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John Nurminen Foundation

Technical Audit of the Kohtla-Järve Wastewater Treatment Plant



Establishment of the Most Feasible Way for Accelerated
Phosphorus Removal

Competence. Service. Solutions.

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1 INTRODUCTION

1.1 Background

The John Nurminen Foundation, the Union of the Baltic Cities Commission on Environment and HELCOM (Baltic Marine Environment Protection Commission) have agreed to work together in order to improve the state of the Baltic Sea. In that purpose the Parties have applied and received financing from EU Baltic Sea Region Programme 2007-2013 for a three-year project called “PURE” (Project on Urban Reduction of Eutrophication), started in December of 2009. The objective of the project “PURE” is to reduce phosphorus discharges to the Baltic Sea by enhancing phosphorus removal at municipal wastewater treatment plants and also improve the knowledge on best available techniques on phosphorus removal in cities and water companies around the Baltic Sea. The “PURE” project was approved by the EU BSRP Monitoring Committee on 16th September 2009.

The ‘PURE’ project partners also include the following cities and/or water companies: Brest Vodokanal, city of Gdansk, Jurmala Water, Kohtla-Järve Water Company (Järve Biopuhastus OÜ) and Szczecin Water Company. The cities and/or water companies have agreed that a technical audit will be carried out at their wastewater treatment plants in order to assess the feasibility and cost efficiency of enhanced, chemical phosphorus removal and other low cost options to reduce phosphorus discharges to receiving waters. The Project Partners intend to achieve an average annual concentration of 0.5 mg phosphorus / litre in effluent waste water on continuous basis. Also investments to achieve this value are included in the project ‘PURE’ at the wastewater treatment plants of Brest and Jurmala, starting in 2011. The Project will be carried out in harmony with the national legislation, rules and environmental regulations of each participant and EU.

1.2 Objectives of the project

The overall objectives of the assignment are

- to review the current wastewater and sludge treatment processes especially in terms of phosphorus removal,
- to develop the most cost effective plan to enhance phosphorus removal to the level of 0.5 mg/l and
- to estimate additional O&M costs required by the enhanced treatment.

2 KOHTLA-JÄRVE WASTEWATER TREATMENT PLANT

2.1 General

The Kohtla-Järve wastewater treatment plant (WWTP) is situated in the industrial area outside the town of Järve. The plant is owned and operated by Järve Biopuhastus OÜ, which is owned by local municipalities of Kiviõli, Püssi, Kohtla-Järve and Jõhvi. Wastewater led to the plant comes partly from households and partly from industrial load. The catchment area covering approximately 220 km² includes the towns of Kiviõli, Püssi, Kohtla-Järve, Jõhvi, Kohtla-Nõmme, Ahtme, Kukruse and Sompa.

According to the design documents, the overall population of the catchment area is approximately 80,000 inhabitants, and industrial load amounts to PE 110 000 in terms of BOD, resulting in a total PE of approximately 190 000. However, at present, the real number of municipal customers served is about 40 000 and some major industries are in a standstill. In the year 2009, the total PE of the plant was about 87 000 in terms of BOD₇ and 82 000 in terms of total nitrogen. Approximately 50 % of the influent BOD load and 60 % of nitrogen load originated from industry.

The plant has conventional primary treatment without primary settling, activated sludge process for enhanced biological phosphorus and nitrogen removal and sludge treatment by mechanical thickening, hygienisation by heating, and dewatering. Dewatered sludge is composted in windrows in the plant area.

The plant was originally built in 1976 and upgraded for nutrient removal in 2008.

2.2 Wastewater flows and loads

The purpose of the audit is to propose measures to enhance nutrient removal and process stability primarily within the existing process tanks. Therefore, the investments to be planned will be based on the existing wastewater loads and flows, namely the average influent values from the year 2009 – 2010. Data from this year are presented and analysed below.

The basic figures describing the wastewater amount and quality are presented in Table 2.1. Person equivalent values have been calculated on the basis of the BOD₇ load, assuming 1 P.E. = 0,068 kgBOD₇/p.e./d. For comparison, also the original dimensioning values of the plant (WTE, 2008) are given in Table 2.1.

Table 2.1. Wastewater data, Kohtla-Järve WWTP

Parameter	Unit	Dimensioning value*	Average 2009	Average 2010**
Pop. Equivalent (BOD)	ca	197 650	81 965	104 203
Q average	m ³ /d	24 900	15 920	16 745
Q max	m ³ /d	54 240	37 040	79 054
q design/average 2009	m ³ /h	2 260	663	698
BOD7	kg/d	13 400	5 557	7 086
	g/ca/d	68	68	68
	mg/l	538	367	423
CODCr	kg/d	28 000	16 509	21 320
	g/ca/d	142	201	205
	mg/l	1 124	1 107	1 273
Suspended solids	kg/d	5 760	5 050	5 084
	g/ca/d	29	62	49
	mg/l	231	343	304
Total nitrogen	kg/d	1 800	1 348	1 503
	g/ca/d	9	16	14
	mg/l	72	89	90
Total phosphorus	kg/d	200	95	113
	g/ca/d	1.0	1.2	1.1
	mg/l	8.0	6.4	6.7
Temperature, max	°C	20	24	26
Temperature, min	°C	10	11	8

*) WTE / EVN, 2008

**) January - July 2010

All influent loads except suspended solids were substantially lower than the dimensioning values in 2009. For the first half of 2010, the loads and flows have been approximately 10 % higher than the average values of 2009. In the future, AS Nitrofert is expected to start production again, which would increase nitrogen load. The town of Jõhvi is expected to build a separate treatment plant for dairy wastewater currently coming to the Kohtla-Järve WWTP. This would reduce at least the loads of BOD and COD.

The quality data for influent wastewater presented here originates from samples taken in the inlet chamber before mechanical treatment. Rejects from sludge treatment and other plant-internal loads are included in these samples.

2.2.1 Wastewater flow

Influent wastewater flow varied from 9 053 m³/d to 37 036 m³/d. The yearly average was 15 923 m³/d. All daily flow values were clearly below the dimensioning flow value of 45 200 m³/d. The influent flow during 2009 is shown in Figure 2.1.

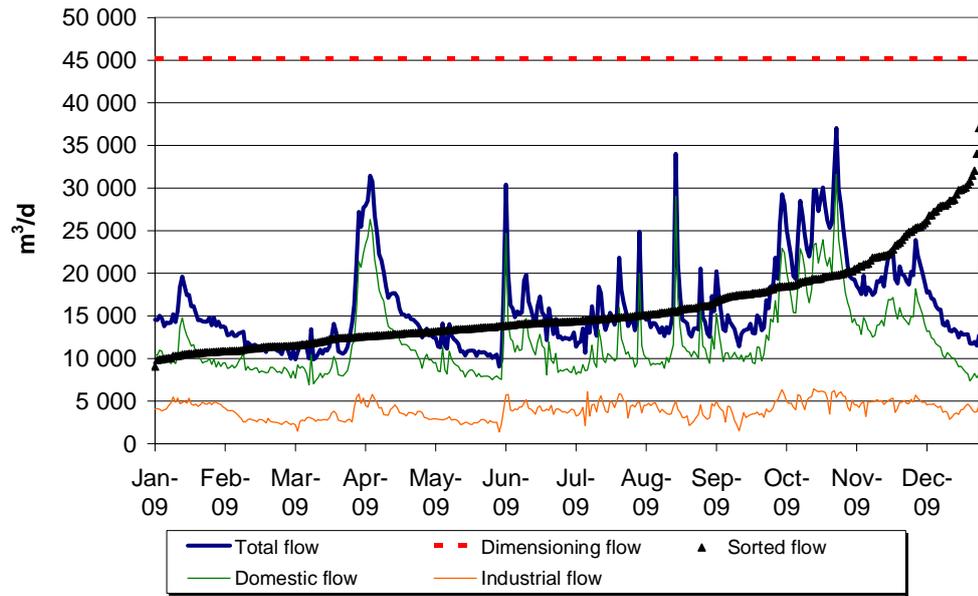


Figure 2.1. Influent wastewater flow, 2009.

The flow time curve shows high variations of domestic wastewater (standard deviation 36 % of average), which are probably due to infiltration of rainwater into the sewers. The catchment are has separate sewer systems for wastewater and rainwater, but the pipes are in need of repair. The flow of industrial wastewater remained more stable throughout the year (standard deviation 26 % of average), although there are two periods of lower flow in February – March and May.

2.2.2 Wastewater loads

Origin of loads

The division of the total load between municipal and industrial sources in the year 2009 is presented in the following figures. From Figure 2.2 it is evident, that about 50 % of BOD and 60 % of nitrogen came from industry, whereas other loads (water flow, phosphorus, suspended solids) were primarily of municipal origin. The most important industrial source was VKG Oil AS (see Figure 2.3 and Figure 2.4). As stated above, AS Nitrofert has previously been a major source of nitrogen load, but their production is currently at a standstill.

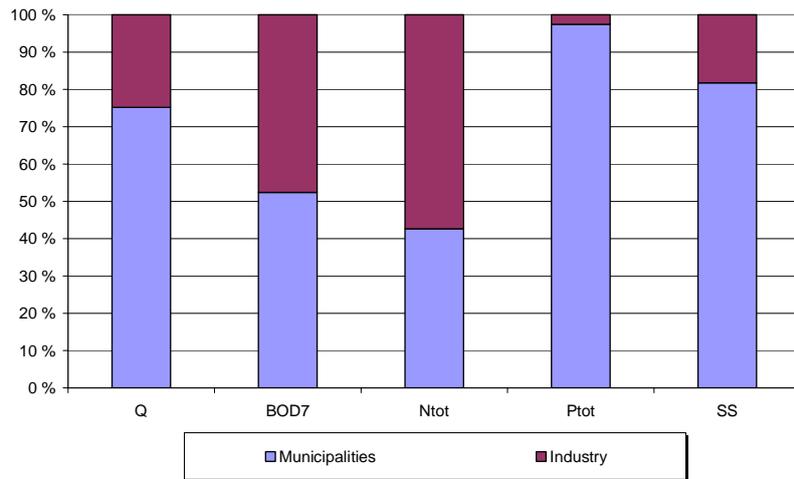


Figure 2.2. Proportional distribution of incoming load, 2009.

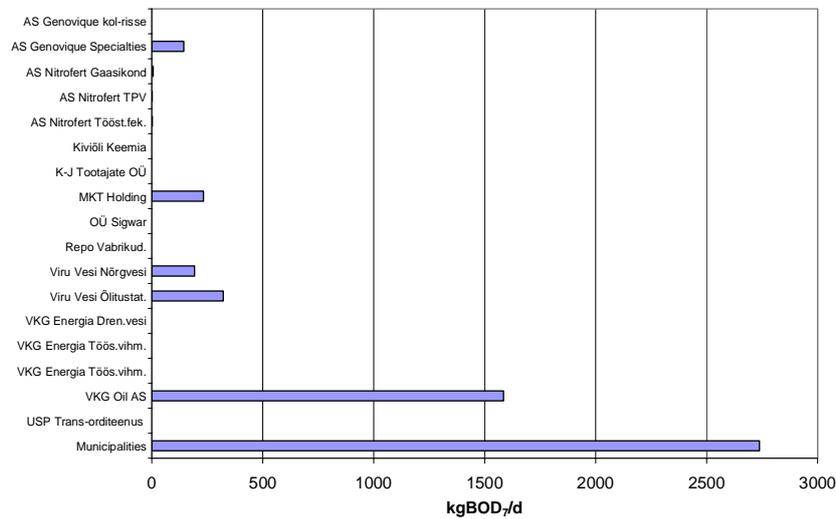


Figure 2.3. Sources of BOD load, 2009.

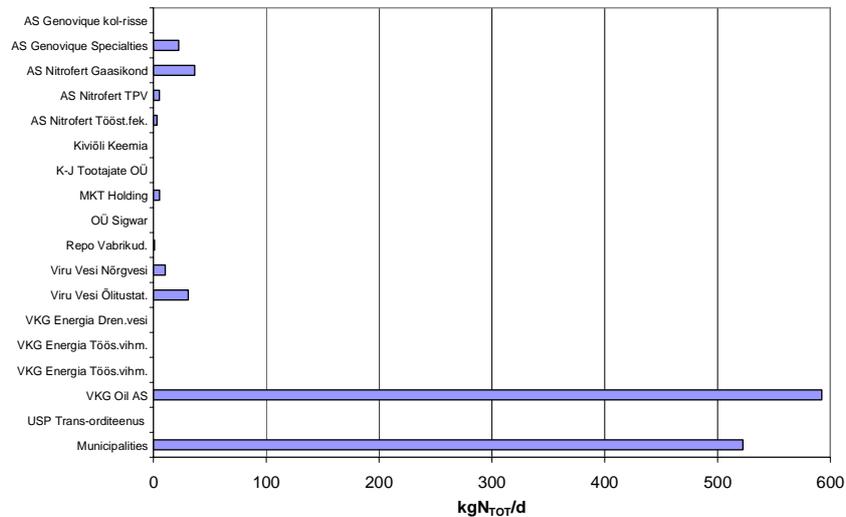


Figure 2.4. Sources of total nitrogen load, 2009.

Organic load

Influent organic load was measured by BOD₇ measurements twice a week and by COD_{Cr}-measurements five times a week. BOD₇ load and concentration varied as shown in Table 2.2. None of the measurements exceeded the dimensioning BOD₇ value of 13 700 kg/d. The influent BOD₇- load during 2009 is shown in Figure 2.5.

