag{2.17}$$

Thus

$$\frac{\Delta p}{w} = \frac{4\tau_w L}{\rho_w gD} \tag{2.18}$$

From Equation 2.14 and 2.18 follows that

$$\tau_w = f\frac{2Q^2\rho_w}{\pi^2 D^4} = f\frac{\rho_w v^2}{8} \tag{2.19}$$

or

$$v = (\frac{\tau_w}{\rho_w})^{\frac{1}{2}}(\frac{8}{f})^{\frac{1}{2}} \tag{2.20}$$

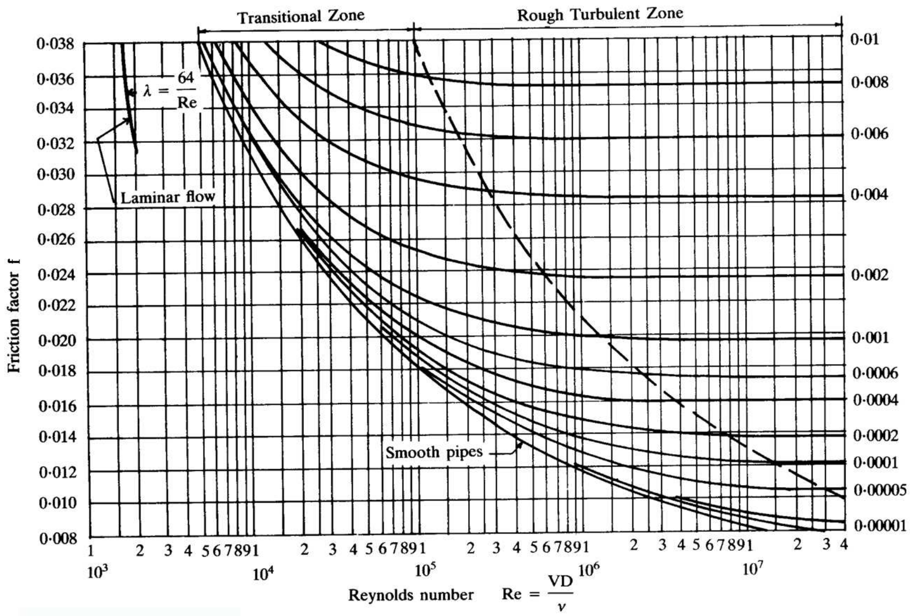

Figure 2.3 The Moody diagram.

2.4 The Effect of Temperature

The viscosity of a fluid is influenced by its temperature. Therefore, the Reynolds number and the shear stress are influenced by temperature.In Figure 2.4, based on [3], the effect of temperature on the viscosity and the density are plotted. Density decreases faster at higher temperatures, while the viscosity decreases faster at a lower temperature. It is important to know the temperature for calculating the Reynolds number or the shear stress during experiments. Viscosity decreases with increasing temperature, which results in a higher Reynolds number (see Figure 2.5) and a lower shear stress (see Figure 2.6). In these figures the effect of the temperature on the Reynolds number and on the shear stress are shown for different velocities and for different pipe diameters. For the bigger pipes and higher velocities this does not cause many problems. However, for the smaller pipes and lower velocities the temperature can be the difference between a turbulent or a laminar flow. Comparison is impossible without using the actual temperature.

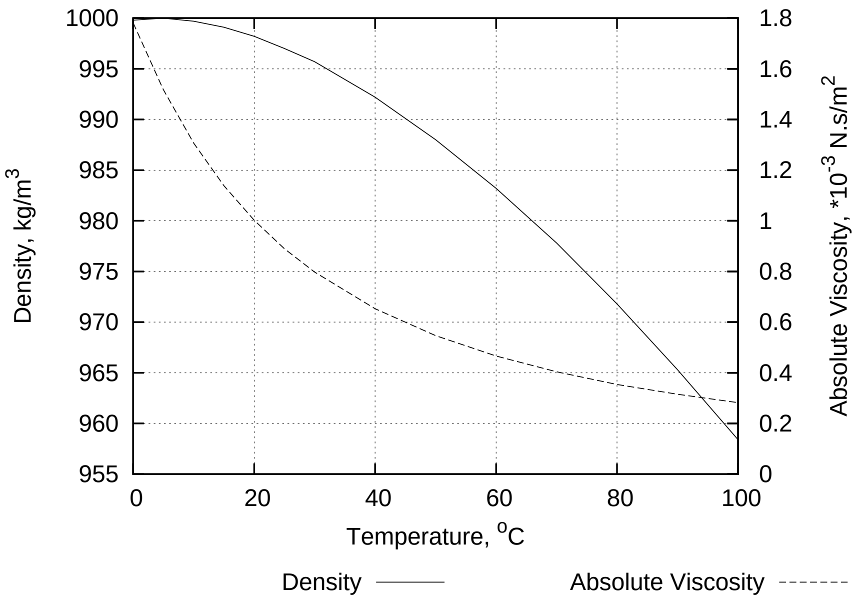

Figure 2.4 The effect of temperature on the density and the absolute viscosity.

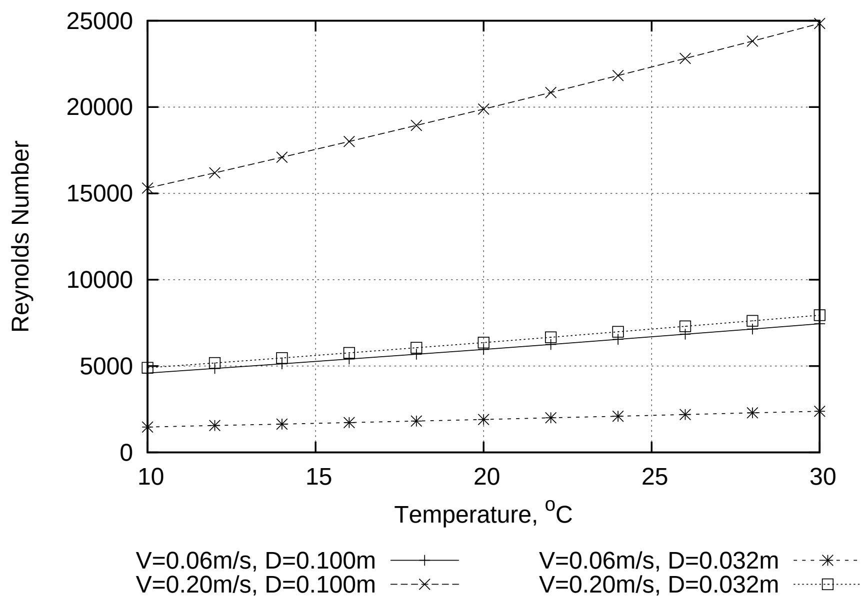

Figure 2.5 The effect of temperature on the Reynolds number.

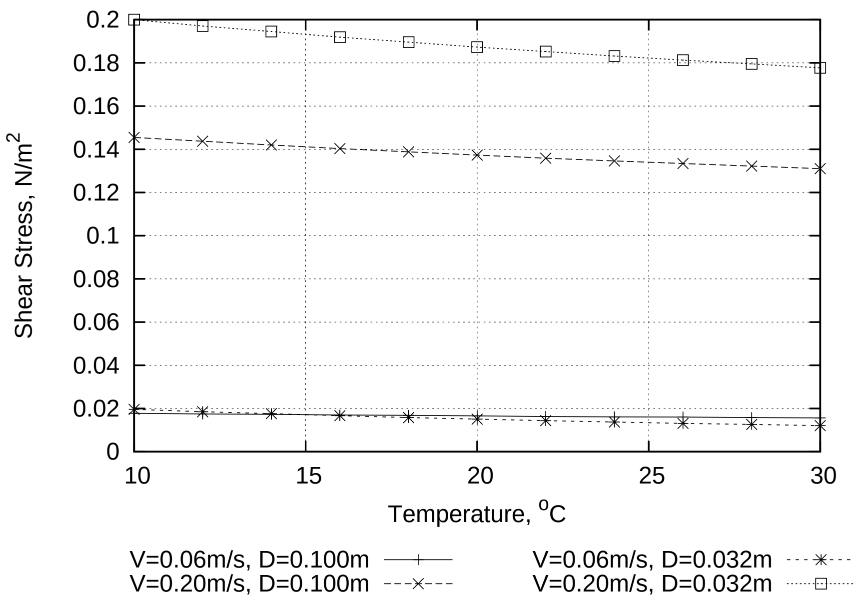

Figure 2.6 The effect of temperature on the shear stress with constant pressure gradient.

chapter

3

Sediment Transport

Even after treatment, there are still particles in the drinking water [14]. These particles can settle and resuspend in the drinking water mains. The first objective of this thesis is verifying of the applicability of particle dynamics. In this chapter, the equations of Stokes, Shields and Berlamont are explained in detail. In Section 3.1, the sedimentation process of the particles according to Stokes is explained. In Section 3.2, the sedimentation process in a horizontal pipe is explained. In Section 3.3, the diffusion process for turbulent flows is given by formulas. Section 3.4 is dealing wirh the resuspension, and explains the equations of Shields and Berlamont.

3.1 Sedimentation

An object in water is exposed to forces. Drag forces are caused by pressure and shear stresses due to friction [10]. The drag force on the object can be expressed by:

$$F = C_s S \frac{\rho v^2}{2} \tag{3.1}$$

In this equation, F represents the drag force (N), S is the surface of the object perpendicular to the flow (m^2), and C_s being the drag coefficient (-). The drag force is directed in the flow direction, as shown in Figure 3.1. Parameter C_s is called the drag coefficient and consists of two components: the effect of the pressure and the friction. Depending on the shape of the object, either the pressure or the friction is more important. The value of the drag coefficient depends on the shape of the object and the Reynolds number. In general, it is not possible to calculate the drag coefficient exactly and therefore it needs to be determined experimentally. In Figure 3.2, the drag coefficient for a cylinder and a sphere are shown relative to the Reynolds number. In Table 3.1, the drag coefficients are given for a sphere. These values are not exact.

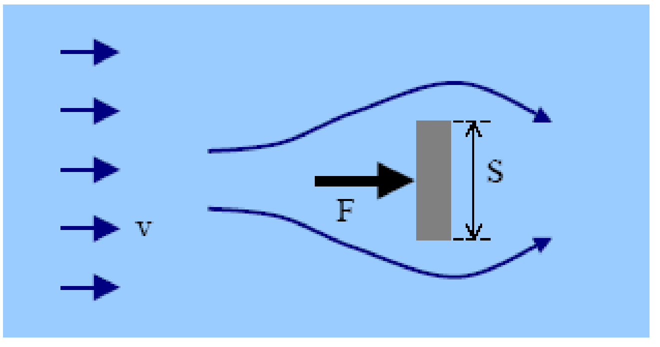

Figure 3.1 The resultant force on an object in a flow [10].

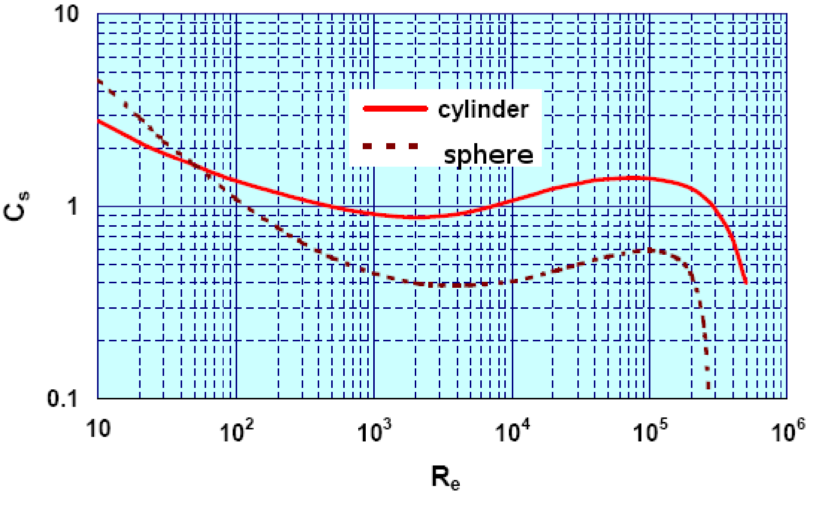

Figure 3.2 The drag coefficient relative to the Reynolds number [10].

When the drag force is known, the stationary settling velocity can be

Table 3.1 Different drag coefficients for a sphere.

C_s	Condition
$\sim 24/Re$	$Re < 1$
$\sim 18.5/Re^{0.6}$	$0.2 < Re < 500$
~ 0.47	$500 < Re < 10^5$

determined for objects that sink in still water. Requiring an equilibrium between the drag force, the buoyancy and the gravity yields:

$$V\rho_s g = V\rho_w g + C_s S\frac{1}{2}\rho_s w_s^2 \tag{3.2}$$

In this equation, V is the volume (m^3). The settling velocity (w_s (m/s)) can be described by:

$$w_s = \sqrt{\frac{2(\rho_s gV - \rho_w gV)}{C_s S\rho_s}} \tag{3.3}$$

Sediment particles have a spherical shape with a diameter d_s (mm) and a density $\rho_s(kg/m^3)$, assuming small particles the occurring Reynolds numbers can be considered small, so C_s is $24/Re$.

$$w_s = \sqrt{\frac{2((\frac{1}{6}\pi d_s^3\rho_s g) - \rho_w g(\frac{1}{6}\pi d_s^3))}{\frac{24\mu}{w_s d_s\rho_s}(\frac{1}{4}\pi d^2)\rho_s}} \tag{3.4}$$

This leads to the Stokes equation:

$$w_s = \frac{g(\rho_s - \rho_w)d_s^2}{18\mu} = \frac{g}{18\nu}\frac{(\rho_s - \rho_w)}{\rho_w}d_s^2 \tag{3.5}$$

The Stokes equation is only valid for Reynolds numbers smaller than 1, $Re = \frac{w_s d_s}{\nu}$.

3.2 Horizontal Flow Sedimentation

Settling is one of the treatment processes of drinking water in settling tanks [12]. According to Stokes, the particles have a certain (vertical) settling velocity and a horizontal velocity ($v = \frac{Q}{BH}$). A tank has a surface load defined as:

$$s = \frac{Q}{BL} \tag{3.6}$$

In this equation s is the surface load (m^3/m^2s), B is the width of the tank (m) and L is the length of the tank (m). After a certain time, t_1 (s), a water package leaves the tank and after a time, t_2 (s), a particle is settled. In the

tank a substantial part of the particles settle when $t_2 \leq t_1$. This means: $t_2 \leq t_1 \Rightarrow \frac{H}{w_s} \leq \frac{L}{v} \Rightarrow \frac{H}{w_s} \leq \frac{BHL}{Q} \Rightarrow \frac{1}{w_s} \leq \frac{1}{s} \Rightarrow w_s \geq s$. This is excluding the effect of turbulence.
The equation for surface load is only valid for rectangular tanks. The drinking water mains are not rectangular but circular. The surface load has to be calculated in a different way. The problem is that a pipe does not have a clear bottom. Kaolinite will accumulate mainly in the bottom half of the pipe [6]. This means that the bottom is the lower half of the pipe. The settling in a pipe can be calculated by:

$$s = \frac{w_s t_1}{t_2} = \frac{w_s \frac{h}{w_s}}{\frac{L}{v}} = \frac{hv}{L} \tag{3.7}$$

In this equation, h represents the distance from the top of the pipe to the bottom. Because the pipe is circular, the maximum distance changes. The average maximum distance is the surface divided by the diameter:

$$h = \frac{\frac{1}{4}\pi D^2}{D} = \frac{1}{4}\pi D \tag{3.8}$$

This gives:

$$s = \frac{\frac{1}{4}\pi D v}{L} = \frac{Q}{DL} \tag{3.9}$$

The surface load is the flow, divided by the projected surface of the pipe. This is also the case at tilted plate settling. The used formulae are only valid for an ideal horizontal flow. This means a uniform composition of the suspension is needed at the inlet over the cross section of the pipe (see Section 3.3). The horizontal velocity is the same in all parts of the pipe and a particle that reaches the bottom is definitively removed from the process. In practise, resuspension of the particles from the bottom can occur, which will be discussed in Section 3.4.

