Optimization of a Wastewater Treatment Plant using modelling tools

Kabd WWTP (Kuwait)

Carla Vázquez Gómara

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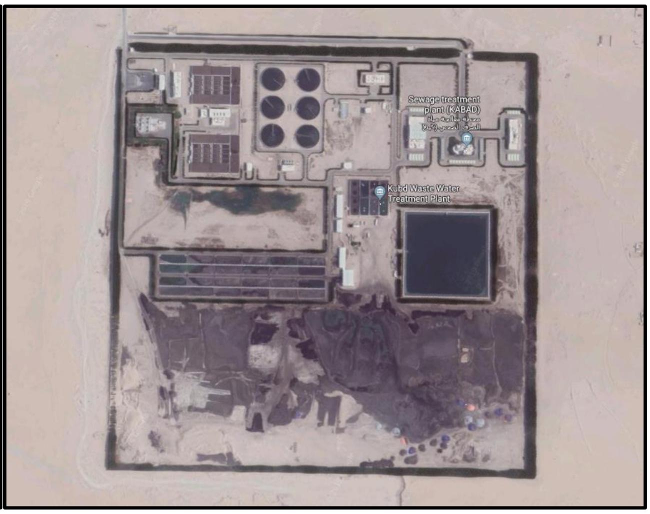

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Master of Science Thesis
by

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Abstract

This study was conducted to evaluate the performance of the Kabd WWTP in Kuwait and find solutions to the current issues they are facing, through the application of a model-based approach using the modelling software BioWin.

Kabd WWTP was constructed in 2010 and started operating in 2012. It was designed to treat domestic wastewater for biological organic matter and nitrogen removal, by using an activated sludge process treatment consisting on four parallel treatment lines followed by a tertiary treatment, the latter required in order to be able to reuse the produced wastewater for agricultural uses.

To develop this study, a protocol was proposed which was based on the STOWA protocol and the guidelines proposed by Meijer and Brdjanovic (2012). The protocol included six steps: project definition, model scheme, data acquisition, data evaluation, calibration and scenario study.

The historical data from previous years was gathered from the lab routine analysis records and the Data Control System records, whereas current data was collected by performing a five day sampling campaign. The results of the campaign showed that the plant was not complying with the effluent standards in terms of COD, BOD, TSS and ammonia. It was identified that the secondary clarifiers were performing poorly and that the nitrification process was not fully happening.

The model was calibrated based on the STOWA guidelines for calibration. The kinetic parameters calibrated in the case of Kabd WWTP were: Maximum specific grow rate for autotrophic biomass, Substrate (NH4) half saturation constant, Aerobic decay rate, Denitrification DO half saturation constant and Denitrification N2 producers.

The model was successfully implemented under steady state conditions and it was used to evaluate the performance of the plant. An increase on the SRT from 5 to 6 days was proposed to solve the problems with high ammonia in the effluent, and also to reduce the daily generated sludge. The model was also used to determine that the plant will be able to comply with the effluent standards until the influent flow reaches Q=200,000 m3/day. Finally, an anaerobic digester was implemented substituting the existing aerobic digester, which showed that the market value of the electricity generated in the plant could reach 686,930 USD per year.

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Abbreviations

Alk	Alkalinity
AD	Anaerobic Digestion
AS	Activated Sludge
ASM	Activated Sludge Model
ASM1	Activated Sludge Model 1
BNR	Biological Nutrients Removal
BOD	Biological Oxygen Demand
BOD5	Biological Oxygen Demand after 5 days
Ca	Calcium
CH4	Methane
CO2	Carbon Dioxide
COD	Chemical Oxygen Demand
CODgf	Glass filtered COD
CODmf	Micro filtered COD
DMC	Data Monitoring Centre
DO	Dissolved Oxygen
EGSB	Expanded Granular Sludge Bed
FB	Fluidized Bed
GCC	Gulf Cooperation Council
GMP	Good Modelling Practice
H2	Hydrogen gas
H2S	Hydrogen sulfide
HRT	Hydraulic Retention Time
IAWPRC	International Association Water Pollution Research and Control
ISS	Inorganic Suspended Solids
IWA	International Water Association
KEPA	Kuwait Environmental Protection Agency
Mg	Magnesium
MLSS	Mixed Liquor Suspended
N	Nitrogen
NO2	Nitrite
NO3	Nitrate
NH3	Ammonia
P	Phosphorus
PAO	Phosphorus Accumulation Organisms
PO4	Phosphate
RAS	Return Activated Sludge
RBCOD	Rapid biodegradable COD
RO	Reverse Osmosis
SBCOD	Slowly biodegradable COD
SC0	Scenario 0
SC1	Scenario 1
SC2	Scenario 2
SC3	Scenario 3
SRT	Sludge Retention Time
SST	Secondary Settling Tank

SVI	Sludge Volume Index
TCOD	Total COD
TDS	Total Dissolved Solids
TKN	Total Kjeldahl Nitrogen
TP	Total Phosphorus
TPmf	Total Phosphorus micro-filtered
TSS	Total Suspended Solid
UASB	Upflow Anaerobic Sludge Bed
USD	United States Dollars
UV	Ultraviolet
VLR	Vertical Loop Reactor
VSS	Volatile Suspended Solid
WAS	Waste Activated Sludge
WWTP	Wastewater Treatment Plant

List of symbols

Symbol	Description	Unit
b_A	Autotrophic biomass decay rate	gVSS/VSS.d
F_{bs}	Readily biodegradable COD fraction	gCOD/g of total COD
F_{xsp}	Slowly biodegradable fraction	gCOD/g of total COD
F_{us}	Unbiodegradable soluble COD fraction	gCOD/g of total COD
F_{up}	Unbiodegradable particulate COD fraction	gCOD/g of total COD
F_{na}	Ammonia fraction	gNH3-N/gTKN
F_{nox}	Particulate organic nitrogen fraction	gN/g Organc N
F_{nus}	Soluble unbiodegradable TKN	gN/gTKN
F_{upN}	N:COD ratio for unbiodegradable particulate COD	gN/gCOD
iN_{SA}	Organic nitrogen fraction in S_A	gN/gCOD
iN_{SF}	Organic nitrogen fraction in S_{SF}	gN/gCOD
iN_{SI}	Organic nitrogen fraction in S_{SI}	gN/gCOD
iN_{XS}	Organic nitrogen fraction in S_{XS}	gN/gCOD
iN_{XI}	Organic nitrogen fraction in S_{XI}	gN/gCOD
K_{NH4}	Substrate NH_4 half saturation constant	gNH4/m³
K_{O2}	Substrate O_2 half saturation constant	gO2/m³
S_F	Readily biodegradable COD	mgCOD/1
S_I	Slowly biodegradable COD	mgCOD/1
TKN_S	Soluble TKN	mgN/1
TKN_X	Particulate TKN	mgN/1
X_S	Unbiodegradable soluble COD	mgCOD/1
X_1	Unbiodegradable particulate COD	mgCOD/1

Chapter 1 Introduction

1.1 Background

It is undeniable that the past century brought huge progress in the field of wastewater treatment, promoting new technologies to avoid contamination both in the environmental and public health spheres. Biological treatment is nowadays widely used for municipal and industrial wastewater treatment (Von Sperling 2007). The recent tendency of looking to wastewater not just as a source of waste but as a source of beneficial resources is leading many countries to question the sustainability of their water management and the efficiency of the wastewater treatment carried out in their facilities.

The possibility of recovering benefits, economic and environmental, from something that used to be just an 'expensive issue that needs to be taken care of', has captured many stakeholders' attention, who are now directing their efforts towards the most pressing needs, such as energy recovery or preservation of hydrological resources.

In the case of Kuwait, the whole Arabian Gulf has a long history of serious problems of water scarcity. They started to produce freshwater from desalination back in 1951, which became fast the major source of freshwater of the states in the Gulf (Aleisa et al. 2011). However, desalination is an expensive and non-environmentally friendly practice, and that is why their next effort was to move towards the promulgation of water reuse and reclamation. For this approach to be successful, a reliable and well-functioning wastewater treatment system is needed.

Kuwait's sanitation coverage has been ranked fifth globally (Aleisa and Al-Shayji 2017). Approximately 90% of the population has access to water and sanitation services, and the treatment facilities have a capacity of around 239 million m^3/year, from which 58% of the water is reused for agriculture (Aleisa and Al-Shayji 2017). The country is aware of the vital importance this field has in the region, and because of that, great efforts are being made towards an environmentally and economically sustainable scenario.

Kabd Wastewater Treatment Plant was built in 2010 and started its operation in 2012, to replace the Jahra WWTP. It is a public facility operated by a private contractor. It is located on the desert area of the country, 40 km away from the city. A model-based study of the Kabd WWTP can help to understand better the performance of the plant while optimizing the conditions for its operation. Both advantages can be of good help in future problem solving and decision making, becoming the model a very useful tool for the managers of the wastewater treatment system.

1.2 Problem statement

Kabd WWTP was designed for an average flow of 180,000 m^3/day, a peak flow of 270,000 m^3/day, and a load of 54,000 kg/day in terms of BOD. Currently, the average influent flow the plant is receiving is 175,000 m^3/day, but this figure is expected to increase in a short term, the same way it has been increasing since the opening of the treatment plant in 2012. This is probably caused by the population growth in the country as well as the higher water demand resulting in a higher wastewater generation. According to that, the plant is expected to exceed its capacity in a short to medium period of time.

To cope with this, an extension of the plant has already been approved, with two more treatment lines. However, the construction of the extension will take some time, and meanwhile, the plant will probably reach its average design flow of 180,000 m^3/day. In this situation, the plant needs to keep running according to the standards for as long as possible, guaranteeing good effluent quality while the extension starts functioning.

Since the intention for most of the treated wastewater is to reuse it for irrigation after tertiary treatment, quality in the effluent needs to be as good as possible and comply with the standards established by the Kuwait Environmental Protection Agency (KEPA) not to be a risk for agriculture nor a threaten for the environment, in case it is directly discharged with no reuse. To do so, the plant needs to improve its performance, so good effluent results can be guaranteed under any circumstances and at any time of the year.

Finally, the plant is also facing issues with the handling of the sludge. Kabd WWTP is producing an average of 20-30 tons of sludge per day. Previously, this sludge was sent to the farms around the country to reuse it as a fertilizer for their crops, but with the new environmental law, this practice is not allowed anymore, since the sludge does not reach the standards for agricultural reuse of sludge by the KEPA that requires a sludge “class A “for reuse.

Currently, the sludge is being piled up on the plant area, being composted by itself on the disposal area, and without any reuse. The plant managers aim to reuse the sludge, either for energy recovery or to put it through a disinfection treatment using ozone in order to reach the standards and be suitable to use as a fertilizer again. The plant is looking for options to avoid incineration and impulse resource recovery, but no decisions have been taken yet.

Chapter 2 Literature review

- This chapter introduces the basics of the most important topics. The project is based on, covering the topics of activated sludge process, water reclamation and reuse, anaerobic digestion for energy recovery, and wastewater treatment plant modelling, with an emphasis on BioWin simulator.

2.1 Activated sludge process: General description

The activated sludge process is very commonly used around the world to treat domestic and industrial wastewater (Von Sperling 2007). The preferred circumstances for its selection are a high effluent quality requirement and small space availability (Von Sperling 2007). Up to the present years, it is been mostly used as a principal treatment, although in the past few years is starting to become a post-treatment option for the effluent of other kinds of treatments, such as anaerobic reactors. (Von Sperling 2007).

The activated sludge involves the production of an activated mass of microorganisms that stabilize the waste under aerobic conditions. The microorganism mass consists principally of bacteria, but also protozoa and other cells (Metcalf et al. 2003). This mass is kept in suspension in the reactor by using appropriate mixing methods, and oxygen is provided to fulfil the aerobic conditions of the system (Metcalf et al. 2003).

The wastewater influent is kept in contact with the biomass for a certain period (Hydraulic Retention Time) after which the mixture of both substances, the so-called mixed liquor suspended solids (MLSS), flows to the secondary clarifier where the microbial mass is settled and thickened (Metcalf et al. 2003). Part of this settled solids are recirculated back to the aerobic reactor (return activated sludge, RAS) in order to keep the concentration of biomass high, an important factor to ensure the efficiency of the system. The other part of the sludge is discarded every day from the system (waste activated sludge, WAS) into the sludge treatment stage (Von Sperling 2007, Metcalf et al. 2003)

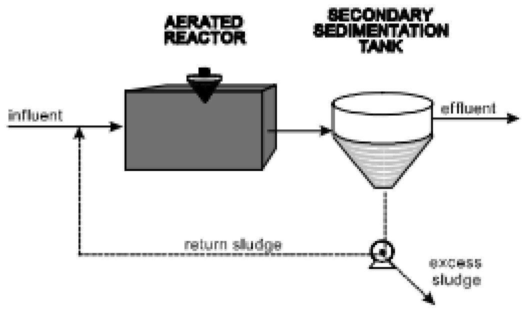


Figure 1 Activated Sludge process units. Von Sperling, 2007

Prior to the activated sludge process, preliminary treatment is required in order to retain bigger solids, along with sand and other substances such as fats and oil. The preliminary treatment consists generally of screening, grit removal and primary sedimentation.

The principal role of the screening is to retain the big solids entering the system in order to avoid the possible damages or the loss of efficiency caused by these materials. Sometimes a fine screen can be also placed after the coarse screen, in order to retain smaller particles. Many factors such as the degree of screenings' removal or disposal options have to be taken into account to decide the type and dimensions of the screening required (Metcalf et al. 2003).

Grit can be removed from wastewater in different ways, although the most commonly used is the grit chamber. Grit chambers are designed to remove grit or other heavy solid materials that have a greater settling velocity than organic solids (Metcalf et al. 2003). Solid materials settle in the grit chamber and are withdrawn, while the rest of the influent flows into the primary clarifier.

The primary clarifier removes readily settleable solids, reducing the number of suspended solids entering the AS system (Metcalf et al. 2003). This way, it reduces the amount of energy required for aeration and mixing, but also the required volume for the biological reactor (Von Sperling 2007). The efficiency of primary sedimentation tanks ranges between 50-70% removal of suspended solids, based on correct design and operation (Metcalf et al. 2003). The sludge generated in this stage is then withdrawn and derived to the sludge treatment line, where it still needs to be stabilized since it contains a high level of biodegradable organic matter (Von Sperling 2007).

As previously stated, activated sludge is a suspended growth system that requires continuous mixing to keep its particles in suspension (Metcalf et al. 2003). This mixing is generally carried out by mechanical mixers. In biological reactors, the devices used for mixing can provide also the oxygen required for the process (Metcalf et al. 2003). These devices are generally surface aerators or coarse/fine bubble diffusers.

There are two parameters related to time in the system: Sludge retention time (SRT) and hydraulic retention time (HRT). First one refers to the time that particulate material stays in the reactor, while the second one reflects the time that the liquid remains in the reactor. In the case of systems that do not have any solid/liquid separation (like an SST), these two values are the same, but when there is a solid-liquid separation the retention times for both are different and SRT> HRT. The relationship between the two parameters is neither proportional nor linear and depends on factors like wastewater organic (COD or BOD5) concentration and reactor suspended solids concentrations (TSS). For biological nutrient removal systems, sludge age is approximately 10 to 25 days and HRT around 10 to 24 hours (Henze et al. 2008)

The sludge age is an essential factor in the design and operation of AS systems and is determined by variables such as reactor volume, production of solids, oxygen consumption, sludge settleability, stability of the system, and most importantly the quality of effluent required (whether only COD removal is required or also other nutrients like nitrogen and phosphorus). (Henze et al 2008, Von Sperling 2007).

Although most of the wastewater treatment plants worldwide are still only focused on organic material removal, there is a growing trend in many countries to incorporate a biological nutrient removal on the treatment (Von Sperling 2007). The decision to incorporate nitrogen and phosphorus removal depends also on the sensitivity of the receiving water body and the final effluent quality standards (Von Sperling 2007).

Nitrogen removal is driven by two processes: nitrification and denitrification. In the case of a nitrogen removing plant, the SRT has to be increased for nitrification to occur, since the growth of the nitrifying bacteria is much lower than other bacteria (Von Sperling 2007). Thus, nitrification is the process that governs the sludge retention time. In order for denitrification to happen, unaerated zones have to be incorporated into the system (Henze et al 2008). In addition, a recycle flow from the aerobic reactor to the anoxic reactor has to be included (Henze et al 2008).

The concept of phosphorus removal is the chemical conversion of phosphorus in phosphates that precipitate and can be removed or the biological incorporation of phosphorus to cell biomass that is later withdrawn of the process as sludge waste (Metcalf et al. 2003). In the case of chemical removal, salts coming from aluminium, iron and calcium are most commonly added. In the case of biological phosphorus removal, anaerobic and aerobic zones are required in the treatment line (Von Sperling 2007) along with a recycle flow through both phases (Meijer 2004). Phosphorus accumulating bacteria (PAOs) uptake the phosphorus under anaerobic conditions, and this phosphorus is then removed from the system with the excess sludge (Von Sperling 2007).

Even though the processes explained above can guarantee a high-quality effluent in conditions of proper design and operation, more and more countries are considering the need of reusing treated wastewater, as water scarcity is becoming a more visible issue day by day. This expanding trend requires a post-treatment for the treated effluent that can guarantee a better biological quality that does not compromise public health in further uses of water.

2.1.1 VLR reactor

A vertical loop reactor, VLR, is an activated sludge treatment process, where the wastewater circulates in a vertical loop around a horizontal baffle. Normally, the design consists of a concrete or steel tank with a depth of approximately 6 m. and a horizontal baffle that extends the entire width and almost the entire length (Smith 1992).

The key factor of this design is that it allows a DO stratification, and therefore it is suited for simultaneous nitrification/denitrification and phosphorous removal without internal recirculation. Typically, the design has two or more rectangular tanks operated in series. The first tank is used as an aerated anoxic reactor, the aeration and mixing provided by surface discs, and the DO level is kept close to zero. The last tank is operated with a DO level higher than 2 mg/l and the aeration provided by bubble diffusers. The horizontal baffle prevents the bubbles from rising to the surface immediately, so they must travel the full length of the reactor to be later released through a perforated air release plate. This increases the aeration efficiency and reduces the cost of the blowers (Siemens 2006).

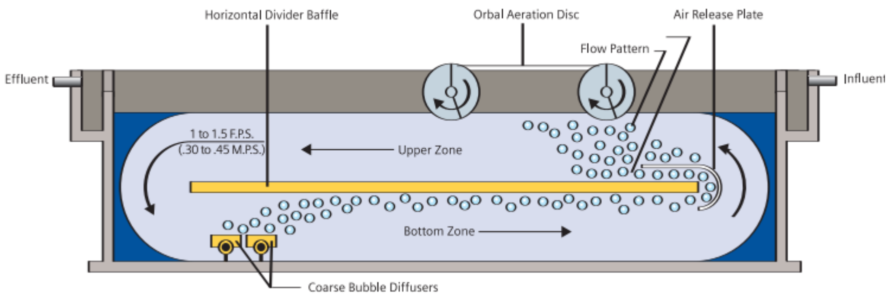


Figure 2 VLR reactor scheme. (Siemens 2006)

2.1.2 Toxicity and bacteria inhibition

Since activated sludge processes use microorganisms for their treatment and these organisms can suffer from inhibition by toxic agents, this can result in a loss of efficiency on the overall system (García Orozco 2008). Inhibition can be caused either by physical-chemical agents such as pH, temperature etc. or any other agent present in wastewater. The effects of toxicity on a cell are: difficulty to absorb nutrients, decrease of the growth rate, and finally the death of the cell. Moving this onto treatment processes, it can be reflected on a lower degradation rate of organic matter (García Orozco 2008).

Cells, depending on the exposure period, can get adapted to the toxic compound and develop the capacity to use it as a substrate (García Orozco 2008). This can be a possibility in case of long periods of exposure, and it is an approach used in certain occasions for industrial treatment applications (García Orozco 2008).

According to Figuerola and Erijman (2010), inhibition of nitrifying bacteria is often related to the presence of phenol, a compound discharged into wastewater from several industrial processes, such as petroleum refineries, coal processing, pharmaceutical, plastic, wood products etc. The inhibition of the nitrification process followed a concentration-dependent manner to phenol, showing 90% inhibition at concentrations higher than 4.7 mg/l.

According to Cortés-Lorenzo et al. (2014), high content of salt in the influent of a wastewater treatment plant can also be a cause for nitrifying bacteria inhibition, since nitrification process is particularly susceptible to salt variations. Concentrations of NaCl greater than 24.1 g/l resulted in a considerable reduction of ammonia oxidation capacity (Cortés-Lorenzo et al. 2014).

Besides the loss of efficiency on treatment processes, toxic compounds can also be the cause for later issues with respect to settleability in the secondary settling tanks (García Orozco 2008).

2.2 Water reclamation and reuse

The increasing pressure on water resources is forcing to find new resources to meet the rising demand. One of the most important sources currently is reclaimed and reused wastewater (Salgot and Folch 2018). Even though wastewater reclamation has been practised for a long time, it has started to be considered as a technology-based practice only from the 20th century (Salgot and Folch 2018). To reuse the wastewater, there is always a need for further treatment besides the traditional secondary treatment (Salgot and Folch 2018). Those treatments are known as tertiary, advanced or reclamation treatments and the technology selected for it depends on the level of treatment required and the later application of the treated wastewater (Salgot and Folch 2018, De Gisi et al. 2017)

Water reuse has many applications that vary from agriculture to industrial and urban options (De Gisi et al. 2017). Since agriculture has historically been the sector needing greater amounts of water, the majority of the wastewater reuse is diverted to activities such as crop irrigation (De Gisi et al. 2017).

The main areas of concern when using treated wastewater are pathogenic microbial contamination and chemical contamination from inorganic salts, heavy metals and detergents. Health risks are associated with the direct or indirect exposure to microbial pathogens, while environmental risks are associated to pathogenic exposure but also to chemical contamination.

(De Gisi et al. 2017). Ultimate concerns on reused wastewater focus on emerging contaminants, such as pharmaceutical products, personal care products, household chemicals, food additives etc. (Salgot and Folch 2018, De Gisi et al. 2017).

2.2.1 General overview of the tertiary treatment

Most treatment lines include a disinfection stage since one of the main risks of treated wastewater comes from the risk of pathogen microbial contamination to public health (Salgot and Folch 2018). The most common tertiary treatment flow diagrams are composed of a combination of these stages (Metcalf et al. 2003): coagulation, flocculation, clarification, filtration and disinfection. The following figure shows a typical combination of technologies (Metcalf et al. 2003):

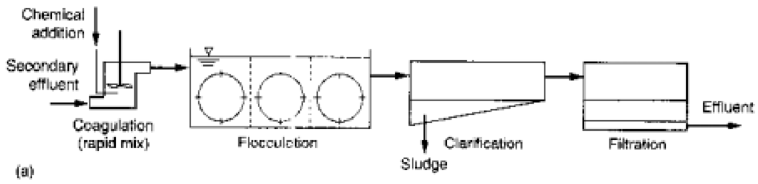


Figure 3 Tertiary treatment process units. Metcalf et al. 2003

Besides the more conventional technologies stated above, some other treatment systems are being implemented and can provide a higher quality effluent, such as reverse osmosis. This technology is a common option for tertiary treatment, it acts as a physical removal process and in theory, can completely remove viruses (Shannon et al. 2008).