Table 2.2 BOD₅ load and concentrations, 2009

BOD ₇	Ave	Min	Max
Concentration (mg/l)	367	184	840
Load (kg/d)	5 557	3 013	12 365

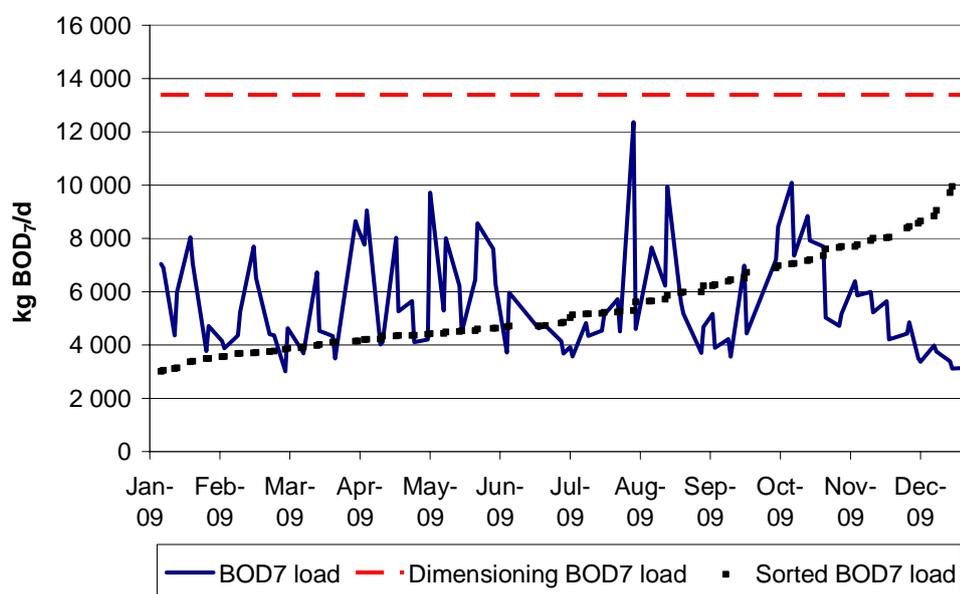


Figure 2.5. Influent BOD₇ load, 2009.

COD_{Cr} load and concentration varied as shown in Table 2.3. The highest 2 % of the measurements exceeded the dimensioning COD_{Cr} value of 28 000 kg/d. The influent COD_{Cr} load during 2009 is shown in Figure 2.6.

Table 2.3 COD_{Cr} load and concentrations, 2009.

COD _{Cr}	Average	Min	Max
Concentration (mg/l)	1 107	215	2744
Load (kg/d)	16 509	5 408	41 630

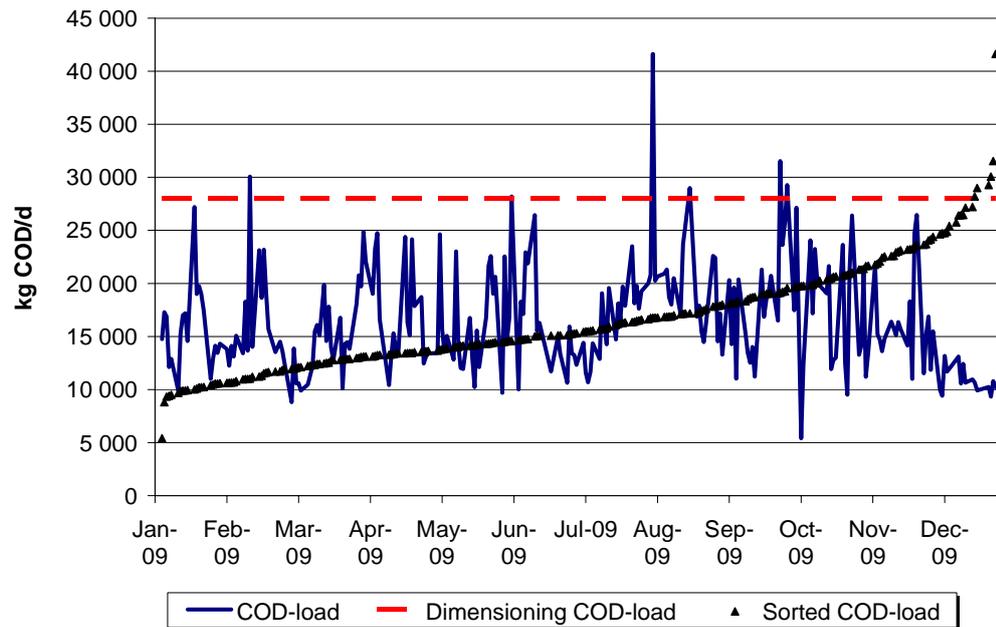


Figure 2.6. Influent COD_C load, 2009.

Apparently the COD/BOD ratio of the influent wastewater is higher than expected in the dimensioning calculations, since the dimensioning values of COD are exceeded more often than the dimensioning values for BOD. As shown in Figure 2.7, the COD/BOD ratio varies very much. The average value of 3,1 indicates that the organic carbon in the influent is primarily of slowly biodegradable type.

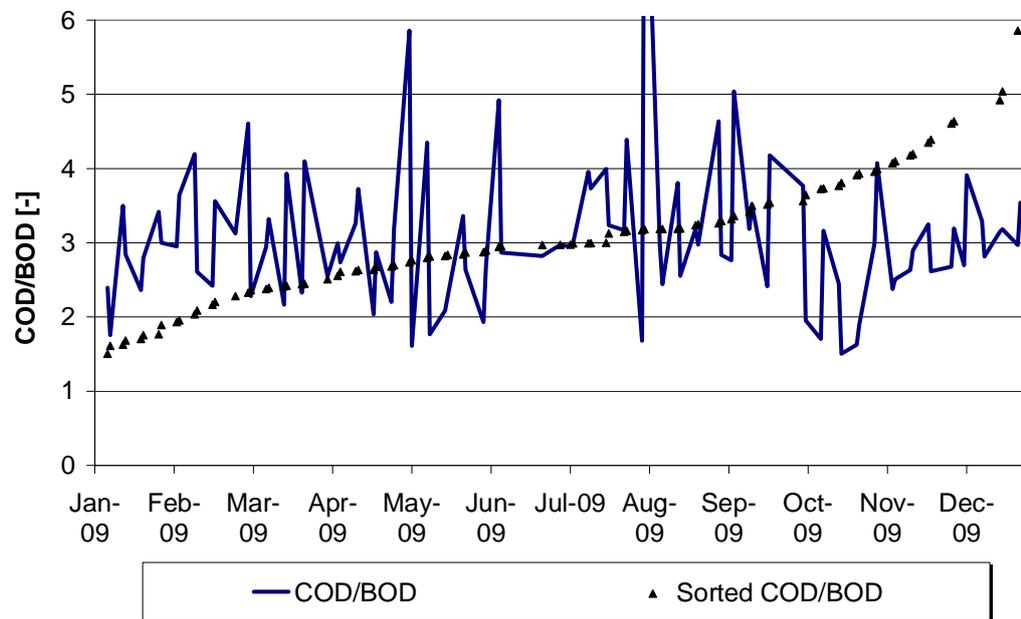


Figure 2.7. COD/BOD ratio, 2009.

The high day-by-day fluctuation of organic load and COD/BOD ratio may be connected to industrial loads. Septic tank sludges are an unlikely reason, because their amount is very small, according to personnel. There may also have been occasional problems with the COD analysis in the laboratory.

Nutrient loads

Influent nutrient concentrations were measured approximately twice a week. Influent phosphorus concentrations were close to typical values of municipal wastewater. Phosphorus load varied as shown in Table 2.4. The measured phosphorus load exceeded the dimensioning value of 200 kgP/d on only one sampling day. The load fluctuated periodically; two periods of significantly higher-than-average loading were recorded in 2009. The peaks may be induced by sludge handling problems and the resulting heavy plant-internal loads. The influent P load during 2009 is shown in Figure 2.8. Typical BOD₇/P_{TOT} ratio in influent wastewater was 59.

Table 2.4. Influent phosphorus load and concentrations, 2009

Phosphorus	Average	Min	Max
Concentration (mg/l)	6.4	2.0	10.2
Load (kg/d)	95	36	205

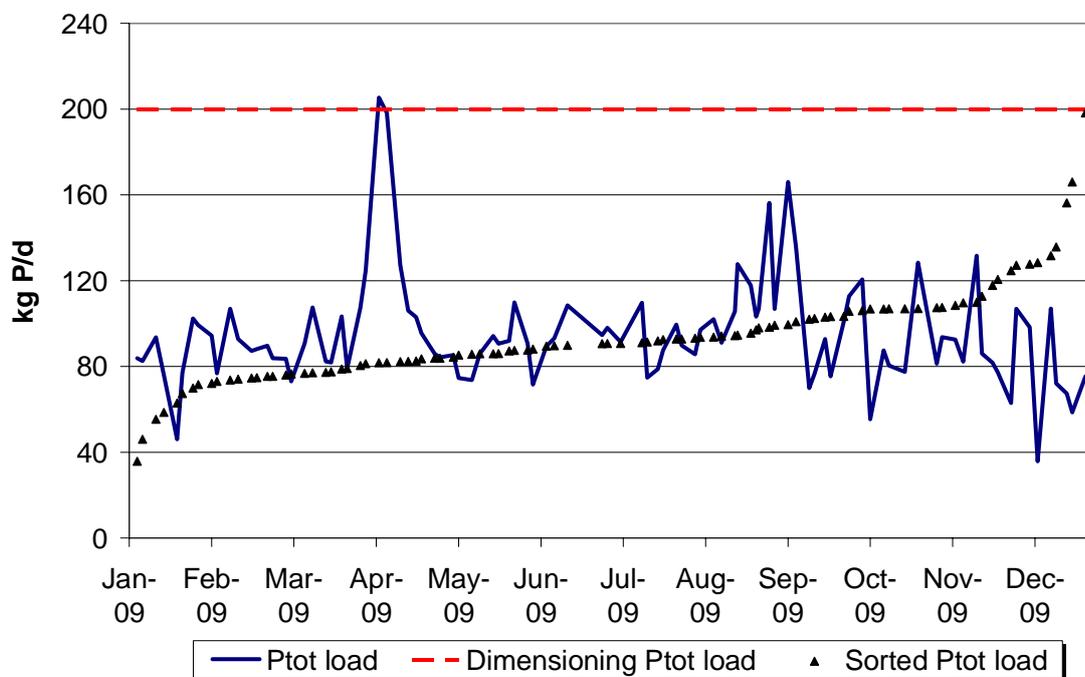


Figure 2.8. Influent total phosphorus load, 2009.

Influent nitrogen concentrations varied as shown in

Table 2.5. The nitrogen concentrations were approximately twice as high as typical values for municipal wastewater, and about 15 % of the recorded loads exceeded the dimensioning load of 1800 kgN/d. Wide variations occurred throughout the year, with a longer elevated period between mid-August and November. The influent N_{TOT} load in 2009 is shown in Figure 2.9. Typical BOD₇/N_{TOT} ratio in influent wastewater was 4,1

Table 2.5. Influent nitrogen load and concentrations, 2009

Nitrogen	Average	Min	Max
Concentration (mg/l)	89	40	161
Load (kg/d)	1 348	698	2 294

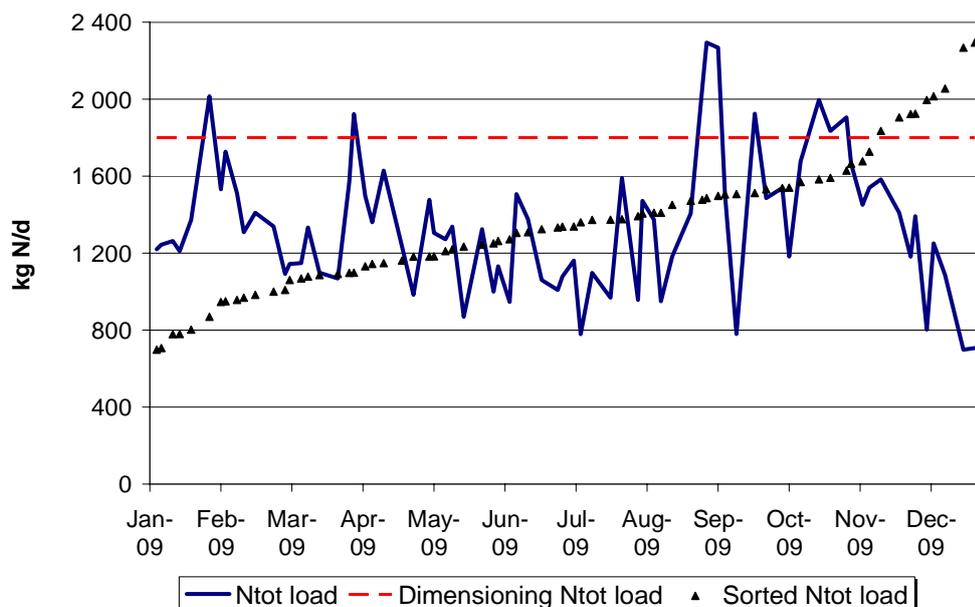


Figure 2.9. Influent total nitrogen load, 2009.

Suspended solids

Influent suspended solid concentrations were measured two to five times a week. The loads and concentrations varied as shown in Table 2.6. The dimensioning value of 5 760 kg SS/d was exceeded in almost 40 % of the measurements. The high variation and high loads may be connected to problems in sludge thickening and, consequently, heavy internal SS loads. The influent SS load during 2009 is shown in Figure 2.10.

Table 2.6. Influent suspended solid loads and concentrations, 2009

Suspended solids	Average	Min	Max
Concentration (mg/l)	343	79	970
Load (kg/d)	5 050	959	13 800

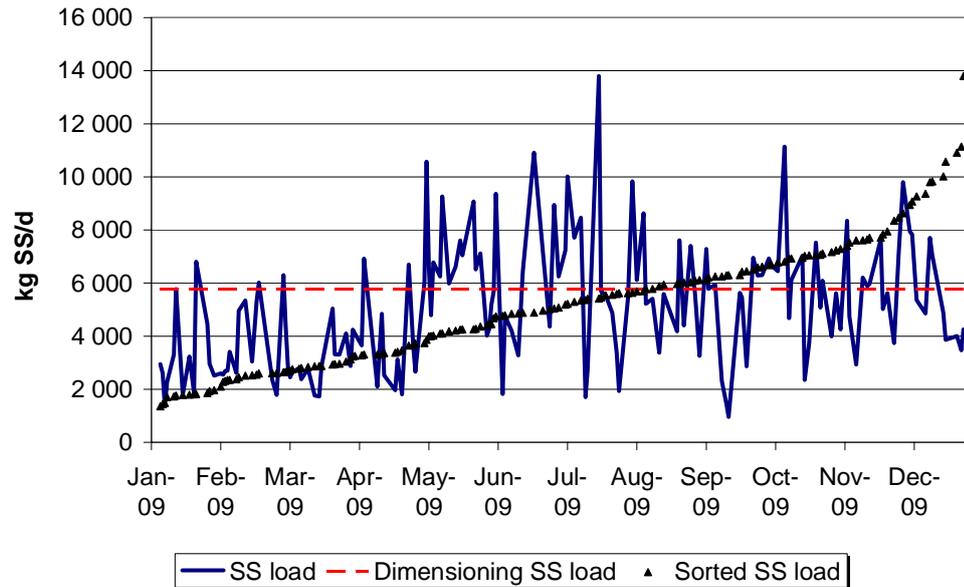


Figure 2.10. Influent suspended solids load, 2009.