3.3 The Diffusion Process

When particles are injected at a point, they are spread by turbulent motions while being advected with the mean motion [11]. The spreading can be seen as a broadening of the distribution in the direction perpendicular to the mean transport. Sediment can, in a simplified form, be considered as a passive tracer that follows the turbulent fluctuations, but with a constant settling velocity w_s with respect to the flow. For a uniform flow, the transport equation for the sediment concentration (c (kg/m^3)) over the vertical is obtained by:

$$\frac{\partial \bar{c}}{\partial t} - w_s \frac{\partial \bar{c}}{\partial z} - \frac{\partial}{\partial z}\left(D_{zz}\frac{\partial \bar{c}}{\partial z}\right) = 0 \tag{3.10}$$

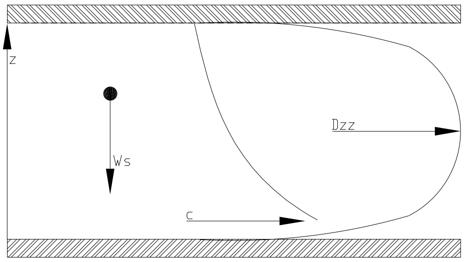

Figure 3.3 Rouse distribution.

When considering a stationary flow, this can be simplified to:

$$w_s\overline{c} + D_{zz}\frac{\partial \overline{c}}{\partial z} = 0 \tag{3.11}$$

D_{zz} is the eddy diffusivity (m^2/s).

$$D_{zz} = \kappa u_* z\left(1 - \frac{z}{h}\right) \tag{3.12}$$

Using this expression in the above transport equation to integrate over z results in the Rouse distribution:

$$\overline{c} = c_0\left(\frac{h-z}{z}\right)^{\frac{w_s}{\kappa u_*}} \tag{3.13}$$

These equations become more clear in figure 3.3. In equation 3.13, c_0 represents an integration constant that follows from a reference concentration, for example near the bottom. The Rouse parameter (the exponent in the above equation) denotes the ratio of the downwards sedimentation velocity and the upward velocities caused by the turbulent fluctuations. In this equation, u_* is the shear velocity (m/s), which can be considered as the product of the mixing length l_m (m) and the velocity gradient $\frac{dv}{dy}$. The proximity of the wall suggests that the mixing length is proportional to the distance from the wall, with the von Karman constant $(\kappa \approx 0.4)$ as the coefficient of proportionality. The differential equation is described as:

$$u_* = \kappa y\left|\frac{\partial \overline{v}}{dy}\right|' \Longrightarrow \frac{v(y)}{u_*} = \frac{1}{\kappa}ln\left(\frac{y}{y_0}\right) \tag{3.14}$$

This results in a logarithmic velocity profile. Since the expression is not valid at the wall $(y = 0)$, an integration constant y_0 is needed to set the velocity somewhere near the wall at a realistic value. For smooth pipes y_0 is determined by:

$$y_0 \approx 0.11\frac{\nu}{|u_*|} \tag{3.15}$$

In the region near the wall, the total shear stress can be assumed to be constant without a big error up to a distance of about one fifth of the pipe radius. For the sub layers, the following equation applies:

$$\tau_w = -\rho \left| u_* \right| u_* \tag{3.16}$$

When the flow is stationary, the mean vertical velocity is zero, as well as the gradient of the longitudinal velocity. Only one term remains, which implies the shear stress to be constant.

$$u_* = \sqrt{\frac{\tau}{\rho}} \tag{3.17}$$

Equation 3.16 and 3.17 are related to Equation 2.19 by:

$$\frac{v}{u_*} = \frac{1}{\sqrt{\frac{f}{8}}} \tag{3.18}$$

3.4 Resuspension

Resuspension of particles is like negative settling. Resuspension is caused by shear forces from the water flowing over the bottom. Different relations between the velocity and the sediment transport exist. Every relation is valid for a different situation. Shields law is the most commonly used method. If the diameters of the particles in the water are known, the critical shear stress can be found by a iterative calculation from the Shields diagram [2] (see Figure 3.4). In the Shields diagram, the beginning of transport is related to the exceeding of a certain value of the shear stress, expressed by a dimensionless parameter Ψ, called Shields parameter:

$$\Psi = \frac{\tau_w}{(\rho_s - \rho_w) g d_s} \tag{3.19}$$

When Ψ is smaller than shown in Figure 3.4, there is motion but no transport. The viscosity expressed by a specific Reynolds number $Re_* = \frac{u_* d_s}{\nu}$ does affect the value of Ψ for small particles. The start of transport for small particles can be reached at laminar flow. In this case, τ_w is proportional to v, and therefore not to v^2 as in a turbulent flow. In Figure 3.5, it is much easier to read u_* as a function of d_s for a density of ρ_s =2650 kg/m^3 and a kinematic viscosity of $\nu = 1 \cdot 10^{-6} m^2/s$.
The Shields law is valid for rivers and canals, but is not applicable to pipes. For pipes, few relations are known between velocity and sediment transport. In sewerage systems, the formulas of Meyer-Peter and Müller are used, but these are fit for circular sewerage [1]. In estuaries, Migniot [13] is mostly used. Migniot is valid for freshly deposited colloidal material. All traditional sediment transport formulas are derived for steady uniform flow in

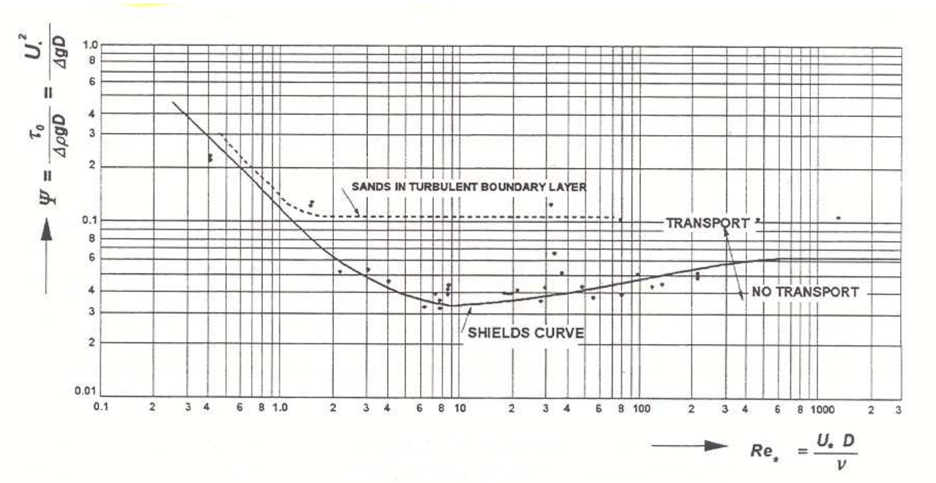

Figure 3.4 Shields diagram for the start of transport [2].

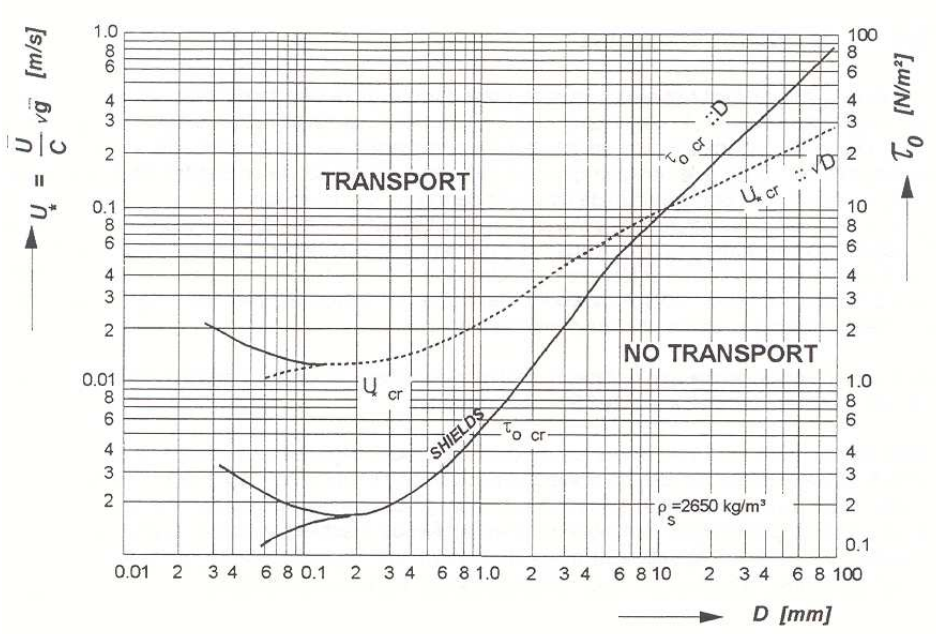

Figure 3.5 Critical shear velocity by Shields [2].

rectangular laboratory flumes and rivers. The conditions in pipes are different than those in the flumes and rivers. Sediment transport is basically controlled by boundary shear stress. From experiments it has become clear that the shear stress distribution is influenced by the filling ratio of the pipe and the presence of sediment. When the filling ratio increases the distribution becomes more uniform. Because the roughness of the wall and sediment bed are different, the mean shear stress of the sediment bed is not equal to the mean shear stress over the cross section. Another experiment demystified that the erosion rate in a pipe with a circular cross section is an order of magnitude larger than in a rectangular flume, when the same excess shear stress is imposed [4]. The formula of Berlamont [13] catered to this symptom. The critical shear velocity $(u_*)_{cr}$ is:

$$(u_*)_{cr} = \sqrt{\alpha_{Berlamont} g \frac{(\rho_s - \rho_w)}{\rho_w} d_s} \tag{3.20}$$

Two values are given for $\alpha_{Berlamont}$, 0.04 and 0.8. The first value is only valid when a particle has to stay in suspension and the second value is to resuspend a particle.

chapter

4

Test Rig

A multiple pipe test rig is necessary to investigate the accumulation of sediment in time. In order to approximate a real-life situation, a test rig with one pipe with a diameter of 100 *mm* has been built by Lut [6]. This test rig will be extended with two small pipes. It is not the multiple pipe test rig, but a test rig to verify whether it is possible to simulate a flow in a big main by means of a flow in a small pipe. In Section 4.1, the single pipe test rig is described that was built by Lut. In Section 4.2, the new installation with the two extra pipes is described. During the experiments with the test rig, a particle counter and turbidity meter are used for the determination of the sediment concentration. Also the deposited material left in the pipes is measured. In Section 4.3, the measuring equipment is described. Some experiments are done with the new test rig to see whether the hydraulic condition in the big pipe and those in the small pipes are comparable. In Section 4.4, the results of these experiments are given.

4.1 Single Pipe Setup

In Figure 4.1, a picture of the single pipe setup, as made by Lut, is shown. The single pipe setup consists of 2 pipes each with a length of 2 meter and a diameter of 100 *mm*. The pipes are connected by a valve. The first two meter is meant to create a fully developed flow. The second two meter is the measuring section. This section can be taken out, in order to weigh the mass of the sediment in the pipe. The pipes are transparent to give the opportunity to visually observe the process in the pipe. A disadvantage of these transparent pipes is the possible growth of biofilm, but this can be neglected as the experiments are performed during a short time only. After the water passed the pipe, it is returned to a vessel. From this vessel the water is pumped into a high, constant head, reservoir. From this reservoir, the water will flow under gravitate back through the pipe. This means that

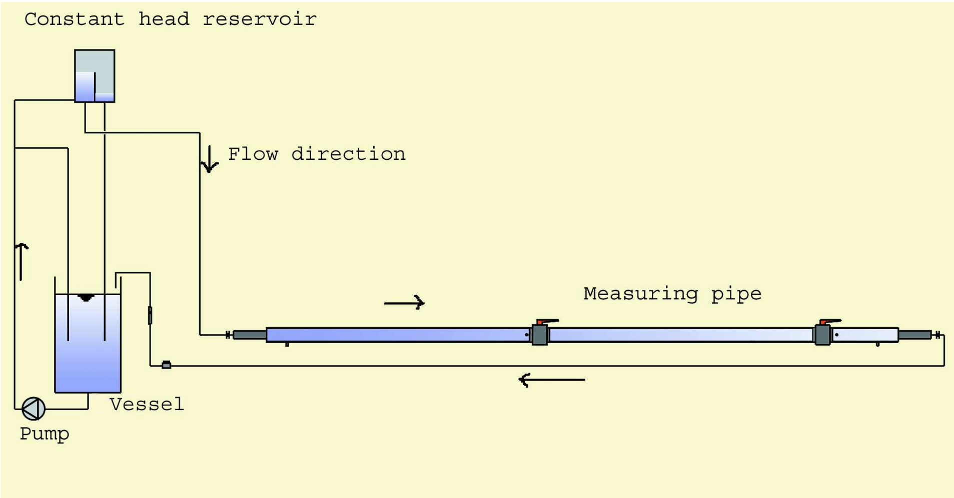

Figure 4.1 Single pipe setup.

the water is recirculated through the system. This is because the amount of water used for the experiment would be too much, when a flow-through system would be used. A normal velocity in a drinking water main is about 0.1 m/s, which is equal to a flow of 2.8 m^3/h. The experiments of Lut took approximately 5 days. One flow-through experiment would take about 330 m^3, while this recirculation system takes about 330 l. The maximum velocity of the water in this test rig is about 0.25 m/s, which are normal conditions for a distribution system. Only for resuspension the velocity should be higher. Resuspension Potential Methods (RPM) are executed with velocities of more than 0.35 m/s [15].

The pipe that returns the water is of a diameter of 50 mm. This is smaller than the big pipe, so the velocities in the return pipe are higher than in the measuring pipe. This will keep particles from settling in the return pipe. The vessel used in this test rig, has a volume of 250 l. In this vessel the water is mixed to avoid settling. A membrane valve is used in combination with two flow meters to regulate the flow. One flow meter for higher flows, up to 7 m^3/h and one for lower flows up to 2500 l/h.

4.2 Multiple Pipe Setup

The single pipe setup described in Section 4.1 already exists and was built by Lut. To study the build up of a sediment layer in a distribution system multiple pipes are necessary, because a pipe can be taken out without disturbing the build up of sediment in the other pipes. By taking out a pipe every day or week or month, the build up can be traced over time. This is

Figure 4.2 Multiple pipe setup.

only possible when the hydraulic circumstances are equal in every pipe.

Building more big pipes is not a good idea, because it will take too much water and the costs for building more big pipes will be too high. The problem with scaling down is that the Reynolds number will drop. This could imply that the flow is not turbulent, but laminar, which gives results that are not representative for the real pipe with a diameter of 100 mm. In Figure 4.4, this is illustrated. For example, the flow is turbulent in the big pipe at a velocity of 0.05 m/s, while the flow is laminar at that velocity for a small pipe. To study this phenomenon, the test rig is extended with two small pipes with a diameter of 32 mm in a different setup (see Figure 4.2 and Figure 4.3). The water is going from the vessel through a feed pipe to the pump. By a pump the water goes to the high reservoir. In this reservoir a construction is made to prevent shortcuts. From this reservoir the water is going under gravitational forces to a branch. Here the water is split. The biggest amount is going to the big pipe. Before it enters the big pipe, the water passes a valve which gives the opportunity to close the big pipe. If that is the case all the water goes to the next branch, were it is divided over the two small pipes. The pipes have a valve at the start of each pipe as well, to close one pipe when necessary. At the end of the big pipe, the two flow meters are used as described in Section 4.1. After the two small pipes only one flow meter is used. An assumption is that the flow is equally divided over the two small pipes. After passing the flow meters, the water

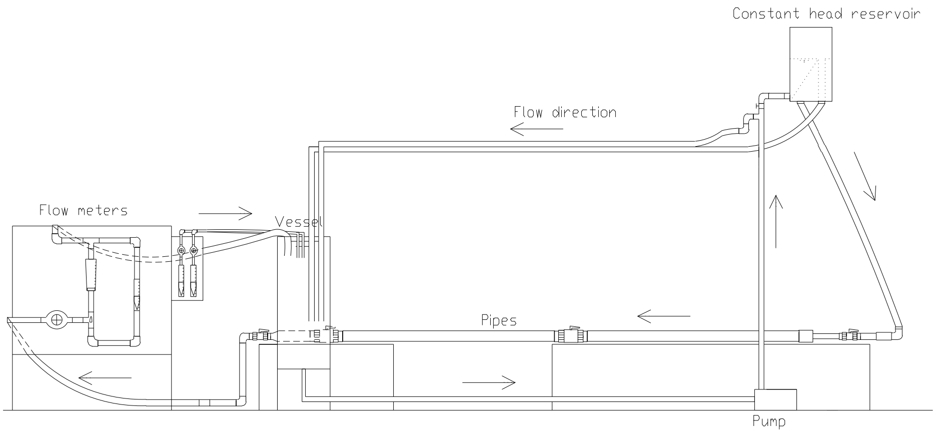

Figure 4.3 Side view multiple pipe setup.

is going back in the vessel and can be recirculated. In the vessel a stirrer is placed, to make sure the water is mixed and particles will not settle in the vessel. In the whole system approximately 330 l of water is used. The vessel contains about 200 l, and the big pipe, its supply and outlet pipes together have a volume of about 50 l. The small pipes have a volume of 10 l of water and the rest of the installation has a volume of 70 l. Manometers are placed on the small pipes, to measure the head loss. This makes it possible to see whether the flows in both small pipes are the same.