2.2.2 Water reuse in Kuwait

Kuwait is located in a very arid area, characterized by high temperatures, low rainfall and a high evaporation rate (Aleisa et al. 2011). The water resources of the country are quite limited, mostly coming from brackish groundwater sources, so Kuwait's water demand relies on desalination (Aleisa et al. 2011). The growth of the water demand in the last decades has directed the country towards the reutilization of wastewater (Abusam 2008). This decision has both environmental and economical benefits for the country, and that is the reason why the government is aiming to reach total reuse of the wastewater (Aleisa and Al-Shayji 2017). Following this path, all the secondary treatment plants were updated to tertiary treatment by 1984, and by 2017 it was estimated that approximately 75% of the wastewater is treated, of which 58% is reused (Aleisa and Al-Shayji 2017)

Most of the treated wastewater is treated up to a level of reverse osmosis quality, the highest treatment rate among the Gulf Cooperation Council (GCC) (Aleisa and Al-Shayji 2017). The WWTP that do not achieve the RO level use treatment schemes such as filtration (via granular media filtration) and disinfection (via chlorination or UV). Currently, most of the treated wastewater effluent is reused for irrigation purposes, especially the RO quality level effluent.

Data from wastewater treatment plants has shown that the reuse of treated wastewater in Kuwait has reduced significantly the number of pollutants discharged into the sea, by reducing more than 50% the volume of wastewater discharged into the sea between 2000 and 2010 (Al-Anzi et al. 2012).

2.3 Anaerobic digestion

The Anaerobic digestion process has been widely used since its first developments, mostly combined with activated sludge processes. The sludge produced during the activated sludge treatment is later treated in anaerobic digesters, stabilizing it and reducing its volume. The possibility of recovering energy from this process is turning it into a more attractive option, due to the high energy consumption of most of the plants and the increasing energy prices.

2.3.1 Concept of anaerobic digestion

The anaerobic degradation route for the organic matter is a multi-stage process with multiple reactions happening in series and parallel. The process is carried out in four consecutive stages, known as: hydrolysis, acidogenesis, acetogenesis and methanogenesis (van Lier et al. 2008).

The anaerobic digestion process comprises a complex food chain, in which several microorganisms are involved in the sequential degradation of the organic matter. The microbial aggregates involved, convert the organic matter and mineralize it into methane (CH4), carbon dioxide (CO_2), ammonia (NH3), hydrogen sulphide (H_2S) and water (H_20) (van Lier et al. 2008).

The bacteria groups carrying out the principal reactions of this process are: fermentative bacteria, hydrogen consuming acetogenic bacteria, hydrogen-producing acetogenic bacteria, carbon dioxide reducers and aceticlastic methanogens. The process is shown below (van Lier et al. 2008):

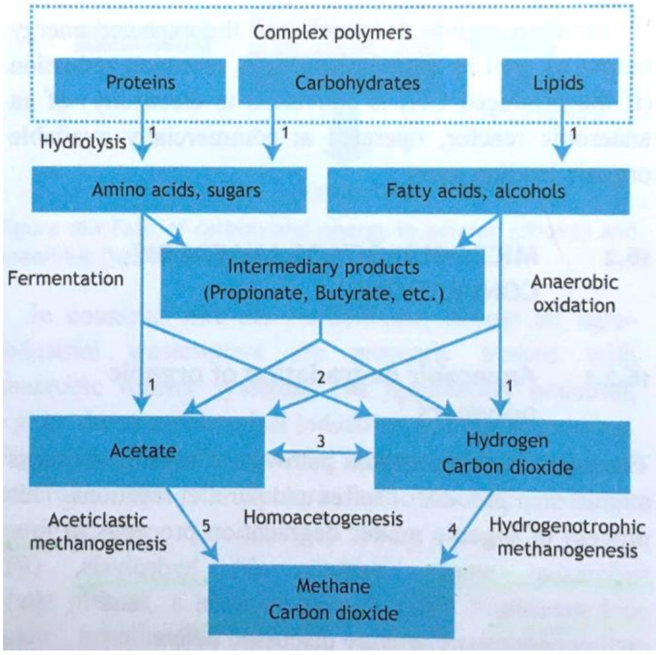


Figure 4 Anaerobic digestion of organic matter (van Lier et al. 2008).

Hydrolysis: Is the process where the enzymes excreted by fermentative bacteria, so-called exo-enzymes, turn the complex undissolved molecules into soluble monomer molecules that can go through the cellular walls and membranes of the fermentative bacteria. The hydrolysis process is very sensitive to temperature (van Lier et al. 2008).

Acidogenesis: Is the process where the previously dissolved compounds (such as amino acids, sugars) present in the fermentative bacteria, are converted to acids (such as propionate, acetate, butyrate) via fermentation or anaerobic oxidation (van Lier et al. 2008).

Acetogenesis: In this process, digestion products are converted into acetate, carbon dioxide (CO2), hydrogen (H2) and new cell material by acetogenic bacteria. Butyrate and propionate are the most important substrates for the acetonic process (van Lier et al. 2008).

Methanogenesis: The final state of the overall anaerobic conversion of organic matter is carried out by the methanogenic bacteria, by converting acetate, hydrogen plus carbonate or methanol into methane, CO2 and new cell material (van Lier et al. 2008).

2.3.2 Anaerobic digestion process

First anaerobic processes were used during the 19th century when sewage tanks were designed to accumulate the sewage solids. The first anaerobic reactor was developed in 1905, the Imhoff tank, where solid sediments were stabilized in a unique tank. All these designs, developed at this early stage, are under the category of low-rate systems since there were no special properties added to the designs to increase their anaerobic catabolic capacity. These processes depended principally on the growth rate of the anaerobic aggregates, and as a result, reactors tended to be big and very fragile to operate. Anaerobic lagoons can also be considered part of the low-rate anaerobic systems. Biggest issues of these systems are related to odour nuisance, overload, and loss of energy and greenhouse gas pollution because of the CH4 escape to the atmosphere (van Lier et al. 2008).

One of the biggest developments on the anaerobic treatment of wastewater was the introduction of the high-rate reactors, in which biomass retention and liquid retention are uncoupled. In the high-rate anaerobic system, physic retention or anaerobic sludge immobilization are used to obtain high concentrations of sludge. This enables the application of high loads of organic matter, and at the same time, relatively short SRT and HRT are kept. During the last three decades, some high-rate anaerobic systems have been developed, including anaerobic filters, UASB reactors, and FB and EGSB reactors (van Lier et al. 2008).

To accommodate an anaerobic reactor to high organic loads, the following conditions have to be fulfilled (van Lier et al. 2008):

- High retention of sludge under the operating conditions.
- Enough contact between the bacterial biomass and the wastewater.
- High reactions rates and absence of serious transportation limitations.
- Biomass needs to be adapted enough to the specific characteristics.
- Prevalence of favourable environmental conditions for the organisms in the system under all the possible operational circumstances.

The main advantages of using anaerobic systems compared to aerobic systems for municipal wastewater treatment are listed below (van Lier et al. 2008):

- Important savings, reaching 90% in operational costs due to the non-requirement of energy for aeration.
- Up to 40-60% of saving on the investment, since fewer treatment units are built.
- When implemented on an appropriate scale, the CH4 recovered can be significant for energy recovery or electricity production.
- The process is robust and able to deal with high hydraulic and organic loads.
- The produced sludge amount is small and well stabilized, so it does not require further treatment.
- Nutrients (N and P) are preserved, providing a high potential for crop irrigation.

2.4 Wastewater treatment plant modelling

2.4.1 General overview of modelling

A model can be defined as a description of the reality that helps to understand and predict certain parameters of a system. Since natural processes are normally very complex, simplification is required to describe them, and therefore models can be a useful and simplified tool to get an understanding on the system of interest (Meijer, 2004).

The level of detail in each model can vary and will be in line with the level of detail needed for the outcome and the data available. Often it is unpractical to go for a high detailed model when it will not provide deeper knowledge on a system. In fact, it is not possible to develop a model that describes every single molecule of a process. Thus, models never reflect the exact reality (Meijer and Brdjanovic 2012).

Modelling has been widely used in many different areas of science. The approximation to reality has been used to predict future scenarios or to analyse the performance of certain systems. Regarding wastewater treatment, application of modelling, which started in the early 1980s, has greatly developed since then into more and more complex options and better understanding.

In 1987, after five years of research and discussion, the IAWPRC (today IWA) launched the Activated Sludge Model no 1, also known as ASM1, with the idea of not only presenting a model but also a guideline for wastewater characterization and a set of default values that have been proven to provide realistic results, only needing minor adjusting (Henze et al. 2000). Since then, the ASM model family has been growing, and more models have been developed as an improvement of the previous ones.

The first one developed, ASM1, can be considered the reference model since it opened the door for a general acceptance of the wastewater treatment modelling (Meijer and Brdjanovic 2012). ASM1 was developed for municipal activated sludge plants, to describe the removal of COD and N. The variety of organic carbon compounds and nitrogenous compounds were subdivided into a limited number of fractions. The model also meant to provide a good description of the sludge production (Meijer and Brdjanovic 2012).

ASM3 was developed to correct the deficiencies of ASM1. They both describe the same processes, but ASM3 introduced the endogenous respiration process and a compound for the storage of organic substrates. These two changes introduced the approach of three different oxygen consumption rates in the process: The rapid rate of oxygen consumption for degradation of RBCOD, the slow rate for degradation of SBCOD and the even slower rate for endogenous respiration. ASM1 only considered a unique oxygen-consuming process, making it more difficult to calibrate the model, since the processes indirectly influence one another through oxygen consumption (Henze et al. 2000, Meijer and Brdjanovic 2012), see Figure 5.

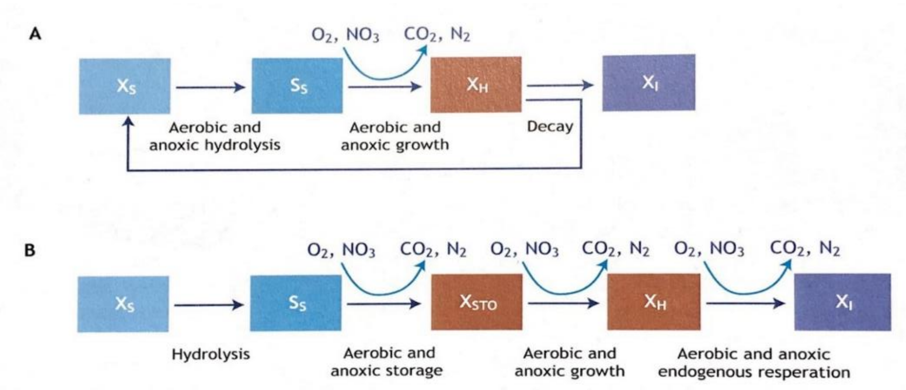


Figure 5 Degradation of COD in ASM1 and ASM3. Meijer and Brdjanovic, 2012

ASM2 and ASM2d were developed to include the process of biological phosphorus removal. The complex nature of this process increases the complexity of these two models. Neither ASM2 nor ASM2d distinguish between the composition of individual cells but consider the average composition of the biomass (Henze et al. 2000)

One important limitation of the ASM models is that they do not take into account the removal of micropollutants, such as metals or xenobiotics. They neither take into account the removal of pathogens (Meijer and Brdjanovic 2012).

One of the first things to consider is the required level of detail of a microbial model. Typical consideration of WWTP design just takes into account the influent and effluent characteristics, ignoring what happens inside the WWTP. That approach is known as the black-box approach. Next step, known as the grey-box model, takes into account the organisms and the functional aspects of the sludge (ASM1, ASM2 and ASM2d). When the metabolism of the organism is also described, the approach becomes close to a glass-box model (ASM3, TUPD model) (Meijer and Brdjanovic 2012).

The use of one or another depends on the objectives and purpose of the model. For activated sludge systems, for instance, black box or grey box models can be accurate enough, but when it comes to plant-wide models, integrating ASM with AD models, more accuracy is required and thus a glass-box approach needs to be used (Meijer and Brdjanovic 2012).

2.4.2 Modelling protocols and procedures

The purpose of generating a protocol, in any field, is to establish a stepwise standardised procedure that can make the experience of the users easier when approaching a project or a task. In the field of wastewater treatment plant modelling, some different protocols have been proposed, most of them focusing on certain parts of the process.

The STOWA protocol, based on work by Hulsbeek et al. (2002) and Meijer et al. (2001) offers a stepwise procedure focused on activated sludge modelling. However, the guidelines by Meijer and Brdjanovic (2012), as well as the HSG guidelines (Langergraber et al. 2004), cover the process of the entire study.

In 2009, the IWA task group for Good Modelling Practice (GMP) proposed a GMP unified protocol for activated sludge modelling, based on most of the guidelines published on previous years (Gillot et al. 2009).

2.4.2.1 Characterization of the main flows

Along with the activated sludge modelling protocols, guidelines for the characterization of the flows began also to be necessary. Thus, at the request of STOWA, standardized guidelines were published based on the work of Roeleveld *et al*. (2002).

Meijer and Brdjanovic (2012) proposed other characterization guidelines based on the previous ones by Roeleveld *et al.* (2002), but with some simplifications in order to make them more practical and financially feasible.

2.4.2.2 Calibration

Calibration is the step of the modelling process when the model parameters are tuned until the results fit with the measured data. The parameter changes applied in different projects are highly variable, and there is not a unified idea of which parameters need to be changed (Meijer and Brdjanovic 2012).

For this stage, some standardized protocols have been proposed, such as BIOMATH (Vanrolleghem et al. 2003) or WERF (Melcer et al. 2004).

Other guidelines, like STOWA and HSG, include the calibration procedure protocol among their more extensive guidelines.

2.4.3 Simulators

A simulator is a software that helps the modeller to simulate the wastewater treatment plant performance (Meijer and Brdjanovic 2012). On a wide level, simulators can be divided into general-purpose simulators and specific wastewater treatment simulators. General-purpose simulators are normally more flexible, but require more knowledge of the user, since they do not have a specific interface on wastewater treatment plants, and models have to be provided (by programming) by the user (Meijer and Brdjanovic 2012, Olsson and Newell 1999). Examples of general-purpose simulators are:

- MATLAB/SIMULINK
- SystemBuild
- Easy-5

On the other hand, specific wastewater treatment simulators, already offer a wide library of predefined process unit models, so the configuration can easily be built by connecting the process unit blocks. The counterpart of this is that it allows the user to simulate process configurations without a full understanding of the model structure (Meijer and Brdajnovic 2012). Examples of specific simulators are:

- AQUASIM
- BioWin
- EFOR
- GPS-X

- SIMBA
- STOAT
- WEST
- SSSP
- DSP

Since there a lot of options in the market for simulators, Olsson and Newell (1999) provide a checklist of criteria to choose the proper simulator that fits the purpose of the study. Some of those criteria are the following: Models available, model library, modularity of the models, validated models, graphical representations, real-time simulation, measurement data…

2.4.4 BioWin: Applications and limitations

BioWin is a commercial software package developed by Envirosim Ltd., in Canada. It is an especially user-friendly software, MS Windows-based, and it is based on modelling principles according to the IAWQ model standards (Meijer and Brdjanovic 2012).

The software includes activated sludge modelling based on ASM1, ASM2d and ASM3, as well as ADM model for anaerobic digestion. Other process units such as primary sedimentation, secondary sedimentation, sludge dewatering, biofilm systems, membrane systems, pH calculation, lime dosage and precipitation reactions are also considered by other sub-models included on the software (Meijer and Brdjanovic 2012). Hydraulics of the plant is modelled as flow in pipes, channels, or tanks inlets and outlets (Olsson and Newell 1999).

Since BioWin is a software developed for engineers rather than scientists, the interface is more user-friendly than other software and focuses more on practical (case-related) conditions. Therefore, the software translates the activated sludge results into practical operational measurements, meaning that the user can use measured data as input data (TSS, COD…) and the software will translate this data into ASM parameters (Xi, Xs…) (Meijer and Brdjanovic 2012). Biowin includes two modules: steady-state model solver, assuming constant flows and compositions, and dynamic model solver, assuming input variations as a function of time (Olsson and Newell 1999).

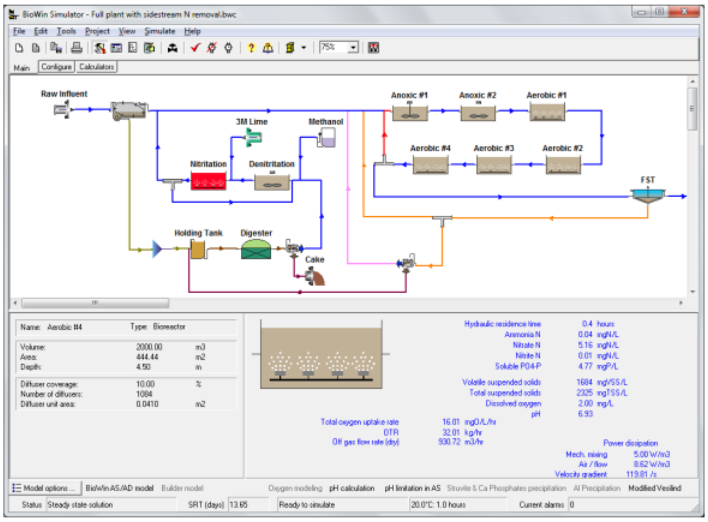


Figure 6 BioWin simulator interface. BioWin help manual

As stated previously, BioWin focuses more on practical-related matters, and therefore it allows non-experienced users to build up configurations quite easily. Since models are not exact copies of reality, same process configuration can lead to different outcomes when using different simulators, as each of them will use different default values for parameters, kinetics and stoichiometry. Users need to be able to understand the model structure and the implications of changing default parameters (Sedran et al. 2006).

Chapter 3 Kabd Wastewater treatment plant description

- This chapter describes the detailed functioning of the Kabd WWTP.
- The chapter includes sections explaining the outline of each part of the treatment, as well as the overall performance of the plant and current issues that is facing.

3.1 Geographical situation and wastewater management overview

The State of Kuwait is a country located in the Arabian Peninsula, sharing borders with Iraq and Saudi Arabia. It is a relatively small country, with a surface of around 17,000 km² and a population of around 4 million people. The capital city, Kuwait city, holds the majority of the inhabitants of the country.

Kuwait is a very arid country, with hardly any water resources available. The temperature during summer can range between 38-46 degrees on average, and precipitation is quite scarce, around 100 mm/year. These climatological conditions make wastewater reuse a key factor in the water management of the country.

Figure 7 Kabd WWTP location map. Google Earth

The wastewater management depends mainly on the Ministry of Public Works in Kuwait, and wastewater treatment is performed in six plants: Sulaibiya, Riqqa, Kabd, Um-Al-Haiman, Alwafra and Alkhairan. The treated wastewater is later forwarded through two distribution networks to multiple agricultural areas around the country to its reuse.

The Kabd WWTP is located 40 km away of the City of Kuwait, to the west. It was constructed in 2012 to replace Jahra WWTP, which was overloaded. The main source of the wastewater arriving at the plant is domestic wastewater, from the areas of Al-Jahra governorate, including Saad Al-Abdulla, Jaber Al-Ahmed, Sulaibkhat, Gharnata and Al-Gurain City.

3.2 Process description

The Kabd WWTP, which receives municipal wastewater from the Jahra Pumping Station, was designed for an average daily flow of 180,000 m^3/day and a peak flow of 270,000 m^3/day. Currently, the plant is receiving an average daily flow of around 175,000 m^3/day. The operation of the WWTP is fully computer-controlled and monitored using the Distributed Control System (DCS). The layout of the plant can be found in Appendix B.

Besides the facilities where the treatment takes place, the plant also includes the following:

- Administration and Control Building
- Workshop Building
- Laboratory Building
- Guard Houses
- Blowers Building
- A Sludge Pumping Station to handle the RAS & WAS flow.
- Sub-stations Building
- Chlorine Building for storage of cylinders and operation units.
- Separate buildings for the Sludge Thickening Equipment, Sludge Dewatering Equipment and Odour Control System for the Inlet area and the Sludge processing units.
- Sludge drying beds with the capacity to handle a minimum of 50% of the total sludge produced in the secondary process, only to be used when the mechanical dewatering system malfunctions.
- Effluent Storage Reservoir with a capacity of 80,000 m^3.
- Storage tanks and pumps for handling wash water requirements of the plant and for the internal irrigation system.

3.2.1 Preliminary treatment

The first stage of the treatment, the preliminary treatment, intends to remove the inorganic solids and large solids entering the plant. It includes the following steps:

- Coarse/Floating material removal
- Fine material/Grit removal
- Oil/Grease removal

Wastewater enters the Inlet Head works through two 1400 mm diameter force mains and is then divided into four pre-treatment channels, operating as 3 duty channels and 1 stand-by. Each channel is composed of: An inlet motorized penstock, bar screen (screening size of 6 mm), Vortex grit chamber and an outlet penstock.

The influent sewage, after removal of coarse and fine solids, passes into a common channel before entering into eight Oil Skimmers, for the removal of oil, grease and other fatty impurities. The treated effluent passes into the common outlet channel through the Parshall flume and flows into the Aeration distribution chamber, through two 1400mm diameter twin pipes. The Parshall flume is installed at the outlet of the Inlet Headwork to measure the inflow, equipped with an ultrasonic flow meter unit on top of the Flume.

The material from the screening and grit removal is transferred into two Compactors, to dewater and compress it, and is then dumped for manual disposal. The oil removed flows into the Primary Scum Pump Station by gravity and is then pumped to the Sludge Drying beds.

3.2.2 Secondary treatment

After the preliminary treatment, sewage flows into the Aeration distribution chamber and it gets equally distributed into two treatment trains by a manual penstock: North and South Aeration tanks.

Each train has an approximate volume of 35,000 m3 and is also divided into two systems. Each system has two VLR tanks (Vertical Loop Reactor) in series, one fine bubble reactor (with two cells in series) and one RAS aeration tank.

The VLR aeration process is a hybrid aeration process using mechanical aeration upfront and diffused aeration in the second half (fine bubble reactor). In the case of Kabd WWTP, the VLR is designed to achieve a denitrification rate of 80% without internal recycle, by performing a high degree of simultaneous nitrification-denitrification in the VLR tanks and a small degree of simultaneous nitrification-denitrification in the first bubble cell.

Each train is a three-stage BNR reactor. Approximately 50% of the bioreactor volume is used by aerated-anoxic zones (VLR tanks) while the rest is aerobic zone (fine bubble diffusers tank). Disc aerators are provided in the top of the VLR tanks to provide both oxygen delivery and mixing in the top part while the bottom part receives a very small oxygen amount to perform almost as an anoxic zone. The VLR tanks are followed by two fine bubble cells in series where fine bubble diffusers are used for providing the required oxygen and mixing.