2.2.3 Temperature

Influent wastewater temperature ranged from 11,0 to 24,0 °C and was on average 14,8 °C during the period January – June and October - December 2009. Data from the rest of the year was not available. No values were under the dimensioning temperature of 10 °C (Figure 2.11). Of individual sources of wastewater, the minimum temperature for municipal wastewater was 7 °C and maximum for industrial wastewater 65 °C.

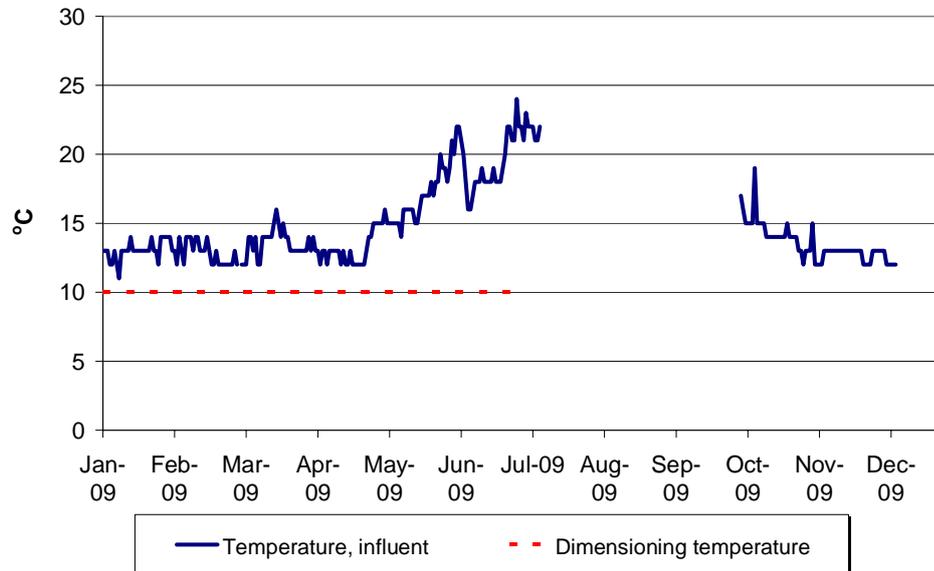


Figure 2.11. Influent wastewater temperature, 2009.

2.3 Treatment requirements and nutrient removal performance

The required quality of effluent wastewater (design values) and the achieved average values from the year 2009 are shown in Table 2.7. The aim is to reach the required results at all times. In this table, the results are compared both to the current demands and to the recommendations of HELCOM.

Table 2.7. Achieved effluent quality compared to demands and HELCOM recommendations, 2009

Parameter	Limit	HELCOM recommendations		Achieved (average 2009)		N:o of days when limit exceeded
	mg/l	mg/l	%	mg/l	%	
Suspended solids	15	-	-	16	95	85
COD _{Cr}	125	-	-	83	93	17
BOD ₇	15	15	80	9.0	98	8
Total nitrogen	10	10	70 - 80	4.4	95	7
Total phosphorus	1.0	0.5	90	0.88	86	13

On average, the effluent limits were reached for all parameters except suspended solids. However, the limits for all parameters were occasionally exceeded in individual daily samples.

Effluent nitrogen concentrations are shown in Figure 2.12.

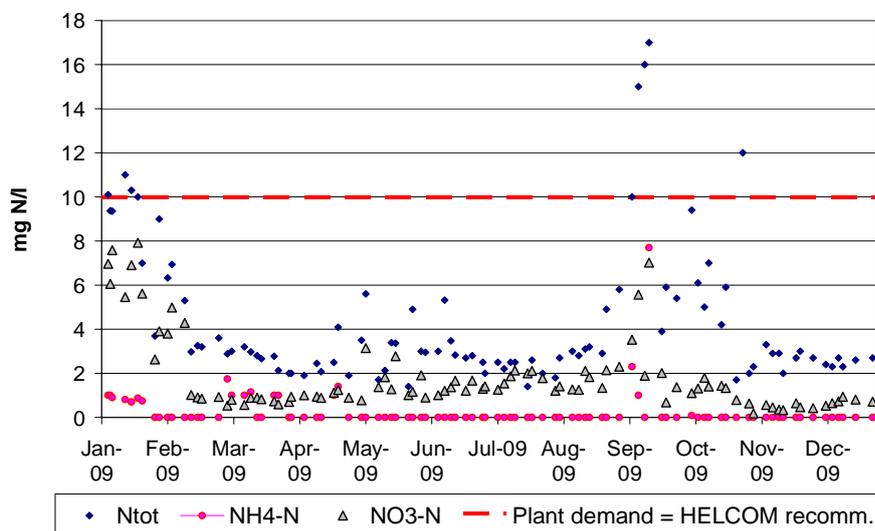


Figure 2.12. Effluent nitrogen concentrations, 2009.

Effluent total phosphorus concentrations (laboratory analysis) in 2009 are shown in Figure 2.13 and phosphate phosphorus concentrations (on-line analysis) for 2010 in Figure 2.14.

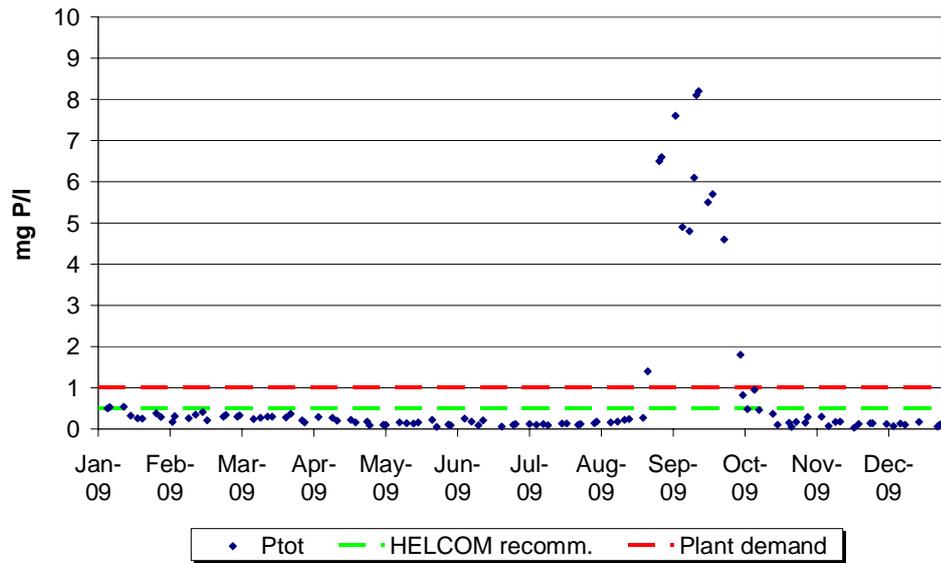


Figure 2.13. Effluent total phosphorus concentrations, 2009.

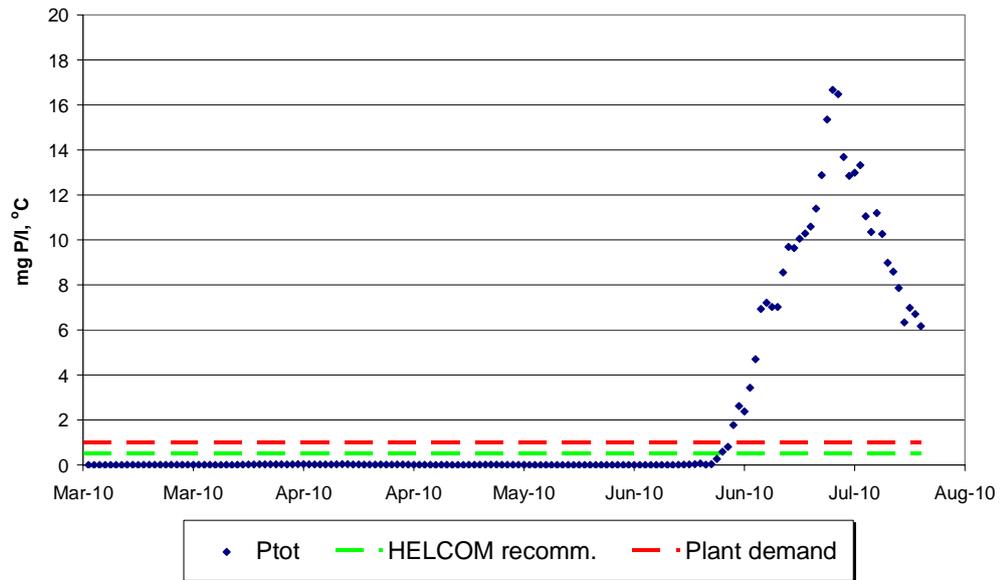


Figure 2.14. Effluent phosphate phosphorus concentration, March – August 2010.

Effluent values of suspended solids often exceeded the requirement of 15 mg/l (Figure 2.15). Due to these situations, the limits for BOD (Figure 2.16) and COD (Figure 2.17) were sometimes exceeded, too. It is evident that at least part of the highest effluent concentrations of nitrogen and phosphorus in 2009 was due to washout of solids. However, the solids concentrations are not high enough to totally explain the high nutrient concentrations.

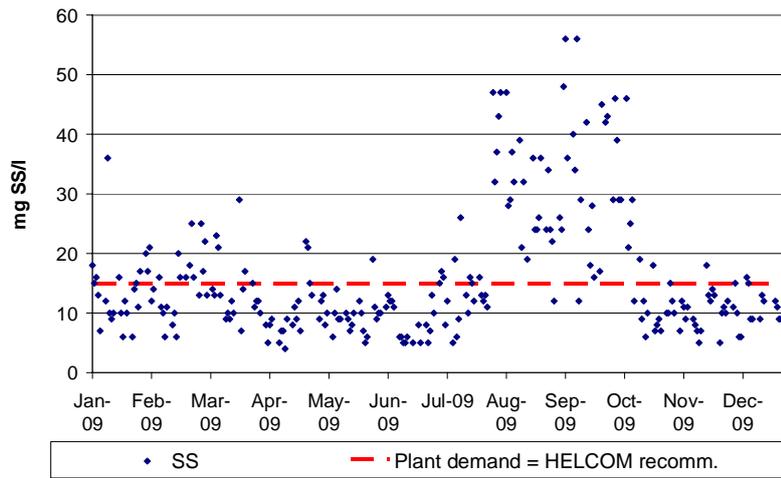


Figure 2.15. Effluent suspended solids concentrations, 2009.

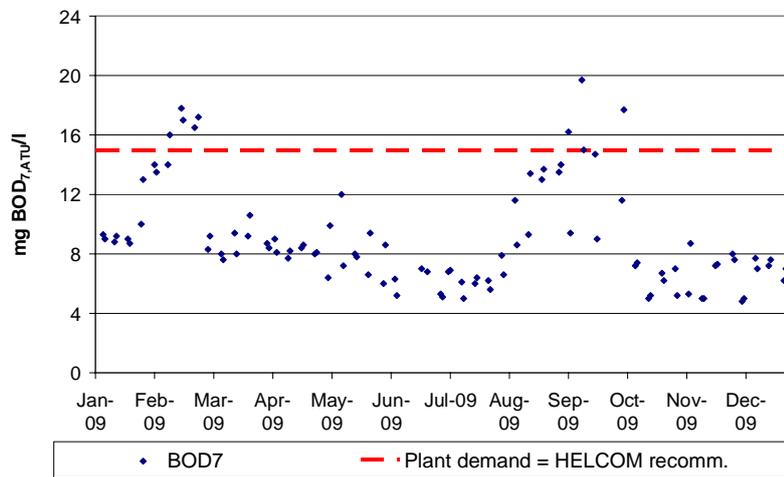


Figure 2.16. Effluent BOD_{7,ATU} concentrations, 2009.

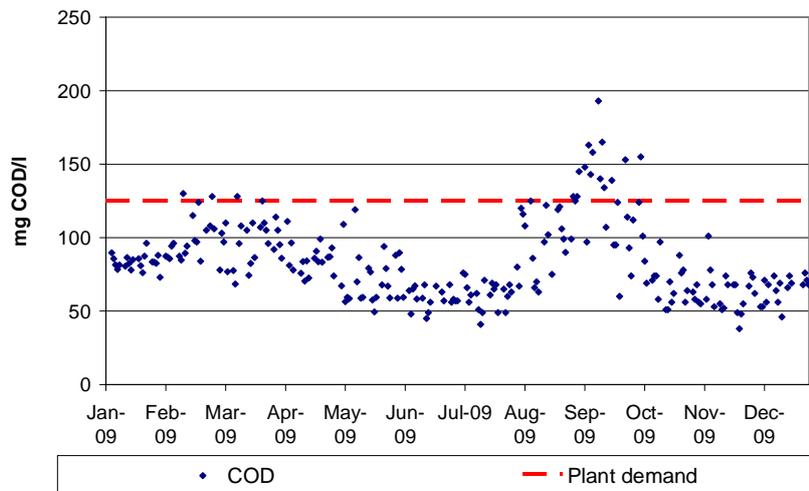


Figure 2.17. Effluent COD concentrations, 2009.

2.4 Process description

The technical aspects of the treatment process are described here concisely. A more detailed description can be found in the process manual written by the constructor.

2.4.1 Primary treatment

There are altogether 17 individual pipes coming into the influent chamber. Two of these are plant internal pipes, conveying rejects from sludge treatment and runoff from the composting field. All pipelines are equipped with electrically driven valves and online measurements of flow rate. In addition, there are online measurements of e.g. turbidity and ammonium nitrogen for individual pipes. Each wastewater source (14 altogether) can be sampled by an automatic flow-proportional sampler. The samplers and analysators are located in a measurement room on top of the inlet chamber.