4.3 Measuring Equipment

The test rig is used for several experiments. During these experiments equipment is used for measuring the turbidity (see Section 4.3.1), the particles (see Section 4.3.2), and the deposited material (see Section 4.3.3).

4.3.1 Turbidity Meter

During the experiments two turbidity meters are used. The Sigrist turbidity meter, CT65 VIS, (see Figure 4.5) and he HACH turbidity meter (see Figure 4.6). The HACH measured the turbidity at the beginning of the pipes. The Sigrist measured the turbidity at the end of one of the pipes. All pipes

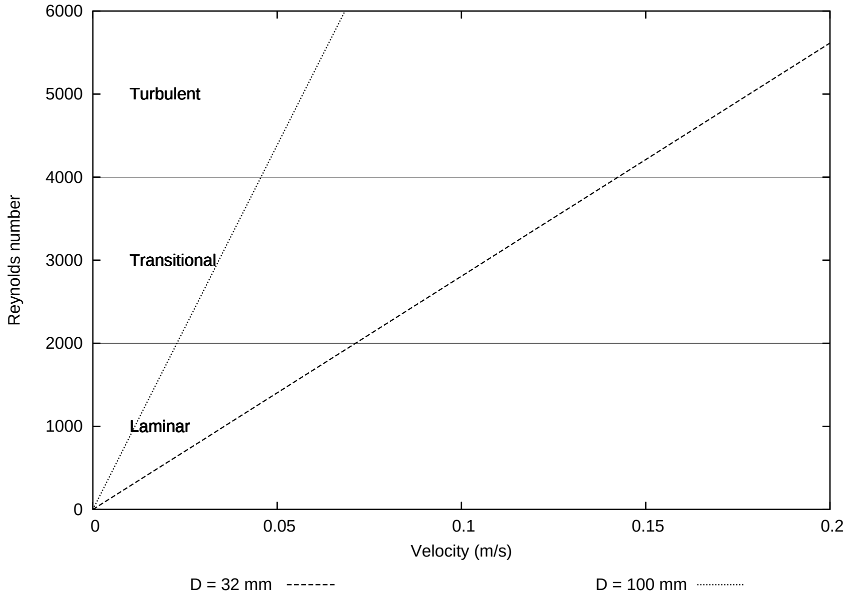

Figure 4.4 Reynolds number plotted against velocity for two pipe diameters.

have connections for the turbidity meter. The HACH turbidity meter can measure turbidities in the ranges of 0 to 2 FTU, 0 to 20 FTU, and 0 to 200 FTU. The Sigrist is only able to measure turbidity in the range of 0 to 2 FTU, and 0 to 5 FTU. At the start of the experiment, the turbidity is much higher than 5 FTU, so the HACH was used for the higher ranges. The Sigrist and the HACH are not comparable. The HACH gives a turbidity of 30 to 40% higher than the Sigrist.

Figure 4.5 Sigrist

Figure 4.6 HACH

4.3.2 Particle Counter

The particle counter counts the number of particles. It determines the size of a particle and it classifies the particles in ranges, which can be determined before starting the experiment. For the first experiment a MET ONE particle counter of Delft University of Technology (DUT)is used. The ranges of this particle counter are: 2-3, 3-4, 4-5, 5-7, 7-9, 9-12, 12-15, 15-20, all in micron. For the other experiments a MET ONE particle counter from KIWA Water Research (KIWA) is used. First, ranges of 2-3, 3-5, 5-7, 7-10, 10-15 and bigger than 15 micron were used. For the last experiments, the ranges are changed to 2-3, 3-4, 4-5, 5-6, 6-10 and bigger than 10. This makes it easier to see which particles are resuspended at different flow velocities. Both particle counters measure particles accurately from a concentration less than 10,000 particles per ml. When the concentration is above 10,000 particles per ml, the chance that more particles together can be seen as one big particle, is more than 10%. When this happens the particle counter counts more big particles and less small particles than the water actual contains.
Analysing the data output of the particle counter is difficult, because the particle counter generates a large amount of data. Little is known about the best way of analysing the data. Kivit [5] introduced the ERF (Extreme Range Factor).

$$ERF = \frac{\text{particles of the smallest size range}}{\text{particles of the largest size range}} \tag{4.1}$$

With the ERF a clear graph can be plotted of the assumed fast decrease in the concentration of large particles and the relatively slow decrease in the concentration of small particles. Another way of analysing the particle data is developed by Haarhoff. Haarhoff developed a model that produces a best-fit line through the discrete numbers produced by the particle counter. This model is still in development.

4.3.3 Suspended Material

One of the purposes of this thesis is to make a quantitative mass balance. To measure the amount of sediment in the pipes, one part of the pipes is taken out and emptied. The water and solids are filtrated as described in Appendix C. After filtration, the filter is dried and weighed.

4.3.4 Manometer

On the two small pipes manometers are placed. With these manometers the head loss in the pipes could be measured, which makes it possible to see whether the flows in both small pipes are the same. Whit a known head loss it is also known if the flow in a pipe is turbulent or laminar. For low

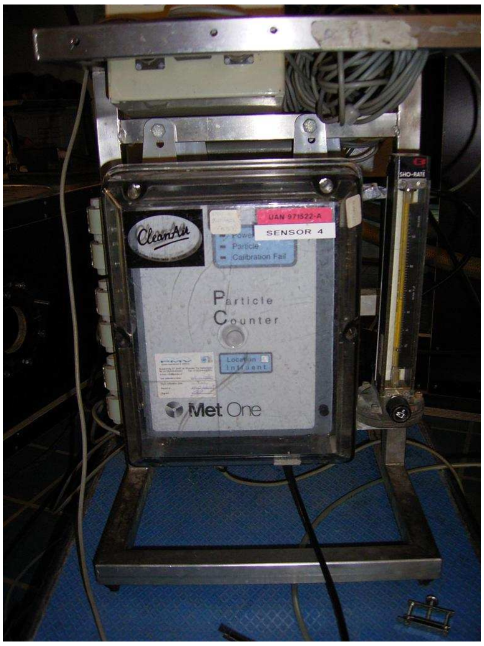

Figure 4.7 MET ONE, particle counter.

velocities there is not much head loss, which makes the values not accurate. The difference between 0.3 mm and 1 mm hight difference of the water columns is not clear on the used manometers (see Figure 4.8).

4.4 Hydraulic Behaviour

In this section the experiments are described, which are done to check whether the flow distribution between the small pipes is equal. Three experiments are conducted with a constant discharge of 5 m^3/h going through the big pipe, and a fourth experiment without any flow through the big pipe. For different velocities in the small pipes the head losses are read from the manometers. The flow meter measured the flow through both pipes, so no distribution could be made between the two tubes.
In Figure 4.9, the head losses at different flow velocities are given for pipe 1. The bold line is the calculated line for turbulent flow, according to equation 2.14. The dashed line with crosses is the calculated line for laminar flow, according to equation 2.11. The flow in pipe 1 is turbulent for all velocities.
In Figure 4.10, the head losses are provided for pipe 2. For velocities lower than 0.15 m/s, it is unclear whether the flow is turbulent or laminar. For velocities higher than 0.15 m/s, the flow is considered turbulent, although

Figure 4.8 Manometers used for measuring the head loss.

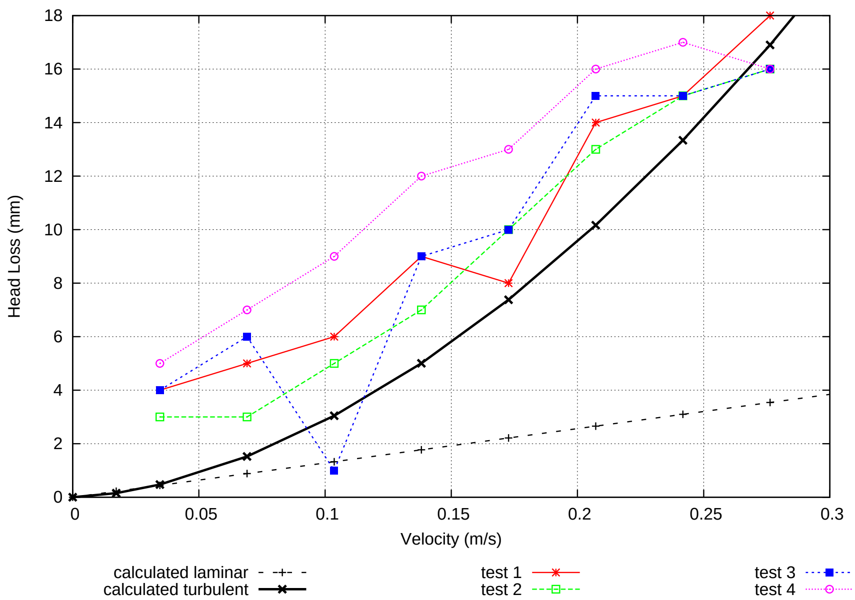

Figure 4.9 Head loss for different velocities in pipe 1.

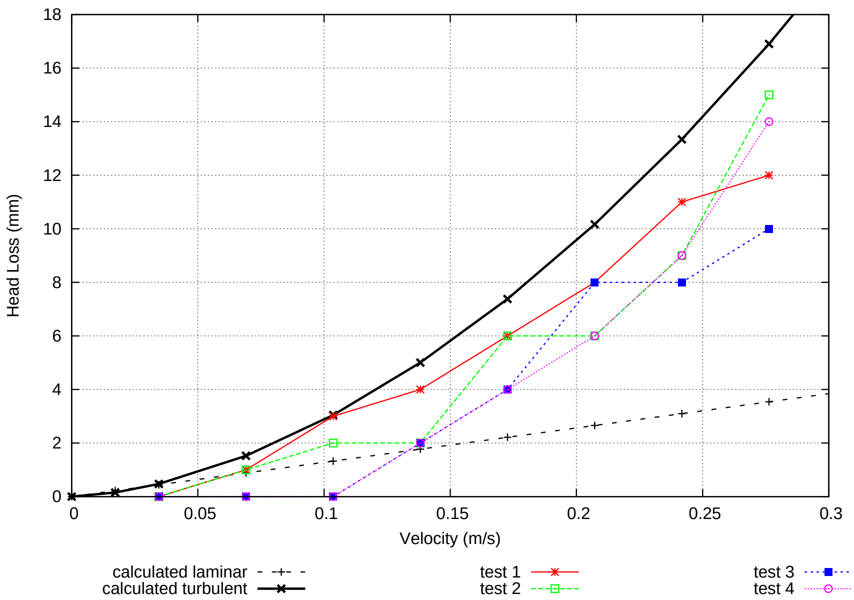

Figure 4.10 Head loss for different velocities in pipe 2.

the head loss is less than in pipe 1 for the same velocities. In both figures, every experiment gives different results. No conclusion can be drawn from these results about the flow regime. The two small pipes share a manometer and a flow meter. Separating the two pipes by giving them both a flow meter and a manometer, results in both pipes having the same flow regime at given velocities (see Figure 4.11). This figure makes clear that from a velocity of 0.1 m/s onwards the flow is turbulent.

One of the questions was whether a small pipe is comparable to a big pipe as used for drinking water mains. The flow in drinking water mains is almost at all times turbulent. In Table 4.1, the Reynold numbers are given for a pipe with a diameter of 32 mm at different flow velocities.

According to these calculated values the flow will be laminar for velocities

Table 4.1 Reynolds numbers and shear stresses for different velocities in pipes with a diameter of 32 mm

$D = 0.032m$; $\varepsilon = 0.00005$; $T = 10°C$

V(m/s)	Re	τ $(N/m^2)(lam)$	τ $(N/m^2)(tur)$
0.06	1470	0.0196	0.0255
0.09	2204	0.0294	0.0506
0.14	3429	0.0457	0.1078
0.20	4898	0.0653	0.2000

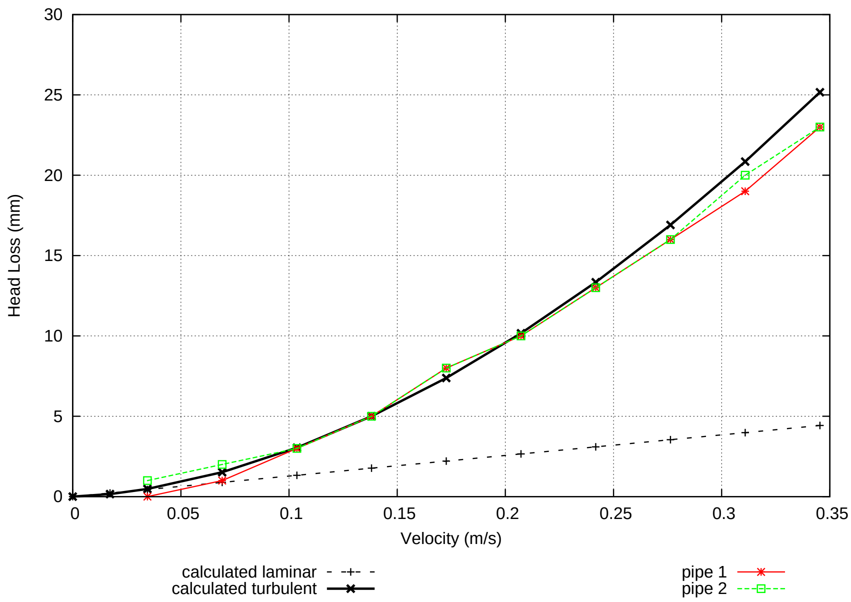

Figure 4.11 Head loss for different velocities in the new situation.

lower than 0.06 m/s . For velocities between 0.06 m/s and 0.20 m/s the flow is transitional. Only for velocities higher than 0.20 m/s the flow will be turbulent, because the Reynolds number is bigger than 4,000. To compare the big pipe to the small pipe the shear stress has to be the same. In Table 4.1, the difference between calculating the laminar shear stress and the turbulent shear stress are provided. For a velocity of 0.14 m/s the difference between laminar and turbulent flow is bigger than 100%. This table proves that it is impossible to compare the big pipe to the small pipe, based on a calculated shear stress. In Figure 4.11 was already proven that the flow in the small pipes is turbulent for velocities higher than 0.1 m/s in stead of the calculated velocity of 0.20 m/s.

With the measured head loss it is possible to calculate the actual shear stress in the small pipes by using Equation 2.13[1], explained in section 2.2. This equation is valid for laminar and turbulent flow. In table 4.2 the Reynolds numbers and shear stresses for different velocities in the big pipe are provided. Because the Reynolds numbers are bigger than 4,000, the flow is turbulent for all velocities. The shear stresses can be calculated exactly. The head loss and velocities needed in the small pipes, to get the same shear stress as in the big pipe, can now be calculated for velocities higher than 0.1 m/s, see Table 4.3.

[1] $\tau_w = \frac{(h_L)\rho g D}{4L}$

Table 4.2 Velocity, Reynolds number, Shear stress in a big pipe.

$D = 0.1m$; $\varepsilon = 0.00005$; $T = 10°C$

V(m/s)	Re	τ (N/m^2)
0.06	4592	0.0178
0.09	6889	0.0359
0.14	10715	0.0776
0.20	15308	0.1455

Table 4.3 Shear tress, head loss and velocity in a small pipe.