The mixed liquor from the biological treatment flows into 6 clarifiers through a Clarifier distribution chamber. The chamber is equipped with manual penstocks for the isolation of each clarifier, and an overflow weir for a possible plant bypass. The clarifiers have an internal diameter of 46 m. and an average water depth of 5.0 m. They have been designed to cope with a maximum flow of 270,000 m3/day and to achieve a rate of suspended solids lower than 15 mg/l in the secondary treatment effluent. After the clarification process, secondary effluent flows into the disc filters via the clarifier collection chamber for further filtration.

The RAS/WAS sludge is withdrawn from the bottom of the clarifier. RAS is first returned to the RAS re-aeration tank, equipped with coarse bubble diffusers. This tank is maintained with a very low aeration, just to ensure the mixing, in order to facilitate the reduction of nitrates and increase the total biomass inventory of the system. From there, the RAS is recycled to the first VLR aeration tank of the line. The WAS is derived into a sludge holding tank before entering the sludge treatment line.

The scum collected in the clarifiers is skimmed into the Scum box and is then transferred via the Secondary Scum Lift Station into the sludge drying beds.

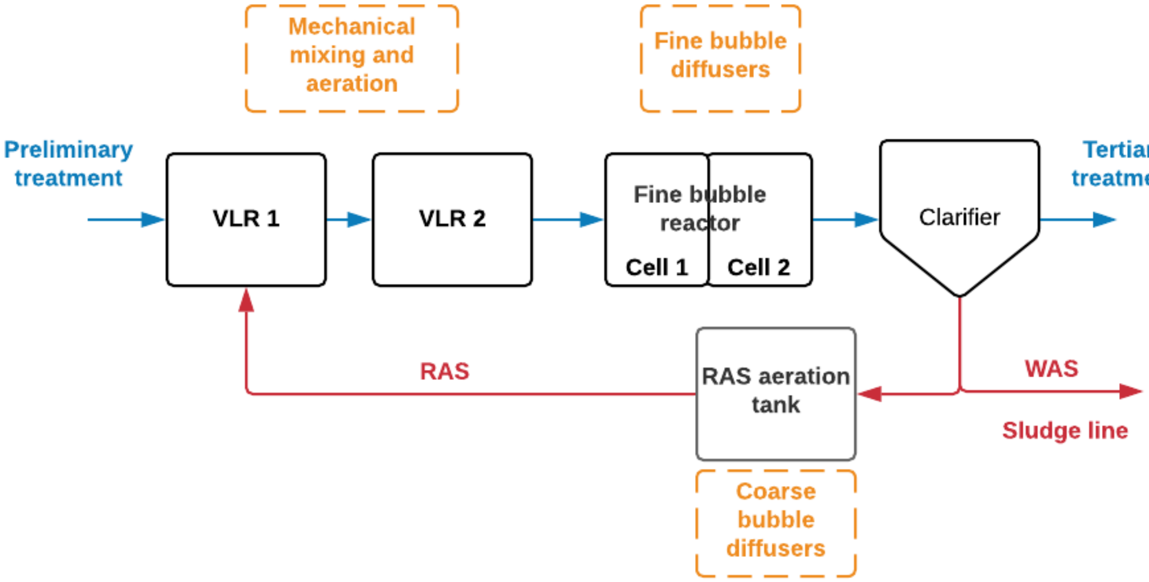


Figure 8 Secondary treatment scheme

3.2.3 Tertiary treatment

The purpose of this stage is to treat the effluent from the secondary treatment to a further level to be able to reuse the treated wastewater. The tertiary treatment of the Kabd WWTP is composed by a filtration stage followed by a disinfection stage.

The effluent from the clarifiers flows through the clarifier collection chamber to the Disc filter influent channel. The filtration step consists of eight rotating Disc Filters, filtration media is Polyester, with a pore size of 10 µm. Filtration occurs from the inside of the filtration panels and towards the outside. The wastewater is fed by gravity to the inside of the drum and into each filtration panel. The suspended solids are retained on the inside of the filter panels while the filtrate discharges into the chamber.

For the disinfection stage, the UV System is used, handling the flow with 2 channels in operation, while the other 2 channels are redundant units.

After disinfection through the UV Channels, the treated effluent flows by gravity into the Overflow Chamber. Under normal flow conditions, the treated effluent flows through the Effluent collection chamber and is discharged in the Data Monitoring Center (DMC, outside the plant) by gravity. During peak flow conditions the excess effluent overflows into the Effluent Storage reservoir through the overflow chamber. This reservoir has an approximate capacity of 80,000 m3. When the peak flow is over, the effluent in the reservoir will flow to the DMC through the effluent collection chamber. Once the treated effluent has reached the DMC, it is derived from there to the different locations for reuse.

A Chlorination System is used to maintain a residual chlorine level of 0.5 ppm in the Treated Effluent from Kabd WWTP to DMC. The continuous chlorine dosing is provided at the Effluent Overflow Chamber, before discharge to DMC. In case of malfunction of the UV disinfection system, the chlorine dosage can be increased for total disinfection of the effluent.

3.2.4 Sludge treatment line

This phase of the treatment intends to treat and dewater the sludge that is daily withdrawn from the water treatment line. The treatment is composed of three steps: Sludge thickening, sludge digestion and sludge dewatering.

The Waste Activated Sludge (WAS) held in the Sludge Holding tank, is carried to the belt thickeners for the sludge thickening process. The Gravity Belt Thickener system consists of 3 lines, with a 2 on-duty/ 1 standby operation mode.

The thickened sludge from the Gravity Belt Thickeners is then transported to the Aerobic Digesters. The Aerobic Digesters are operated as continuous flow, completely mixed aeration reactors and designed based on volatile suspended solids (VSS) reduction. This unit consists of 8 aerobic digesters, each of them with 2 floating surface aerators and a volume of 3200 m^3 approximately.

Sludge from the aerobic digesters discharges into the Sludge dewatering step. The Sludge Dewatering system has 3 lines, in an operating mode of 2 working/ 1 standby. In case of an event of sludge dewatering system malfunction, the sludge is discharged into the drying beds.

The liquid generated in the thickening and dewatering processes is collected and brought back to the Aeration distribution chamber via the Return liquor Pump station.

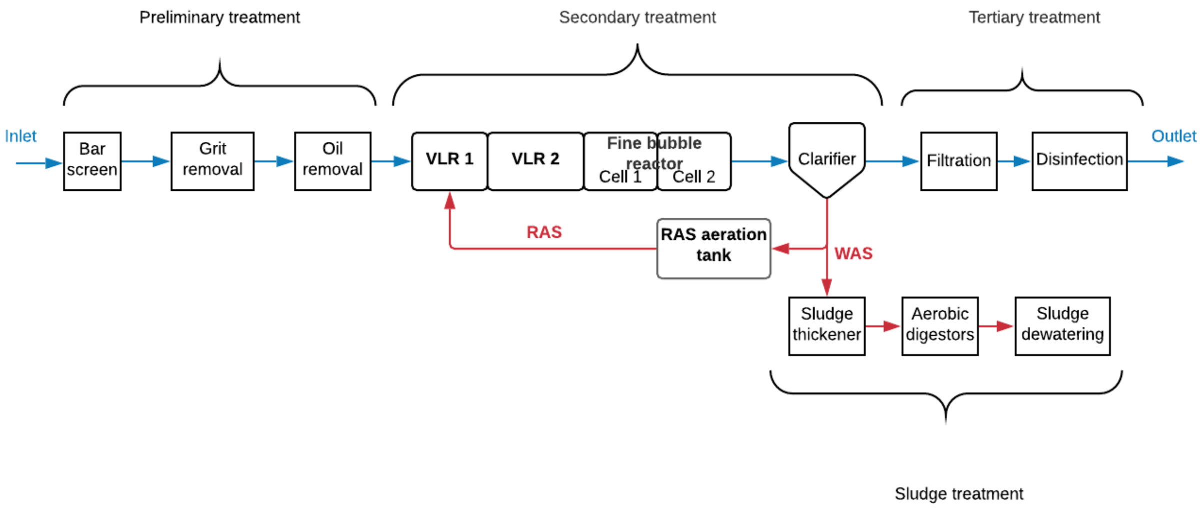


Figure 9 Overall treatment overview

3.3 Effluent discharge standards

The plant needs to comply with the effluent standards from the Kuwait Environmental Public Authority (KEPA) as well as the standards established on the contract between the Ministry of Public Works and the private contractor operating the facility, for irrigation reuse. According to a previous MSc thesis research study (Al-Buloushi 2018) the effluent discharge standards are the following:

Table 1 Effluent discharge standards. Al-Buloushi (2018)

Parameters	Symbol	Unit	(KEPA) irrigation water standards	Kabd contract standards
Temperature	**Temp.**	°C	-	45
pH	**pH**	-	6.5-8.5	6.5-8
Biological Oxygen Demand	**BOD_5**	mg/L	20	10
Chemical Oxygen Demand	**COD**	mg/L	100	100
Conductivity		µs/cm	-	2000
Total Dissolved Solids	**TDS**	mg/L	-	1500
Total Suspended Solids	**TSS**	mg/L	15	10
Phosphate	**PO_4**	mg/L	30	25
Ammonia	**NH3-N**	mg/L	15	15
Nitrate – Nitrogen	**NO3-N**	mg/L	35	20
Dissolved Oxygen	**DO**	mg/L	>2	
Coliform Count		MPN/100 mL	400	400
Faecal Coliform		MPN/100 mL	20	0
Salmonella		MPN/100 mL		0

3.4 Operational issues

According to the information received from the WWTP engineers and the interviews done while in the field, currently, the plant is facing the following operational issues:

First one is related to the filtration process of the tertiary treatment. The opening of the filters is 10 microns, and due to the amount of TSS on the secondary effluent, filtration is difficult and the efficiency of the process decreases. Besides that, algae growing on clarifiers mixes with the secondary effluent TSS and also contributes to the loss of efficiency in the filtration. Currently, chlorine is being dosed prior to the disc filters, and periodical cleaning of the panels with sodium hypochlorite is being done to try to enhance the filtration capacity.

The second issue the plant is facing is related to sludge production. The WWTP produces on average 20-30 tons per day of sludge, which has been piled up for a long time now, composting by itself in the disposal area.

Finally, the plant is also facing issues with the amount of ammonia in the effluent, which is not reaching the standards. Engineers claim it’s a seasonal problem that has been happening every winter when temperatures drop.

Chapter 4 Research question and objectives

4.1 Research questions

- How is the WWTP currently performing? Is the plant meeting the required effluent standards? How can the operation of the plant be changed in order to guarantee a compliance with the standards?
- How much excess flow can the plant handle while still meeting the effluent standards?
- How can the produced amount of sludge in the system be reduced?
- What are the effects, in terms of methane production, energy recovery and cost saving, of replacing the aerobic digesters of the sludge treatment line with anaerobic digesters?

4.2 Objectives

Kabd WWTP is very interested in a model-based analysis of the plant. Up to now, no modelling has been done in the plant or any other wastewater treatment plant in the country, so this model could be a starting point to a new line of study for this plant and others in the region. Having a model of the plant can be useful for future performance analysis, or decision making. According to that, the main objectives of this study are the following:

- To develop a model that simulates the current situation of the plant, and that can be used as a basis for future applications.
- To assess the current performance of the plant.
- To analyse the performance of the plant and its compliance with the effluent standards in case of an increase of influent flow and load.
- To propose operation changes to reduce the amount of sludge generated in the system.
- To evaluate anaerobic digestion as an option to recover energy while dealing with the sludge.

Chapter 5 Materials and methods

- This chapter describes the methodology used on the modelling process of the plant.
- The aim of the chapter is to present a clear and easily understandable protocol that can help the reader to follow the stepwise procedure used for the project.
- Each section of the chapter includes a more detailed explanation of the phases.

5.1 Kabd WWTP model protocol

One of the decisive factors for the successful development of the model-based design wastewater treatment in the Netherlands was the early standardization of the methodologies (Meijer and Brdjanovic 2012). The standardization was based on the development of protocols and guidelines for experienced and new modellers to reduce the complexity of the model-based design projects.

The protocol for the modelling of the Kabd WWTP has been established based on the work developed by Meijer and Brdjanovic (2012). This work, at the same time, was originated from the STOWA protocol for model design, developed after the work published by Hulsbeek *et al.* (2002) and Meijer *et al.* (2001).

The protocol developed for this project can be seen in detail in the diagram in *Figure 16 Kabd WWTP Proposed Modelling Protocol*, at the end of the chapter. It consists of six main steps that are subsequently divided into more detailed tasks. This chapter contains a brief explanation of each of those phases.

5.2 Project definition

The first step of the project was to establish the main objectives and the general overview. It also required to define the boundaries of the project, in order to be clear with the information that was needed and the scope of the model. Some initial information about the plant and the general characteristics of the surrounding environment were also collected at this early stage.

All the information required for this phase has already been presented in previous sections, as well as the objectives of the research. The information about the plant was gathered from a previous MSc thesis research study carried out in 2018 by Bushra Yousef Al-Buloushi on the Kabd WWTP, and data provided by the engineers and managers of the Kabd WWTP.

In the received information, all the plant functioning was explained in detail, including operation of the units, and the current issues that the operation is facing were mentioned. Design data gathered in this phase can be found in Appendix C.

Regarding the boundaries of the project, according to the objectives stated previously, the model was constructed to simulate the performance of the secondary treatment and sludge treatment line. The tertiary treatment, including filtration and disinfection and the later storage of the treated wastewater, was considered out of the scope of the project.

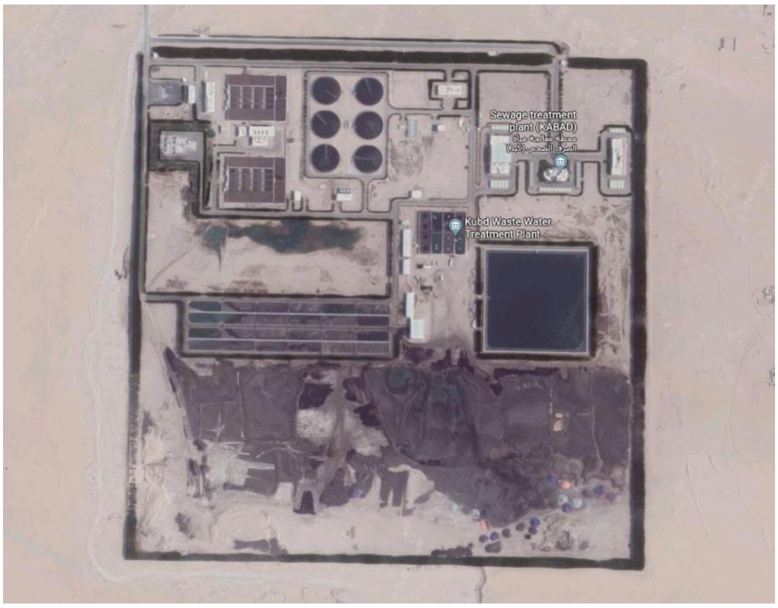


Figure 10 Top view of the Kabd WWTP. Google Earth

5.3 Model flow scheme

Using the information gathered in the previous step, a preliminary plant model was built, based on the process diagram of the plant. Once the model was ready, it was fed with the preliminary flow data and a first simulation was run. At this stage the simulation was not calibrated.

The diagram below, shows a simplification of the plant scheme and the boundaries of the project.

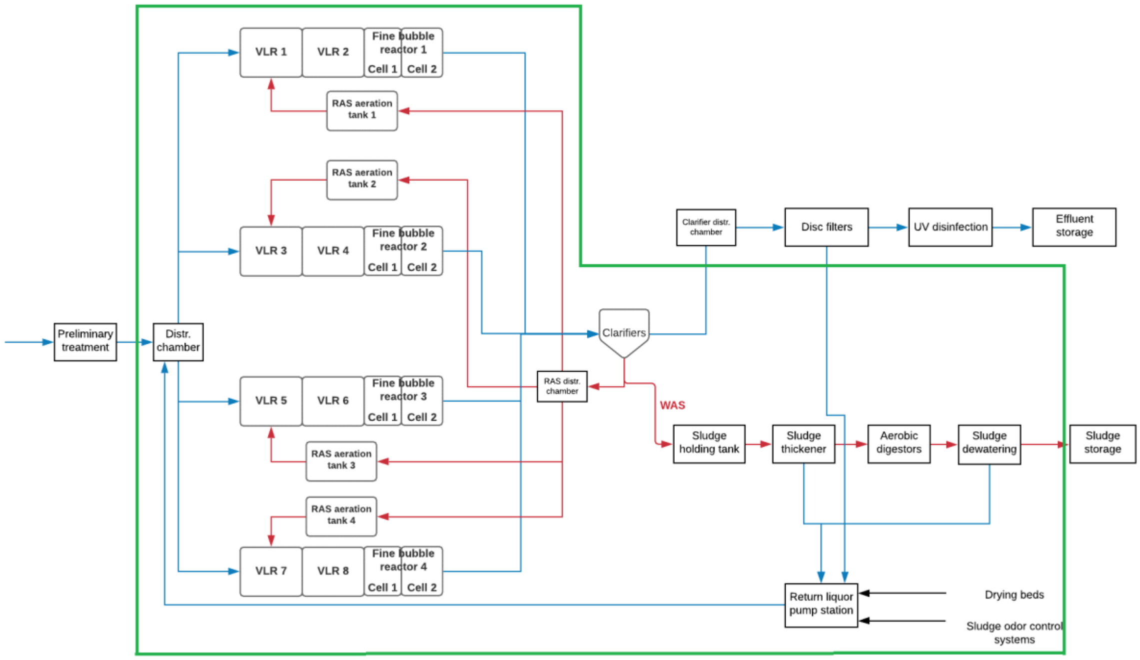


Figure 11 Process flow diagram with the boundaries of the project

Since the wastewater in the plant flows by gravity and all the gates of the distribution chamber are always equally opened, it was assumed that the four lines of treatment receive the same flow, and therefore it was enough to model one of them to represent the process.

5.4 Data acquisition

The data acquisition stage involves the collection of historical data of the plant as well as additional measured data. This step was done in the field during the months of November and December 2019.

As a first step, data regarding daily operation and performance of the plant was collected. Possible performance issues were also looked at, since often this information is more reliable on site than in the plant documentation. In order to collect all the information, visits to the WWTP were done as well as interviews with the plant operators and engineers.

The plant has an automatic control and data acquisition system (SCADA) from where data was collected. The data logged by this system is the following:

- Pump operating time;
- Energy consumption of mechanical devices;
- Blower capacity;
- Temperature;
- Dissolved oxygen;
- Flow meter readings.

An additional sampling campaign was held during the months of November and December 2019. The objectives of the sampling campaign were the following:

- Verification of historical data;
- Collection of data to check the main operational parameters;
- Characterization of the main flows.

Prior to the start of the sampling campaign, a measurement plan was prepared and sent to the operator, in order to discuss the measurements required and facilitate the work during the campaign. This plan included detailed information on the following: Parameters required to be analysed, sample locations, number of samples and type of sampling.

The duration of the flow measurement campaign was 17 days, while the duration of the chemical parameters campaign was 5 days.

5.4.1 Historical data

During the plant visits, historical data was gathered from 2013 to 2019. This data included the following:

- Flow measurements
- Measurements on the following parameters:
  - Temperature
  - pH
  - Conductivity
  - Turbidity
  - TDS
  - TSS
  - VSS
  - COD
  - BOD5
  - DO
  - Oil
  - NH3
  - NO3
  - NO2
  - PO4
  - H2S

The locations where these measurements are carried out are the following: Influent, secondary effluent, tertiary effluent, aeration tank, WAS flow, and RAS flow. The charts used for daily routine collection of this data can be found in Appendix D.

5.4.2 Sampling campaign

The sampling campaign was carried out during the months of November and December. Flow measurements were monitored for 17 days, starting on the 19^{th} November until 5^{th} December, whereas the chemical parameters were measured on a 5 days long campaign, from the 1st December until the 5th December.

For the flow measurements, flow meters located in different spots of the plant were used. Due to the unavailability of a portable system to measure the flows and the malfunction of one of the flow meters in the plant, some of the flows of the sludge line had to be estimated. The locations where meters were available and therefore measurements were made, are shown in the diagram below, as well as the locations where estimations were made based on other known flows and ratios. The locations for measurements were selected according to the availability of flow meters, as well as to the equations for each subsystem explained in the following section 5.5.2. The estimations made are explained in further detail in the *6.1Sampling campaign results* section.

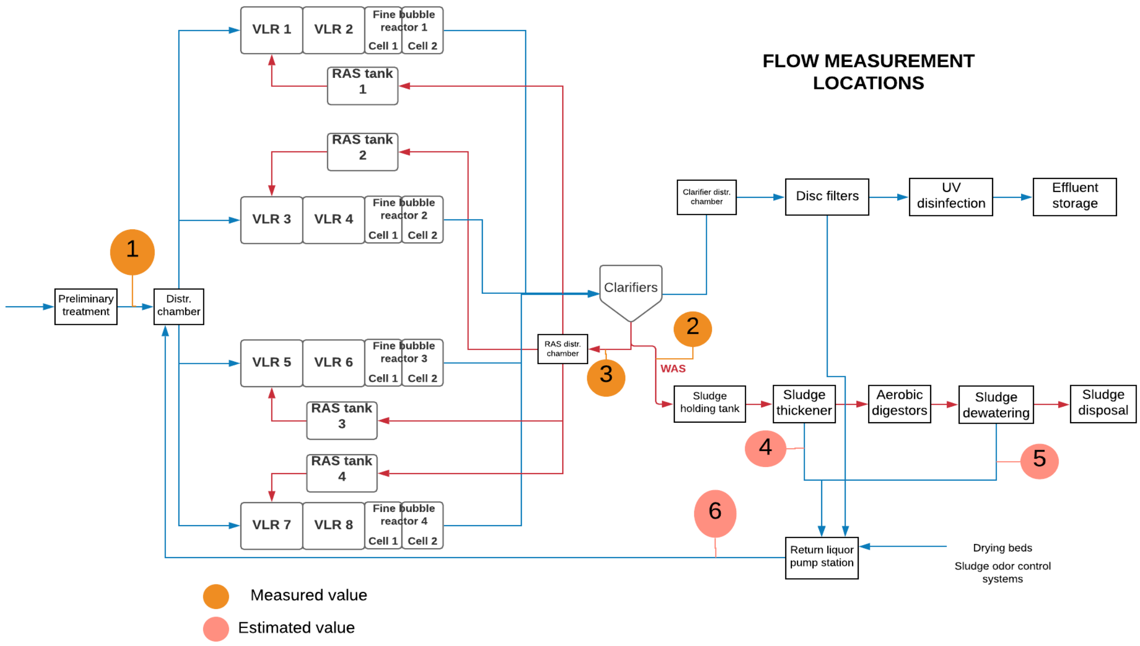


Figure 12. Flow measurement campaign, sampling locations

For the chemical parameters sampling campaign, grab samples and composite samples were taken in 8 different locations for 5 days. Due to the limited staff available, the maximum composite samples that could be taken per day were three, and therefore the locations in the water line were selected for it, since they were considered key locations for the developing of the model, and more accuracy was needed. For the composite samples one sample was taken per shift, every 8 hours (4 pm, 12 am and 8 am). Grab samples were collected at 8 am.