From the inlet chamber the wastewater is directed to 2 x 3 mm automatic MEVA screens in order to remove non-biological parts from the wastewater. The screenings wash presses are placed in the screen building together with containers for screenings and grit.

After screening, the wastewater flows to a rectangular 2-line aerated grit and grease removal unit. The unit has separate corridors for sand and grease. Sand pumps are hanging from a moving bridge over the separation corridor. The pumps move along the separation corridor with the bridge and discharge to a through which directs the sand/water mixture into the classifiers located in the screening building. Floating grease is removed by scraper from a separate corridor. The dimensioning hydraulic retention time of the sand and grease separation is 20 – 30 min (actual HRT ~ 1 h).

Septic sludge from the catchment area is delivered to a reception tank located below the floor of the screen building and pumped from there to the inlet chamber.

All process machinery, channels and basins in the screening building as well as the grit removal unit are covered and equipped with odour removal by air exhaustion.

2.4.2 Flow equalization and emergency controls

From grit removal the wastewater flows to a buffer and equalization tank, from which it is pumped to the distribution chamber of aeration lines with three axial flow submerged pumps (see Figure 2.18). The flow is equalized by controlled pumping to avoid rapid flow changes and peak flows in the biological process.

Peak flows that are too high to be levelled with controlled pumping are directed to two emergency tanks via an overflow weir installed in the equalisation tank. When the inlet flow has normalised the wastewater in the emergency tanks is pumped back to the inlet chamber.

Wastewater from five industrial sources, which may contain oil spills, can be automatically directed to an oil separation unit. The by-pass is intended to be launched

by on-line oil measurement, but due to unreliable measurements the personnel has decided to operate the by-pass manually based on lab analysis. There is a filter in the middle of the rectangular separator basin. Floating oil is collected from the upper end of the filter, whereas heavy fractions descend to the bottom and are collected by gravity. The water flows through the filter. Having passed the oil separator the contaminated wastewater is led to the emergency tank and pumped back to the inlet chamber. The contaminated wastewater can be diluted with wastewater from equalisation tank prior to pumping into the inlet chamber.

All structures mentioned above are covered and equipped with odour removal by air exhaustion.

2.4.3 Biological treatment

Activated sludge process

The activated sludge process is a two-stage step-feed (cascade) DN-process with an anaerobic zone in the beginning of the first stage. The wastewater is divided 50/50 between stages 1 and 2, and all return sludge is directed to the beginning of the first stage. There is internal nitrate circulation in both denitrification-nitrification stages. A simple schematic representation of the process is given in Figure 2.18 and the more detailed PI diagram is shown in Figure 2.18.

There are three identical aeration lines. The total volume of aeration basins is 57 280 m³ and their water depth is 4,5 m.

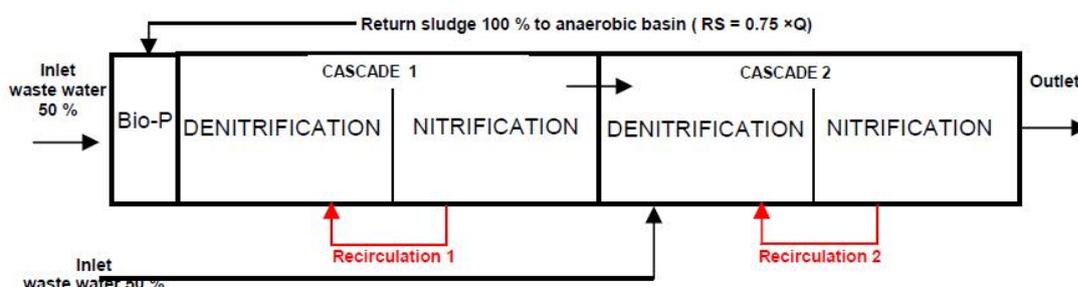


Figure 2.18. Schematic representation of the activated sludge process.

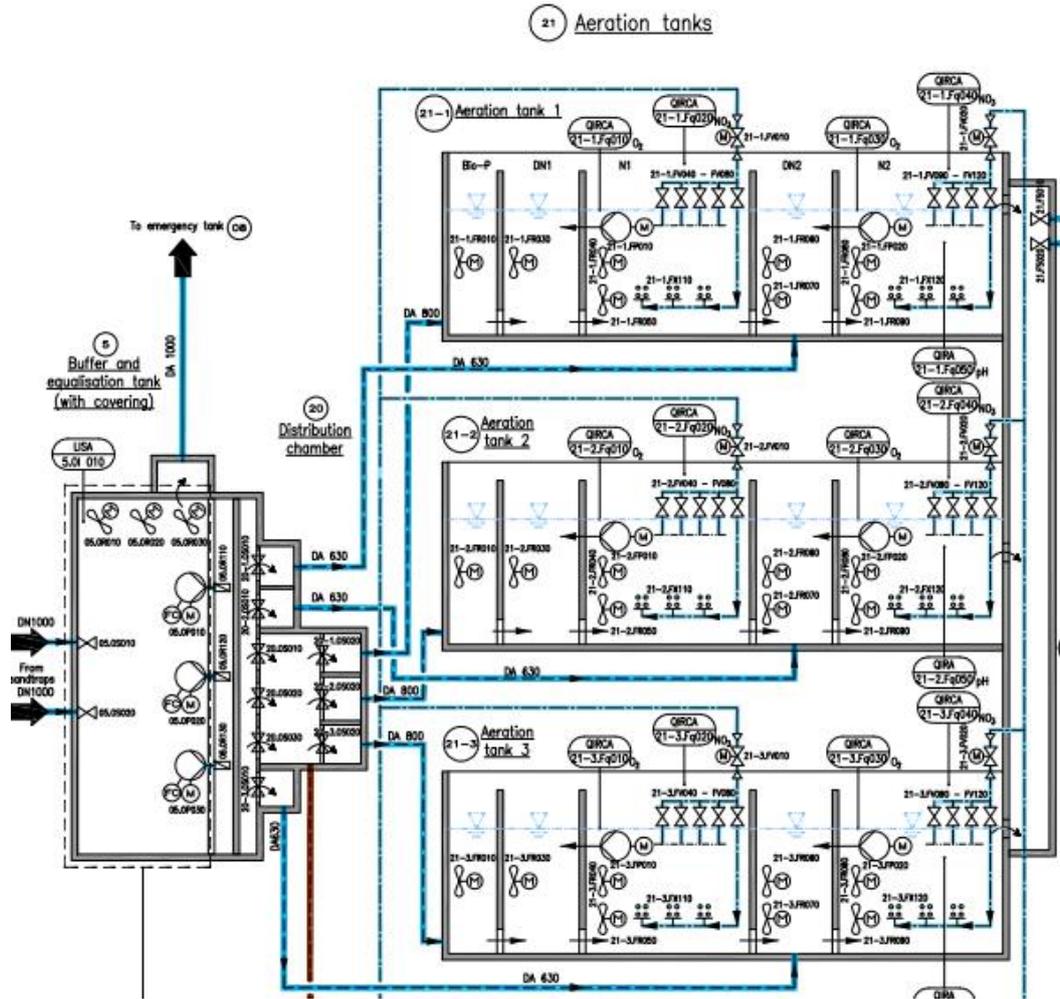


Figure 2.19. Equalisation tank, distribution chamber and aeration lines. PI-diagram.

In the distribution chamber following the equalization tank, the water is divided so that 50 % of wastewater is mixed with 100 % of return sludge, and the mixture is distributed evenly to the beginning of the three aeration lines. The remaining 50 % of wastewater is likewise divided into three equal flows which are directed to the beginning of the second stage of aeration lines. Distribution is kept constant year-round. There are no penstocks, and eventual closing and opening of the water and sludge gates requires the use of set planks.

The anaerobic, anoxic and aerobic zones are not divided into internal zones, and are thus more or less completely stirred reactors. However, at the beginning of both nitrification zones there is a gas removal zone with no aerators and one mixer. Recycling from aerobic to anoxic zone is done by a propeller pump through the wall separating the zones. The recycling pumps do not have frequency converters. According to design documents, they are controlled on/off by nitrate measurements in the aeration basins. According to personnel the pumps are operating constantly. Their capacities per aeration line are, according to as-built documentation, 878 m³/h for Step 1 and 1979 m³/h for Step 2.

Aeration

The bottom aerators are of tubular fine-bubble type. There is one DO measurement and one nitrate measurement per aerobic zone. DO is controlled by a control valve (1 per aerobic zone) and the compressors keep up constant pressure in the air main. There are four HV-turbo KA5S compressors (capacity 9089 m³/h each), located in a separate building. Air flow is measured from each compressor individually and as total flow. The DO setpoint in the aerobic zone is approximately 2 mg/l.

Return sludge pumping and excess sludge withdrawal

The aeration tanks have a common outlet channel from which the activated sludge is directed to a distribution chamber of final sedimentation tanks (see Figure 2.20). MLSS is measured on-line from this chamber. The chamber is located at the return sludge pumping station together with return sludge pumps and excess sludge pumps. Excess sludge is taken from the return sludge pump sump located under the pumping station and pumped with 2+1 frequency-controlled horizontal screw pumps to gravity belt thickeners. In the technical specifications of the renovation project, excess sludge pumps are specified as submerged centrifugal pumps, which is not the case in reality. Excess sludge flow and TSS are measured before the thickeners.

There are two sets of return sludge pumps: one pump in each settler and three pumps in the return sludge pumping station. The pumps in the settler pump the sludge from the settler bottom to the pumping station inlet well. These pumps are controlled by ratio control on the basis of influent flow rate. The pumps in the return sludge pumping station pump the sludge from the inlet well to the mixing chamber before the aeration tanks. These pumps are controlled by sludge level measurement in the inlet well.

Pumps in settling tanks are needed due to the hydraulic profile of the water line. Before the recent renovation project, activated sludge was pumped from aeration basins to distribution of secondary settling. This practice was terminated and replaced with pumping from settlers to return sludge pumping station.

Secondary sedimentation

There are three round secondary sedimentation tanks (d = 40 m, h = 4,0 m). The tanks are equipped with big flocculation wells (d = 14 m), each of which has three slowly rotating mixers. Sludge is collected by suction scrapers and directed to the return sludge pumping station. Each settler has one effluent trough located at the perimeter of the basin.

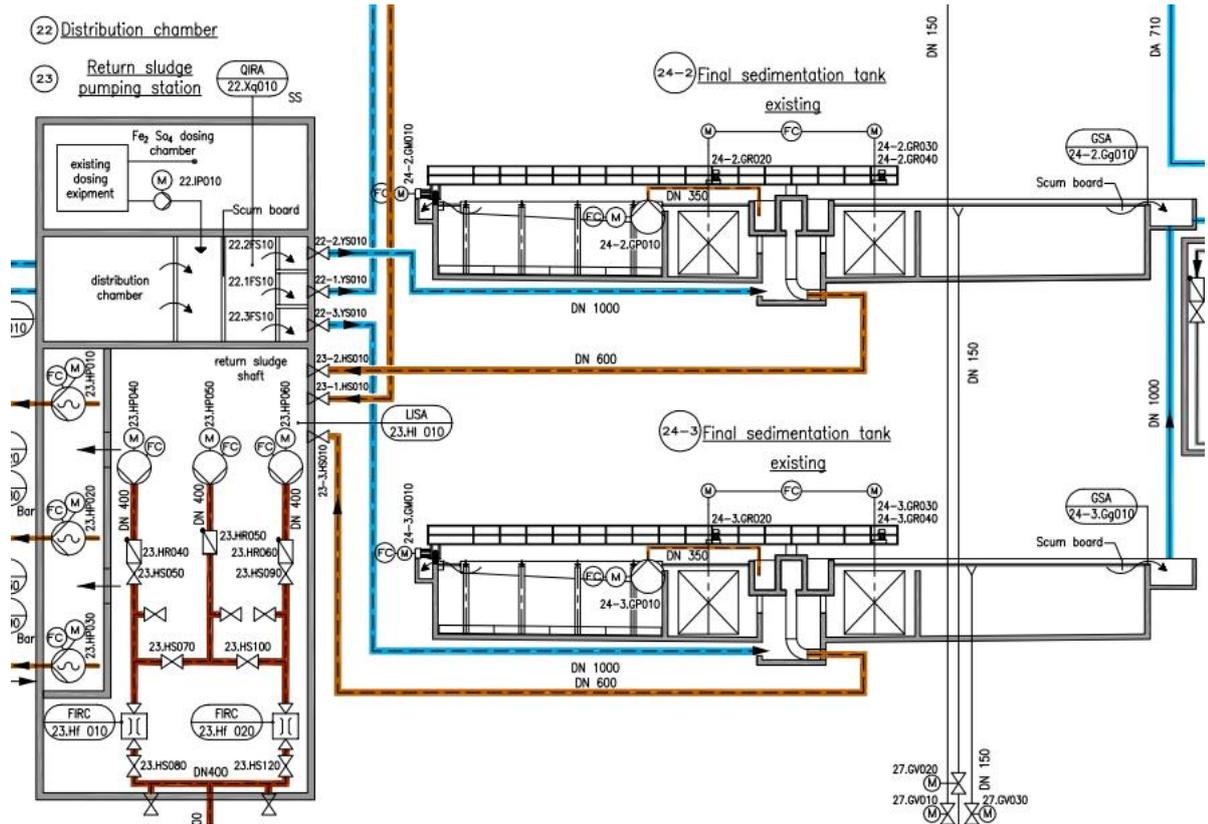


Figure 2.20. Secondary sedimentation (2 tanks of 3) and return sludge pumping station. PI diagram.

Clarified wastewater is directed to the effluent chamber, which is equipped with flow metering, automatic sampler and online monitoring for temperature, TS, pH, PO₄-P, NH₄-N and NO₃-N.

2.4.4 Precipitation of phosphorus

Aluminium sulphate solution is dosed to the return sludge chamber at an approximate rate of 70 l/h to remove the residual phosphorus after biological treatment.

2.4.5 Sludge treatment

Excess sludge is thickened with gravity belt thickeners to a concentration of approximately 6 % TS. The thickeners are completely covered for odour control.