$D = 0.032m$; $\varepsilon = 0.00005$; $T = 10°C$

τ (N/m^2)	$h_L(mm)$	V(m/s)
0.0776	4.0	0.12
0.1455	7.4	0.17

chapter

5

Experiments

In this chapter the experiments as carried out with the test rig are described. For each experiment, the objective is given, followed by a description of the experimental setup. After that the results and a discussion are provided. From the end of February until half May six experiments are carried out. The objective of these experiments is to verify the Stokes equation and to test the applicability of the Shields and Berlamont equations. The test rig is filled with normal tap water and a certain amount of kaolinite. Kaolinite is used because it has about the same particle size distribution as found in drinking water [6]. During the first two experiments, all three pipes, two small pipes, and one big pipe, were used (see Section 5.1). The velocities in both experiments were different, but the amount of kaolinite added to the tap water is the same in both experiments. The focus of these two experiments was on sedimentation. The third experiment was focused on sedimentation, but by accident some data about resuspension were collected (see Section 5.2). The velocity used in this experiment is the same as in the first experiment, the difference with the first experiment is the amount of kaolinite added to the tap water. In experiment 3, it is tried to measure sedimentation, by comparing the inflow and the outflow of a pipe. In the fourth experiment, only the small pipes are used (see Section 5.3). After all the sediment has settled, the velocity is increased and some sediment resuspended. In the fifth experiment, only the small pipes are used. A lot of kaolinite is added. After settling, the velocity is increased in experiment 6 (see Section 5.4). A summary of the experiments is given in Table 5.1 and Table 5.2.

Table 5.1 A summary of all experiments.

Exp.	Velocity (m/s)		Amount	Surface Load (m^3/m^2s)	
	Big Pipe	Small Pipe	Kaolinite (gram)	Big Pipe	Small Pipe
1	0.06	0.055	10.00	$1.18 \cdot 10^{-3}$	$0.35 \cdot 10^{-3}$
2	0.14	0.100	10.00	$2.75 \cdot 10^{-3}$	$0.63 \cdot 10^{-3}$
3	0.06	0.055	5.00	$1.18 \cdot 10^{-3}$	$0.35 \cdot 10^{-3}$
4	x	0.055	4.18	x	$0.35 \cdot 10^{-3}$
5	x	0.055	9.80	x	$0.35 \cdot 10^{-3}$
6	x	0.060	x	x	$0.38 \cdot 10^{-3}$

Table 5.2 A summary of all experiments.

Exp.	Wall Shear Stress (N/m^2)		Flow Regime		Sedimentation
	Big Pipe	Small Pipe	Big Pipe	Small Pipe	Resuspension
1	0.0178	0.0178	Turbulent	Laminar/Transitional	Sedimentation
2	0.0776	0.0766	Turbulent	Turbulent	Sedimentation
3	0.0178	0.0178	Turbulent	Laminar/Transitional	Sedimentation
			x		Resuspension
4	x	0.0178	x	Laminar/Transitional	Resuspension
5	x	0.0178	x	Laminar/Transitional	Sedimentation
6	x	0.0178	x	Laminar/Transitional	Resuspension

5.1 Sedimentation in the Pipes

Experiment 1 and 2

5.1.1 Objective

The objective of the two first experiments is to compare the effect of different velocities on sedimentation. In Experiment 2 a higher velocity is used than in Experiment 1. The higher the velocity, the higher the surface load and the less sediment will settle. The expectation is that in Experiment 2, it will take longer before all the sediment is settled, because of the higher velocity, kaolinite has less time to settle in the pipe. Another objective of these experiments is to get some experience with the test rig and the measuring equipment.

5.1.2 Experimental Setup

In these experiments the big pipe and the two small pipes are used. At the start of the experiment, 10 grams of kaolinite is added in the vessel. In the first experiment the velocity in the big pipe is 0.06 m/s and in the small pipes 0.055 m/s. The two small pipes share one flow meter. After approximately 5 days the pipes are taken out and the amount of deposited material is determined. During the experiment, the turbidity is measured at two different times, in different parts of the installation. This makes it possible to know whether the sediment is homogeneously divided over the installation. A particle counter of DUT is used. This particle counter counted the particles at the end of the big pipe.
In the second experiment the velocity is 0.14 m/s in the big pipe and 0.1 m/s in the small pipes. To measure the particle counts, a particle counter of KIWA is used. For measuring the turbidity the HACH turbidity meter and the Sigrist turbidity meter are used. The water first flows through the HACH and after that through the Sigrist turbidity meter. Both instruments measure the same water. The turbidity meters are placed at the end of the big pipe on a side stream of the particle counter.

5.1.3 Results and Discussion

In Table 5.3, the turbidity on two different times is given for different parts of the installation. The turbidity is measured by a HACH turbidity meter, with samples. This way of measuring the turbidity is not accurate. The results are not clear. There is a difference between the parts of the installation and a difference in time. The latter is clear, because of settling, but the decrease in turbidity is not the same in every part. From this table it is impossible to conclude that the particles are divided equally over the installation.

Table 5.3 Turbidity in different parts of the installation.

Part of the Installation	Turbidity (FTU) 02-03-05 17.00	Turbidity (FTU) 03-03-05 12.00
Vessel	1.52	1.10
Outflow Small Pipes	1.37	0.928
Outflow Particle Counter	1.12	1.12
Outflow Big Pipe	1.79	0.924
Overflow	1.21	0.871

In Figure 5.1, the particle distribution of the first experiment is given. This graph contains a lot of gaps in the data, because the DUT particle counter was only able to measure for 1 day. This is a maximum, a couple of times it crashed within one day. In Figure 5.2, the cumulative particle volume

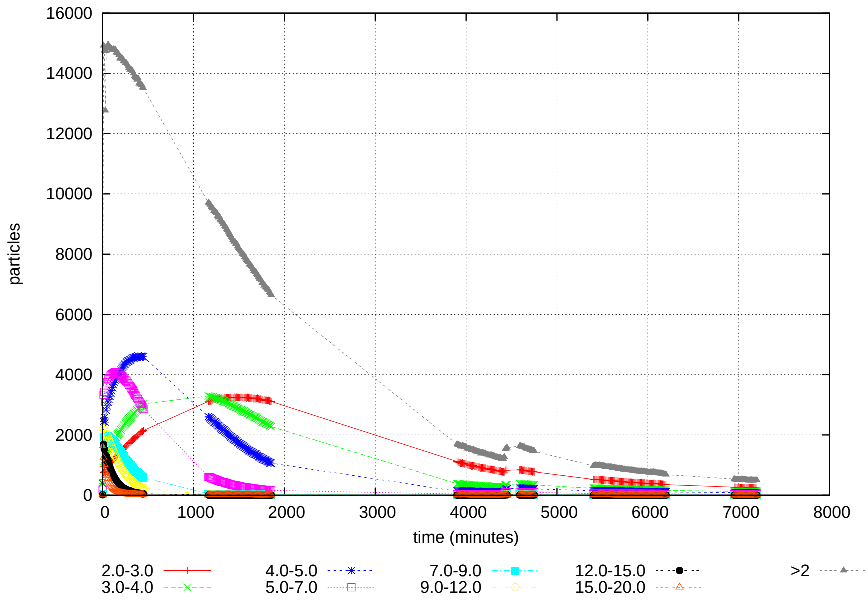

Figure 5.1 Particles in the different ranges, experiment 1.

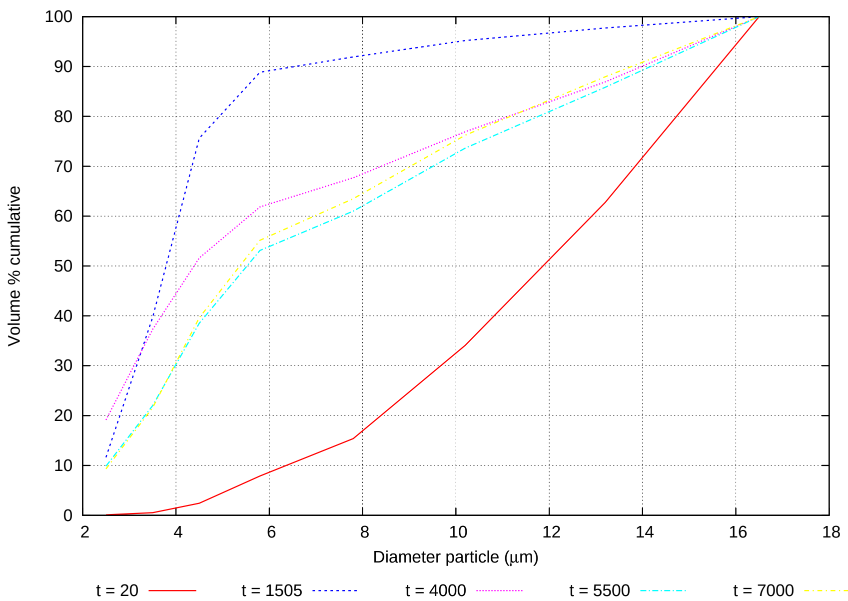

Figure 5.2 Cumulative particle volume distribution at different times.

distribution at different times is plotted. In this graph the volume is ploted on the y-axis and the particle diameter on the x-axis. This graph makes clear that the proportion of big particles increases in time. After 1 day, 90% of the particles is smaller than 6 micron, while after 2.75 days 90% of the particles is smaller than 14 micron. Or after 1 day 95% is smaller than 10 micron and after 2.75 days only 75% is smaller than 10 micron. The bigger particles will settle faster than the smaller particles. Not all the big particles settle, it looks like an equilibrium will be reached between settling and resuspension. The smaller particles settle slowly, while the big particles keep constant, so the ratio big particles got bigger during time. This does not mean that there are more big particles than small particles, because the volume is plotted and not the amount of particles.
At the end of the test, the pipes are taken out. The particles are settled, in the big and in the small pipes, in the bottom half of the pipe. The deposited material is measured as explained in Appendix C. In Table 5.4, the results are given. In the vessel and in the high reservoir, there are no particles, or too few to weigh. Between the two small pipes a difference is found. This is probably because some water and sediment of pipe 1 got lost during pouring the water in a measuring-glass. The same happened with cleaning the big pipe, a lot of water and sediment got lost. It was impossible to take all the sediment out of the big pipe. The sediment stuck on the bottom. The pipe was to heavy to shake with, so a lot of sediment stayed in the pipe. A kind of big spoon is necessary to clean the big pipe, but that was not available. The deposited material in Table 5.4 is not all the material that was in the pipe. The surface load in the big pipe is 3.4 times higher than in the small pipe, whereas the surface on which the sediment settles is 3.125 times higher, so 3.4/3.125 = 1.088 times less sediment per m^2 will settle. Only the sediment in small pipe 2 is measured accurately. In the small pipe 1 there will be the same amount as in the small pipe 2, and in the big pipe should there be 163.1 mg/m [1]. In the big pipe 7% is left, and not filtrated. The total amount of kaolinite in the pipes should be 1.11 grams [2]. At the beginning of the experiment 10 grams of kaolinite is added, this means that 8.89 grams of kaolinite is settled on other places in the installation. Although the velocity in the feed pipe is much higher than the velocity in the measuring pipe, some kaolinite is settled in this pipe. Also kaolinite is settled in the high reservoir, in the vessel and in the tubes (see Appendix F).

In Figure 5.3 the Rouse distribution is given in a small pipe for a kaolinite particle of 5 micron. For calculating the Rouse distribution Equation 3.13 is used. For calculating u_* Equation 3.14 and Equation 3.15 is used and u_* is determined by a iterative calculation [3]. The velocity is assumed constant

[1] $1130mg/1.088 = 1038.6mg/m^2 = 163.1mg/m$

[2] $(56.8 + 56.8 + 163.1)mg * 4m = 1.11g$

[3] $u_* = \frac{0.055*0.4}{(ln \frac{yu_*}{0.11*(8.96 \cdot 10^{-9})})}$

Table 5.4 Sediment in different parts of the installation.

Part of the Installation	Sediment per Meter Pipe (mg/m)	Sediment per m^2 (mg/m^2)
Small Pipe 1	52.7	1048.4
Small Pipe 2	56.8	1130.0
Big Pipe	151.2	962.6
Vessel	0	
High Reservoir	0	

over the height of the pipe. The concentration is about the same over the height of the pipe, because c/c_0 is 1 everywhere.
During the second experiment, the water velocity is higher than in the first experiment. In Figure 5.4, the particle distribution of experiment 2 is given. In this graph, the total amount of particles decreased two times as rapid. This is because of the particle counter. The flow through the particle counter should be 100 ml/min, but due to fouling the flow drops and the particle counter counts the particles for the same time, but in less water. After 2 days, almost all particles have settled.

In Figure 5.5, the cumulative volume distribution at different times for experiment 2 is plotted. Bigger particles settle first and their equilibrium is reached sooner than the smaller particles. After a certain time, the big particles do not settle anymore, while the small particles are still settling.

In Figure 5.6, the turbidity, measured by both turbidity meters, is given. The turbidity meters measured the same water, but the Sigrist turbidity meter measures lower values than the HACH. At the end of the experiment the deposited material is measured. The particles have settled in the bottom half of the pipes. In Table 5.5, the amount of sediment for the two small

Table 5.5 Sediment in the small pipes.

Part of the Installation	Sediment per Meter Pipe (mg/m)
Small Pipe 1	122.0
Small Pipe 2	121.0

pipes are given. Comparing these values with the values of experiment 1 (see Table 5.4), much more sediment is settled in these pipes. The surface load in the second experiment in the small pipes is 1.8 times higher than in the first experiment, but 2.3 times more sediment has settled. The flow through the installation in the first experiment is 2020 l/h and in the second experiment 4580 l/h, this is 2.3 times higher than in the first experiment. The settling

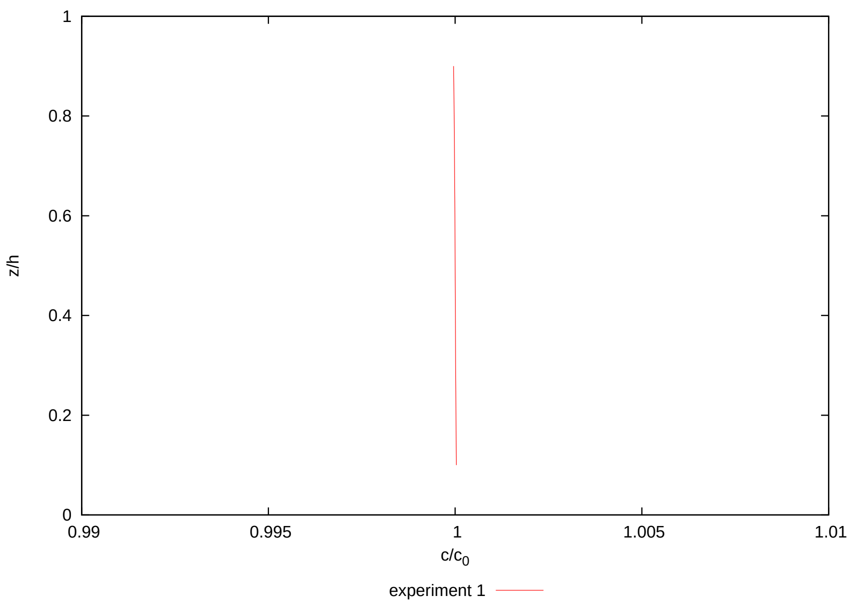

Figure 5.3 Rouse distribution, experiment 1.

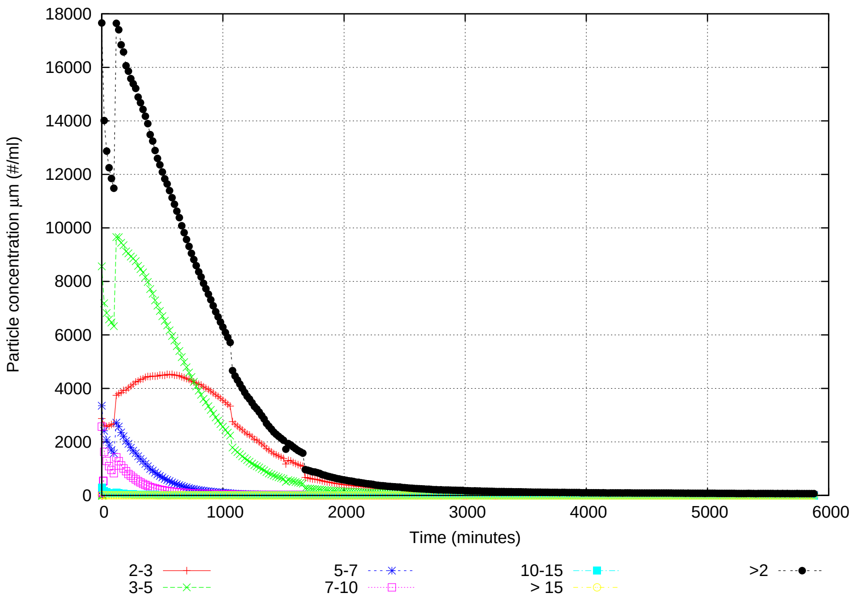

Figure 5.4 Particles in the different ranges, experiment 2.