Due to the unavailability of certain materials and chemicals in the WWTP lab, it was decided to daily transport the samples to two external labs to perform the remaining tests. The NUERS lab, from Kuwait University, performed TP, Calcium and Magnesium tests, whereas the Middle East Environmental Laboratories performed the TKN tests. The location where the samples were taken is shown below:

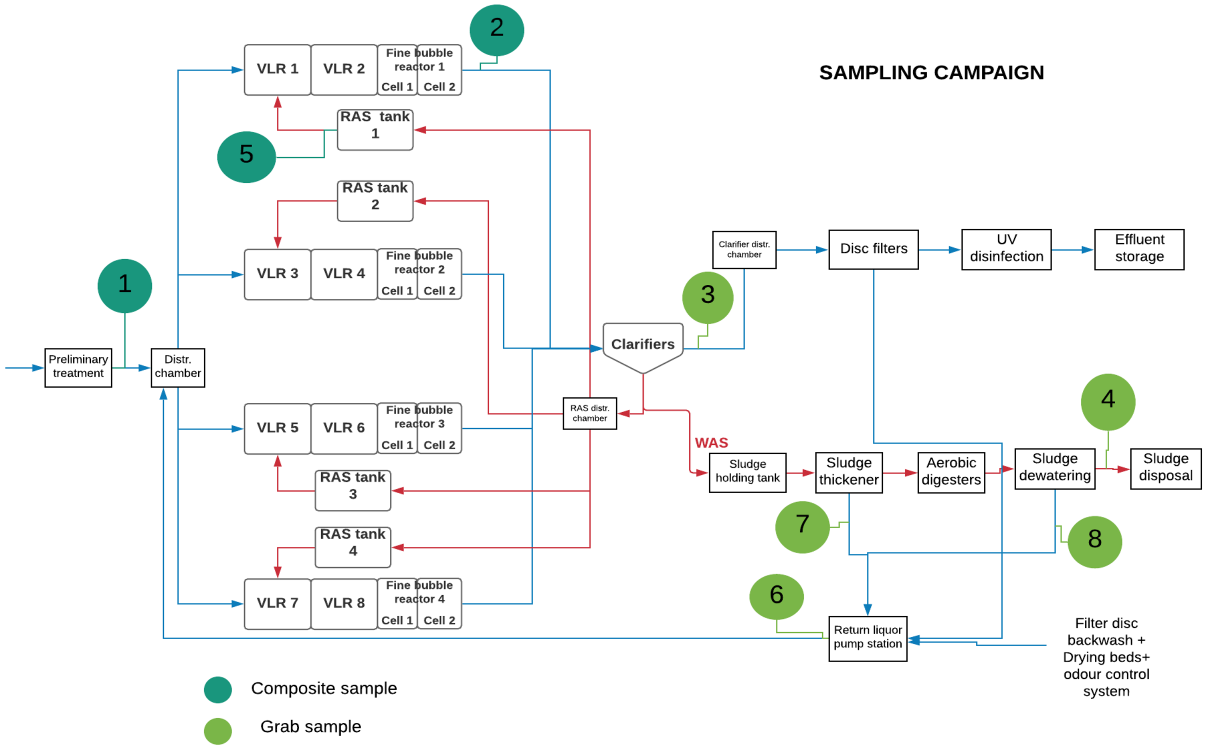


Figure 13. Sampling campaign locations

The parameters analysed on each location are compiled in the following table:

Table 2. Measured parameters during the sampling campaign

Test	**Code name**	**Units**	**Influent**	**Effluent Aer1**	**Secondary effluent**	**Concentrated sludge**	**RAS**	**Return liquor**	**Thickener supernatant**	**Dewatering supernatant**
Location in the PFD			1	2	3	4	5	6	7	8
Type of sample			Comp.	Comp.	Grab	Grab	Comp.	Grab	Grab	Grab
Temperature	T	°C	X	X	X		X	X	X	X
pH	pH	-	X	X	X		X	X	X	X
Total COD	TCOD	mgCOD/l	X		X			X	X	X
COD micro-filtered	CODmf	mgCOD/l	X		X					
COD glass-filtered	CODgf	mgCOD/l	X	X			X			
BOD5	BOD5	mgBOD/l	X		X					
Total Susp. Solids	TSS	mg/l	X	X	X		X	X	X	X
Volatile Susp. Solids	VSS	mg/l	X	X	X		X	X	X	X
Total Kjeldahl Nitrogen	TKN	mgN/l	X	X	X		X	X	X	X
Ammonia	NH3	mgN/l	X	X	X		X	X	X	X
Nitrite	NO2	mgN/l	X	X	X		X		X	
Nitrate	NO3	mgN/l	X	X	X		X		X	
Total Phosphorus	TP	mgP/l	X	X	X		X	X	X	X
Total Phosphorus filtered	TPmf	mgP/l			X					
Dissolved Oxygen	DO	mgDO/l	X	X	X		X		X	X
Alkalinity	Alk	mgCaCO3/l	X		X					
Calcium	Ca	mg/l	X		X			X		
Magnesium	Mg	mg/l	X		X			X		
Moisture content						X				

5.5 Data evaluation

After obtaining all the data from the plant, an evaluation was required in order to check if there were major errors and ensure that the final data set was properly balanced. To do so, flow balances and mass balances were calculated.

Flow balances are used to calculate all the unknown flows of the system. For that, the system was divided into relevant subsystems, and for each of them a balance was done.

Since water is conserved and therefore always accountable (not taking into account the negligible amount that evaporates), flow balances should always close. An inconsistency in these indicates a mistake in one or more flow measurements, or a misunderstanding of the process flow. Detecting these major mistakes in an early stage can save a lot of time and effort.

The calculation of the flow balances was performed by using a matrix system.

Along with the flow balances, the mass balances were also calculated. When no conversion takes place, mass balances perform similar to flow balances and should add up to zero (e.g. TP mass balance). This balance can therefore be used as additional information to calculate missing measurements. When there are more balance equations than number of unknown measurements, the system is “over determined”, and the difference between the number of equations and the unknowns is the “degree of redundancy”. The higher the “degree or redundancy” the higher the accuracy of the calculations, since it provides the possibility to check the balanced data. Due to the limited flow meters and the estimations that had to be made on flow measurements, over determination was not the case in this flow mass balance.

5.5.1 Subsystem division

Flow balances are used to calculate all the unknown flows of the system. For that, the system was divided into relevant subsystems, and for each of them a balance was done. Generally the subsystems correspond with the main areas of interest in the plant (e.g. water line, sludge line), but according to the objectives of the project, some other subsystems may be found relevant for the case. The 9 subsystems for this case are the following: Aeration distribution chamber, Aeration train 1, aeration train 2, aeration train 3, aeration train 4, sludge line, return liquor pump station, clarifiers and RAS distribution chamber.

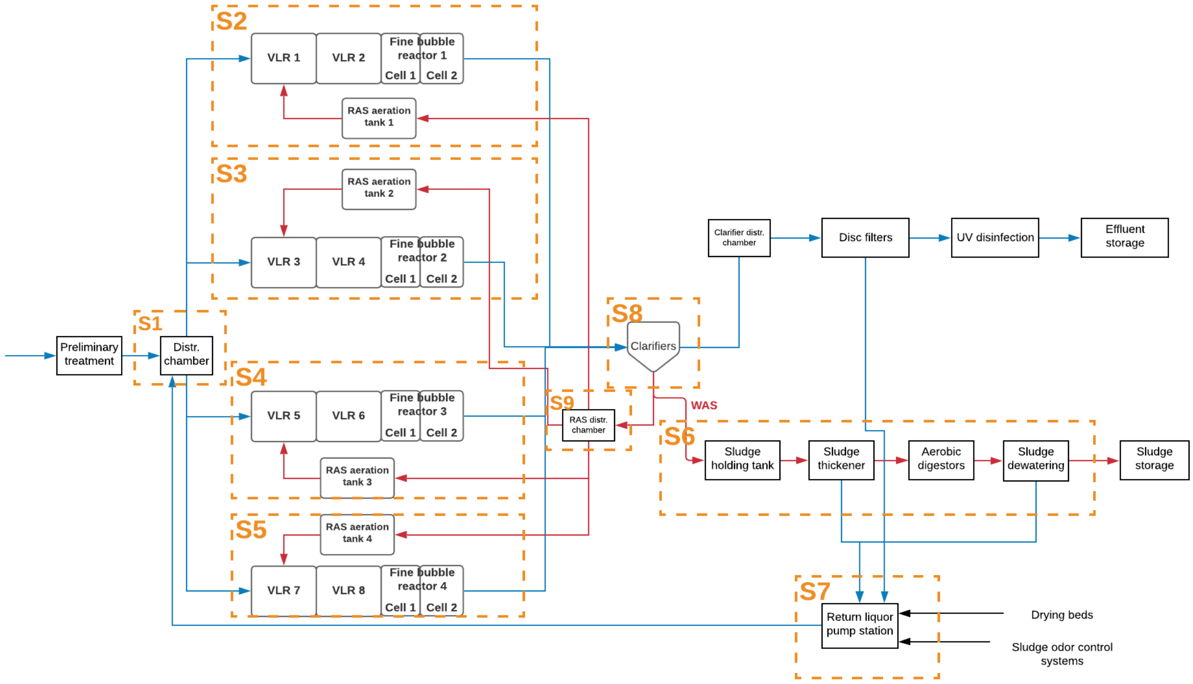


Figure 14. Flow scheme of the WWTP with subsystem boundaries

5.5.2 Flow balances

The process flow diagram with the different subsystems is shown below:

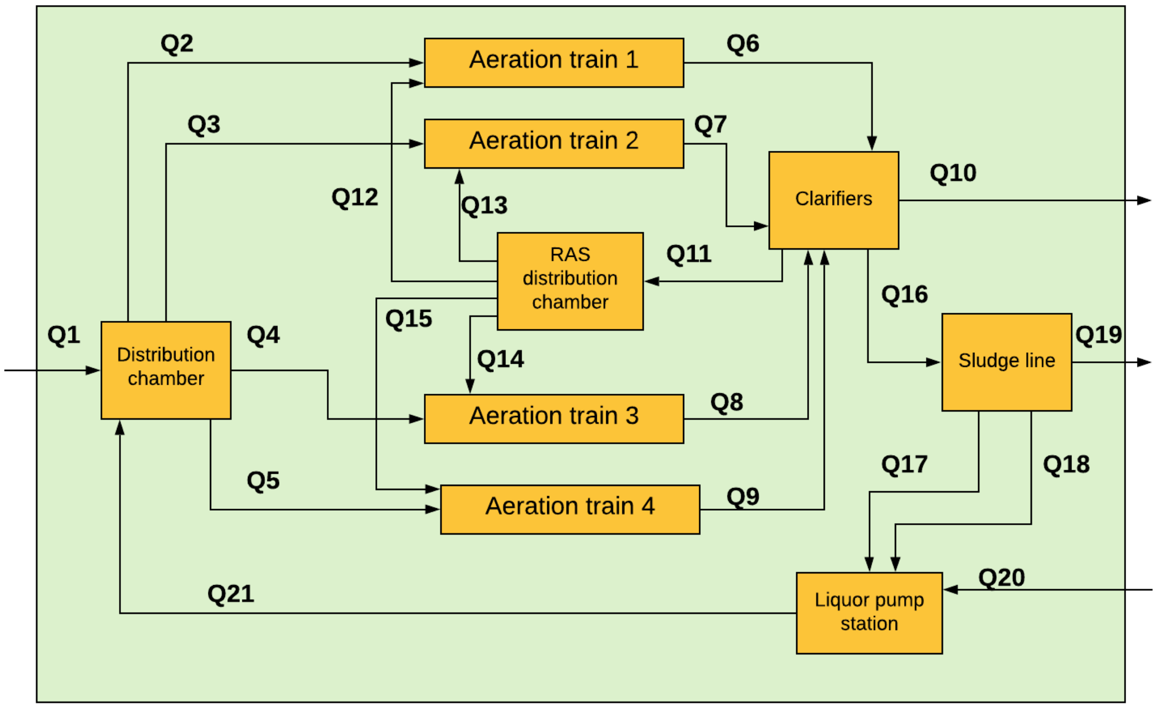


Figure 15. Process flow diagram over subsystem boundaries

The flow balance equations for the individual subsystems derived from the diagram are the following:

Distribution chamber: Q1= Q2+ Q3+ Q4+Q5-Q21

Aeration train 1: Q2+Q12= Q6

Aeration train 2: Q3+Q13=Q7

Aeration train 3: Q4+Q14=Q8

Aeration train 4: Q5+Q15=Q9

Sludge line: Q16= Q17+ Q18+ Q19

Return liquor pump station: Q20+Q17+ Q18 =Q21

Clarifiers: Q6+Q7+Q8+Q9= Q10+Q11+Q16

RAS distribution chamber: Q11= Q12+Q13+Q14+Q15

Overall system: Q1= Q10+ Q19- Q20

Since all the gates on both the aeration distribution chamber and the RAS distribution chamber are fully opened and the wastewater flows by gravity, it was concluded that the flow was equally divided over the four trains, and therefore the equation for those subsystems are the same.

Based on the previous equations, the flow matrix designed was the following:

Table 3. Flow balance matrix

Code	Description	S1: Distribution chamber	S2: Aeration train 1	S3: Aeration train 2	S4: Aeration train 3	S5: Aeration train 4	S6: Sludge line	S7: Return liquor pump	S8: Clarifiers	S9: RAS distribution chamber	Overall System
Q1	Influent to the distribution chamber	+									+
Q2	Flow to VLR1	-	+								
Q3	Flow to VLR3	-		+							
Q4	Flow to VLR5	-			+						
Q5	Flow to VLR7	-				+					
Q6	Flow from train 1 to clarifiers		-						+		
Q7	Flow from train 2 to clarifiers			-					+		
Q8	Flow from train 3 to clarifiers				-				+		
Q9	Flow from train 4 to clarifiers					-			+		
Q10	Clarifiers to tertiary treatment								-		-
Q11	RAS from clarifiers								-	+	
Q12	RAS to RAS tank 1		+							-	
Q13	RAS to RAS tank 2			+						-	
Q14	RAS to RAS tank 3				+					-	
Q15	RAS to RAS tank 4					+				-	
Q16	WAS to sludge holding tank						+		-		
Q17	Liquor from sludge thickener						-	+			
Q18	Liquor from sludge dewatering						-	+			
Q19	Sludge effluent						-				-
Q20	Liquor from disc filters+ drying beds+ odour control							+			+
Q21	Liquor to distribution chamber	+						-			

5.5.3 Mass balances and SRT calculation

With the results gathered from the sampling campaign, a TP mass balance was carried out in order to check the collected data and the equations proposed. The TP is an endurable parameter as P does not disappear from the system in a gaseous phase, and therefore can be used to check the reliability of the equations and the data. The matrix used for that is the same as the one used for the flows, but in this case the mass load (Concentration*Flow) is applied.

In order to calculate the SRT, the following formula was used:

Equation 1. SRT calculation based on TSS

$$SRT = \frac{V_{AT} * TSS_{AT}}{Q_{WAS} * TSS_{WAS} + Q_{EFF} * TSS_{EFF}}$$

In order to double check the results obtained with the formula above and assess the reliability of the data, SRT was also calculated based on TP, using the formula below:

Equation 2. SRT calculation based on TP

$$SRT = \frac{V_{AT} * (TP_{AT} - TP_{S,eff})}{Q_{INF} * (TP_{INF} - TP_{S,eff})}$$

V_{AT}: Activated sludge tank volume (m3)

TP_{AT}: Total phosphorus concentration in aeration tanks

Q_{inf}: Influent flow

TP_{inf}: Total phosphorus concentration in the influent (kg/m3)

$TP_{s,eff}$: Soluble phosphate concentration in the effluent and aeration tanks (kg/m3)

5.5.4 Characterization of the main flows

In this phase of the study, wastewater influent characterization was carried out by calculating the different fractions of COD and nitrogen in the influent.

For the COD characterization, first the soluble inert fraction was calculated based on the effluent micro-filtered COD and the effluentBOD_5:

Equation 3. Soluble inert COD calculation

$$S_I = 0.9 * COD_{mf,eff} - 1.5 * BOD_{5,eff} \quad \text{(Meijer and Brdjanovic 2012)}$$

Where: $S_I = Soluble\ inert\ COD$

$$COD_{mf,eff} = Micro - filtered\ effluent\ COD$$

$$BOD_{5,eff} = Effluent\ BOD_5$$

Then the soluble readily biodegradable fraction was calculated, according to the following equation:

Equation 4. Soluble biodegradable COD calculation

$$S_s = COD_{mf} - S_I \text{ (Meijer and Brdjanovic 2012)}$$

Where: $S_s = Soluble\ readily\ biodegradable\ COD$

$COD_{mf} = Micro - filtered\ influent\ COD$

The biodegradable COD fraction (Xs + Ss) was calculated according to the BOD course curve, and by the estimation of the K parameter, since only BOD5 was measured and this was not enough to correctly estimate the BOD curve.

The equation used to calculate the particulate biodegradable fraction:

Equation 5. Biodegradable COD

$$X_S + S_S = \frac{BOD_5}{1 - e^{-5k}} * \frac{1}{1 - f_{BOD}}$$

According to Meijer and Brdjanovic (2012), k_{BOD} values range between 0.15 and 0.8 d^{-1}, and f_{BOD} values between 0.1 and 0.2.The assumed values of these parameters are:

$$k_{BOD} = 0.35$$

$$f_{BOD} = 0.15$$

Finally, the inert particulate COD was calculated using the following equation:

Equation 6. Inert particulate COD calculation

$X_I = COD_t - COD_{mf} - X_S$ (Meijer and Brdjanovic 2012)

Where $X_I = Inert\ particulate\ COD$

$COD_t = Influent\ total\ COD$

Regarding the nitrogen fractions, the soluble TKN was calculated according to the following equation:

Equation 7. Soluble TKN calculation

$TKN_s = S_{NH4} + (iN_{SF} * S_F)$ (Meijer and Brdjanovic 2012)

Where: $S_{NH4} = Influent\ ammonia\ in\ mgN/l$

$(iN_{SF} * S_F) = Organic\ soluble\ nitrogen\ fraction$

The particulate TKN fraction was calculated based on the following equation:

Equation 8. Particulate TKN calculation

$TKN_X = TKN_{mf} - TKN_S$ (Meijer and Brdjanovic 2012)

For the organic nitrogen values, assumed fractions (iN) were used according to typical ranges stated by Meijer and Brdjanovic (2012):

Table 4. Typical ranges for organic nitrogen fractions. Meijer and Brdjanovic (2012)

Symbol	Unit	Typical range	Default
iN_{SI}	gN/gCOD	0.01-0.02	0.01
iN_{SA}	gN/gCOD	0	0
iN_{SF}	gN/gCOD	0.02-0.04	0.03
iN_{XI}	gN/gCOD	0.01-0.06	0.03
iN_{XS}	gN/gCOD	0.02-0.06	0.04

5.6 Model calibration

At this stage, the model scheme was fed with the data obtained in previous phases of the project, such as wastewater characterization fractions, sampling results data, DO concentration of aeration tanks, clarifiers efficiency etc.

Model calibration was then carried out, following the protocol by Meijer and Brdjanovic (2012), also based on the STOWA protocol. Protocol scheme can be found in Appendix E.

Firstly, phosphorus was calibrated and with it the SRT was fixed. After that, COD and solids (TSS and VSS) were calibrated, and finally the nitrification and denitrification processes were adjusted.

To do so, some kinetic parameters and influent fractions were adjusted:

- Sludge amount and composition: Xs, Xi,
- Nitrification: μ_A , K_{NH4}
- Denitrification: K_{O2}, N2 producers

Even though, in most modelling protocols the calibration phase is followed by a validation phase to evaluate the model under other data set, in this case it was decided not to carry out the validation stage, due to the inaccuracies on the data gathered. The validation phase intends to evaluate the applicability of the model to different operating conditions, but since the reliability of the data available was not good, it was decided not to perform it, since it would not be a reliable analysis.

5.7 Model scenarios

Based on the main objective of this study, a base scenario was built in which the plant complied with all the effluent standards required by the contract and by the Environmental Agency (KEPA). For this scenario, attention was focused on the operating SRT and the efficiency of the clarifiers, both identified as the main sources of the plant's issues.

Once the minimum conditions for the plants well-functioning were stablished, another three scenarios were proposed, related to other issues faced by the plant.

First scenario intended to reduce the amount of sludge generated daily. The second scenario analysed the situation of the plant with a gradual increase of influent flow, and the last scenario replaced the aerobic digesters of the plant by anaerobic digesters and assessed its consequences in terms of energy saving and sludge production.

Finally, an optimal scenario was proposed based on the joint analysis of the previous scenarios.

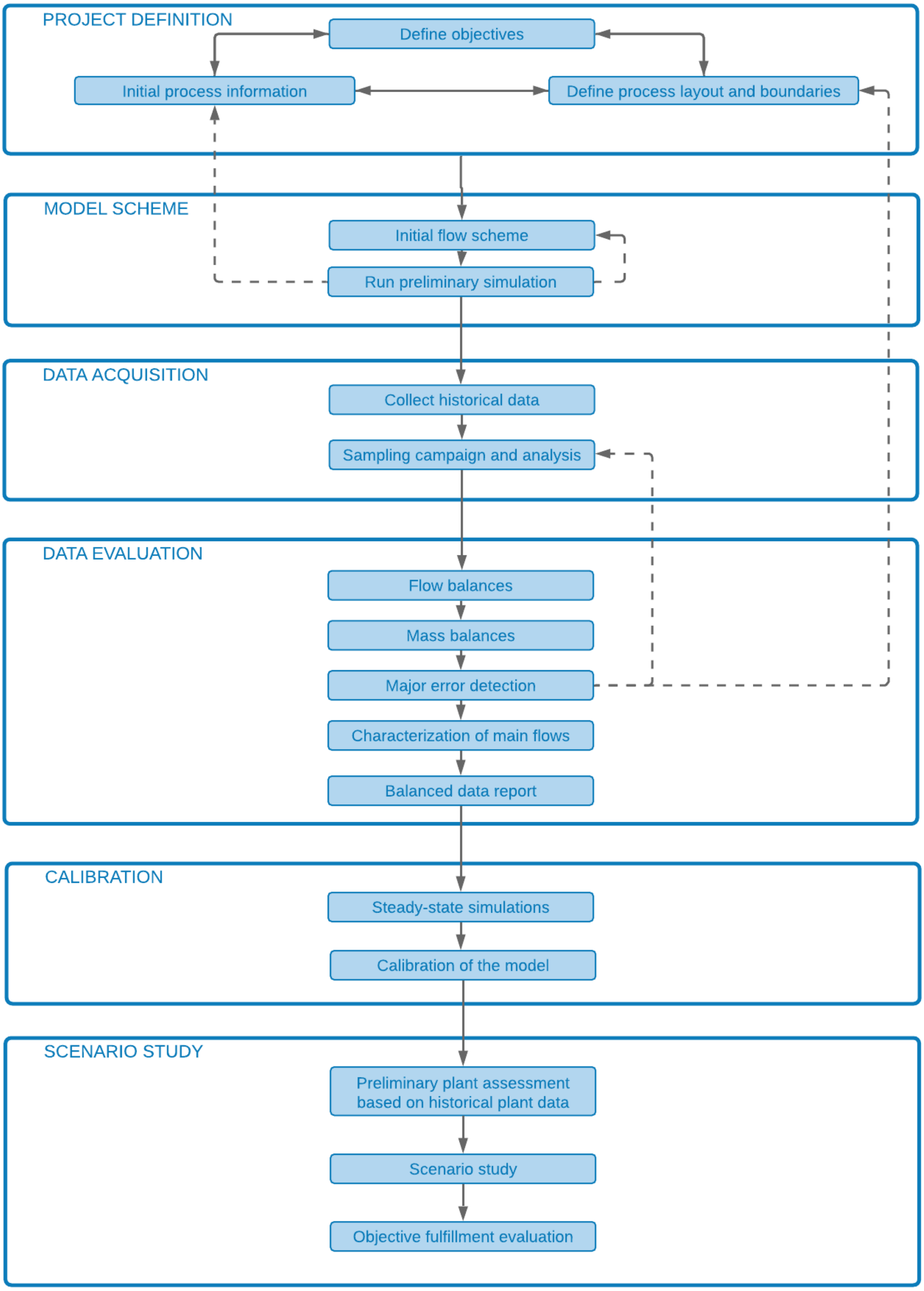


Figure 16 Kabd WWTP proposed modelling protocol

Chapter 6 Results

6.1 Sampling campaign results

The flow measurement campaign was carried out from 19th of November to the 5th December. The measured data is presented in *Table 5*. The highest deviation values were found on the sludge line flow, probably due to continuous control and variation over the waste sludge.