Thickened sludge is pumped to hygienisation. The main components of the hygienisation plant are 3 process tanks, 1 storage tank (after hygienisation), heat exchangers and pumps. Hygienisation takes place at a minimum temperature of 55 °C for 20 - 24 hours in batches, without addition or removal of substances during treatment.

After hygienisation the sludge is pumped to final dewatering by centrifuges. The dewatered sludge cake with a target dry solids content of >20 % is conveyed to an automatic mixer and mixed with shredded wood. The mixture of sludge and structure

material is transferred to an open-stack composting plant located next to the sludge treatment building.

Rejects from the sludge treatment facilities and rainwater from the composting plant are pumped to the inlet chamber.

2.4.6 Odour removal

The odour removal plant based on an activated carbon filter eliminates the odour extracted from the structures and buildings at the mechanical treatment and the sludge treatment facilities. The plant is located outdoors in a container.

2.5 Operation

2.5.1 Operation practice

Activated sludge process

At the time of visit, line 3 was operated as a chemostat with wastewater feed to the second stage only and no return sludge feed at all. In addition, a flow corresponding to the influent of line 3 was pumped from the anaerobic compartment to the neighbouring line 2. This approach was taken due to low influent loads.

The excess sludge flow rate is kept constant for most of the year. This is a practical obligation. Sludge thickening is a bottleneck, and high MLSS concentrations in aeration are a continuous problem (see Item 2.5.3). Therefore, excess sludge is removed at the maximum achievable rate. Sludge retention time is calculated but not used for process control. Decisions on excess sludge flow rate changes are based on MLSS in activated sludge.

The design sludge retention time (SRT) recommended by the constructor is 27 d.

In 2009, excess sludge concentration was measured 5 times per week and the flow was measured continuously. Average amount of removed sludge was 4 583 kg TS/d (design 7 453 kgTS/d). Specific sludge production was 0,82 kgTS/kgBOD₇ and 0,66 kgVS/kgBOD₇. Chemical sludge is included in these figures, so the actual specific sludge production is a little lower. Mixed liquor suspended solids (MLSS) concentration in the aeration basins was measured 3 times per week.

The average MLSS concentration was 9,5 g/l in step 1 and 8,4 g/l in step 2, varying mostly between 6 and 12 g/l as shown in Figure 2.21.

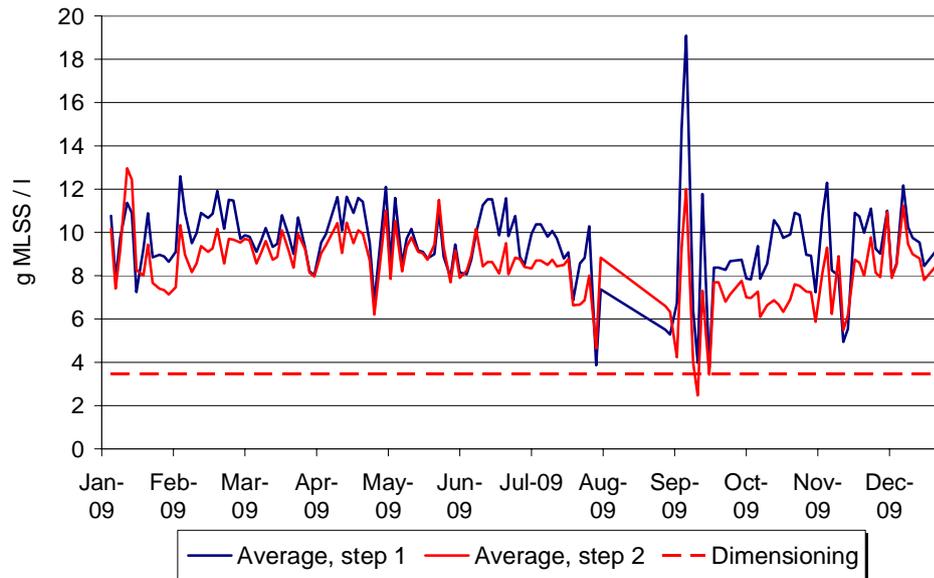


Figure 2.21. MLSS concentration. Average of lines 1-3 at the end of steps 1 and 2, 2009.

The average total sludge retention time (SRT) in 2009 was approximately 100 d. Values between 90 and 150 d can be calculated depending on the approach. The 14-day moving average (which is, in case of SRT, a more informative parameter than daily values) ranged from 54 to 384 d as shown in Figure 2.22.

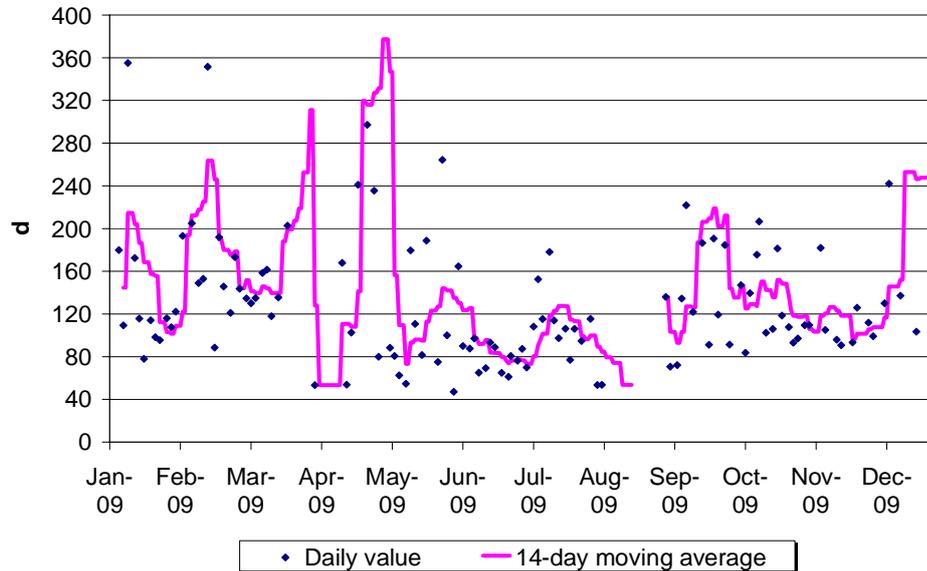


Figure 2.22. Sludge retention time, 2009.

SRT has been very high throughout the year, and it has varied very much. During the first half of the year 2009, the SRT varied within a range of over 200 d. Towards the end of the year the variation was somewhat smaller, and also the excess sludge removal flow rate was kept more constant than before.

In 2010, the SRT and MLSS have been lowered successfully until June, but problems with sludge treatment made sludge removal difficult again in July, causing SRT and MLSS to rise (see Figure 2.23).

The exceptionally high SRT and MLSS are a result of problems in sludge thickening. This issue is explained in more detail in Item 2.5.3.

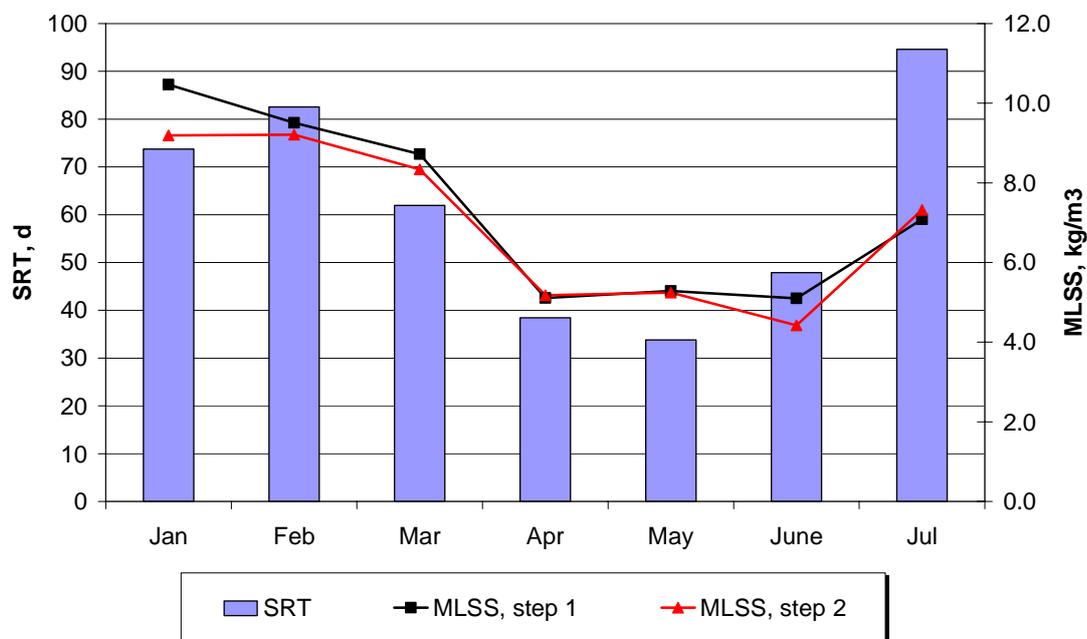


Figure 2.23. Sludge retention time (SRT) and mixed liquor suspended solids (MLSS) as monthly averages, 2010.

Return sludge and nitrate circulation flow rates

The applied return sludge ratio has been 118 % (30 - 230 %). Nitrate circulation pumps operate at constant speed; flow is not measured. According to the pump capacities given in the as-built documents, the total nitrate recycling rate has been, on average, 1 290 %.

Secondary sedimentation and sludge settleability

Two secondary settlers are currently in use. Surface load in secondary sedimentation was, on average, 0,30 m/h and sludge load 2,7 kgMLSS/m²/h in the year 2009. Hourly flow rates were not available. Estimating from the maximum daily flowrate, and not taking flow equalization into account, the maximum surface load would have been 0,84 m/h.

Precipitation of phosphorus

Chemical dosing was originally operated as constant dose (g/m³) and controlled according to wastewater flow rate, with feedback compensation from online analysis of phosphate phosphorus in the effluent. However, this did not prevent the occurrence of occasional high effluent phosphorus concentrations. According to personnel, the

original control strategy did not permit adequate dosing. Consequently, chemical feed was changed to constant, manually determined feed.

2.5.2 Loading of the activated sludge process

BOD₇ load

The F/M-ratio (organic load) and volumetric load to aeration for the year 2009 are presented below in Figure 2.24. The average values were 0,012 kgBOD₇/kgMLSS/d and 0,10 kgBOD₇/m³/d. The dimensioning values are 0,070 kg BOD₇/kg MLSS/d and 0,24 kg BOD₇/m³/d, respectively. The values have been calculated according to the volume of the activated sludge basins without the anaerobic zone.

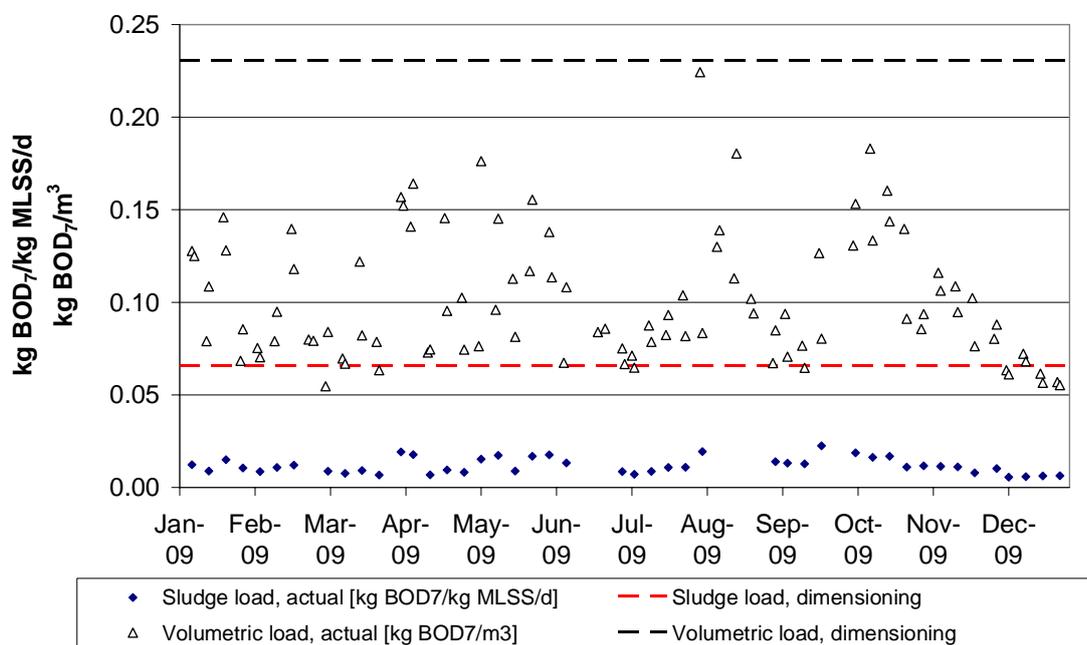


Figure 2.24. Organic load to aeration, 2009.

Total nitrogen load

Total nitrogen load per kgMLSS and per aeration basin volume is plotted in Figure 2.25. The average values for 2009 were 0,003 kgN/kgMLSS/d and 0,022 kgN/m³/d). The dimensioning values are 0,008 kg N/kgMLSS/d and 0,03 kgN/m³/d, respectively. The values have been calculated according to the volume of the activated sludge basins without the anaerobic zone. Typical dimensioning values for plants which have to operate in temperatures of >12 °C are on the order of 0,03 kgN/kgMLSS/d and 0,15 kgN/m³/d.