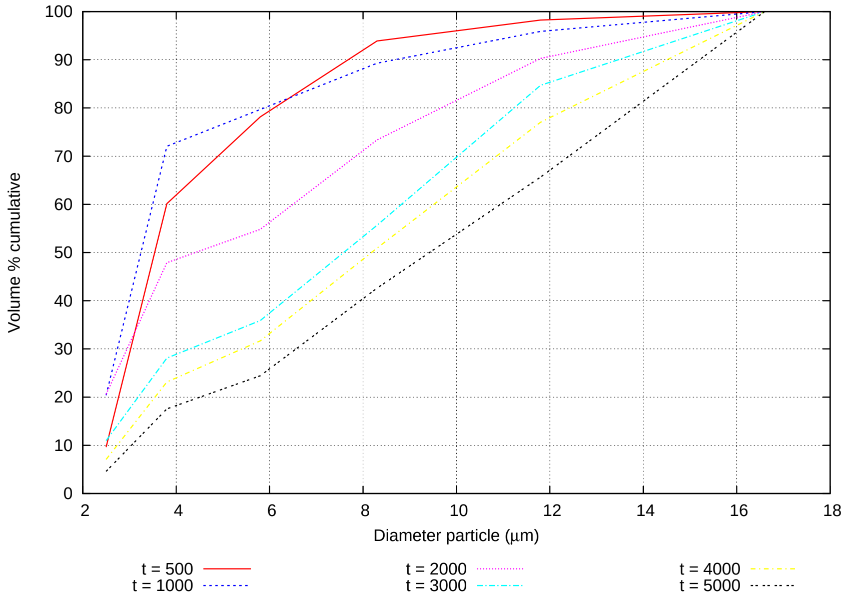

Figure 5.5 Cumulative volume distribution at different times.

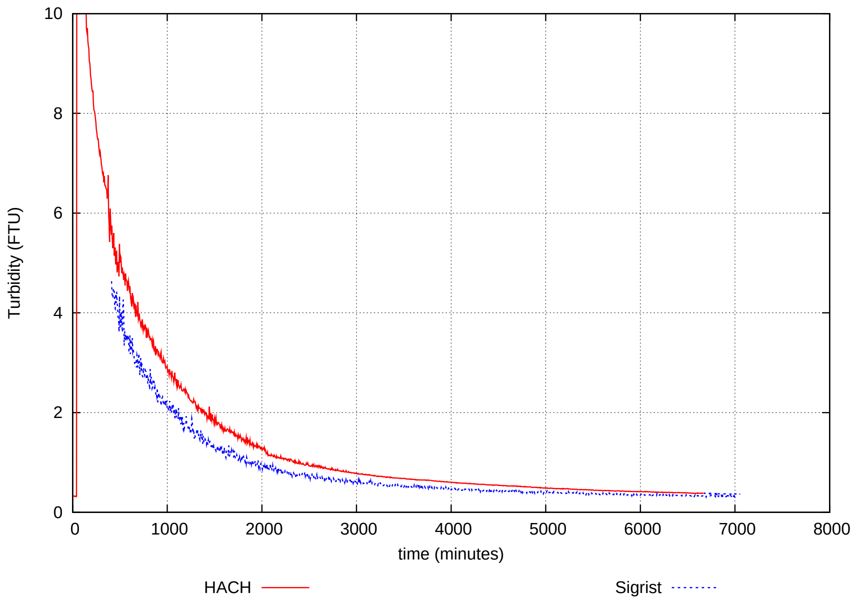

Figure 5.6 Turbidity meters HACH and Sigrist, experiment 2.

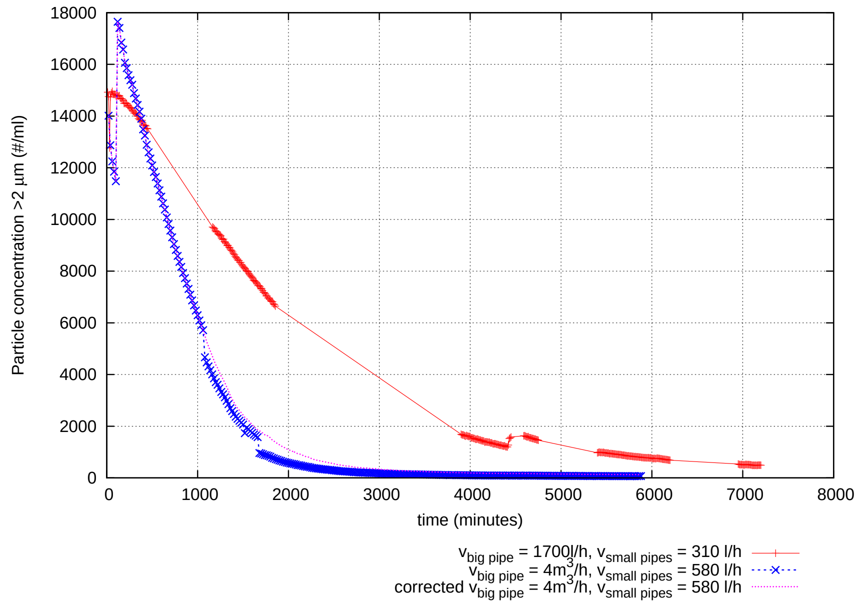

Figure 5.7 Total amount of particles, experiment 1 and experiment 2.

velocity of a particle of 5 micron is $2.45 \cdot 10^{-8} m/s$. The efficiency of settling in a small pipe is in experiment 1: $\frac{w_s}{s} = \frac{2.45 \cdot 10^{-8}}{0.35 \cdot 10^{-3}} = 7 \cdot 10^{-5}$. In experiment 2 is the efficiency: $\frac{2.45 \cdot 10^{-8}}{0.63 \cdot 10^{-3}} = 3.89 \cdot 10^{-5}$, but this particle passes a pipe 2.3 times as often as a particle in experiment 1. The efficiency is calculated by: $1.0000389^{2.3} = 1.000089$. The efficiency in the second experiment is thus $8.9 \cdot 10^{-5}$ which is 1.3 times higher than experiment 1. This does explain the decrease in sediment, but not the amount, probably because the effects of big pipe are not taken into account in these calculations. In Figure 5.7 the total amount of particles of experiment 1 and 2 are plotted in one graph. This figure also makes clear that the particles in experiment 2 settle faster than in experiment 1.

5.2 Difference Between the Inlet and the Outlet of a Pipe

Experiment 3

5.2.1 Objective

The objective of this experiment is to measure sedimentation in a four meter long pipe, by measuring the particle counts from the inlet of a pipe and the outlet of a pipe.

5.2.2 Experimental Setup

Before starting this experiment, the test rig was upgraded. Both small pipes were equipped with a flow meter. At the inlet and at the outlet of the pipes, tap points are made for connecting the particle counter and/or turbidity meter. At the start of this experiment, 5 grams of kaolinite is added. The velocity in the small pipes was 0.055 m/s and in the big pipe it was 0.06 m/s. On different times, the particles are counted at the inlet and at the outlet of a pipe. The inflow and outflow were not measured at the same time, but right after each other, because there was only one particle counter available. Even if there were another particle counter it would not have been possible to compare them without first callibrating them. The turbidity at the inlet is measured by the HACH turbidity meter and the outlet is measured by the Sigrist turbidity meter. During this experiment the particle counter is moved from the inlet to the outlet of small pipe 2, back to the inlet, after that to the outlet of small pipe 1, again back to the inlet and the last measurements were done at the outlet of the big pipe. When it is possible to measure sedimentation in the pipe, a different particle concentration should be measured at the inlet of the pipes and at the outlet of the pipes. In this experiment less kaolinite is added than in the experiments before.

5.2.3 Results and Discussion

In Figure 5.8, the particle distribution of experiment 3 is given. After approx 3.5 days, almost all particles are settled. After 2 days a peak is visible in the amount of particles. At this time, a tap point of the big pipe was pulled away causing an increase in the velocity. It was impossible to put the particle counter at the end of the big pipe, so it was moved to the inlet. At the inlet it measured the small peak. The peak would be much higher if it was measured at the end of the big pipe, were the increased velocity caused the resuspension of particles. Now the resuspended particles are mixed in the vessel and this mixture is measured at the inflow.

In Figure 5.9, the cumulative volume distribution at different times for experiment 3 is given. The percentage of small particles decreases. The bigger particles settle faster than the small particles. Not all big particles settle but they reach an equilibrium.

In Figure 5.10, the turbidity measurements are given. Again there is a difference between the Sigrist and the HACH meter. It is strange that the resuspension is only measured by the HACH. In Figure 5.11, the difference in turbidity measured by the Sigrist and the HACH turbidity meter is plotted for experiments 2 and 3. During experiment 2 the turbidity meters measured the same water and during experiment 3 the HACH turbidity meter measured at the inlet and the Sigrist at the outlet. Figure 5.11 makes clear that there is almost no difference between the two experiments. The

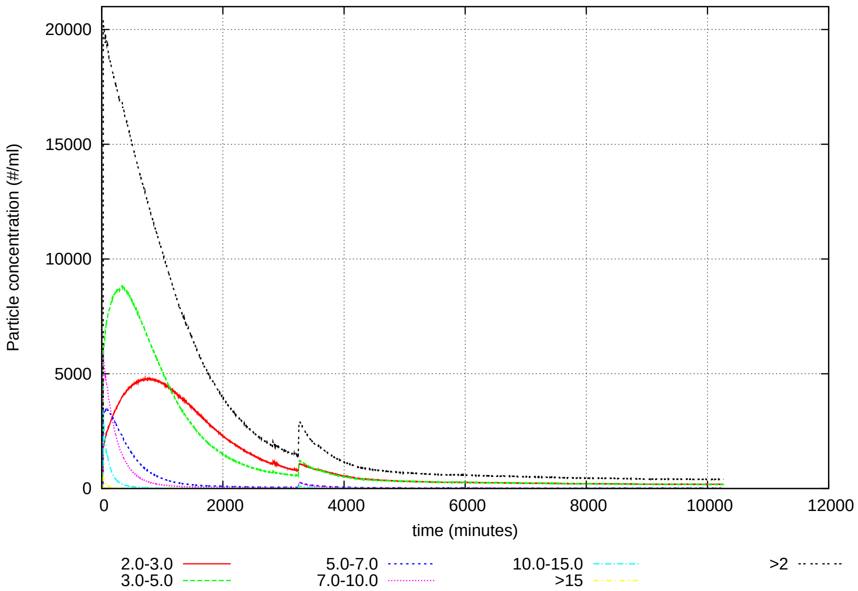

Figure 5.8 Particles in the different ranges, experiment 3.

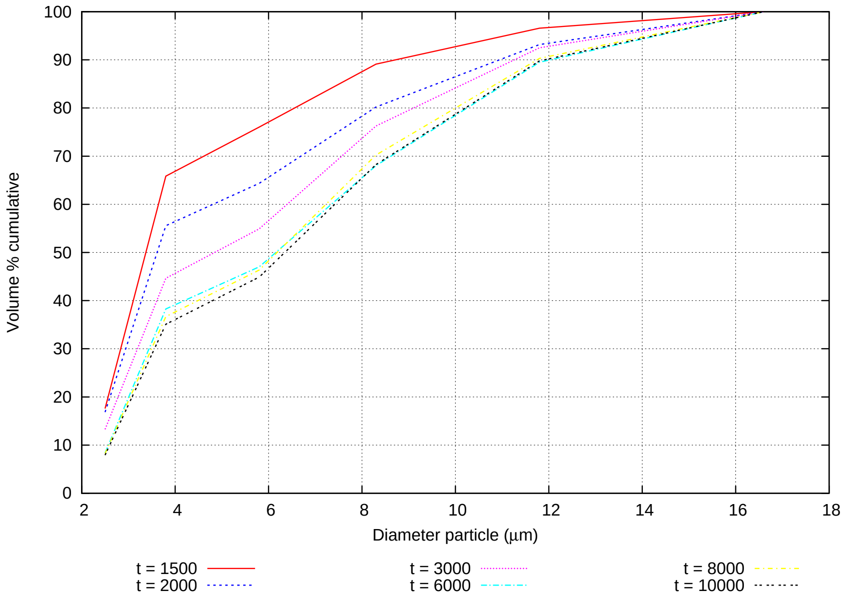

Figure 5.9 Cumulative volume distribution at different times.

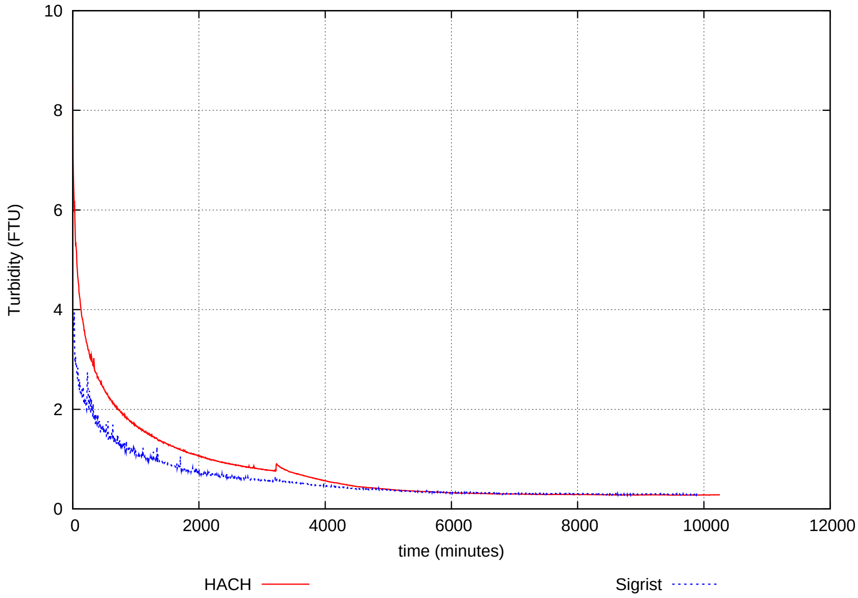

Figure 5.10 Turbidity meters HACH and Sigrist, experiment 3.

difference between the HACH and Sigrist turbidity meter in experiment 3 is not caused by sedimentation of particles. It is the normal deviation, thus the turbidity meters are not suitable to measure sedimentation in the pipes.

In Figure 5.12, the values measured by the particle counter are plotted. During the test, the concentration of particles slowly decreased. Every set of dots gives the data from the particle counter for a measuring point. If settling would be measurable in these pipes a difference should be seen between the inlet and the outlet of the different pipes. For this test and also for the three other tests done, no difference can be seen. A best fit through the inlet data has been determined with the least-square method. This gives a (calculated) inflow value, which can be compared with the measured outflow value for the same time. In Table 5.6, the calculated settling is given for the four tests carried out. A negative number means no settling has occured (but resuspension). From this table, it is clear that it is impossible to measure sedimentation in a four meter long pipe. Calculations with Stokes equation and the surface load gives a settling percentage of 2.5% [4] in a pipe with a diameter of 32 mm and a length of 500 m. The velocity of the water needs to be 0.05 m/s and a density for the particles of 2600 kg/m^3 and a diameter of 10 μm, at a temperature of 10 ^oC. Longer pipes are needed to measure a difference between inlet and outlet.