Table 5. Flow measurements

DATE	INFLUENT	RAS FLOW	SAS FLOW	THICKENER SUPERNAT.	AEROBIC DIGESTER OUTLET	DEWATERING SUPERNAT.	RETURN LIQUOR
	Q1	Q11	Q16	Q17		Q18	Q21
	m3/d	m3/d	m3/d	m3/d	m3/d	m3/d	m3/d
19/Nov	185178	119887	1004	612	141	127	2866
20/Nov	181268	119852	999	609	405	364	2624
21/Nov	174601	121348	998	609	386	347	2512
22/Nov	185116	119808	846	516	133	120	2576
23/Nov	170230	120935	535	326	395	356	2157
24/Nov	175347	119935	1097	669	444	400	2673
25/Nov	179088	120334	1014	619	422	380	2689
26/Nov	181987	119749	996	608	460	414	2673
27/Nov	180974	119662	1016	620	232	209	2657
28/Nov	174942	119209	917	559	378	340	2882
29/Nov	178453	118968	858	523	189	170	3091
30/Nov	180247	118478	1043	636	415	374	2785
01/Dec	182910	118655	902	550	334	301	2914
02/Dec	181504	119102	723	441	351	316	2705
03/Dec	180587	118906	847	517	565	509	2882
04/Dec	181415	118317	1013	618	510	459	2898
05/Dec	179572	118129	1009	615	403	363	2721
AVG	**179613**	**119487**	**930**	**568**	**363**	**326**	**2724**
STD DEV	**2.18%**	**0.75%**	**14.87%**	**14.87%**	**33.80%**	**33.80%**	**7.59%**

Due to the malfunction of one of the flow meters in the sludge line, and the impossibility of getting a portable flow meter for those locations, the flows of the thickener supernatant (Q17), dewatering supernatant (Q18) and return liquor (Q21) had to be determined in a different way, as follows.

Sludge thickener supernatant was calculated using the measurements of the flow meter located after the aerobic digesters, and assuming that the flow entering and leaving the aerobic digester were the same. It was concluded that the supernatant of the thickener unit was 61% of the flow entering the unit. This ratio was also double-checked using the following criteria:

- WWTP Process engineer's recommendations
- Historical data from before the flow meter in the thickener broke down. This data analysis can be found on Appendix G.

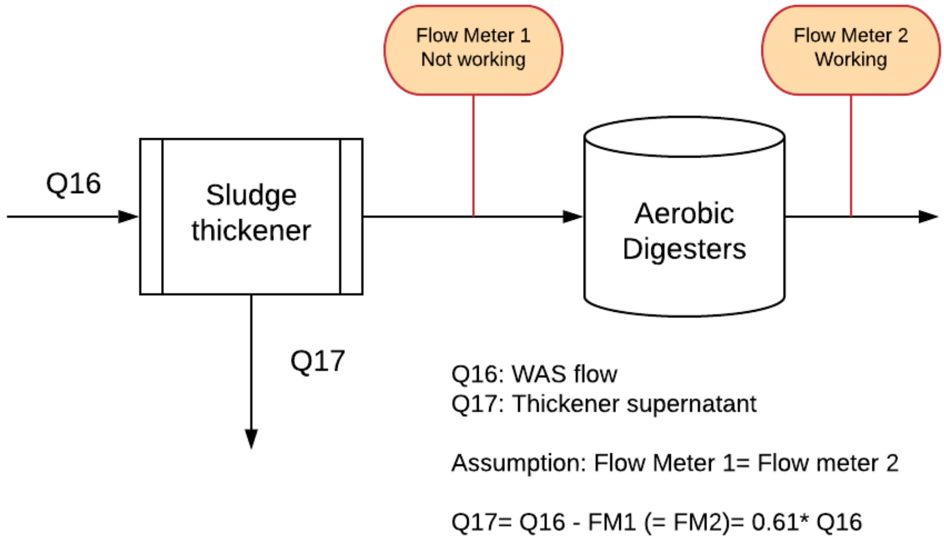


Figure 17 Scheme of the assumption for the thickener supernatant flow

Sludge dewatering supernatant was estimated based on the process engineer's recommendations, assuming the supernatant to be a 90% of the flow entering the dewatering stage.

Return liquor pump flow was estimated based on the pump capacity and running hours, this last data coming from the DCS monitoring system. Efficiency of the pumps was assumed to be 70%. Running hours of the pumps are presented in Appendix F.

Regarding wastewater parameters, *Table 6* below shows the average values of the chemical parameters measured during the 5 day sampling campaign, from the 1st December to the 5th December. Grab samples were taken at 8 am every morning, and composite samples were formed out of three samples taken at 4 pm, 12 am and 8 am each day.

Table 6. Average concentration values during the sampling campaign

Code name	**Units**	**Influent after prelim. Treatm.**	**Effluent Aeration line**	**Second. effluent**	**Concentr. sludge**	**RAS tank**	**Ret. liquor**	**Thick. supernat.**	**Dewat. Supernat.**	**WAS**
		1	2	3	4	5	6	7	8	*
		Comp.	Comp.	Grab	Grab	Comp.	Grab	Grab	Grab	
T	°C	24	25	28		25	27	26	21	
pH	-	7.30	7.1	7.0		7.0	7.2	7.2	7.4	7.02
TCOD	mgCOD/l	531		62			586	1259	113	
CODmf	mgCOD/l	169		25						
CODgf	mgCOD/l	198	21			27				
BOD5	mgBOD/l	273		27						
TSS	mg/l	241	3210	145		7704	485	851	74	7698
VSS	mg/l	117	1774	90		4506	255	445	28	3976
TKN	mgN/l	36	231	28		552	68	101	17	
NH3	mgN/l	29	16	17		18	20	19	9	
NO2	mgN/l	0.0	0.01	0.1		0.0		0.1		
NO3	mgN/l	10	0.16	0.1		0.1		0.1		
TP	mgP/l	4	37	4	1800	86	10	24	1	
TPmf	mgP/l			2						
DO	mgDO/l	0.6	2.8	2.2		0.2		3.8	3.0	0.19
Alk	mgCaCO3/l	340		197						
Ca	mg/l	78		76			80			
Mg	mg/l	22		21			23			
Moisture content	%				86%					

**The values of the WAS sludge were obtained from the WWTP lab routine analyses*

Looking at the results obtained on the secondary effluent samples, it can be concluded that the plant does not comply with the effluent standards specified above in *Section 3.3*, in terms of BOD and TSS.

6.2 Flow mass balance

Initially, the flow mass balance was carried out using the average flows measured during the measurement campaign, but during the model simulation phase, major errors were found, mainly on the aeration tanks regarding MLSS. When checking the data with historical data from 2019 and 2018, it was found out that the WAS flow measured during the sampling campaign was much lower than usual values, causing an increase in the calculated SRT and the accumulation of solids in the aeration tanks. This was adjusted by assuming a misreading on the WAS sludge flow, which was then slightly increased according to average values from historical data in order to fit better in terms of solids in the tanks.

The measured and calculated flows are presented in the following table:

Table 7. Flow values for the overall plant

Code	Description	Flow type	Equation	Value (m3/day)
Q1	Raw influent	Measured		179613
Q2	Influent to Line 1	Calculated	(Q1+Q21)/4	45584
Q3	Influent to Line 2	Calculated	(Q1+Q21)/4	45584
Q4	Influent to Line 3	Calculated	(Q1+Q21)/4	45584
Q5	Influent to Line 4	Calculated	(Q1+Q21)/4	45584
Q6	Effluent Line 1	Calculated	Q2+Q12	75456
Q7	Effluent Line 2	Calculated	Q3+Q13	75456
Q8	Effluent Line 3	Calculated	Q4+Q14	75456
Q9	Effluent Line 4	Calculated	Q5+Q15	75456
Q10	Clarifiers outlet	Calculated	Q6+Q7+Q8+Q9-Q11-Q16	180887
Q11	RAS flow	Measured		119487
Q12	RAS to RAS tank 1	Calculated	Q11/4	29872
Q13	RAS to RAS tank 2	Calculated	Q11/4	29872
Q14	RAS to RAS tank 3	Calculated	Q11/4	29872
Q15	RAS to RAS tank 4	Calculated	Q11/4	29872
Q16	WAS flow	Measured		1450
Q17	Thickener supernatant	Estimated	0.61*Q16	885
Q18	Dewatering supernatant	Estimated	0.9*Sludge flow	509
Q19	Sludge effluent	Calculated	Q16-Q17-Q18	57
Q20	Influent backwash*	Calculated	Q21-Q17-Q18	1330
Q21	Return liquor	Estimated	70% pump efficiency	2724

**Liquor from disc filter backwash, drying beds and odour control system.*

With the results presented in the previous table the following flow mass balance was carried out:

Table 8. Flow mass balance matrix

Descript.	**Distr. Chamb.**	**Aeration 1**	**Aeration 2**	**Aeration 3**	**Aeration 4**	**Sludge line**	**Return liquor**	**Clarifiers**	**RAS distr. Chamber**	**Overall System**
Raw influent	179613									179613
Influent to Line 1	-45584	45584								0
Influent to Line 2	-45584		45584							0
Influent to Line 3	-45584			45584						0
Influent to Line 4	-45584				45584					0
Effluent Line 1		-75456						75456		0
Effluent Line 2			-75456					75456		0
Effluent Line 3				-75456				75456		0
Effluent Line 4					-75456			75456		0
Clarifiers outlet								-180887		-180887
RAS flow								-119487	119487	0
RAS to RAS 1		29872							-29872	0
RAS to RAS 2			29872						-29872	0
RAS to RAS 3				29872					-29872	0
RAS to RAS 4					29872				-29872	0
WAS flow						1450		-1450		0
Thickener supernat.						-885	885			0
Dewat. Supernat.						-509	509			0
Sludge effluent						-57				-57
Influent backwash							1330			1330
Return liquor	2724						-2724			0
Balance	0	0	0	0	0	0	0	0	0	0

6.3 TP mass balance

During modelling phase it was concluded that TP values were not consistent with the rest of the parameters and the flow mass balance values. A TP mass balance was performed too, but the errors on it were significant.

Additionally, when checking lab routine analysis data from the days of the sampling campaign, it was found out that phosphate values measured on the WWTP lab were much higher than the TP values, which were measured by an external lab.

Due to all these inconsistencies it was decided not to use the TP values, therefore not presenting a TP mass balance and relying on flow measurements and the other parameters measurements.

6.4 SRT calculation

Since the TP values were found unreliable, the SRT was only calculated by using the method based on the TSS mass balance, stated above in Equation 1.

Initially, it was calculated by using the flow values measured during the measurement campaign, but during model configuration phase, simulated values did not coincide with measured values in a few parameters throughout the process, and therefore a correction was made in the WAS flow, which also affected the SRT calculation.

The results are shown below:

Table 9. SRT calculation results

	Initial flow measurements	Corrected flow measurements
SRT (TSS)	5.7	5.0

6.5 Wastewater characterization

Wastewater characterization was performed according to the STOWA protocol and influent parameters measured during the sampling campaign. Organic matter fractions were calculated in the first place, followed by nitrogen fractions. The latter ones were calculated by assuming typical ranges for organic nitrogen fractions, as discussed in the previous section 5.5.4.

The following table shows the influent data that are required and used for the wastewater characterization:

Table 10. Parameters used for wastewater characterization

		Value	**Unit**	**Load**	**Unit**
Influent parameters					
Flow	Q			179613	m3/d
Total COD	TCOD	531	mgCOD/l	95375	kgCOD/d
Micro-filtered COD	CODmf	169	mgCOD/l	30355	kgCOD/d
BOD5	BOD5	272	mgBOD/l	48855	kgBOD/d
Total Suspended Solids	TSS	241	mg/l	43287	kg/d
Volatile Suspended Solids	VSS	117	mg/l	21015	kg/d
Total Kjeldahl Nitrogen	TKN	36	mgN/l	6466	kgN/d
Ammonia	NH3	29	mgN/l	5209	kgN/d
Nitrate	NO3	10	mgN/l	1796	kgN/d
Nitrite	NO2	0	mgN/l	0	kgN/d
Effluent parameters					
Flow	Q			180877	m3/d
Micro-filtered COD	CODmf	25	mgCOD/l	4521	kgCOD/d
BOD5	BOD5	4*	mgBOD/l	755	kgBOD/d

**BOD5 value in the effluent was too high (due to a high content of particulate BOD in the effluent) and therefore a standard value was used for the purpose of the characterization.*

For the calculation or organic matter fractions, the Equation 3 Equation 4, Equation 5 and Equation 6 were used.

For the calculation of nitrogen fractions, the Equation 7 and Equation 8 were used.

The characterization reached is shown in the following table:

Table 11. Wastewater characterization

		Value	Unit
Organic matter fractions			
Soluble inert COD	Si	16.5	mgCOD/l
Soluble biodegradable COD	Ss	152.5	mgCOD/l
Particulate biodegradable COD	Xs	235	mgCOD/l
Particulate inert COD	Xi	127	mgBOD/l
Nitrogen fractions			
Soluble organic nitrogen	iNsf*Sf	3	mgN/l
Particulate organic nitrogen	iNxs*Xs+ iNxi*Xi	5	mgN/l
Soluble TKN	TKNs	32	mgN/l
Particulate TKN	TKNx	5	mgN/l

The wastewater fractionation of the influent was calculated based on the previous characterization, and is shown in the table below:

Table 12. Wastewater fractions

Name	Default	Value
Fbs - Readily biodegradable (including Acetate) [gCOD/g of total COD]	0.16	0.287
Fac - Acetate [gCOD/g of readily biodegradable COD]	0.15	0.15*
Fxsp - Non-colloidal slowly biodegradable [gCOD/g of slowly degradable COD]	0.75	0.44
Fus - Unbiodegradable soluble [gCOD/g of total COD]	0.05	0.03
Fup - Unbiodegradable particulate [gCOD/g of total COD]	0.13	0.24
Fcel - Cellulose fraction of unbiodegradable particulate [gCOD/gCOD]	0.5	0.5*
Fna - Ammonia [gNH3-N/gTKN]	0.66	0.81
Fnox - Particulate organic nitrogen [gN/g Organic N]	0.5	0.62
Fnus - Soluble unbiodegradable TKN [gN/gTKN]	0.02	0.02*
FupN - N:COD ratio for unbiodegradable part. COD [gN/gCOD]	0.07	0.07*
Fpo4 - Phosphate [gPO4-P/gTP]	0.5	0.5*
FupP - P:COD ratio for unbiodegradable part. COD [gP/gCOD]	0.022	0.022*

**Default values used*

6.6 Model configuration

The initial plant model was constructed using the BioWin software, with the following process flow:

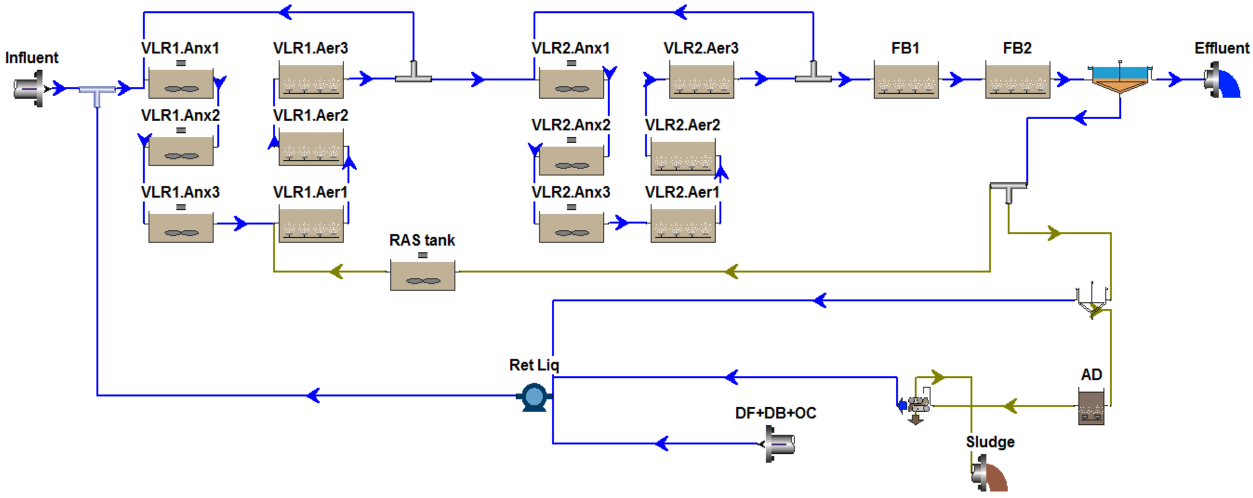


Figure 18 Plant model configuration (one treatment line)

The model represents one of the four lines existing in the plant. Flows were divided and the elements of the configuration that are shared among the four lines (clarifiers, sludge line) were divided by four in terms of size.

The model was updated with flow measurements, influent parameters characterization, filter backwash parameters characterization, DO concentration in the aeration tanks, as well as all the physical characteristics of the reactors and other elements.

In the case of the clarifiers, they were modelled as an ideal clarifier, and the efficiency was estimated based on the parameter measurements in the effluent.

The model was simulated using the default kinetic and stoichiometric values provided by BioWin, and the comparison between the measured values and the values provided by the model before calibration are shown in the following table.

Table 13. Comparison of simulated and measured values before calibration

			INFLUENT	AERATION	RAS TANK	EFFLUENT
COD	**mg/l**	BioWin	531			194
		Measured value	531			62
		Deviation	0.0%			-212.9%
COD filtered	**mg/l**	BioWin		17	17	
		Measured value		21	27	
		Deviation		19.0%	37.0%	
BOD5	**mg/l**	BioWin	242			35
		Measured value	273			27
		Deviation	11.4%			-29.6%
TSS	**mg/l**	BioWin	278	4977	12074	215
		Measured value	241	3170	7704	145
		Deviation	-15.4%	-57.0%	-56.7%	-48.3%
VSS	**mg/l**	BioWin	153	2797	6760	121
		Measured value	117	1908	4506	90
		Deviation	-30.8%	-46.6%	-50.0%	-34.4%
TKN	**mg/l**	BioWin	36	181	436	9
		Measured value	36	231	552	28
		Deviation	0.0%	21.6%	21.0%	67.9%
Ammonia	**mg/l**	BioWin	29	0.02	0.12	0.07
		Measured value	29	16	18	17
		Deviation	0.0%	99.9%	99.3%	99.6%
Nitrate	**mg/l**	BioWin	10	12.7	10.5	12.7
		Measured value	10	0.16	0.1	0.1
		Deviation	0.0%	-7837%	-10400%	-12600%

The results in the table show some deviation in almost all the parameters, especially on nitrogen related parameters. TSS and VSS values in all the locations, as well as COD and BOD values in the effluent seem quite high compared to measurements.

Regarding nitrogen, the table shows high differences. In the BioWin model, nitrification seems to be happening completely whereas in reality it only happens partially. According to Henze et al. (2008), under normal behaviour conditions of the bacteria and the existing average temperature of 25°, the minimum SRT for nitrification is:

Equation 9. Minimum SRT for nitrification

$$SRTm = \frac{1}{\mu_{A,25} * (1 - f_{xt}) - b_{A,25}} = \frac{1}{0.63 * (1 - 0.25) - 0.045} = 2.3\ days$$

$$\mu_{A,25} = \mu_{A,20} * \theta_n{}^{(25-20)} = 0.35 * 1.12^5 = 0.63$$

$$b_{A,25} = b_{A,20} * \theta_b{}^{(25-20)} = 0.04 * 1.029^5 = 0.045$$

$$f_{xt} = Anoxic\ fraction\ of\ the\ system = 0.25$$

The calculated SRT is longer than this, hence the system should nitrify completely, and this not being so can be related to an inhibition of the nitrifying bacteria caused by an external agent.

6.7 Calibration

The model was calibrated following the protocol by Meijer and Brdjanovic (2012), which is based on the STOWA protocol. As stated before, measured values of TP were found unreliable and therefore the model was not calibrated for P.

According to the previously calculated value and in order to adjust the solids content, SRT was fixed to 4.7 days.

Then X_I and S_I parameters were adjusted to fit the solids and COD concentrations as much as possible, even though a close fit on those parameters was not possible.

After that, the nitrification process was calibrated to match the inhibited performance of the nitrifying bacteria. To do so, kinetic parameters such as Maximum specific growth rate and Substrate (NH4) half saturation constant were adjusted.

To calibrate denitrification, the DO half saturation constant and the Denitrification N2 producers were adjusted.