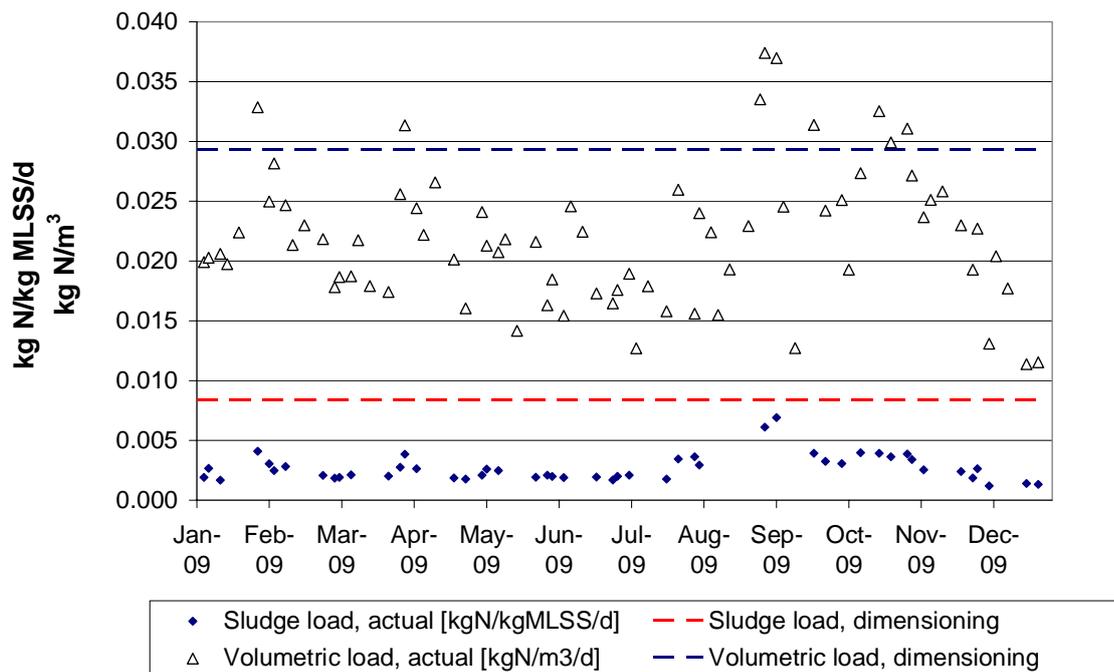


Figure 2.25. Nitrogen volumetric load to aeration, 2009.

Summary

The most important operational parameters of the activated sludge process and secondary sedimentation are presented concisely in Table 2.8.

2.5.3 Typical operational problems

The plant has had a problem of rising effluent phosphorus during summer, when process temperatures may be up to 27 °C. This phenomenon is visible in Figure 2.13 for the year 2009 and was encountered again in August of 2010. The resulting breaches of effluent requirements are an economical issue because of liability payments.

Operating of mechanical sludge thickening has been problematic because of variations in excess sludge SS concentration. The concentration used in the dimensioning calculations was 7,2 kgSS/m³. According to data from September 2010, when MLSS in aeration was around 5 kgSS/m³, excess sludge varied between 2 and 10 kgSS/m³. Problems with thickening arise when SS is below the dimensioning value. Return sludge cannot be made any thicker by reducing return sludge pumping, because the pumps located in the settlers are already operated at minimum speed. Consequently, the MLSS and SRT are excessively high, because sludge cannot be removed from the activated sludge process fast enough.

There are problems with sludge dewatering. The heated sludge cannot be cooled down enough with the warm excess sludge in the summer, and polymer performance deteriorates. As a result, only about 15 % DS in dried sludge is achieved.

The constructor has, to this date, not provided training for the oil separator and for a number of other unit processes.

Table 2.8. Basic operational parameters of the activated sludge process and secondary sedimentation, 2009.

Parameter	Unit	Dimensioning	Actual, 2009
Aeration			
MLSS	kg MLSS/m ³	3.5	8.9
F/M ratio	kg BOD ₇ /kg MLSS/d	0.070	0.012
	kg N/kg MLSS/d	0.008	0.003
Volumetric load	kg BOD ₇ /m ³ /d	0.243	0.101
	kg N/m ³ /d	0.029	0.022
BOD ₇ /N to aeration	-	7.4	4.1
BOD ₇ /P to aeration	-	67	59
Sludge retention time			
total	d	27	100
aerobic	d	17	61
Hydraulic retention time			
anaerobic	h	1.5	4.5
anoxic	h	22	35
aerobic	h	31	48
Return ratio	%	75 %	118 %
DN recycle ratio	%	824 %	1288 %
Total recycle ratio	%	899 %	1406 %
Secondary sedimentation			
Number of units in use	pc	3	2
Rising velocity, q _{ave}	m/h	0.31	0.30
Rising velocity, q _{max}	m/h	0.68	0.84
Sludge loading rate, aver.	kg SS/m ² /h	1.1	2.7
Sludge loading rate, max	kg SS/m ² /h	2.4	7.5
HRT, min	h	5.9	5.7

3 ANALYSIS AND PROPOSED IMPROVEMENTS

3.1 Analysis of plant conditions

3.1.1 Technical conditions and performance

The plant has recently been renovated, and is thus in good structural and mechanical condition. The machinery and instrumentation are modern; however, some machinery was broken and some did not operate satisfactorily.

Effluent targets have been reached for most of the time. However, they have been occasionally exceeded especially for SS (almost $\frac{1}{4}$ of the year), and to a lesser extent for BOD, N_{TOT} and P_{TOT} . Phosphorus and nitrogen removal rates are usually excellent, but operational problems sometimes cause deterioration of phosphorus removal.

3.1.2 Design issues

Pre-treatment

Monitoring facilities of influent wastewater are excellent. All major sources of wastewater can be sampled and monitored separately. Treatment process is also extensively monitored by both manual sampling and online measurements. However, the reports make a bit complicated reading.

The measures taken to protect the biological process against toxic effects from hazardous wastewaters (eg. oil spills) and peak flow rates are excellent.

Biological process

Activated sludge process and secondary sedimentation have a lot of excessive capacity, both with regard to the dimensioning values of the upgrading in 2008 and to the actual loads of 2009 and 2010. On the basis of the operational parameters of the activated sludge process (Table 2.8; eg. anaerobic retention time, recycling capacity, specific BOD and N loading, sludge retention time) it is evident, that one line would be enough to treat the current load to the extent required by HELCOM limits and two lines would be enough for the dimensioning loads. Secondary sedimentation is also underloaded for most of the time. However, due to excessively high MLSS concentrations, the sludge loading rates may sometimes have exceeded the dimensioning values, which is the most likely reason for the occasional, elevated SS concentrations in the effluent. (see Figure 2.15)

It is apparent that the activated sludge process has been renovated with minimum investment costs. Plug-flow regime has not been established and nitrate recycling has been implemented without piping and frequency converters, for example. The implementation in essence mimics the regime of an oxidation ditch, but without reaching the necessary flow velocities and economical aeration control. The functioning of the process is based on substantial overcapacity. It is effective, but not efficient: a lot

of energy is wasted in excess aeration and recycling pumping. In addition, due to the absence of plug-flow conditions, nitrate may be transported from the anoxic to the anaerobic zone, undermining enhanced biological phosphorus removal (EBPR).

On the other hand, very-low loaded activated sludge process does have its benefits. The most important advantages are low production of excess sludge and maximal exploitation of wastewater organic carbon for nitrogen and phosphorus removal. However, even when running two lines, the plant could be operated with the dimensioning SRT of 27 d. Moreover, the organic load of the WWTP seems to be high enough for high-grade nutrient removal ($BOD_7/N > 4$ and $BOD_7/P > 50$) even with lower SRT values.

Recycle pumping has an excess capacity of approximately 100 %, taking into account the maximum allowed effluent nitrogen concentration (10 mg/l). The capacity of total recirculation pumping (return + nitrate recycle) is about 1000 % of dimensioning average flow; about 500 % would be enough to reach the target removal rate. The actual total recycling has been about 1400 %, which, theoretically, enables 93 % nitrogen removal. This explains the recorded nitrogen removal rate of 95 %. It must be noted that according to the design documents, the recycling pumps are supposed to be controlled by nitrate measurements in the aerobic basins (on/off switching by threshold value). Apparently this control has been disabled or the threshold value is very low, because the pumps are running continuously.

It must also be noted that the calculation of required recycling capacity in cascade 2 is erroneous (Chapter 9 of operation manual). If nitrate to be denitrified is calculated by concentration, then the nitrate concentration prevailing in cascade 2 must be calculated as kgN_{Nitri} / Q_{IN} , not $kgN_{Nitri} / (Q_{IN} + Q_{RET})$ as in the document. If the latter was the case, the concentration of nitrate and hence the required amount of denitrification would be reduced by increasing Q_{RET} , which cannot be the case in reality.

The mixing power implemented in anaerobic and anoxic zones is inadequate, according to the installed capacity given in the project documents (0,9 – 2,5 W/m^3 installed vs. 4 – 5 W/m^3 required). Moreover, some mixers were broken and not in use. Visual inspection revealed insufficient mixing in line 3, whereas line 2 was seemingly mixed well. It must be noted that visual inspection does not reveal possible insufficient mixing occurring under the surface. Personnel have recently reported of sludge settling in the aeration basins, and effects of inadequate mixing may have caused disturbances in the sampling of activated sludge (see Item 3.1.4).

At the moment, the air required by biomass is insufficient for proper mixing of activated sludge. The specific air feed into aeration basins in the period 8.5. – 31.12.2010 was 0,82 $m^3/m^2/h$, which is less than half of what is required for adequate mixing. This further illustrates the problems of operating with excessive process volumes.

Sludge treatment

Sludge treatment solutions have been chosen, in principle, correctly. Mechanical thickening and hygienisation with short retention time minimize the possibility of

phosphorus and nitrogen release. However, operational costs of mechanical thickening are high due to polymer consumption. Gravity thickening could provide more robust operation without any polymers and with less maintenance, and there are ways to minimize the risk for phosphorus release in such thickeners. The hygienisation is effective, but consumes a lot of energy and causes deterioration of sludge dewatering properties in the summer, when hygienised sludge cannot be cooled adequately. Sludge to be composted is mixed with structure material in a drum mixer, which ensures good mixing – a prerequisite for good composting. The main composting area is dimensioned for a retention time of 10 weeks and the storage area for 48 months.

The need for thermal hygienisation can be questioned, because all sludge is composted. The target of disposal is, according to documents of the renovation project, “deposition as soil structure material”. According to EU regulations and Finnish legislation, which are compatible with each other, WWTP sludge can be used for soil enrichment, including food crops, after adequate composting and storage without a separate hygienisation stage unless by-products originating from animals are included in the wastewater or sludge. According to our knowledge, Estonian requirements are essentially the same. If by-products are present, the sludge must be hygienised in 70°C for one hour (class III products) or in 133 °C and 3 bar pressure for 20 min (class II products). The current hygienisation does not fulfil these requirements.

In summary, it appears that objectives for sludge disposal could be achieved with careful use of the present composting plant, without separate hygienisation. Taking into account the high energy consumption, problems with sludge dewatering and the fact that hygienisation as implemented would not be adequate in case of animal by-products, hygienisation is, in our opinion, not justified.

3.1.3 Operational issues

Problems with thickening and high MLSS

Sludge retention time and MLSS concentration are 3-5 times higher than would be necessary. This creates risks for exhaustion of substrate and consequent release of phosphorus as well as sludge washout due to bad settleability or overloading of secondary sedimentation. Keeping an excessive amount of sludge in the process is dangerous, because the sludge contains a “phosphorus bomb”, which can be rapidly released if serious problems with EBPR arise. Such problems can occur under certain circumstances, such as low organic loading or deficiency of certain ions (e.g. Mg²⁺).

As stated before, the excessively high MLSS and SRT are caused by problems with sludge thickening. These problems, in turn, are induced by the fluctuation of excess sludge concentration. During the plant visit in October 2010, the excess sludge SS (i.e. return sludge SS) was found to fluctuate between 2-10 kg/m³ with a wavelength of three hours, which was equal to the rotation time of scrapers in the sedimentation basins. Excess sludge pumps have been operated continuously at constant speed. Consequently, the problem most probably lies in the settlers. This conclusion is supported by the observation that the thickeners worked better when the settler scrapers were in different “phase”, i.e. in different places of settler at same time.

It can be concluded with good confidence that either the sludge accumulates differently in different parts of settler bottom or the suction scraper does not work the same way in all parts of settler. The most probable explanation is that sludge exits the inlet chamber of the settler on one side only, although it is supposed to flow evenly to all directions. Thus, more sludge would settle on one side of the settler, creating a thicker and denser blanket than on the other side. This is possible, judging from the position and direction of the inlet pipe in the drawings. A less likely but possible reason is that the settler bottom may not be evenly shaped, e.g. water depth may be different on opposite sides of the basin.

According to personnel, variation of excess sludge concentration did not occur before the renovation, in which pumping from aeration to settling was replaced by pumping from settlers to return sludge pumping station. It is possible that the reasons for sludge concentration variation existed already before renovation, but they were not detected, because gravitational suction removed less water with the sludge than the more powerful pumped suction. Finally, periodical turn-off of these pumps because of overheating can also contribute to the variation in sludge concentration.

Theoretically, the variation of excess sludge concentration can also be connected to the variation of the flow of activated sludge from aeration basins. This flow is created by pumping from equalisation basin and return sludge pumping.

It was noted that contrary to the drawings, the mixers in the center well were not placed uniformly around the center point of the basin. In addition, one mixer was slightly higher and rotating faster than the other two. Theoretically, this may have something to do with uneven settling of sludge, but it is less likely than the reasons given above.

High effluent phosphorus concentrations

In the summer of both 2009 and 2010, a period of high effluent phosphorus concentrations occurred (see Figure 2.13 and Figure 2.13). On both occasions, the concentration of effluent phosphorus rose rapidly and decreased equally rapidly with no obvious explanation.

Occasional rapid deterioration and subsequent, equally rapid restoration of EBPR have been reported from almost all WWTPs applying EBPR. The reasons behind these phenomena are not completely known despite extensive, long-time research. However, the most common reasons, according to current knowledge, are the following:

1. washout of sludge from secondary sedimentation
2. sudden lack of organic substrates in influent wastewater
3. exceptionally high influent phosphorus load, exceeding the capacity of EBPR
4. exceptionally high influent nitrogen load, resulting in more nitrate and disturbance of anaerobic zone
5. occurrence of glycogen-accumulating organisms (GAOs)
6. dissolution of cell-internal polyphosphates and consequent release of phosphate into liquid phase in the activated sludge process or in sludge treatment

All the reasons listed above may have contributed to the problem of high effluent phosphorus at Kohtla-Järve WWTP.