In Figure 5.13 the total amount of particles is plotted for experiment 2,

[4] $\frac{w_s}{s} = \frac{6.68 \cdot 10^{-8}}{2.71 \cdot 10^{-6}} = 2.5\%$

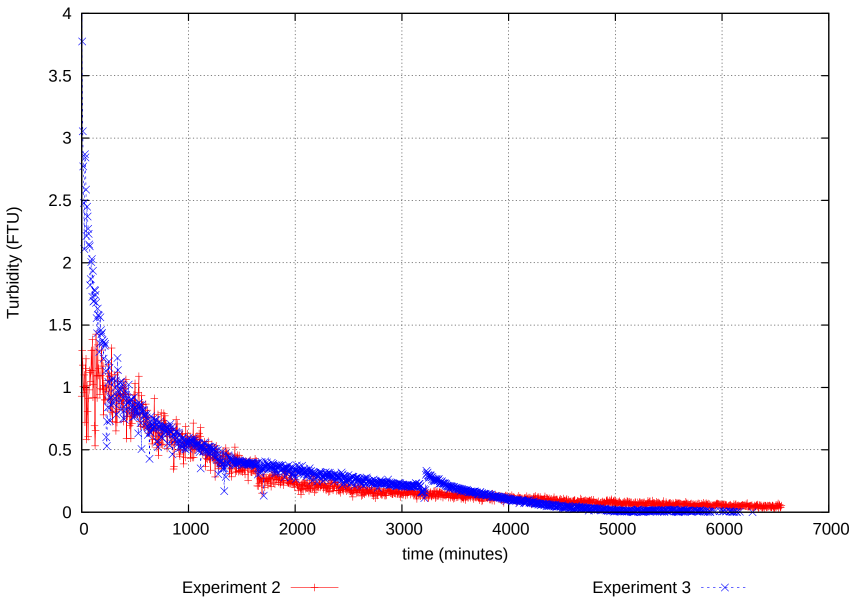

Figure 5.11 Difference between the Sigrist and the HACH, experiment 2 and 3.

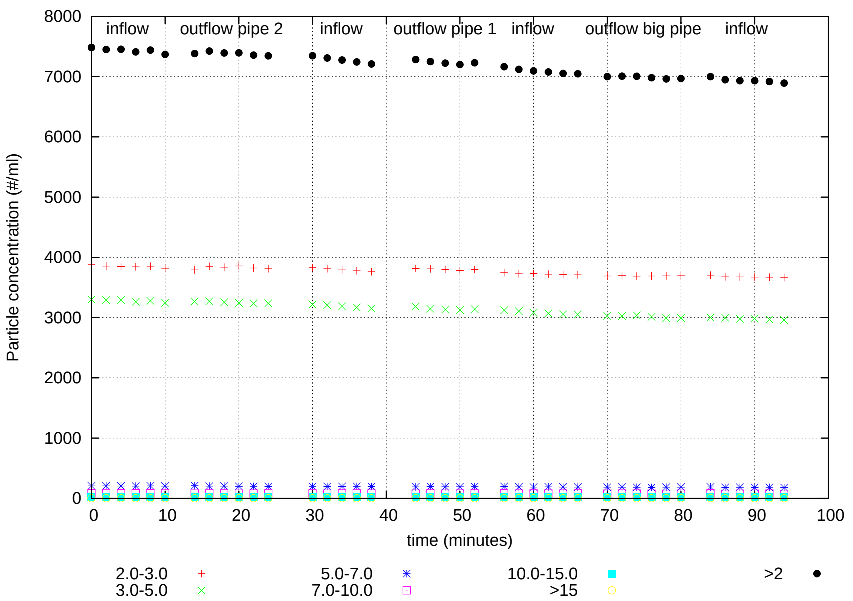

Figure 5.12 Particles counts for the inflow and outflow of the pipes.

Table 5.6 Sedimentation at different tests.

Test	Part of the Installation	Settling Total Amount Particles
1	Pipe 2	$-0.4\% \pm 5\%$
	Pipe 1	$-0.7\% \pm 5\%$
	Big Pipe	$0.4\% \pm 5\%$
2	Pipe 2	$-0.5\% \pm 5\%$
	Pipe 1	$-0.1\% \pm 5\%$
	Big Pipe	$0.7\% \pm 5\%$
3	Pipe 2	$-3.6\% \pm 5\%$
	Pipe 1	$-1.5\% \pm 5\%$
	Big Pipe	$-0.3\% \pm 5\%$
4	Pipe 2	$-0.7\% \pm 5\%$
	Pipe 1	$-0.7\% \pm 5\%$
	Big Pipe	

with a velocity in the big pipe of 0.14 m/s and in the small pipes a velocity of 0.10 m/s. Additionally, the total amount of particles for experiment 3 is plotted. The velocity in the big pipe was 0.06 m/s and in the small pipes 0.055 m/s. In the third experiment, only 5 grams of kaolinite is added, still it gets a higher concentration of particles and settling takes more time. In Figure 5.14, not the amount of particles is plotted but the mass of the particles [5]. In this graph there is a difference visible between experiments 2 and 3. Experiment 3 has more particles and more mass. In Figures 5.13 and 5.14, it looks like more kaolinite is added in experiment 3 than in experiment 2. It could be a weighing error, but this is not expected, because of the size of the error, it would have been noticed. Probably the problem is with the particle counter. From a certain concentration on the particle counter never counts more than a certain amount of particles. It does not matter how many particles are added, it still counts the same amount of particles. That is what is seen in Figure 5.14. The concentration of 5 grams of kaolinite in 330 liters of water is too high, 10 grams of kaolinite in 330 liters of water is higher, but does not give a higher value of total amount of particles.

In figure 5.15, the resuspension part is zoomed in of Figure 5.13. Particles bigger than 15 micron do not resuspend. Nothing else can be concluded, because the velocity is not known.

[5] The density of the particles is 2600 kg/m^3 and with the average diameter of a range of particles has the volume be calculated. Density times volume gives the mass.

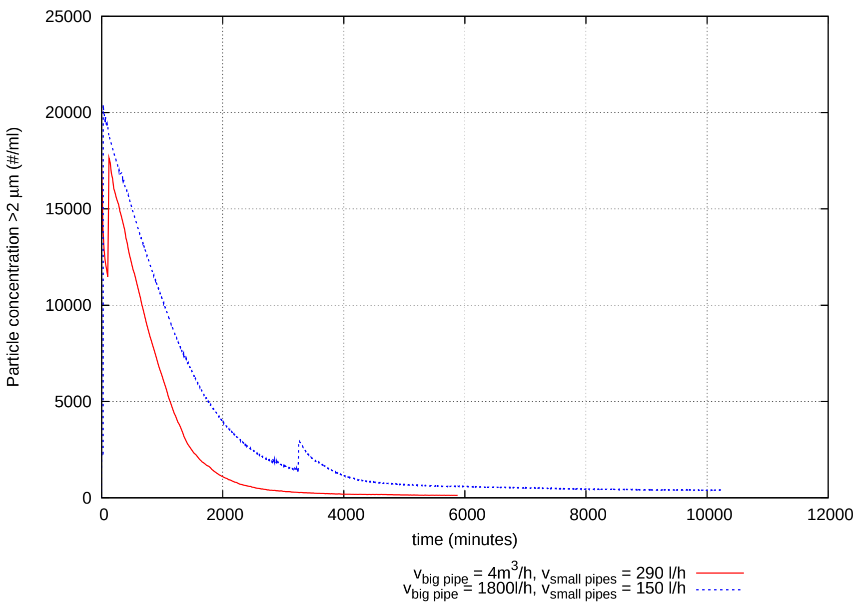

Figure 5.13 Total amount of particles, test 2 and test 3.

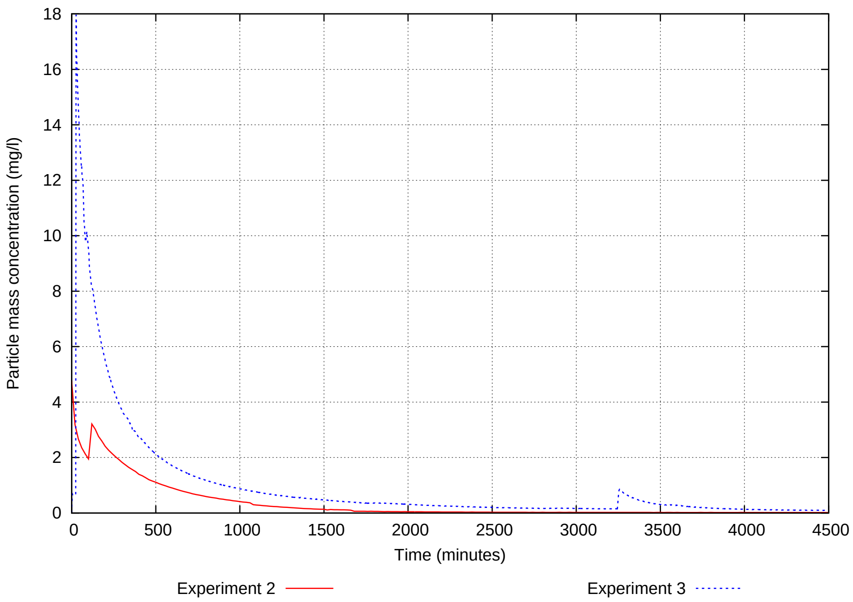

Figure 5.14 Total mass of particles, test 2 and test 3.

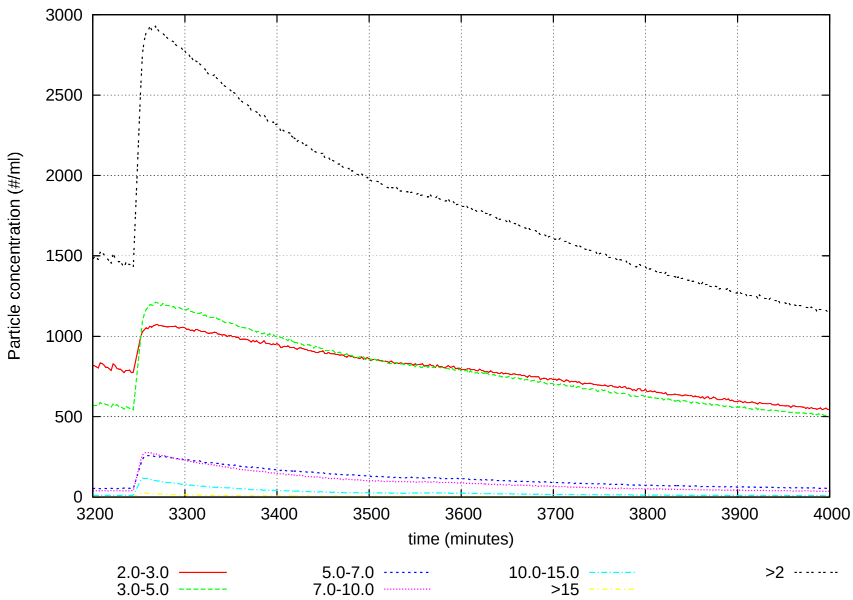

Figure 5.15 Resuspension in the third experiment.

5.3 Resuspension in the Small Pipes

Experiment 4

5.3.1 Objective

The objective of this experiment is to measure the resuspension and to compare the Shields law and the Berlamont law with the measured results.

5.3.2 Experimental Setup

This experiment is a continuation of experiment 3. First, 4.18 grams of kaolinite is added. During this experiment only the small pipes are used. The pipes are not cleaned after experiment 3. The velocity in the small pipes is 0.055 m/s. After the sediment is settled the velocity is increased for one minute to 0.25 m/s. According to Shields law all particles smaller than 25 micron will resuspend. This is because $u_* = 0.016\ m/s$[6] for a velocity of $v = 0.25\ m/s$. The Reynolds number Re_* is 571[7]. According Figure 3.4 the Shields parameter Ψ should be at least 0.06 for getting particles in resuspension, which is for particles smaller than 25 micron [8]. According

[6] $u_* = \frac{0.25 \cdot 0.4}{(ln \frac{0.0032 u_*}{0.11 * (8.96 \cdot 10^{-9})})}$

[7] $Re_* = \frac{u_* D}{\nu} = \frac{0.016 \cdot 0.032}{8.96 \cdot 10^{-9}} = 571$

[8] $\Psi = \frac{\tau_w}{(\rho_s - \rho_w) g d_s}, \tau_w = \rho_w u_*^2, \Psi = \frac{997 \cdot 0.016^2}{(2600-997)9.81/cdot25 \cdot 10^{-6}} = 0.06$

to Berlamont all particles smaller than 20 micron will resuspend [9]. The considerations made by Shields and Berlamont are explained in Section 3.4.

5.3.3 Results and Discussion

In Figure 5.16, the sedimentation and resuspension of the particles is shown. The increase in velocity took place after 3.75 days. There is a small peak in particles and an even smaller peak in turbidity (see Figure 5.17). After this peak a big increase of particles and turbidity is seen. However, this is not because of the increase in velocity. Actually, it is totally unclear. Probably a lot of sediment in the feed pipe has resuspended. In Figure 5.18, a zoom in on the particles of the controlled resuspension part is given. From this graph it is clear that only particles smaller than 15 micron resuspend. According to the laws of Berlamont and Shields particles smaller than 20 micron or smaller than 25 micron will resuspend, respectively. This does not happen in this experiment. The bigger particles do not resuspend. This is because not only the higher velocity causes resuspension but also the suddenness of the acceleration. The both laws does not take this into account. In this experiment the valve is opened slowly. In experiment 3, a tap point was pulled away and the velocity increased rapidly from one moment to the other. The shapes of the resuspension graphs are different. In experiment 3 a steep line is going up, while in this experiment 4 the increase of particles numbers is going slowly. It is hard to control the resuspension, because the number of particles increases much more after the controlled resuspension.

Figure 5.19 makes clear that the settling in experiment 4 is faster than settling in experiment 3. This is because only the small pipes are used in experiment 4. The small pipes have a smaller surface load, so more sediment will settle.

5.4 Sedimentation in Small Pipes

Experiment 5 and 6

5.4.1 Objective

The purpose is to measure the deposited particles before resuspension and after resuspension. The amount of particles that has resuspend can be measured.

5.4.2 Experimental Setup

The two small pipes are used for this experiment. The velocity in the pipes is 0.06 m/s. At the start of the experiment, 9.8 grams of kaolinite is added.

[9] $(u_*)_{cr} = \sqrt{0.8 \cdot 9.81 \frac{(2600-997)}{997} 20 \cdot 10^{-6}} = 0.016 m/s$

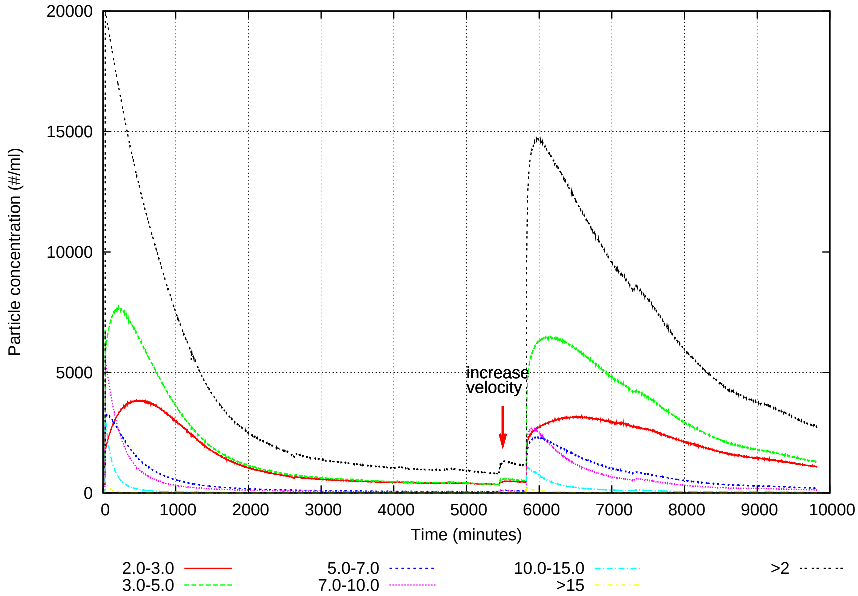

Figure 5.16 Particles in the different ranges, experiment 4.