The kinetic factors adjusted during the process are shown in the following table:

Table 14. Kinetic factors adjusted for calibration

Name	**Parameter**	**Default**	**Value**	**Unit**
Maximum specific grow rate for autotrophic biomass	μ_A	0.9	0.58	1/d
Substrate (NH4) half saturation constant	K_{NH4}	0.7	1.3	mgN/l
Aerobic decay rate	b_A	0.17	0.57	1/d
Denitrification DO half saturation constant	K_{O2}	0.1	0.6	mg/l
Denitrification N2 producers		0.5	1	-

The calibrated fractions for the influent wastewater are the following:

Table 15. Calibrated wastewater fractions

Name	**Default**	**Value**
Fbs - Readily biodegradable (including Acetate) [gCOD/g of total COD]	0.16	0.54
Fac - Acetate [gCOD/g of readily biodegradable COD]	0.15	0.13
Fxsp - Non-colloidal slowly biodegradable [gCOD/g of slowly degradable COD]	0.75	0.8
Fus - Unbiodegradable soluble [gCOD/g of total COD]	0.05	0.033
Fup - Unbiodegradable particulate [gCOD/g of total COD]	0.13	0.05
Fcel - Cellulose fraction of unbiodegradable particulate [gCOD/gCOD]	0.5	0.5*
Fna - Ammonia [gNH3-N/gTKN]	0.66	0.806
Fnox - Particulate organic nitrogen [gN/g Organic N]	0.5	0.5*
Fnus - Soluble unbiodegradable TKN [gN/gTKN]	0.02	0.02*
FupN - N:COD ratio for unbiodegradable part. COD [gN/gCOD]	0.07	0.22
Fpo4 - Phosphate [gPO4-P/gTP]	0.5	0.5*
FupP - P:COD ratio for unbiodegradable part. COD [gP/gCOD]	0.022	0.022*
Fsr - Reduced sulfur [H2S] [gS/gS]	0.15	0.15*

**Default values used*

The initial COD fractions of the influent and the final calibrated ones are shown in the following table:

Table 16. Initially calculated and calibrated COD characterization fractions of the influent

Description	**Parameter**	**Calculated**	**Calibrated**	**Unit**
Soluble biodegradable fraction	Ss	152.5	324.5	mgCOD/l
Soluble inert fraction	Si	16.5	17.5	mgCOD/l
Particulate biodegradable fraction	Xs	235.5	151	mgCOD/l
Particulate inert fraction	Xi	127	38	mgCOD/l

The big differences on soluble and particulate fractions could be related to improper filtering of the sample, during the sampling campaign. In regular lab routine analyses either 0.1 µm or 1.6 µm filters were used, whereas in the case of COD micro-filtered samples in this study, 0.45 µm were required.

The comparison between measured values and estimated values by the model is shown in the following table:

Table 17. Comparison of model and measured values after calibration

			INFLUENT	FB2 tank	RAS TANK	EFFLUENT
COD	**mg/l**	Biowin	531			149
		Measurement	531			62
		Deviation	0.0%			140.3%
COD filtered	**mg/l**	Biowin		20	21	
		Measurement		21	27	
		Deviation		-4.8%	-22.2%	
BOD5	**mg/l**	Biowin	317			46
		Measurement	273			27
		Deviation	16.1%			70.4%
TSS	**mg/l**	Biowin	242	4048	9875	188
		Measurement	241	3210	7704	145
		Deviation	0.4%	26.1%	28.2%	29.7%
VSS	**mg/l**	Biowin	118	1950	4678	91
		Measurement	117	1908	4506	90
		Deviation	0.9%	2.2%	3.8%	1.1%
TKN	**mg/l**	Biowin	36	229	529	28
		Measurement	36	231	552	28
		Deviation	0.0%	-0.9%	-4.2%	0.0%
Ammonia	**mg/l**	Biowin	29	17	20	17
		Measurement	29	16	18	17
		Deviation	0.0%	6.3%	11.1%	0.0%
Nitrate	**mg/l**	Biowin	10	0.03	0	0.03
		Measurement	10	0.16	0.1	0.1
		Deviation	0.0%	-81.3%	-100.0%	-70.0%
Nitrite	**mg/l**	Biowin	0	0.12	0	0.12
		Measurement	0	0.01	0.1	0.1
		Deviation	0.0%	1100.%	-100%	20.%

According to the table, some of the measured values seem inconsistent with the rest, such as the COD and BOD5 values in the effluent, and therefore it can be assumed that there were some unreliable measurements during the sampling campaign. Further discussion about it can be found in *Section 7.2.*

6.8 Scenario study

6.8.1 SC0: Base scenario to comply with the effluent standards

The main objective of this study is to assess the performance of the treatment plant and come up with a solution to comply with the effluent standards required by the KEPA and the contract of the WWTP. After the analysis of the parameters measured in the effluent during the sampling campaign, two main problems were found affecting the performance:

First, the secondary settling tanks are not performing as required, and this causes very high TSS, VSS, COD and BOD concentrations in the effluent.

Second, probably due to the inhibition caused by a toxic compound that reduces the nitrifying capacity, ammonia in the effluent is higher than the maximum standard.

The current situation measured during the sampling campaign compared to the effluent standards is shown in the following table:

Table 18. Effluent measured values and WWTP standards

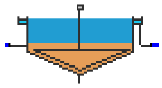
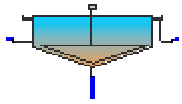
	Unit	**Effluent**	**Standards**
COD	mg/l	149*	100
BOD5	mg/l	27	10
TSS	mg/l	145	15
NH3	mg/l	17	15
NO3	mg/l	0.1	20

** Since the measured effluent COD value from the sampling campaign is considered unreliable, the value used for this comparison is the one obtained from the calibrated model.*

In order to evaluate the capacity of the clarifiers according to the current influent flow and load, the same model configuration was built, but only changing the ideal clarifier of the initial configuration for a model clarifier. The model clarifier is based on a 1D flux model. Under the exact same influent values and clarifier dimensions, the model showed favourable results that complied with the standards in terms of solids, BOD and COD in the effluent. This proves that the clarifiers' capacity has not been exceeded, and therefore the current inefficiency is caused by some other cause, which could be related to maintenance, bulking sludge or other issues. This could not be proven using the ideal clarifier from the model configuration, since in this one the efficiency is manually adjusted and has nothing to do with the clarifier's dimension.

The average SVI (Sludge Volume Index) of the secondary sludge during the sampling campaign was found to be SVI=305 ml/g, which is a quite high value that can be an indicator of bulking sludge.

Table 19. Comparison between calculated values in the effluent for ideal clarifier and model clarifier

	Unit	Ideal clarifier	Model clarifier
			
TSS	mg/l	187	8.5
VSS	mg/l	89	4
COD	mg/l	147	26
BOD	mg/l	44	3

Coming back to the base scenario model, different efficiency values for the clarifier were tested, starting from the initial value of 97.2% used for the calibration of the model. It was concluded that the minimum efficiency required to comply with the standards is 99.8%.

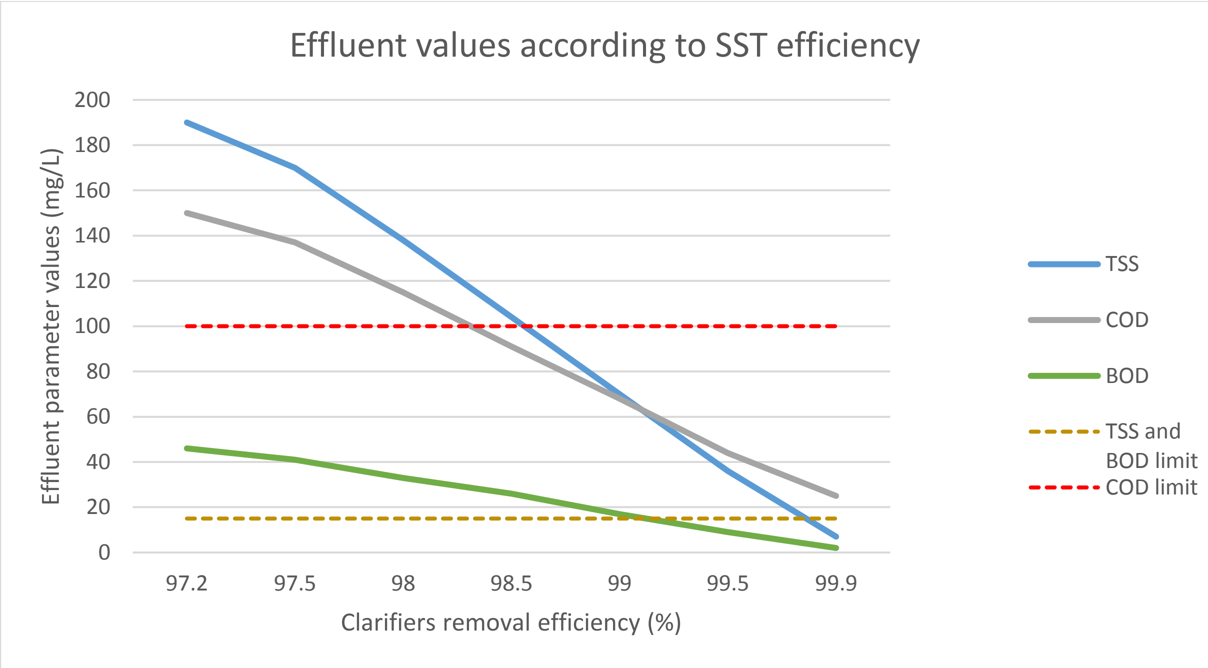


Figure 19. Relationship between effluent TSS, BOD and COD values and clarifier removal efficiency

Regarding the excess of ammonia, the SRT was gradually increased starting in the 4.7 days from the calibrated model. It was concluded that ammonia values in the effluent are acceptable from a SRT of 6 days onwards.

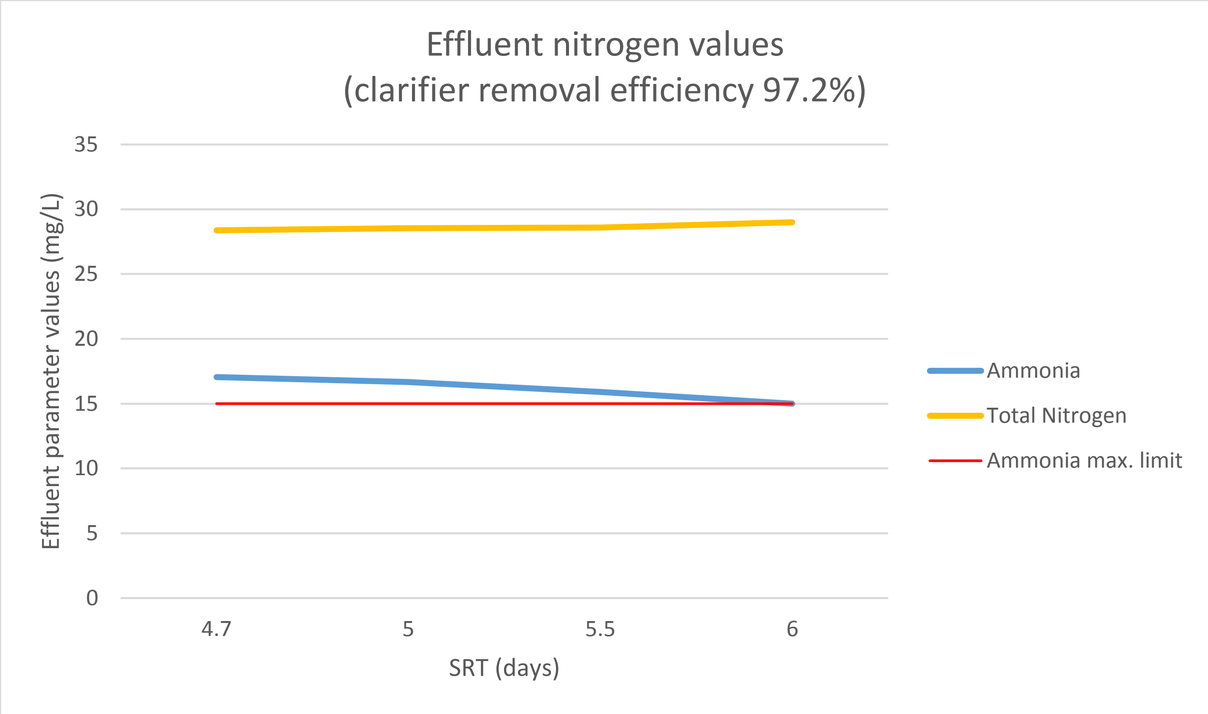


Figure 20. Effluent nitrogen values in relation with SRT

Lastly, the DO concentration measured on the fine bubble diffuser tank was DO= 2.9 mg/l, which is a little bit high and can be optimized to DO=2 mg/l for energy saving. This adjustment did not cause any changes in the effluent quality values.

Merging both analysis, ammonia and clarifiers' efficiency, the base scenario proposed that complies with the effluent standards is shown in *Table 20*.

The efficiency of the clarifiers assumed in this scenario is based on the assumption that the problems faced by the clarifiers are fixed (maintenance and/or bulking sludge) and that they are working properly according to their design capacity.

Table 20. Parameters for Base Scenario (SC0)

	Unit	**Value**
SRT	days	6
Clarifiers removal efficiency	%	99.9
WAS (per treatment line)	m3/d	892
DO in the FB aeration tanks	mg/l	2
Effluent results		
COD	mg/l	26.3
BOD5	mg/l	2.5
TSS	mg/l	8.9
Ammonia	mgN/l	13.8

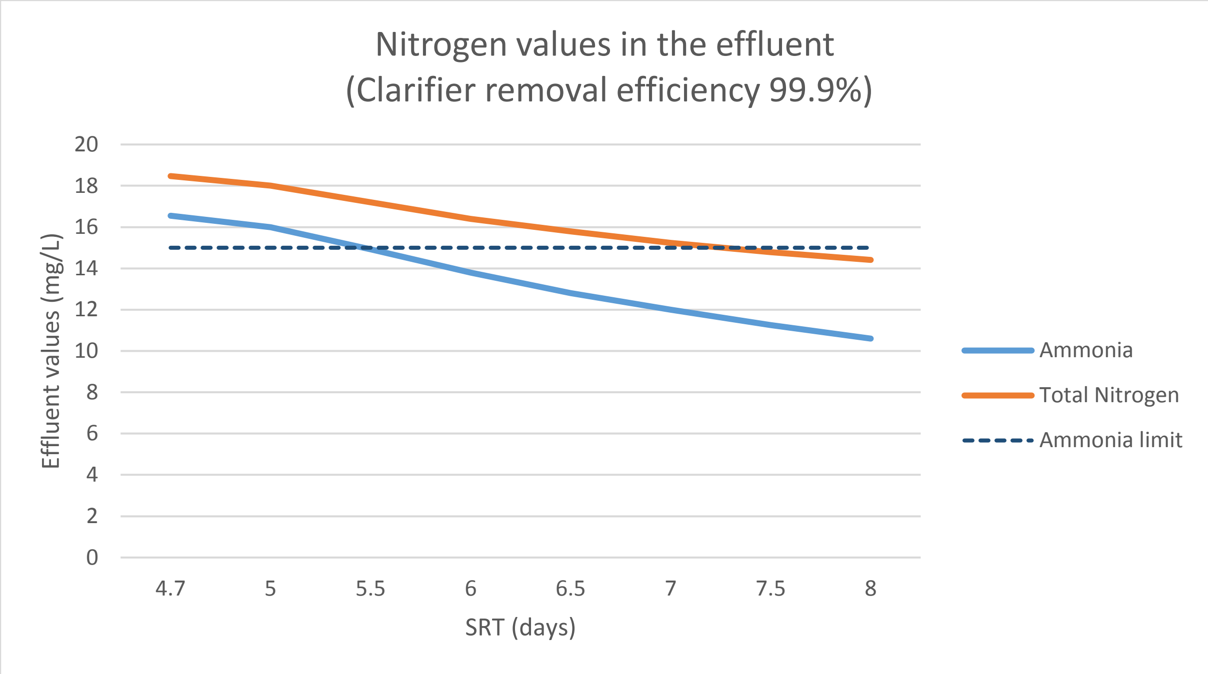


Figure 21. Relationship between ammonia and TN values in the effluent and SRT

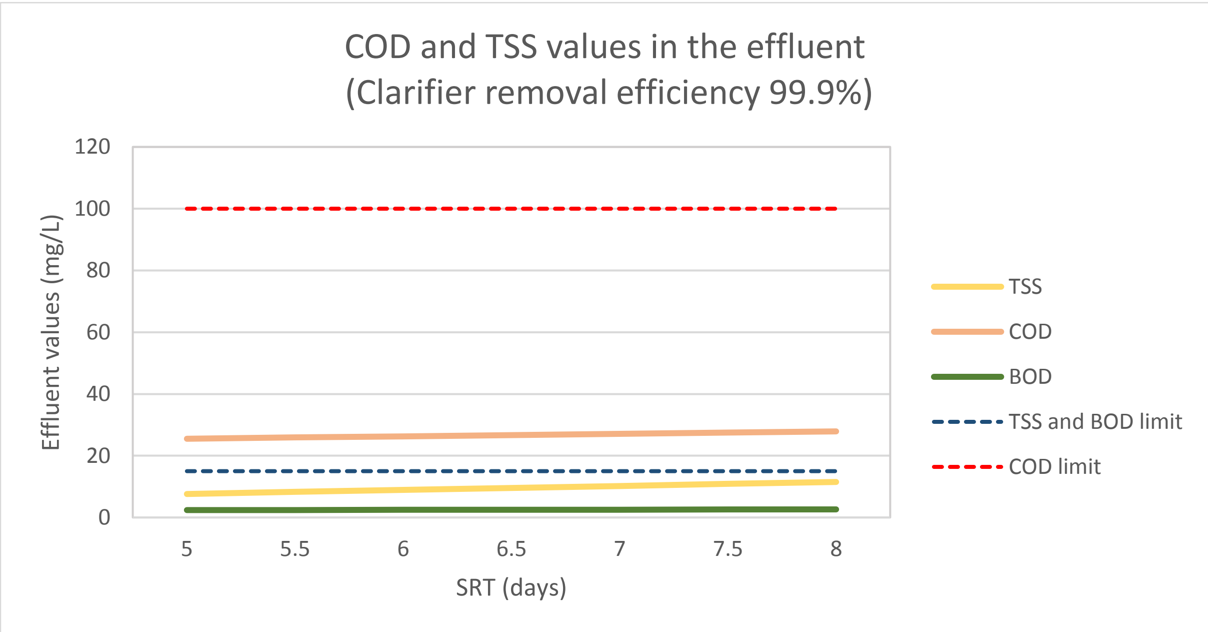


Figure 22. Relationship between COD, BOD and TSS values in the effluent and SRT

6.8.2 SC1: Reduction of the amount of sludge generated

One of the issues faced by the Kabd WWTP is the daily generated waste sludge, which is nowadays being piled up inside the WWTP's property, close to the drying beds and with no control.

An increase in the SRT will reduce the amount of WAS and therefore also the final sludge. However, the increase in SRT causes also an increase in the MLSS of the aeration tanks, which, according to literature, should normally be in the range of 2000 mg/l to 5000 mg/l, and can generate settleability issues and a deterioration of the oxygen transfer to biomass on high concentrations (Henze et al 2008, Krampe and Krauth 2003).

Besides that, the higher the SRT the higher the concentration of the final sludge, as well as being more stabilized and easier to dewater, which is an advantage for some of the options for final use, such as incineration or even sometimes landfill disposal (Andreoli *et al.* 2007)

The following table and graph show the increase of MLSS and decrease of sludge produced in terms of flow and mass, in relation with the increase in SRT. According to this, the optimum scenario for sludge reduction without compromising the MLSS concentration, is a maximum SRT of 6 or 6.5 days.

Table 21. Behaviour of MLSS and sludge related parameters with SRT increase

SRT	**MLSS FB2 (mg/l)**	**WAS* (m3/d)**	**Flow sludge* (m3/d)**	**Sludge mass* (kg/day)**
4.7	4296	1167	45.53	9560
5	4559	1087	42.42	9462
5.5	4991	976	38.07	9318
6	5380	891	34.78	9204
6.5	5765	820	32	9103
7	6147	760	29.6	9013
7.5	6527	706	27.5	8932
8	6904	660	25.76	8859
8.5	7279	620	24.17	8792
9	7652	583	22.76	8729

**Per treatment line*

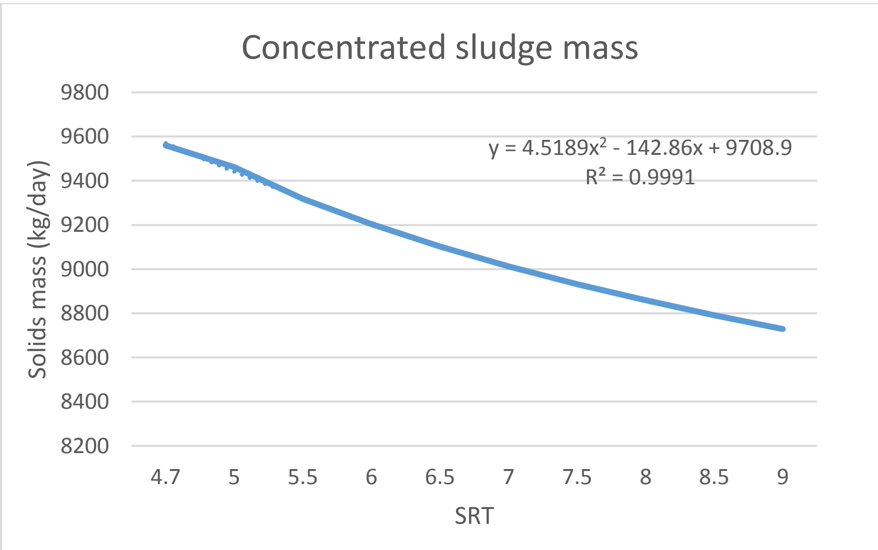


Figure 23. Behaviour of concentrated sludge (in terms of mass) in relation with SRT

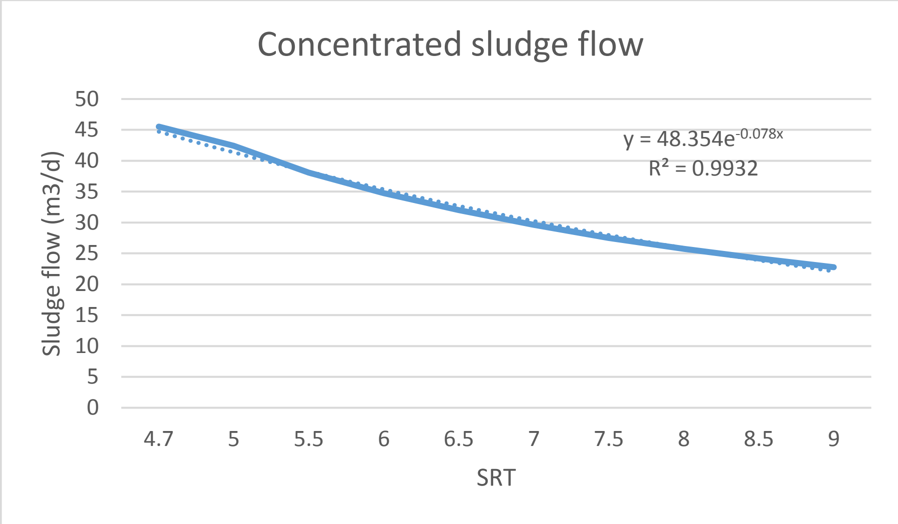


Figure 24. Behaviour of concentrated sludge (in terms of flow) in relation with SRT

In this scenario, with a selected SRT of 6 days, the sludge in terms of flow is reduced by 23.6% compared to the current situation , while in terms of mass is reduced by 9.6%.