Some exceptionally high influent phosphorus and nitrogen loads did coincide with the problems of high effluent phosphorus. These peak loads did not, however, last as long as the effluent problems and were not continuous. They may have contributed to the problem, but were probably not the only reason. On both occasions, effluent phosphorus was on average higher than influent phosphorus, meaning that phosphorus stored in activated sludge was escaping either through solids washout or dissolution of cell-internal polyphosphates.

In the year 2009 the high effluent P_{TOT} concentrations coincided with high effluent SS concentrations, i.e. sludge was escaping from the secondary settlers at the time of phosphorus problems. A part of the elevated phosphorus was certainly due to the escaping solids. However, the phosphorus concentration was so high compared to the SS-concentration, that some phosphorus may have been released from the sludge into liquid phase. This was certainly the case in June 2010, when effluent $PO_4\text{-P}$ was over 16 mg/l at maximum, while effluent SS remained 4 – 6 mg/l.

According to personnel, the phenomenon occurs in conjunction with high wastewater temperatures. Data from years 2009 (Figure 3.1) and 2010 (Figure 3.2) confirm this: the release seems to coincide with temperature elevation, and temperature and effluent phosphorus peak at the same time.

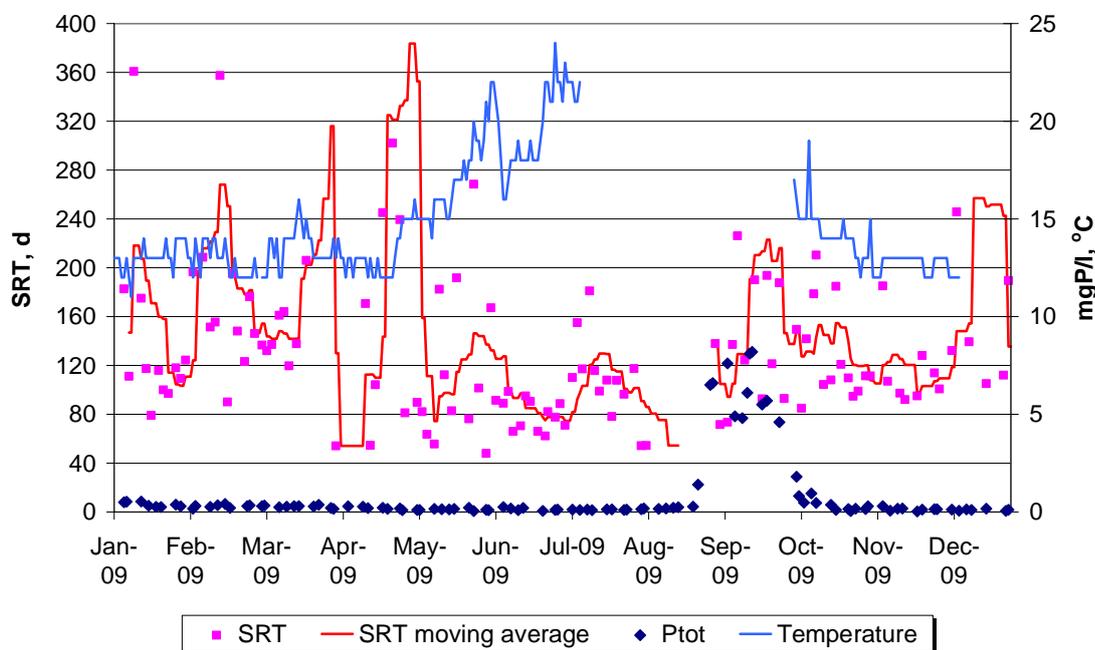


Figure 3.1. Sludge retention time, wastewater temperature and effluent P_{TOT} , 2009.

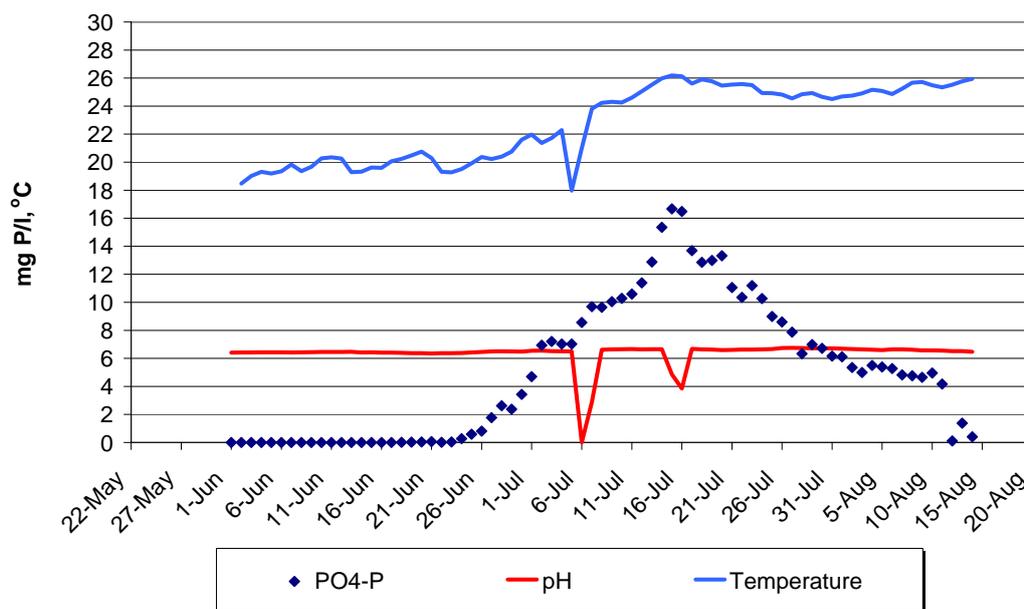


Figure 3.2. Effluent PO₄-P, process pH and temperature, June – August 2010

A very probable reason for the sudden release of phosphates and decline of phosphorus removal is the proliferation of so called glycogen-accumulating organisms (GAOs). These organisms take up volatile fatty acids in anaerobic conditions like phosphorus-accumulating organisms (PAOs), but unlike PAOs, they do not take up phosphate in anoxic and aerobic conditions. GAOs derive the energy required for substrate uptake in anaerobic conditions from cell-internal glycogen, whereas PAOs use cell-internal polyphosphate chains as their energy source. Hence, GAOs compete with PAOs over the available substrate, but do not contribute to phosphorus removal in any way. If GAOs outcompete PAOs, phosphorus removal deteriorates rapidly.

According to recent research¹, the following conditions favour GAOs:

- high temperature (20 - 30 °C)
- pH < 7,0
- low P/VFA ratio in influent wastewater
- one type of VFA (e.g. acetate or propionate) as the sole carbon source

As can be seen in Figure 3.2, process pH was, on average, 6,5 in the summer of 2010. The two pH drops recorded in July may be measurement errors, but they do coincide with sharp increases of effluent phosphate concentration. Seasonal deterioration of EBPR process performance as wastewater temperature increased during the summer has been reported also e.g. in the USA². The high loads of industrial wastewater in Kohtla-Järve certainly induce high process temperatures, and they may also contribute to low pH and unusual composition of VFAs.

¹ Lopez-Vasquez *et al.*, 2009. Modeling the PAO-GAO competition: Effects of carbon source, pH and temperature. *Water Research* 43 (2009), pp. 450-462

² Gu *et al.*, 2005. In: WEF (ed.). Investigation of PAOs and GAOs and their effects on EBPR performance at full-scale wastewater treatment plants in US, October 29 – November 2, WEFTEC, Washington DC, USA, 2005.

As noted above, phosphorus removal not only deteriorated in the summer of 2010, but release of phosphate from PAO cells must also have occurred, because more phosphorus left the plant than entered it. GAOs may have been so abundant that PAOs were left without substrate and resorted to consuming their own cell mass, which triggered phosphate release.

Phosphorus uptake started again in 2009 when temperature dropped below 20 °C. This may be induced by PAOs becoming dominant again because of the temperature drop. In 2010, PO₄-P decreased after the peak by increasing chemical dosing. Whether PAOs regained superiority over GAOs or all phosphate was removed by precipitation is uncertain; temperature remained over 25 °C, so the conditions were still favourable for GAOs.

However, phosphate problems and high temperature have not always occurred simultaneously. In July 2009 (Figure 3.1), wastewater temperature was almost 25 °C, but phosphorus removal worked fine. Due to absence of some temperature data, there is no certain information on process temperature during the phosphorus peak of 2009. In 2010, phosphorus release began already almost 1 month before the highest temperature occurred. Thus, although GAOs probably were the most important reason to the problems of phosphorus removal, other phenomena may also have been involved.

A sudden increase of SRT and simultaneous high temperature may have contributed to phosphorus release. Looking at Figure 3.1 and comparing Figure 3.2 and Figure 2.23 we see that both occasions of high effluent phosphate were preceded by a steep decrease in excess sludge removal and, consequently, steep increase in the SRT. This makes biomass accumulate in the process at a time when biological reactions proceed fast. The PAO microbes may have run out of substrate and resorted to consuming their own cell mass, or a prolonged retention time in sludge treatment may have induced phosphorus release there. Whatever the direct reason for phosphorus release was, it occurred at a time when a lot of sludge was retained in the process and, consequently, there was a lot of phosphorus to be released.

Finally, it must be said that the existing dosing of aluminium sulphate is obviously not functioning as it should, because high effluent concentrations of PO₄-P have been recorded. The phosphorus peaks could probably have been cut with the existing chemical precipitation, if it was controlled by effluent flow rate and online analysis of effluent PO₄ as described in the constructor's documents.

Future prospects

If the expected changes in incoming load will take place (see Item 2.1), the BOD/N ratio and, to a lesser extent, also BOD/P ratio of the influent wastewater will decrease. This will undermine nitrogen removal and possibly also phosphorus removal. The wastewater of AS Nitrofert contains a lot of nitrogen, whereas the dairy wastewaters from Jõhvi contain a lot of readily biodegradable BOD, but not very much nutrients. The WWTP needs the "good" BOD from dairy wastewaters for denitrification and EBPR. Therefore, it is in the interests of Järve Biopuhastus Oü to receive the dairy wastewaters and not allow their treatment in a separate plant.

3.1.4 Phosphorus balance

A special sampling and analysis program was executed between 8.-29.9.2010 to calculate the phosphorus balance of the WWTP and the internal phosphorus loads coming from sludge treatment. The results of the balance are shown in Table 3.1.

Table 3.1. Phosphorus balance, Kohtla-Järve Wastewater Treatment Plant 8. - 29.9.2010

Item	Unit	Value
Municipal and industrial sources	kgP/d	82
Filtrate from sludge treatment	kgP/d	7
Equalisation tank	kgP/d	97
Excess sludge	kgP/d	39
Effluent	kgP/d	9
Balance, equalization tank	kgP/d	-8
accuracy	%	91
Balance, biological process	kgP/d	49
accuracy	%	49
Balance, total plant	kgP/d	41
accuracy	%	49

In Table 3.1, average daily flows of phosphorus during the monitoring period are given and balances are calculated over three sub-processes as follows:

$$\begin{aligned} \text{balance [kg/d]} &= \text{kgP}_{\text{in}} - \text{kgP}_{\text{out}} \\ \text{accuracy [\%]} &= 100\% - \text{ABS}(\text{balance} / \text{kgP}_{\text{in}}) \end{aligned}$$

In an ideal case, $\text{kgP}_{\text{in}} = \text{kgP}_{\text{out}}$ and accuracy = 100 %. If this is not the case, either phosphorus accumulates in the calculated process or is released from it, or there are errors in phosphorus analyses, sampling or flow measurements. As a general rule, accuracy > 90 % is a good result, indicating that measurements and analyses are correct and no significant accumulation or release takes place.

Internal phosphorus load, i.e. proportion of phosphorus load from sludge treatment filtrate and other plant-internal sources was 7,3 % of total load to the biological process, and the ratio $\text{PO}_4\text{-P}/\text{P}_{\text{TOT}}$ in the plant-internal wastewater was 44 %. These figures indicate that biologically bound phosphorus was not released in the hygienisation process.

Phosphorus balance over the equalization basin is 91 % accurate, indicating that influent flows are correctly measured, sampled and characterized.

Phosphorus balances over the biological process and the entire plant are 49 % accurate, indicating either grave measurement or analysis errors or – more likely – that phosphorus accumulated in the process during the monitoring period. Total phosphorus concentrations measured from aeration basins varied very much, indicating at maximum an accumulation of more than 700 kgP/d. This being impossible, the sampling from aeration basins was probably not representative. This, in turn, may be caused by incomplete mixing due to too low mixing power or aeration intensity.

Finally, the monitoring period may have been too short. Monitoring should span a period of at least one sludge retention time, preferably two. The SRT of the plant at the time of monitoring was about 25 d on the basis of the phosphorus data. In summary, phosphorus was probably accumulating in the biological process, but this phenomenon was masked by incomplete mixing of activated sludge.

3.2 Recommended actions

3.2.1 General

The capacity, level and condition of the facilities and the competence of the personnel are sufficient for high-grade removal of all relevant substances – down to the HELCOM level or even more. However, certain technical bottlenecks and operational problems are causing occasional disturbances, which affect treatment results. Below, we propose

1. immediate measures to find the causes of some problems and to diminish their effects
2. modifications to the current process units, with preliminary implementation plans and cost estimates

The most important goal is to make the sludge thickening work properly, and thus enable controlled and adequate excess sludge removal. At present, the sludge retention time and MLSS concentration are practically out of control. These being the most important parameters of the activated sludge process, nothing else can be expected to work properly before they are fixed. For example, there is a continuous risk for sludge washout and even minor spontaneous phosphorus release affects the treatment results heavily.

Advanced tools and consultation to support process operation are strongly recommended. The personnel has to make important decisions related e.g. to the predicted increase in industrial wastewater load, transition from winter to summer settings and possible transition from three to two-line operation. For example, a dynamic process model would be a good tool to support these decisions. The conditions for constructing and calibrating a reliable model are good: the personnel are skilled and the plant has good instrumentation and laboratory facilities, especially with regard to influent monitoring.

3.2.2 Operation of existing units

Thickening

The compatibility of thickener performance with technical specifications and employer's requirements should be checked and the constructor should be demanded to fix the problem. A letter of claim to the constructor has already been written. If the constructor does not comply, we recommend that Järve Biopuhastus Oü seek legal advice to press the case forward.