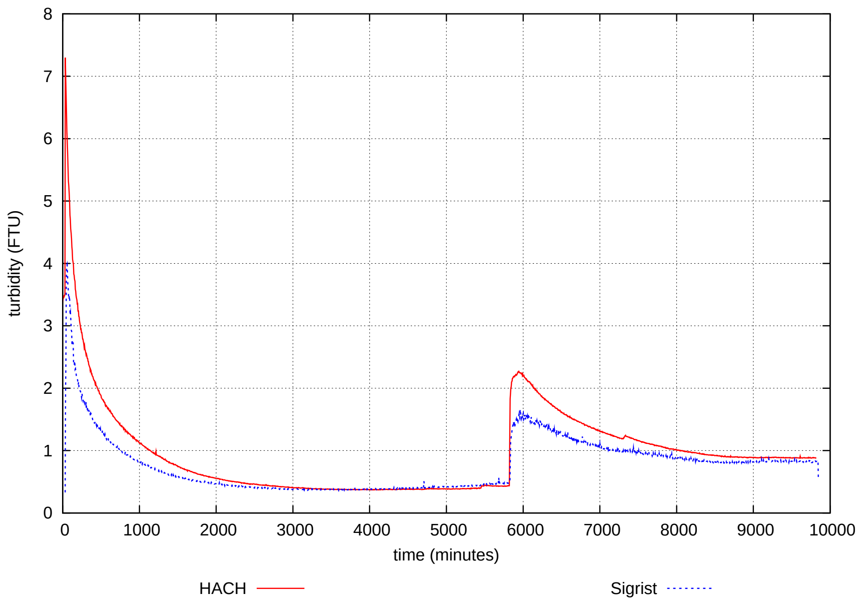

Figure 5.17 Turbidity meters HACH and Sigrist, experiment 4.

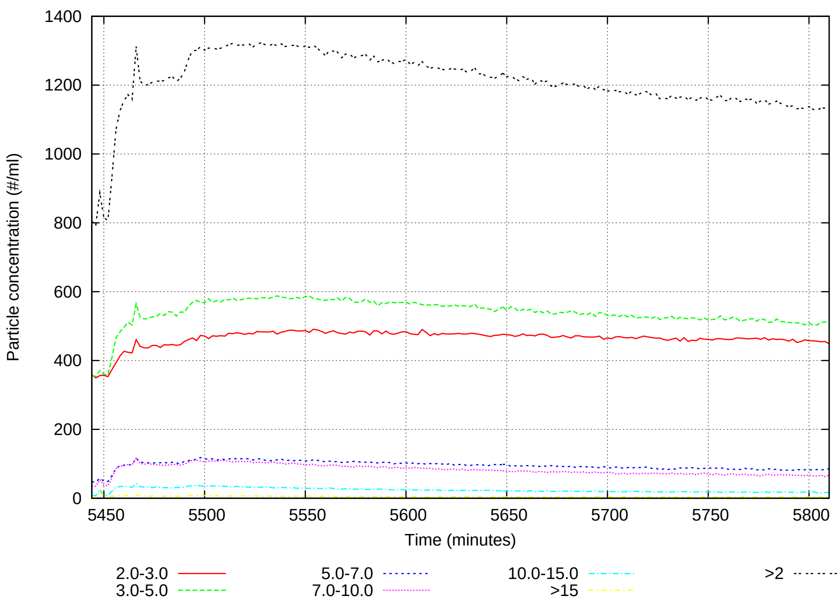

Figure 5.18 Particles in the different ranges, resuspension experiment 4.

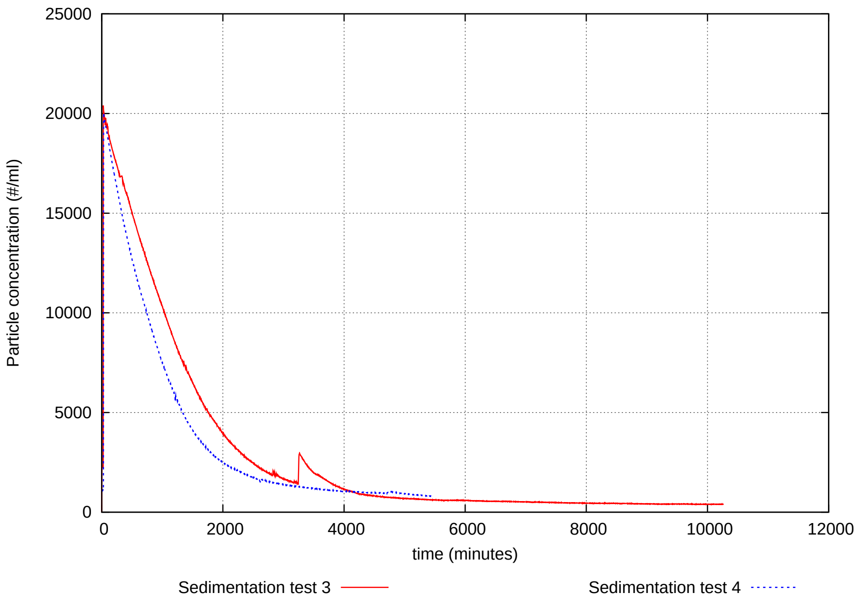

Figure 5.19 Settling experiment 3 and experiment 4.

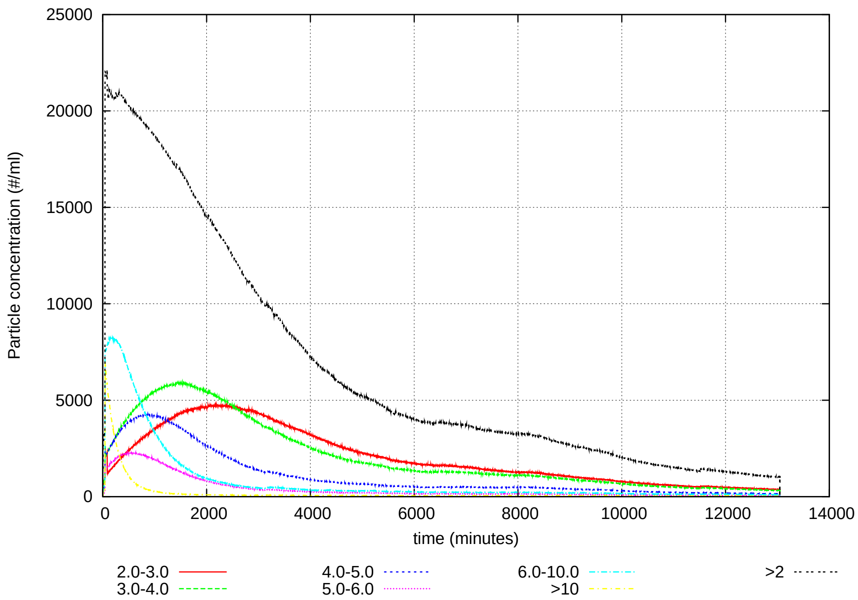

Figure 5.20 Particles in the different ranges, experiment 5.

The kaolinite settles to the bottom of the two small pipes. During the first part of the experiment (experiment 5), the pipes are filled with kaolinite. After all the sediment is settled, one pipe is taken out and the deposited particles are weighted. In the other pipe, an increase in velocity will take place and particles will resuspend (experiment 6). The velocity will be increased from $0.06m/s$ to $0.35\ m/s$ for one minute. The velocity of $0.35\ m/s$ is used for the Resuspension Potential Method (RPM) [15]. With this method an indication can be given about the discoloured water risk of a main. The increase in velocity will cause a resuspension of particles smaller than 35 micron according to Berlamont. According to Shieldsparticles smaller than 40 micron will resuspend. The maximum range the particle counter measures is bigger than 10 micron. Thus, particles from all ranges will resuspend.

5.4.3 Results and Discussion

In Figure 5.20, the sedimentation process is shown. The lines are not as smooth as in the earlier experiments. The installation, is polluted with sediment from the previous experiments. The measuring pipes are cleaned between the experiments, but the feed pipe, and also the vessel and the high reservoir, are not cleaned. When all the sediment was settled, after about 9 days, one pipe is taken out. The last two meter is emptied, which contained 0.6958 grams. It is assumed that in all parts of the pipes the same amount of particles will settle, thus 2.8 grams kaolinite is settled in the small pipes. The big pipe was not used, so 7 grams of kaolinite, almost 70%, is settled

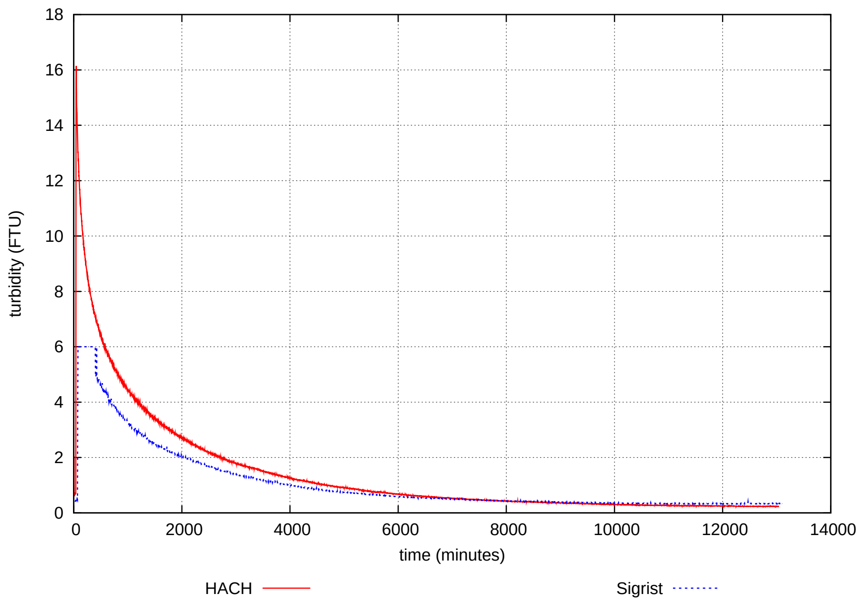

Figure 5.21 Turbidity meters HACH and Sigrist, experiment 5.

in or stuck on parts of the installation! For fhotos, see Appendix F
In Figure 5.22, the velocity at t=0 is increased from 0.06 m/s to 0.35 m/s for one minute. This gives a small peak in particles. After this small peak, a much bigger peak is visible. The increase of velocity in the measuring pipe also caused an increase in velocity in the feed pipes, which resuspended particles in these pipes. After one day, the flow in the high reservoir is not overflowing anymore. The constant pressure may not have been constant during this experiment. During the rest of the experiment, the particle concentration is fluctuating. In Figure 5.23, the turbidity is plotted against the time. This is only done for the Sigrist turbidity meter, because something went wrong with the HACH turbidity meter. The turbidity meter gives the same pattern as the particle counter.

Figure 5.24, zooms in on the controlled resuspension part of Figure 5.22. Particles in all ranges are resuspended, which confirms the expectations. The deposited material is not weighted. A lot of sediment from other parts of the installation than the measuring pipe is resuspended and the determination of deposited material will be inaccurate.

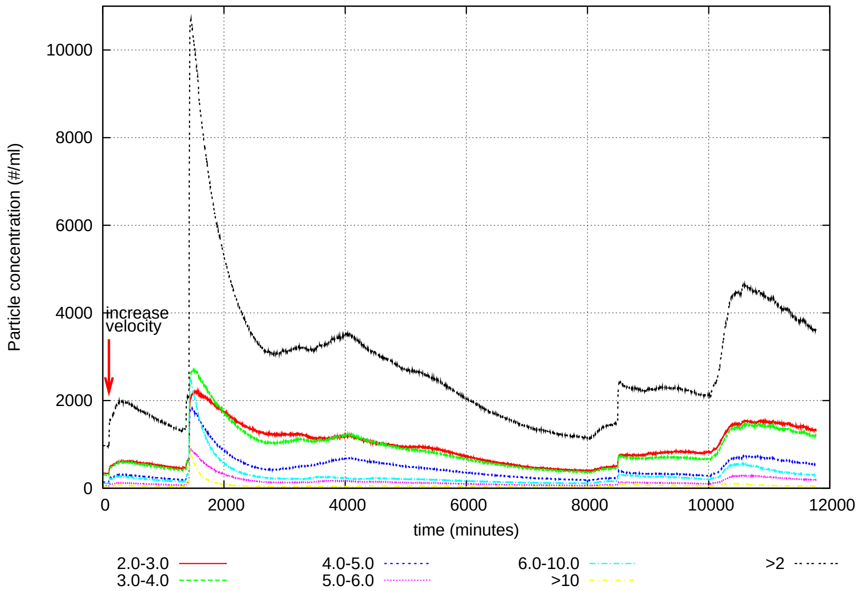

Figure 5.22 Particles in the different ranges, experiment 6.

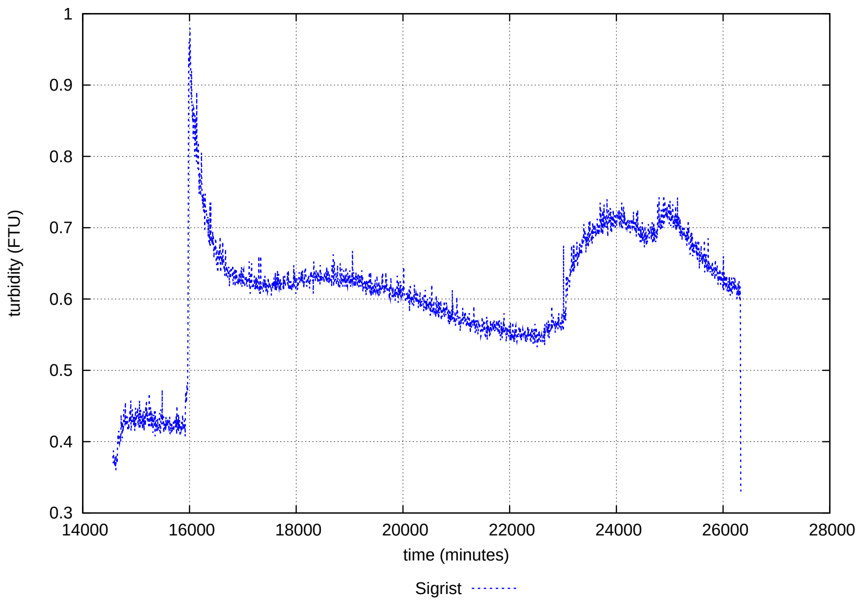

Figure 5.23 Turbidity meter Sigrist, experiment 6.

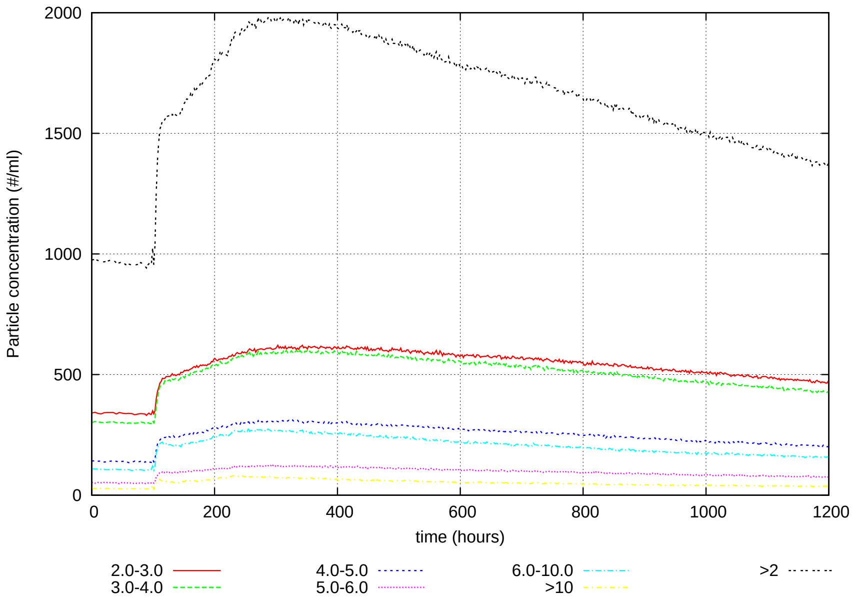

Figure 5.24 Particles in the different ranges, resuspension experiment 6.

chapter

6

Conclusions and Recommendations for Future Research

6.1 Conclusions

6.1.1 Hydraulic Behaviour

The flow in transport mains is turbulent. The pipes in the test rig with a diameter of 32 mm are comparable with transport mains with a diameter of 100 mm, when the flow in the small pipes is turbulent. On the small pipes of the test rig is a manometer placed. With this manometer it is proven that for velocities higher than 0.1 m/s the flow in a small pipe is turbulent.