6.8.3 SC2: Simulation of an increase in the influent flow

The influent flow arriving to Kabd WWTP is increasing yearly, and is currently almost reaching the plant's design capacity. This scenario study shows the effect of a gradual increase of the influent flow on the behaviour of the plant, assuming the influent parameter concentrations stay the same.

In order to do so, the ideal clarifier of the model was replaced by a model clarifier, so it shows when the clarifier collapses due to its capacity being exceeded.

The plant was designed for a maximum capacity of 180,000 m3/day, with a peak flow of 270,000 m3/day.

The scenario was run for the current situation where 4 treatment lines are available, but also for a future upgrade of the plant that has already been planned, with two more identical treatment lines. This upgrade will increase the capacity of the plant by 50%. Both situations are run with an SRT of 6 days, according to the base scenario proposed earlier.

The results for the current situation with four treatment lines are shown below:

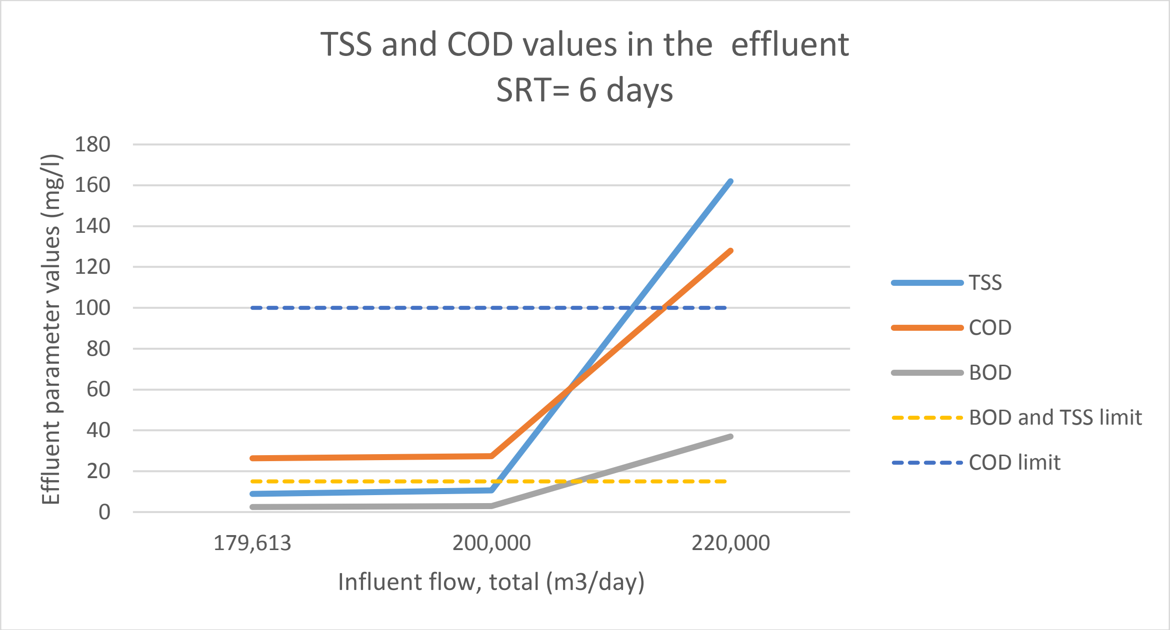


Figure 25. Relationship between effluent parameters and influent flow

For a SRT of 6 days, the plant can perform well until approximately Q= 200,000 m3/day, where effluent parameter values increase sharply. The plant is designed to handle a flow of 270,000 m3/day hydraulically speaking, but in terms of load the treatment will most likely collapse when a flow of 200,000 m3/day.

The results for the upgrade scenario with 6 treatment lines are shown below:

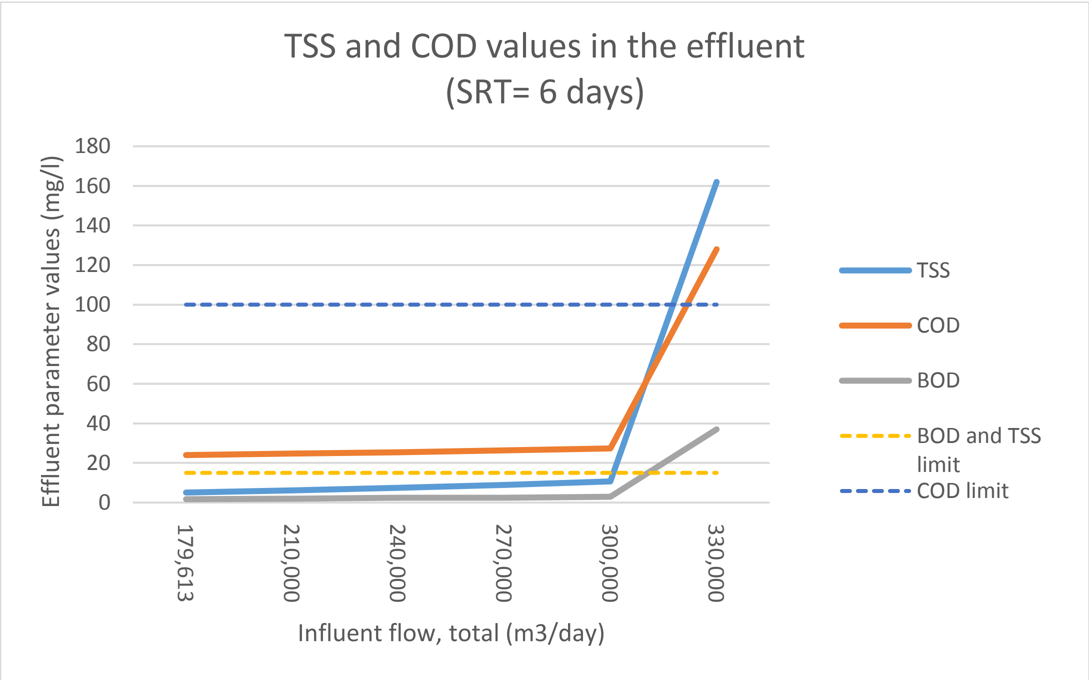


Figure 26. Relationship between effluent parameters and influent flow

With the upgrade of the new treatment lines, in the case of a SRT of 6 days, the influent flow can be increased up to approximately Q=300,000 m3/day, when effluent values highly exceed the standards.

For further increase of the influent flow, SRT would have to be decreased in order to reduce the amount of solids in the system, so the clarifiers are able to perform according the standards required for the effluent in terms of solids and organic matter.

6.8.4 SC3: Introducing an anaerobic digester in the system

The last scenario of this study intends to assess the possibility of replacing the aerobic digesters of the system by anaerobic digesters in order to recover energy and save costs by the production of biogas.

The design calculations of the anaerobic digester are based on the base scenario's (SC0) WAS flow and thickener efficiency, with a safety factor of 1.2.

Table 22. Design parameters of the anaerobic digesters

Parameter	Value	Unit
Maximum inlet flow (per treatment line)	424	m3/d
Minimum HRT	15	days
Safety factor	1.2	-
Volume (per treatment line)	7631	m3
Total volume	30,524	m3
Height	5.25	m
Surface area (per treatment line)	1453	m2
Total surface area	5812	m2

Based on the SC0 scenario with and SRT of 6 days, the production of biogas is 4732 m3/d, while the production of CH_4 is 3507 m3/day, for the whole plant (the four treatment lines).

The following table and graph show how the production of methane is affected by the variation of the SRT. An increase in the SRT reduces slightly the CH_4 production.

Table 23. Biogas and methane production according to SRT

SRT	WAS (m3/day)	Biogas (m3/d)	CH4 (%)	CH4 (m3/day)
4.7	4660	5257.92	75.58%	3973.9
5	4344	5129.28	74.45%	3818.7
5.5	3940	4940.16	74.26%	3668.6
6	3568	4735.68	74.05%	3506.8
6.5	3280	4559.04	73.87%	3367.8
7	3036	4391.04	73.70%	3236.2
7.5	2824	4231.68	73.55%	3112.4
8	2640	4081.92	73.40%	2996.1
8.5	2476	3940.8	73.25%	2886.6
9	2332	3807.36	73.12%	2783.9
9.5	2204	3682.56	72.99%	2687.9
10	2088	3568.32	72.86%	2599.9

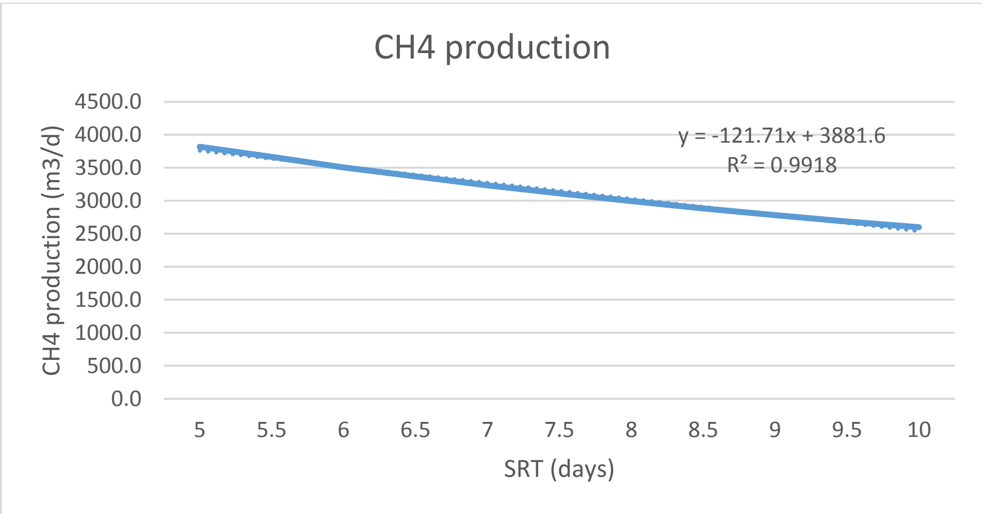


Figure 27. CH4 production according to SRT

There is no information available to do an estimation of the annual costs of the plant in terms of energy. According to the methane generated and assuming a conversion factor of 10.95 kWh/m3, the energy recovery is calculated to be 38,399 kWh/d. Assuming the market price of the electricity in Kuwait for industries is 0.049 US dollars, an approximated value of 1882 USD/day can be estimated in daily energy savings, once the investment costs have been paid back.

6.8.5 Optimal scenario

The optimal scenario is developed based on the previous scenarios, aiming for the optimum results, but always complying with the minimum standards required in the effluent.

As stated in previous sections, the efficiency of the clarifiers assumed in this scenario is based on the assumption that the problems faced by the clarifiers are fixed (maintenance and/or bulking sludge) and that they are working properly according to their design capacity.

According to that, the optimal model is selected as follows:

Table 24. Optimal scenario parameters

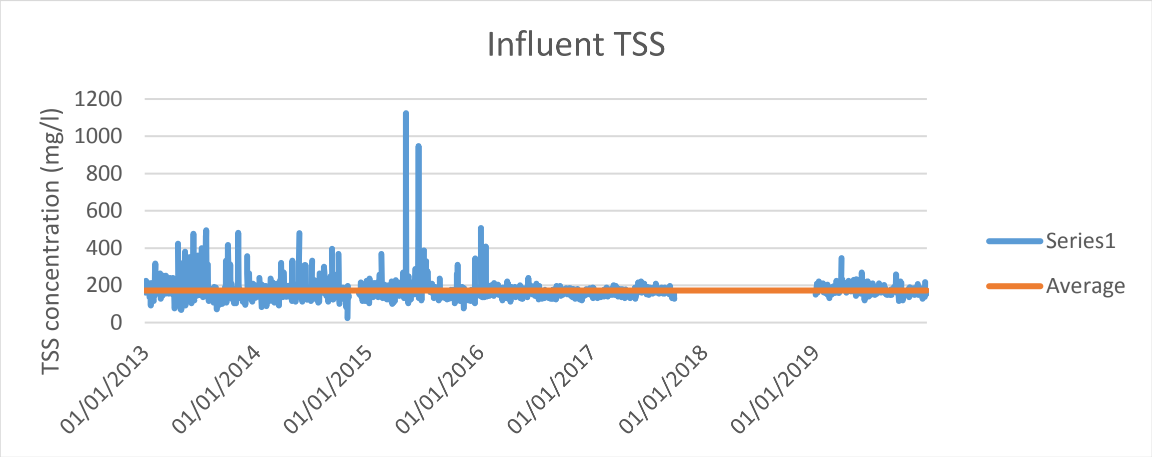
	Unit	**Value**
SRT	days	6
Clarifiers removal efficiency	%	99.9
WAS	m3/d	3568
Concentrated sludge flow	m3/d	139.1
Concentrated sludge mass	kg/d	36,816
CH4 production	m3/d	3507
Effluent results		
COD	mg/l	26.2
BOD5	mg/l	2.5
TSS	mg/l	9.0
Ammonia	mgN/l	13.8

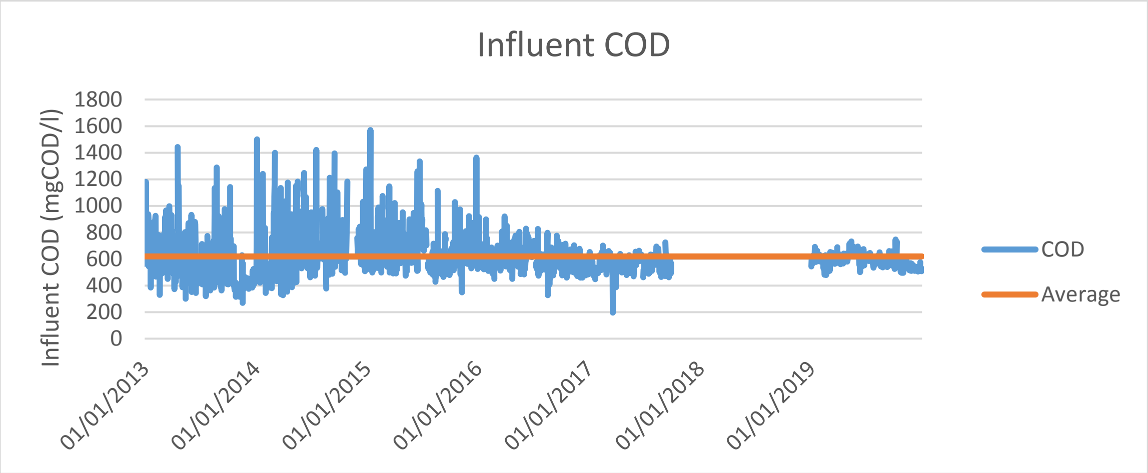
In this scenario, the influent flow can increase by 11% until the plant reaches its maximum performance capacity, while in case there is an upgrade of the WWTP with another two lines, influent flow can increase by 67% of the current flow.

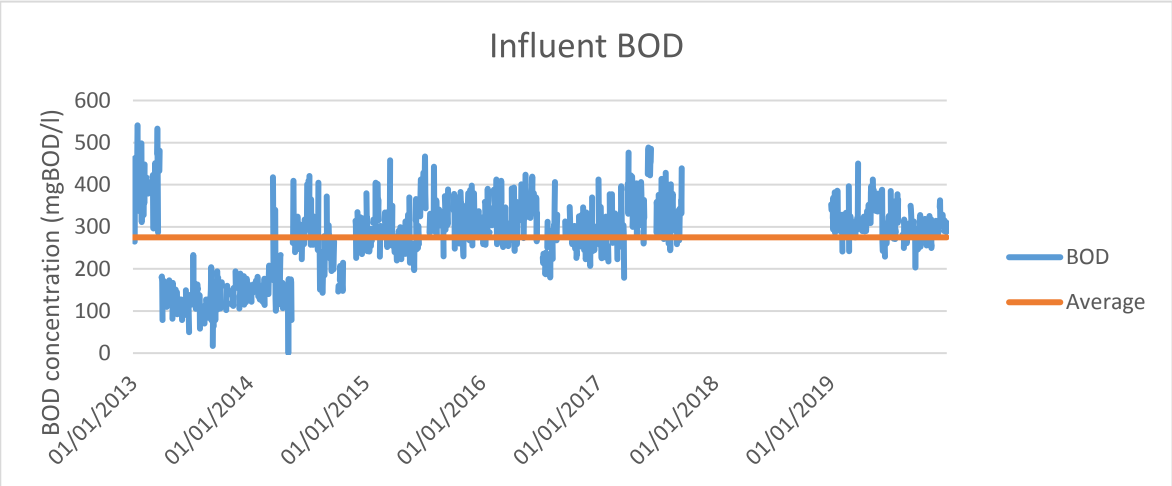
6.9 Historical data analysis

The following sections presents the historical data gathered from the lab routine analysis charts and the flow measurement charts of the DCS. The data records go back to 2013, but there is no information available on 2018 regarding influent parameters concentration.

The results show that influent TSS have mostly maintained on a range between 150 to 400 mg/l, with a few peaks. ISS shows a similar trend and an average value of 118 mg/l. COD and BOD varied more in the initial years, and show more constant values over 2019. These parameters are presented in *Figure 28*, whereas influent temperature, pH, ammonia, conductivity and VSS can be found in Appendix G.


Influent TSS
TSS concentration (mg/l)
1200
1000
800
600
400
200
0
01/01/2013
01/01/2014
01/01/2015
01/01/2016
01/01/2017
01/01/2018
01/01/2019
Series1
Average


Influent COD
Influent COD (mgCOD/l)
1800
1600
1400
1200
1000
800
600
400
200
0
01/01/2013
01/01/2014
01/01/2015
01/01/2016
01/01/2017
01/01/2018
01/01/2019
COD
Average


Influent BOD
BOD concentration (mgBOD/l)
600
500
400
300
200
100
0
01/01/2013
01/01/2014
01/01/2015
01/01/2016
01/01/2017
01/01/2018
01/01/2019
BOD
Average

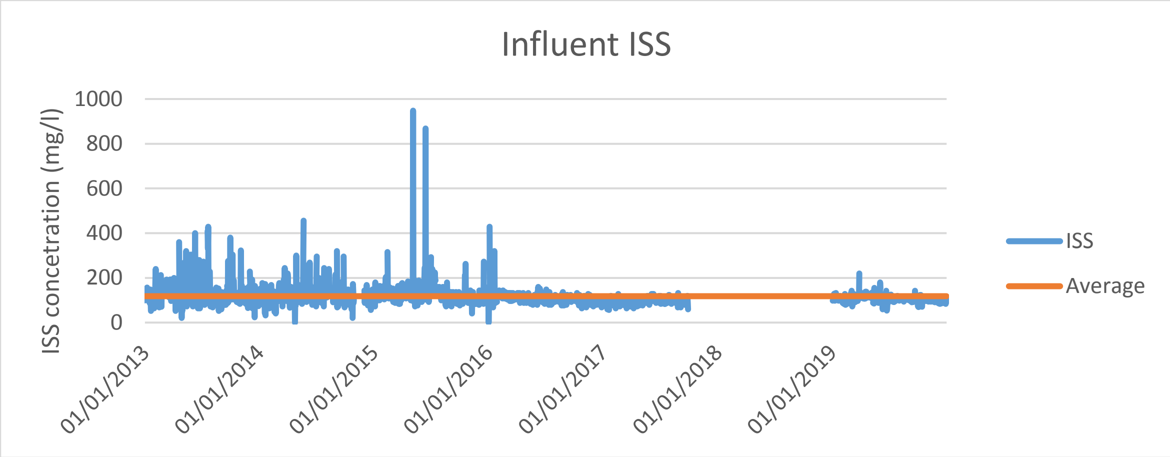


Figure 28. Influent parameters

Regarding the aerated reactors the following figure shows the MLSS and DO concentration. The former shows an increase until 2017, reaching peak values of 8000 mg/l, whereas in 2019 values range between 2500 and 6500 mg/l. The DO concentration chart shows some peaks at the beginning which can probably be related to misreading, but overall the concentration is maintained between 1 and 4 mg/l. See *Figure 29*.

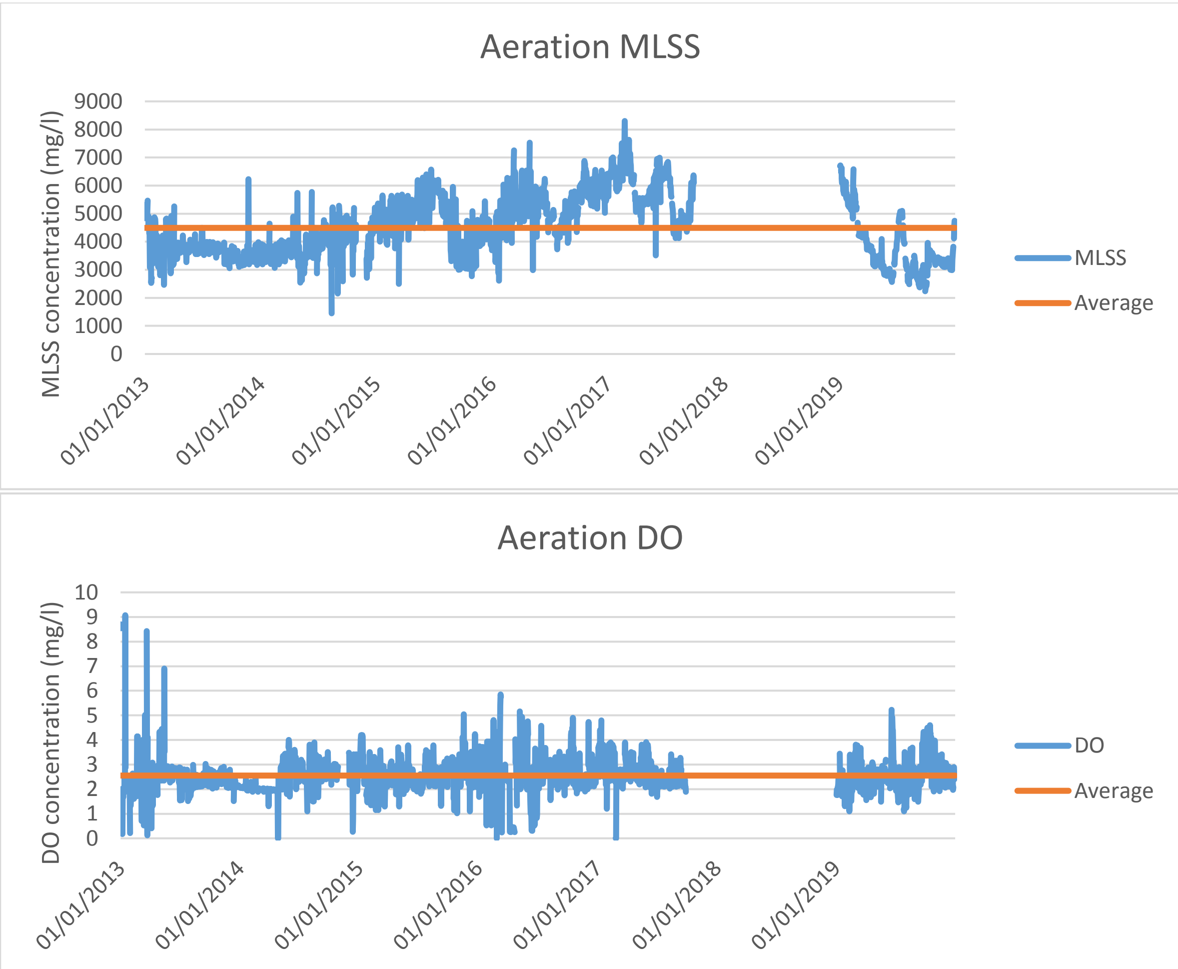


Figure 29. Aeration tank parameters

Regarding influent flow, daily information is available until 2018 when only monthly information is available. The following chart shows the gradual increase of the influent flow that it's already been discussed in previous sections.