As an immediate solution, try changing the type of polymer used in the thickeners.

Secondary sedimentation and sludge pumping

The following actions are recommended to establish the reason for variation of excess sludge concentration. Stopping or slowing down of scrapers and the effect of center well mixers were ruled out during the course of this study.

1. Check that the variation is not connected to the flow of activated sludge from aeration. Draw a graph of the sum of pumping from equalisation basin and return sludge pumping and combine this graph with measurement of excess sludge SS
2. Stop one scraper for e.g. 1,5 h to make the scrapers be in different places at different times. If both settlers have the same erroneous design, this should change the wavelength and/or amplitude of thickness variation
3. Try to identify the settler sector from which the diluted sludge is coming from by matching the SS measurement of excess sludge with visual inspection of scraper position. Verify the identification by stopping the return and excess sludge pumps (and thickeners) for the time when the scraper passes the “problem sector” and start them again, when the scraper is again in the “good sector”. The excess sludge SS should not decrease or should at least decrease less than before. Stopping return sludge pumping for e.g. 1 hour is not dangerous for the activated sludge process, as the retention time in aeration basins is several days. However, this is not a good permanent solution
4. If the scraper pumps allow passing of flow through the pump while the pump is not operating, or the water (sludge) column is not otherwise made discontinuous by e.g. entry of air into the pipe: set sludge level setpoint in the suction well of return sludge pumping station so low (e.g. +56.00) that sludge suction can work on gravitation, without the scraper pump. Stop the scraper pumps and monitor excess sludge SS behaviour to identify, whether the variation was induced by pumped suction

As an immediate action to decrease the concentration variation, close 1 or 2 outer pipes of the suction scraper, which retrieve the most diluted part of the return sludge. Open them only for 1 or 2 rounds in the morning to collect the sludge settled at the perimeter. Be aware that peak flows can push the sludge upward along the outer wall of the basin and to the effluent through. This risk should, however, be minor, because of the flow equalization arrangements applied before aeration and because the settler is comparatively deep.

Operation of biological process

When the problems with sludge removal have been solved and SRT can be controlled, the plant should shift to operating two aeration lines instead of three during the time of non-freezing air temperatures. This is done to achieve the following benefits:

- save aeration and pumping energy

- create concentration gradients of dissolved BOD and nitrogen in the process, thus improving reaction rates
- reduce unnecessary accumulation of sludge into the process.

With present loading, one line would be enough to treat the whole influent load most of the time. However, two lines should still be used in normal operation to provide for excess capacity in emergency situations. The line which is not used should be filled with clean water to protect the aerators and other equipment, and the equipment should be operated periodically for a short time (i.e. let some air to the aerators, operate mixers and circulation pumps for 1 hour etc.). Treated wastewater can also be used to fill the line, but it is not recommended. It may promote growth of algae, which can be very problematic to remove. In winter, three lines must be used to avoid freezing.

Sludge retention time should be kept below 30 d and MLSS below 5 g/m^3 in wintertime, inasmuch this can be done without endangering nitrification. In summertime, 15 d and $2,5 \text{ g/m}^3$ are probably sufficient. However, the optimal values must be determined on site through empirical observation. Changes in SRT shall be smooth and carefully controlled.

Calculation of the SRT is more complicated in a step-feed process than in a normal activated sludge process. However, it is still fairly straightforward and can be done with at least two ways.

Method 1: use the TSS concentration measured on-line in the distribution chamber. This is the average MLSS concentration in Step 2 of all aeration basins. The average MLSS concentration in Step 1 can be calculated from the solids balance of the aeration basin, when concentration in Step 2, total influent flow rate, division ratio of influent to Step 1 and Step 2 and return sludge flow rate are known. When average concentrations in Step 1 and Step 2 are known, the total amount of sludge in the system can be calculated. Divide the total amount of sludge by the product of excess sludge flow rate and excess sludge TSS (which are both measured online) to get the SRT.

This method is the easier one to use and can easily be automated. In practice, it assumes that the division ratio of flow to Step 1 and Step 2 is the same for all aeration lines; if it is not, the formula must be adjusted accordingly.

Method 2: measure MLSS individually in all aerobic reactors to get the concentrations in Step 1 and Step 2 of each aeration line. Then follow the procedure of Method 1 individually for each treatment line. This method is more elaborate and can be automated only if MLSS is measured online in all aerobic reactors (6 measurements in total). However, it gives you the individual sludge ages of each treatment line and, thus, a more accurate picture of what is happening in the process.

It must be noted that MLSS concentration should be the same in the denitrification and nitrification reactors of each single step, especially when there is a high pumping of internal circulation between these two zones. The concentration should change significantly only at the point where the second part of the influent is introduced to the beginning of Step 2. If the MLSS concentrations of D and N zones inside one step are

different, mixing is inadequate in one or both zones. In this case, use Method 1; it will give you a good approximation of the total sludge age.

MLSS concentration in the anaerobic zone is approximately the same as (or a little lower than) in the D and N zones of Step 1. However, anaerobic SRT is not very important for sludge growth, because practically no biomass synthesis occurs in this zone.

Finally, neither one of the methods described above takes into account the sludge contained in secondary sedimentation basins. However, this sludge amount is a fraction of the amount contained in the aeration basins, and it is usually neglected.

Removal of residual phosphate

Occasional deterioration of phosphorus removal must be considered inevitable in any EBPR process. Controlled chemical precipitation is the only certain method to cut the resulting peaks of effluent phosphorus concentration. Precipitation must be controlled by effluent flow rate and online analysis of effluent $\text{PO}_4\text{-P}$ as described in the project documents. However, care must be taken not to dose too much precipitant, since overdosing may lead to a “vicious circle” of decreased EBPR activity and continuously increasing need of chemicals. The control settings must be tuned to fulfil these requirements. If the period of guarantee is still ongoing, this tuning should be claimed from the constructor.

The occurrence of GAOs can be investigated by analysing the glycogen content of the activated sludge in all aeration basin zones. These analyses should be done during normal process behaviour and at a time when high phosphate concentrations are recorded in the effluent. If glycogen levels especially at the end of the aerobic zone are higher during disturbances of phosphorus removal, GAOs probably are the main reason for these disturbances.

Moreover, the VFA profile of the influent wastewater should be analysed. If influent VFA consists e.g. only of acetate or propionate, it may support proliferation of GAOs.

Because warm temperature supports GAOs, we advise Järve Biopuhastus Oü to investigate, whether the warmest industrial wastewaters could be equalized or cooled in the summertime before they are discharged to the treatment plant. This way, process temperature could perhaps be lowered by a few degrees. In winter, of course, warm waters benefit the biological process. It may also be possible to inhibit the occurrence of GAOs during warm temperatures by elevating the pH in the aeration basins. This can be done e.g. by dosing sodium hydroxide or calcium hydroxide to the process.

Composting

For adequate composting and removal of pathogens like *Salmonella* and *E. Coli*, the composting has to be done in two phases:

- active composting, at least 6 months, during which the rows are mixed once a month
- maturing, at least 6 months, during which the rows are mixed one or two times

Good mixing must be ensured on every mixing occasion. Temperature inside the windrows must be continuously monitored and recorded, and remain over 55 °C for a minimum of 14 d. To be used for all food crops, the end product must have a CO₂ production of less than 3 mg CO₂-C/g VS/d and NO₃-N/NH₄-N ratio of more than 1.

3.2.3 Modifications

Hygienisation and sludge equalisation

We propose that Järve Biopuhastus Oü end the practice of sludge hygienisation and use the hygienisation tanks for equalisation of sludge flow and concentration before mechanical thickening. One or two hygienisation tanks, depending on the amount of excess sludge, will be used to equalize the flow before thickening and one tank will be in reserve. Excess sludge shall be directed to the inlet pipes of the hygienisation tanks with a newly constructed pipeline, by-passing thickeners. From the outlet pipes of the hygienisation tanks, a new pipeline will transport the sludge to the inlet of sludge thickeners. After thickening, sludge will be pumped to the storage tank with existing arrangements, using the current by-pass of hygienisation.

Two hygienisation tanks will be supplied with a propeller mixer and flow direction baffles. According to preliminary calculations, the existing pipes of thickened sludge can be used also for non-thickened sludge; flow rates will not exceed recommended maximum. It may be possible to direct the sludge from the equalisation tanks to thickening gravitationally, in which case no new pumps are necessary. However, this cannot be determined for certain at the moment, because mechanical drawings from the hygienisation plant were not available for this study. If gravitational transport from equalisation tanks to thickeners is not possible, the pumps 41.YP001 and 41.YP002 must be replaced by pumps with larger capacity.

The pumping of sludge to the equalisation tanks and withdrawal from them will be continuous, i.e. the tanks act as flow-through vessels. Retention time in these tanks will be determined by sludge level. Storage tank and dewatering will be operated as they are today. Retention time in the equalisation and storage tanks should be as short as possible to avoid phosphate release from sludge.

We justify this proposition with the following arguments

- careful composting yields acceptable pathogen reduction according to EU legislation
- hygienisation consumes a lot of energy and causes problems for sludge dewatering. Consequently, ending hygienisation will bring savings in operational costs (energy and polymer consumption)
- equalisation of sludge flow yields more certain results than changing the inlet structures of secondary sedimentation

Other measures (optional)

If uneven settling of sludge in secondary sedimentation is confirmed, the distribution of inflow to the settler can be improved. This can be done e.g. by modifying the end of the feeding pipe or assembling baffles in the inlet chamber or to the lower edge of the center well. However, these measures are laborious, because they require emptying the

settler basin. This, in turn, may require temporary lowering of groundwater levels; according to personnel, the basins cannot be emptied because of the buoyancy induced by groundwater. Moreover, the true effect of these measures is difficult to predict and cannot be guaranteed.

The performance of EBPR could be supported and formation of surface sludge tackled by dividing the anaerobic zone of each aeration line into two or three sub-zones, thus creating plug-flow conditions in the anaerobic zone. This can be done with light-structured separation walls and additional mixers.

If the problems of sludge thickening cannot be solved in any way, consider building two gravity thickeners and stop belt thickening altogether. Gravity thickening is a robust, simple process which does not require any chemicals, and thus has minimal operational costs. Risk of phosphate release is fairly small, because the plant has no primary settling and hence no raw sludge, which would bring VFAs to the thickeners. However, to avoid phosphate release, retention time in the thickeners should not exceed 24 h. In addition, the thickeners should be equipped with aeration pipes, so that anaerobic conditions can be inhibited, when risk for phosphate release is high.

3.2.4 New investments

The estimated costs for transforming the hygienisation tanks to equalisation tanks are 68 000 EUR. This figure includes the following items (assembled and ready for use):

- new pipework (DN100, DN80, stainless steel 316L)
- mixers (2 x 3 kW)
- flow guide vanes in two tanks
- electrics, instrumentation and automation
- planning
- contingencies

The estimated investment cost for modifications of the settler center well and inlet chamber is 20 000 – 40 000 EUR.

The estimated investment cost for division of anaerobic zone in all aeration lines to two consecutive sub-zones is 75 000 EUR.

The estimated investment cost for construction of two gravity thickeners (diameter 7 m, water depth 4 m) is 140 000 EUR.

4 SUMMARY

The capacity, level and condition of the facilities and competence of the personnel at Kohtla-Järve WWTP are sufficient for high-grade removal of all relevant substances down to the HELCOM level or even more. The treatment results fulfil HELCOM requirements for most of the time. However, certain technical bottlenecks and operational problems are causing occasional disturbances and violations of effluent limits.

The most important bottleneck is sludge thickening. Thickener problems are disturbing excess sludge removal and, consequently, inhibit proper control of the activated sludge process.

Inadequate mixing of activated sludge is disturbing the control and efficient operation of the activated sludge process. The reasons for this are broken mixers and insufficiently dimensioned mixing capacity.

Sludge hygienisation consumes a lot of energy and creates problems for sludge dewatering. In our opinion, hygienisation is not needed to reach the relevant EU-requirements for sludge quality.

Phosphorus removal has deteriorated during warm process temperatures in 2009 and 2010. The main reason for this is probably the occurrence of glycogen-accumulating organisms (GAOs), which compete with phosphorus-accumulating organisms (PAOs) over substrate and proliferate in warm temperatures. Also other reasons, such as washout of sludge and excessive sludge age, may have contributed to the problems of phosphorus removal.

The consultant proposes that Järve Biopuhastus Oü, in the first hand, demand the constructor to improve the performance of belt thickeners and fix the mixers. The consultant also proposes that Järve Biopuhastus Oü end the practice of sludge hygienisation and convert two hygienisation tanks to equalisation tanks of sludge flow and concentration before mechanical thickening. The estimated costs for this modification are 68 000 EUR. When hygienisation is not used, the composting plant must be operated and monitored with special care to ensure adequate reduction of pathogens.

Modification of inlets of secondary sedimentation, dividing anaerobic zones of aeration basins into sub-zones, and using gravitational thickening instead of mechanical thickening are proposed as optional measures to improve the performance of the WWTP. Their investment costs are on the order of 20 000 – 40 000 EUR, 75 000 EUR and 140 000 EUR, respectively.

Possibilities for equalizing or cooling the warmest industrial wastewaters in the summertime to lower process temperatures and inhibit GAO proliferation should be investigated. It may also be possible to inhibit the occurrence of GAOs during warm

temperatures by elevating the pH in the aeration basins. This can be done e.g. by dosing sodium hydroxide or calcium hydroxide to the process.

Precipitation of residual phosphate should be used more effectively. It is the only certain method to cut the peaks of effluent phosphorus and to guarantee continuous effluent concentrations of less than 0.5 mg/l (HELCOM demand). Precipitation must be controlled by effluent flow rate and online analysis of effluent PO₄-P as described in the project documents. The control settings must be tuned to fulfil the project requirements. If the period of guarantee is still ongoing, this tuning should be claimed from the constructor.

Energy consumption of the activated sludge process and probability of effluent limit violations can be decreased with certain operational measures, e.g. ceasing to operate the process with excessive capacity. These measures are described in detail in this technical audit.

The effect of expected changes in the amount and composition of influent wastewater must be considered in due time. Especially the variables which affect nutrient removal, e.g. changes of BOD/N and BOD/P ratios, VFA profile and substances inhibiting nitrification, must be considered.

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