6.1.2 Test Rig

While carrying out experiments, some problems are noticed with the test rig and the measuring equipment. Determining the deposited material of the big pipe was impossible. The sediment sticks to the bottom of the pipe. To get the sediment come off the pipe, the pipe needs to be shaken, but the pipe is to heavy. A kind of spoon is needed to get all the sediment out of the pipe.

During the experiments, the particles settle. Approximately 70% of the particles do not settle in the measuring pipes but in other parts of the installation! The recirculation system causes this problem. There is no control on the feed pipe and the vessel, these parts of the installation are a “black box”, and should be carefully look at.

6.1.3 Sedimentation

At lower velocities, more particles should settle, because the surface load is less. The amount of particles added, should be settled faster at a lower velocity. This is not found for the experiments carried out, because the test rig is a recirculation system. The water goes more often through the measuring pipe at a higher velocity, which causes a faster settling of sediment.

From the cumulative particle volume distribution graphs it is clear that big particles settle first, but they do not settle all. It takes more time before small particles are settled. The ratio big particles and small particles changes during the experiment. The volume of big particles increases and the volume small particles decreases. The increase in volume in big particles during time can also be caused by coagulation.

In experiment 3 it is proven that it is impossible to measure sedimentation in a pipe of 4 meters directly. No difference is measured between the inlet and the outlet of a pipe.

6.1.4 Resuspension

From the shapes of the different resuspension graphs, it is clear that not only the increase in velocity is important, also the suddennes of acceleration is important. The faster the increase takes place, the more particles will resuspend.

The results of the experiments show that it is hard to control the resuspension. After increasing the velocity, particles resuspended. But after a while much more particles resuspended.

More increase in velocity, causes resuspension of bigger particles. This is measured and in accordance with the theory.

The particles that resuspended in experiment 4 are smaller in size than predicted by the Shields and the Berlamont equations. According to the Shields, bigger particles should resuspend than predicted by Berlamont.

6.1.5 Final Conclusions

The objective of this thesis was to verify the applicability of particle dynamics theory. The conclusions are:

- The Stokes equation can not be verified with the test rig used and the experiments carried out. Particles did settle, but they did not only settle in the measuring pipes. Using the data to verify Stokes, will not be possible because too much sediment settles in other parts of the installation than the measuring pipes.

- According to experiment 4, the Berlamont equation for deposited particles fits better than the Shields equation for particle settling for these data. Only one experiment is done with clear results. This is not enough to conclude that Berlamont or Shields does describe the resuspension of particles in drinking water mains.

The second objective was to check the feasibility of the parallel recirculating test rig. This test rig should be used for quantifying the mass balance, which stems from the general question of how the sediment builds up in a drinking water main. The conclusions are:

- The small pipes in the test rig are the first step to a multiple pipe system. These pipes can be used for the multiple pipe system, but no experiments are carried out to answer the question.
- Quantifying the mass balance was not possible with this test rig. Too much sediment settled in other parts of the installation, than in the measuring part.

With the test rig build and used during this master thesis, it is not possible verify the applicability of particle dynamics theory. Also it is not possible to quantify the mass balance.

6.2 Recommendations for Future Research

6.2.1 Hydraulic Behaviour

The flow in the small pipes is turbulent for velocities higher than 0.1 m/s. This is based on the measured head losses. If in a next test rig other pipes, or pipes of a different length will be used, new experiments have to be conducted, to know at what velocity the flow is turbulent in these pipes.

6.2.2 Test Rig

A lot of sediment settles in other parts of the test rig than in the measuring pipes. Even by building a flow through test rig, sediment can settle in supply pipes. The upstream conditions have to be carefully looked after. The incoming concentration of the measuring part have to be known exactly, for making a mass balance over the measuring part only.

It was not possible to collect the sediment in the big pipe. Collecting this sediment is important to quantify the mass balance. A kind of spoon has to be made to clean the big pipe.

The particle counter is not accurate for particle concentrations higher than 10.000 particles per ml. Different particle counters give different results. The last years, the particle counter is being used more often. Also more information about the effectiveness will be become known. At the moment not much is known about applying the data of a particle counter on a useful way. Investigation of methods for applying the data should be done.

6.2.3 Sedimentation

The Stokes equation could not be verified with the test rig. In a pipe with a length of 500 m and a diameter of 32 mm, 2.5% of the sediment will settle. A test rig with a longer pipe is requierd.

Comparing the big pipe to the small pipe was done at the same shear stress. Better is to take the surface load as parameter for comparing settling in the big pipe and the small pipe, but only if the surface load is of a significant dimension. Otherwise, $\frac{w_s}{u_*}$ is a good parameter for comparing pipes of different sizes.

Coagulation is neglected in this research. Probably coagulation needs to be taken into account for higher concentrations kaolinite. It is also possible to carry out some experiments with small sand particles. Sand does not coagulate.

From the first experiments a cumulative particle volume distribution in plotted. A longer experiment needs to be done, to see how the particle volume distribution is after a long period of settling. Is it possible that the volume distribution change, while the data of the particle counter gives a constant total amount of particles?

The small pipes are not used for investigating the build-up of sediment. An experiment can be devised in which the two pipes will be taken out at different times. The deposited material will be determined at each time. A manual for such an experiment is provided in Appendix D.

6.2.4 Resuspension

The experiments carried out in this thesis are not distinctive enough to allow conclusions with respect to the validity of Shields or Berlamont equations. More resuspension experiments are needed, using different velocities. The particle counter can give data in different ranges of the particle diameter. These ranges need to be chosen in a way that the particle counter data can be compared to calculated values from Shields or Berlamont law.

The increase in velocity in these experiments lasted only for one minute. After one minute the lower velocity is used to let the particles settle. It will be interesting to know whether the particles settle, when the higher velocity is maintained.
With a new flow-through test rig a RPM can be carried out. This RPM can be compared to RPMs carried out in the distribution network.

List of Symbols

a	acceleration (m/s^2)
B	width (m)
c	concentration (kg/m^3)
C_s	drag coefficient (-)
d_s	diameter of a particle (mm)
D	pipeline diameter (m)
D_{zz}	eddy diffusivity (m^2/s)
f	Darcy-Weisbach friction factor (-)
F	force (N)
g	gravitational acceleration constant (m/s^2)
h_L	head loss due to friction (m)
k	internal pipe roughness (m)
L	length of the pipe (m)
m	mass (kg)
m_0	mass of the membrane filter (mg)
m_1	mass of the membrane filter and the non dissolved material (mg)
M	mass of an object (N)
N	number of particle per ml ($\#/ml$)
p	pressure (N/m^2)
Re	Reynolds number (-)
s	surface load (m^3/m^2s)
S	surface of the object at right angles to the flow (m^2)
u_*	shear velocity (m/s)
v	average fluid velocity (m/s)
V	volume (m^3)
V_s	volume of a sample (l)
w	specific weight (N/m^3)
w_s	settling velocity (m/s)
Q	water flow (m^3/s)

κ	von Karman constant (-)
μ	absolute viscosity ($kg/m/s$)
ν	kinematic viscosity (m^2/s)
ρ_w	fluid density (kg/m^3)
ρ_s	sediment density (kg/m^3)
ρ_d	amount of non dissolved material (mg/l)
τ	shear stress ($kg/m/s^2$)
τ_w	shear stress on the wall ($kg/m/s^2$)
Ψ	shields parameter (-)

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appendix

A

Contents Test Rig

Boodschappenlijstje installatie

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B

Data Analysis for Haarhoff Equations

appendix

C

Determination of the Not-Dissolved Material

For the determination of the content of not dissolved material the Dutch norm NEN 6484 is taken as starting point. The amount of not dissolved material is determined by filtrating, drying and weighting.

Equipment

- A filtration appliance (?), suitable for vacuum filtration and filters with a diameter of 50 mm.
- A membrane filter, porie size of 0.45 micron, made of cellulose-acetate
- A droogstoof (?)
- An exsiccator (?), filled with silicagel (?) with a colour indicator. The silicagel (?) can be used as long as it has a blue colour.
- A petri dish from glass or aluminum.
- Kwarts- of platina kroes (?)
- Usual laboratory glassware

Analysing the sample

Analyse the sample as soon as possible after sampling. Shake the sample. Use as much of the sample that at the determination not less than 10mg and not more than 100mg not dissolved material will be found. The best is to filtrate the whole sample, and not to filtrate a part and calculate back how much not dissolved material is in the amount of water you sampled.

Procedure

- Pretreatment of the membrane filter.
  Wash the membrane filter in the filtration appliance (?) with about 100 ml spirits (?) water. Dry this in a petri dish for 2 hours at 105 $\pm 2^o C$ in a droogstoof (?). Cooling in a exsiccator (?) and weight the membrane filter. (According the norm you should repeat this till a constant weight is reached.)
- Determination of the not dissolved material.
  Use the pretreated membrane filter to filtrate the sample. Rinse the bottle from the sample with 100 ml spirits (?) water and filtrate this water as well. Dry the membrane filter in a petri dish for 2 hours at 105 $\pm 2^o C$ in a droogstoof (?). Cooling in a exsiccator (?) and weight the membrane filter. (According the norm you should repeat this till a constant weight is reached.)

Calculation

Calculate the amount of not dissolved material using equation C.1. In this equation ρ_d is the amount of not dissolved material (mg/l), m_0 is the mass of the membrane filter (mg), m_1 is the mass of the membrane filter and the not dissolved material (mg) and V_s is the volume of the sample (l). Round off on 1 mg/l.

$$\rho_d = \frac{m_1 - m_0}{V_s} \tag{C.1}$$

Figure C.1 Water samples, taken out of the pipes.

Figure C.2 Filtration of water and remained sediment of the pipes.

Figure C.3 Sediment remaining after filtration.

appendix

D

Manual Build Up of Sediment, Experiment 7

D.0.5 Objective

The objective of this experiment is to decide if the velocity does effect the sedimentation. Is more sediment settling at a higher velocity? That is one of the questions that need to be answered. Another subject of research in this experiment is the build up of sediment. By taking out the two pipes on different times it should be possible to measure a difference between the two times and so the build up in time is measured.

D.0.6 Experimental Setup

This experiment makes only use of the two small pipes. At the start of the experiment 5 grams of kaolinite will be added. On two different times a pipe will be taken out and the not-dissolved material will be determined. This will be done for 3 different velocities. The first velocity will be 0.05 m/s. The flow will be about 150 l/h. The second velocity will be 0.10 m/s, the flow is 290 l/h. The third velocity will be 0.20 m/s, which gives a flow of 580 l/h. Between each experiment the pipes need to be cleaned and new fresh water need to be used.

D.0.7 Expected Results

In figure D.1 is the expected graph plotted. The amount of sediment in the second pipe will be higher. At a higher velocity, sediment will settle faster and more sediment will settle. In experiment 4, 4.18 grams of kaolinite is settled at a velocity of 0.055 m/s. This experiment is used as a starting point for experiment 7. From the particle counts is the particle concentration calculated. From the particle concentration can the amount of particles in

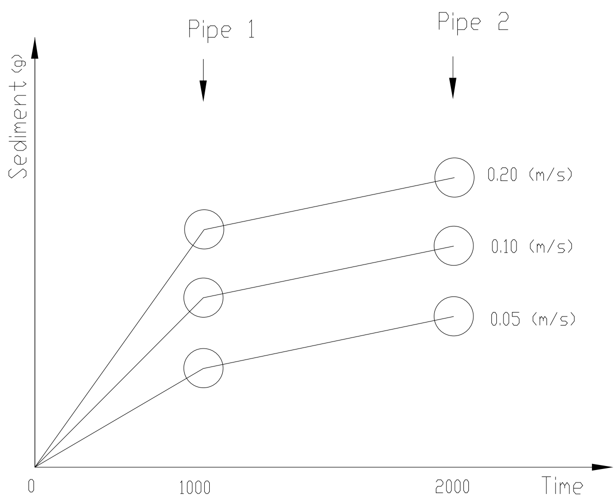

Figure D.1 Expected results experiment 7

grams be calculated. In table D.1 are the values given. At the start is 4.18 grams added, 4.80 grams is measured. The difference is because the concentration is too high and the particle counter does not measure accurate. From time 0 until time 500 minutes the total amount of particles is decreased

Table D.1 Sedimentation experiment 4

Time (minutes)	Total Particle Concentration (mg/l)	Total Amount of Particles in the water (g)
0	17.12938491	4.79622777
500	2.27754068	0.63771139
1000	0.94027157	0.26327604
2000	0.33232260	0.09305033

from 4.18 grams to 0.64 grams. From experiment 5 is concluded that only 30% settles in the pipes. About 1.06 grams are settled in the pipes. The not-dissolved material on the filter of 2 m pipe will be 0.27 grams. At least three filters need to be used to filtrate the water from the pipe. When the second pipe is taken out at 1000 minutes only 30% x 0.37 grams is 0.11 grams will settle. This will settle in the one pipe that is left. In this pipe was already 1.06 grams divided by 2 is 0.53 grams settled. In total is in this pipe 0.64 grams settled. The not-dissolved material in 2 m pipe is about 0.32 grams. To filtrate this at least four filters are necessary. The difference between 500 and 1000 minutes is not big.

The same calculations can be done for taking out the second pipe at 2000 minutes or taking out the first pipe at 1000 minutes and the second pipe at 2000 minutes. The values of the amount of particles that are settled are given in table D.2 and table D.3. The conclusion is that the first pipe has to be taken out after 500 minutes. The second pipe has to be taken out after 2000 minutes. The difference in particles between these two times is 81.7 mg. In the second column of table D.2 the amount of particles in the water of the pipe is given. These particles are not settled but will be measured in the not-dissolved material. At 500 minutes will be about 3.7 mg of kaolinite measured that is not settled. At 2000 minutes this is 0.5 mg. This means that between 500 minutes and 2000 minutes 3.2 mg of kaolinite is measured that is not settled. Thus the deviation is about 4%. In figure D.2 the calculated results are plotted.

Table D.2 Expected sedimentation, first pipe

Time (minutes)	Amount of Particles in the Water of 2 m Pipe (mg)	Total Amount of Particles) in 2 m Pipe (mg) Taking Out the First Pipe
0	27.6	0
500	3.7	265.7
1000	1.5	293.8
2000	0.5	306.5

Table D.3 Expected sedimentation, second pipe

Time (minutes)	Total Amount of Particles in 2 m Pipe (mg), Taking Out the First Pipe at 500 Minutes	Total Amount of Particles in 2 m pipe (mg) Taking Out the First Pipe at 1000 Minutes
0	x	x
500	265.7	x
1000	321.9	293.8
2000	347.4	319.3

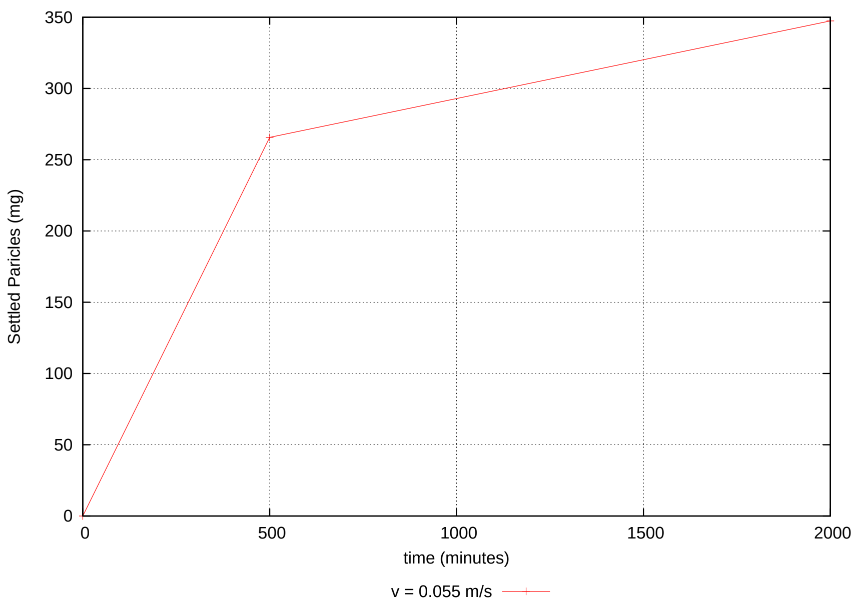

Figure D.2 Expected results experiment 7 for v = 0.055 m/s

appendix

E

Design of the Flow Through Multiple Pipe Test Rig

appendix

F

Photos of kaolinite in the installation