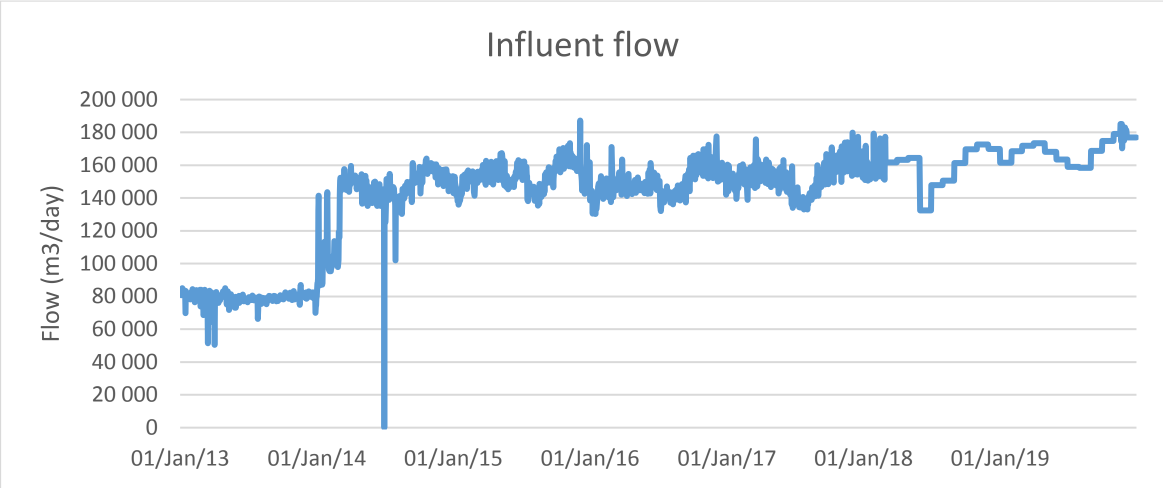


Figure 30. Influent flow

Chapter 7 Discussion

7.1 WWTP performance

After the analysis carried out in this study on the performance of the Kabd WWTP, the first thing that arises is that the plant does not comply with the standards in terms of ammonia, COD, BOD and TSS. The first parameter, ammonia, is related to an issue with nitrification.

When checking the historical data regarding ammonia, it shows a slight improvement during warmer months, but it does not last throughout the year. The current operating SRT in the plant is 5 days, which is suggested to be increased to 6 in order to mitigate this problem.

The rest of the parameters, COD, BOD and TSS are related to the poor performance of the clarifiers. When checking historical data to look for patterns, it was found out that according to routine analysis data of 2019 by the WWTP lab, all these parameters seems to be complying with the effluent standards, which is inconsistent with the current situation of the WWTP observed during the sampling campaign and the visits to the plant. The rest of the parameters checked in the routine analysis data seem to be consistent with what was obtained during the sampling campaign.

Regarding flow measurements, the plant is reaching its design capacity of 180,000 m3/day and currently is performing with values around 179,000 m3/day, as measured in the sampling campaign. The deviation of these data, as well as the data from the RAS flow, are quite low, with values of 2.1% and 0.75% respectively. The influent flow values have been increasing gradually since the opening of the plant, with an approximate increase over 6 years of 26%. The RAS has been increasing accordingly, maintaining a proportionate recycling ratio of around r=0.70. The flows measured on the sludge line showed higher deviations over the days, probably due to operational changes.

The concentration of DO observed in the fine bubble aeration tanks was quite high, with values around 2.9 mg/l. During the modelling phase, this concentration was optimized to 2 mg/l, which did not cause any changes on the performance of the plant, and can save energy costs of the blowers.

The solids accumulated in the biological reactor are quite high. The solution for this would be to decrease the SRT to decrease the MLSS value of the reactor. The counterpart of this is that, if the SRT is decreased, ammonia would most likely reach higher values than permitted by the standards of the effluent.

Finally, the daily generated sludge is being piled up close to the WWTP buildings and with no control. Other solutions should be found in order to reuse this sludge or at least transport it to a specific location specially prepared for that.

7.1.1 Clarifiers' performance

The performance of the clarifiers in the WWTP is currently not good, causing a high concentration of solids in the effluent, which is causing problems in the following stage of the process, filtration in the disc filters. The hydraulic capacity has not been exceeded, nor the load capacity, as proved previously on scenario SC2 on section 6.8.3. Therefore, the reduction in efficiency is probably caused by other problems.

When checking the SVI of the sludge in the settling tanks it was found out from the lab routine analysis charts that the average value during the sampling campaign days was SVI= 305 ml/g, and the average value for 2019 was SVI= 230 ml/g. Even though there is not a commonly established critical value above which bulking sludge occurs, and it depends a lot on local practises in settler design, in The Netherlands for instance an SVI higher than 120 ml/g is already considered bulking sludge (Henze et al. 2008). The values obtained in the lab are twice this value, which makes it probable that bulking sludge is the cause for the clarifiers' poor performance.

Bulking sludge, a term used to describe sludge with an excess growth of filamentous bacteria, causes difficulties for the sludge to settle, since the sludge flocs are porous and open and they contain a lower solids content. This flocs settle lower, so they require larger settlers in order to prevent solids escaping with the effluent (Henze et al. 2008). Since the WWTP it's almost reaching its design capacity, there is no much extra space in the clarifiers to reverse this situation of poor settleability.

There is not a clear solution to bulking sludge. When trying to remediate it, chlorination, ozonation and application of hydrogen peroxide is suggested (Henze et al, 2008), the former already being carried out by the Kabd WWTP in the effluent arriving to the disc filters. In terms of prevention, selectors are recommended before the influent reaches the biological reactor.

Besides poor settling, some algae were observed growing in the clarifiers. This may be caused by a high content of nutrients arriving to the settlers, and can also be related to a poor maintenance of the units. Algae may cause an increase on BOD and TSS results when sampling. Chlorination can be an option to remedy it, as well as covering the clarifiers or a more frequent maintenance (Henze et al. 2008).

7.1.2 Bacteria inhibition

The nitrification process is very much related to the SRT of the plant and the temperature (Henze et al 2008). The local average temperature of the WWTP is 25 °C on average, which can be considered quite high, and therefore favourable for nitrification. The minimum SRT required for nitrification, calculated in *Equation 9*, is 2.1 days, a lower value than the operative SRT of the plant. This shows that under normal situation for the bacteria, the plant should fully perform nitrification, and the fact that this is not happening indicates there probably is an external agent that inhibits the process.

After analysing the context and location of the plant, the possible causes were narrowed to two options: Inhibition by high salinity in the influent and inhibition by industrial discharges.

According to Cortés-Lorenzo *et al.* (2014), under a concentration of NaCl lower than 3700 mg/l nitrification process was not much affected. During the period of the sampling campaign the average concentration of Total Dissolved Solids in the influent was TDS= 710 mg/l, and according to the routine analysis data provided by the WWTP lab, during 2019 the average concentration of TDS was TDS= 933 mg/l. Both values are much lower than the previous condition, and hence it was concluded that the high salinity was not the problematic agent.

Regarding the industrial discharge option, after conversations with operators and lab staff it was stated that the wastewater arriving to the plant shows a blackish colour sometimes. Since the WWTP is located in the desert area of Kuwait, where the oil and petro-chemical industries are located too, discharges coming from these industries were considered as a possible cause for the inhibition of the bacteria. The research done by Figuerola and Erijman (2010), shows that phenol, a compound present in refinery wastewater, is a cause of inhibition for nitrifying bacteria.

According to staff in the plant, these discharges are not on a continuous basis, and come as shock loads, making it more difficult for bacteria to adapt, whichever the toxic compound may be. No toxicity tests were performed to prove the origin of the toxic compound and further research would be required to identify it.

7.1.3 Assessment of the configuration

Kabd WWTP is configured with four parallel treatment lines, each of them with two VLR reactors and one fine bubble aeration reactor. The VLR is a quite uncommon configuration, formed by tanks with a horizontal baffle, forcing wastewater to move on a vertical loop.

One of the advantages claimed by this treatment configuration is that there is no need for internal recycle in order to perform denitrification, since simultaneous nitrification-denitrification is achieved in the VLR tanks. In order to simulate this configuration in the model, a series of tanks were used for each VLR, the first three tanks being anoxic and the last three aerobic, to simulate the bottom part of the VLR which is anoxic and the top part of the VLR which is aerobic. Appendix I shows a table with the performance of these tanks regarding the main parameters.

Besides the VLR tanks, the configuration also includes a RAS tank, where the RAS flows from the clarifier and before going back to the treatment system. The purpose of this tank is to reduce the concentration of nitrate and to increase the total biomass concentration in the system. According to the table showed in Appendix I, the RAS tank does not increase the biomass concentration of the model, and when comparing it to the measured values of the sampling campaign in *Table 6*, where “WAS sludge” equals to the RAS tank influent, and “RAS tank” shows the outcome of the reactor, same results are concluded. Regarding nitrate reduction, the model does not show it since there is almost no nitrate, but it does show an increase of ammonia.

Other examples of this configuration were found in Iowa (USA) and Ohio (USA).

7.2 Reliability of sampling campaign measurements

Flow mass balancing is one of the most important steps of the modelling process and can have a high influence on the model prediction accuracy. The plant has flow meters in the following locations: Influent, RAS pump station, WAS pump station, Effluent of the collection chamber (effluent of tertiary treatment), aerobic digester inlet and aerobic digester outlet. Since one of them did not work (aerobic digester inlet), and another one was out of the boundaries of this study (tertiary treatment effluent), the flows were measured in four locations only, and the rest of the flow values were either estimated as accurately as possible using SCADA information or calculated on a flow mass balance basis. The estimations made and not being able to double-check the flow measurements by having an "over determined" system, in terms of having more unknowns than equations, makes the flow mass balance less reliable.

Regarding the chemical parameters sampling campaign, most of the samples taken were grab samples, with some exceptions, due to the unavailability of staff for more frequent sampling. This, of course, implies a certain level of accuracy loss during the sampling, which often can mean high errors for following stages. Moreover, the analyses were performed in three different labs, the one of the WWTP and two external labs. This also may cause inconsistencies among the data. Overall, the deviation of the average parameters obtained is quite high, especially in grab samples, reaching values close to 70% for certain parameters, such as TSS or nitrate. Composite samples showed less deviation among samples, with maximum values of around 25% in VSS in the influent.

Besides this, after analysing the measured data the effluent values for BOD and COD were found inconsistent with the rest of the effluent values, since a value of VSS=90 mg/l can hardly be related to a value of COD=62 mg/l, as the COD/VSS ratio is normally between 1.2 and 1.6.

Moreover, TP values were also found inconsistent. TP results obtained from the external lab showed lower results than phosphate measurements carried out by the plant lab on the same days. Also, in one of the days of the sampling campaign, the result for the micro-filtered TP in the effluent was higher than the result for the non-filtered TP. These issues, along with the difficulties to close a TP mass balance during the modelling phase, led to the decision to discard these data.

Finally, DO meters located in the aeration tanks were not calibrated and did not perform well, and therefore DO had to be measured manually by using a portable DO meter.

7.3 Data interpretation approaches

During the data evaluation phase and later the model calibration phase, some inconsistencies were found among data, which made the closing of the mass balances problematic as well as the following calibration of the model. TP values and flow values did not combine well, making it difficult to close the TP mass balance or to calibrate the model for P without compromising the flow balance or the other parameters measurements. At this point two approaches were open: To consider the flow measurements reliable and discard the TP measurements, or to consider the TP measurements reliable and make major adjustments on the flow mass balance. As stated in the previous section, when looking more carefully on the P values, some inconsistencies were found. This, together with the fact that measuring flow is more straight-forward than measuring TP, were the reasons why the first approach was selected, relying on flow measurement and discarding the TP measurements.

However, the second approach was also explored, and adjustments were made on the flow mass balance in order to calibrate P in the model. In this approach, the SRT was decreased to 2.5 days. Results on the calibrated model and scenarios can be found on Appendix J.

7.4 Evaluation of the scenarios

After the evaluation of the performance of the plant three scenarios were proposed.

The first one aimed to reduce the daily generated sludge, for which the SRT was increased and the WAS sludge decreased. This reduced the sludge flow generated every day, but at the same time increased both the solids content accumulated in the plant and in the final disposed sludge. The latter can be a positive fact since it improves the conditions of the sludge for dewatering, incineration or sometimes even landfill disposal, but the former can cause settleability issues when the accumulation is too high. Since the plant already has a high solids content, the SRT should not be increased much to avoid this accumulation as much as possible.

The second scenario analysed the consequences a gradual increase of the flow would have on the plant. According to Section 6.9, since 2013, the flow has been gradually increasing, with an increase of around 26% in the past 6 years (2014-2019). Although the concentration of the wastewater does not seem to change much over the years, the increase of flow causes an increase on load which according to the estimation of this scenario, will make the plant collapse in a lapse of 3 years (under an operation SRT of 6 days). The plant is designed for higher peak flows in terms of hydraulics, but not in terms of loads and that is why the settling tanks collapse with the increase of solids in the system and effluent values increase sharply when reaching a flow of 200,000 m3/day. If this situation is reached at some point of time, it will be preferable to bypass the excess of influent in order to avoid the collapse of the systems. Also, by changing the operation and decreasing the SRT, less MLSS would be accumulated in the tanks allowing a better performance of the clarifiers, but on the other hand it could cause issues with high content of ammonia in the effluent.

However, and since the calculated lapse is of 3 years, it is more than probable that the planned upgrade of the plant will be operating by that time.

The third scenario aimed to assess the cost reduction caused by substituting the aerobic digesters for anaerobic digesters. The daily production of methane could probably be increased by optimizing the conditions and operation of the plant towards this direction, such as adjusting pH, SRT, temperature of the process etc. This is out of the scope of this study, and the calculation of generated electricity was done on a base scenario. By converting methane to electricity, the annual saving according to the market price is 686,930 USD. However, the investment that needs to be made to recover the gas is high, not only for replacing the digesters but also for installing an electricity transformation system. Considering that the electricity in Kuwait is quite cheap, this will hardly ever be an advantage on economic terms. The plant management should evaluate the option better as an advantage on sustainability.

7.5 Wastewater characterization and fractions

The wastewater characterization was carried out following the STOWA protocol. First, S_I was calculated with the measured filtered COD of the effluent, and with that the rest of the parameters were adjusted during the calibrations phase (X_I, X_S and S_S). The following table shows a comparison between the Kabd WWTP and other WWTP in different countries:

Table 25. Wastewater characterization comparison

Parameter		Kabd	Brdjanovic et al. (2000) Haarlem (NL)	Lopez-Vazquez et al. (2013) India	Meijer et al. Croatia	Eidroos (2015) Oman
Fbs	Readily biodegradable	0.61	0.318	0.30	0.558	0.486
Fxsp	Slowly biodegradable	0.28	0.425	0.455	0.051	0.207
Fus	Unbiodegradable soluble	0.033	0.066	0.137	0.051	0.032
Fup	Unbiodegradable particulate	0.072	0.19	0.105	0.390	0.275

The table shows that values differ quite a lot among the different cases. In the case of Kabd WWTP, readily biodegradable COD is the highest as well as unbiodegradable particulate COD the lowest, but none of them seem to be too far from the rest of the countries.

Hereafter, an analysis was carried out on the main parameters of the influent, by comparing them to typical ranges stated by Meijer and Brdjanovic (2012):

Table 26. Wastewater influent main parameters ranges

Name	Param.	Unit	Value (mg/l)	Range (mg/l)	Result
Total COD	TCOD	mgCOD/l	531	500- 1200	Low-medium
Micro-filtered COD	CODmf	mgCOD/l	169	200- 480	OUT
BOD5	BOD5	mgBOD/l	273	230- 560	Low-medium
Total Suspended Solids	TSS	mg/l	241	250 - 600	Low
Volatile Suspended Solids	VSS	mg/l	117	200- 480	OUT
Total Kjeldahl N	TKN	mgN/l	36	30- 100	Low-medium
Ammonia	NH3	mgN/l	29	20- 75	Low-medium
Nitrate+ Nitrite	NO3+ NO2	mgN/l	10.01	0.1- 0.5	OUT

A few WWTP were studied in order to compare the influent ratios of different cases. The results are shown in the following table:

Table 27. Wastewater influent ratio comparison

Country	Kabd (Kuwait)	Haarlem (The Netherlands)	India	Varazdin (Croatia)	Oman	Typical ranges
COD/BOD	1.95	3.7	0.22	2.5	1.48	1.5- 3.5
COD/TN	11.64	8.8	18.9	11.4	6.97	6- 16
BOD/TN	5.97	2.4	84.71	4.5	4.71	3- 8
COD/VSS	4.53		1.55	3.1	2.55	1.2- 2.0
VSS/TSS	0.49	0.76	0.70	0.8	0.91	0.4- 0.9

The first table shows that the micro-filtered COD, VSS and nitrate are out of the recommended range.

When comparing with other countries, one of the first thing that pops up is the big difference that the rest of the countries show compared to this study for the VSS/TSS ratio. All of them have values between 0.7-0.9, whereas Kabd WWTP shows quite a low ratio of 0.49.

This is repeated again in the COD/VSS ratio, where most of the plants show values fitting the range, even though Varazdin and Oman WWTP have higher values out of the range, but again the difference with Kabd WWTP is high. Both this ratios, and the results of *Table 26*, show that the VSS content in the influent is quite low compared to the TSS.

This means that the inorganic suspended solids (ISS) value is quite high. Based on the studies checked in other countries, regular ISS values in the influent are in the range of 15-40 mg/l, but Kabd WWTP showed a concentration of ISS= 124 mg/l during the sampling campaign, and similar values in previous years, with an average of ISS= 105 mg/l in the period 2013-2016. This is probably related to the fact that the plant has no primary clarifier. A high content of ISS in the influent may be related to the possible toxic compound arriving in the influent, but also to the fine sand particles from the desert surrounding the plant.

Regarding the micro-filtered COD, the table shows that the measured value was too low, which can be related to uncertainties in the filtering process of the samples. Nitrate also shows a very high value compared to the ranges, maybe caused by nitrification happening in the sewer system due to the presence of oxygen. The rest of the parameters and ratios are in range, though most of them are on the low part of the range.

7.6 Model calibration

The calibration step was carried out following the guidelines by Meijer and Brjdanovic (2012), based on the STOWA protocol.

The X_I andS_I parameters were adjusted to fit the MLSS and COD concentrations in the aeration tanks and effluent as much as possible. However, since the COD and BOD values in the effluent seem inconsistent with the rest of the data, those two could not be fitted. Regarding the solids, VSS was fitted throughout the system, but TSS content was quite high compared to measured values and could not be adjusted to a close fit.

In order to calibrate the nitrification process, first the maximum specific growth rate was decreased in order to reduce the performance of nitrifiers and increase the predicted value of ammonia in the effluent. The aerobic decay rate was increased for the same purpose, but since the predicted value of ammonia was still lower than the measured, substrate (NH4) half saturation constant was adjusted to finally fit the ammonia values.

After that, the Denitrification DO half saturation constant and the denitrification N2 producers were adjusted in order to enhance denitrification in the model and decrease the nitrate values in the effluent.

The sludge line values fitted with some exceptions. Since the flows in that part of the system were mostly estimated and all the samples taken were grab samples, accuracy in the results obtained was not high and therefore it was found useless to try to calibrate the sludge line more accurately based on such high uncertainties.

Chapter 8 Conclusions

8.1 Conclusions

8.1.1 Kabd WWTP performance evaluation

- Kabd WWTP does not comply with the effluent standards and needs to handle some issues in order to be able to comply with them.
- The performance of the clarifiers is very poor and needs to be improved. Since this is most likely related to bulking sludge and/or maintenance of the units, the generated model cannot provide further analysis on the causes or possible remediation.
- There is an external contaminant that comes to the WWTP in shocks, causing an inhibition of the nitrifying bacteria, which causes high contents of ammonia in the effluent, not complying with the standards.
- Aeration in the fine bubble diffuser tanks is quite high and can be optimized to save energy costs.
- The plant is operating with a high content of solids in the reactors, probably caused by a high content of inorganic suspended solids arriving in the influent.
- The daily generated sludge is piled up close to the buildings of the plant, and there is no control or disposal plan for it.
- Inconsistencies were found among the results of the sampling campaign and the lab routine analysis data.

8.1.2 Modelling implementation

- The sampling program implementation and the reliability of the gathered data were the most critical steps of the project.
- There were not enough flow measurements to be able to double-check the results and guarantee more accuracy on the flow mass balance.
- Most of the samples were grab samples, which did not provide good accuracy on the results of most parameters.
- A model of the Kabd WWTP was successfully implemented to simulate the performance of the treatment system.
- The calibration of the model was performed up to the level of accuracy allowed by the gathered data of the sampling campaign and routine analysis data.
- The performance issues of the plant were identified using the implemented model.
- The final disposed sludge per day was reduced by a 23.6% in flow compared to the current situation, while in terms of mass was reduced by a 9.6%, by increasing the SRT from 4.7 to 6 days.
- The treatment system proved to be able to handle an increase of the influent load up to a flow of 200,000 m3/day, in the current situation with four treatment lines.

- The energy cost reduction per year of substituting the aerobic digesters for anaerobic digesters is estimated on 686,930 USD.
- The model proved that the optimal scenario would guarantee compliance of the effluent standards.

Overall, it can be concluded that the study successfully achieved the research objectives. A model of the plant was built which simulates its performance, and the issues faced by the plant were identified, proposing changes to be able to comply with the effluent standards. Moreover, operation changes were proposed in order to reduce the generated sludge, and an analysis of the possible increase of the influent and its consequences was carried out. Finally, aerobic digesters were substituted by anaerobic digesters and an estimation of the potential energy savings was successfully performed.

8.2 Recommendations

- The study concluded there is bacterial inhibition in the WWTP which can be related to the potential existence of a toxic compound arriving with the influent to the plant. Further analysis on this issue was out of the scope of the study, but it is recommended that toxicity lab analysis is performed and further research is made on the influent composition in order to confirm this hypothesis and find out the agent, its concentration and the frequency it arrives to the plant, so further measures can be taken.

- The current performance of the clarifiers needs to be improved in order to be able to comply with the standards. This study concluded that the problem is most likely related to bulking sludge, but further microscopic lab analysis is recommended to confirm it. In that case, it is recommended to continue with the chlorination, and if it proves to be inefficient to try with ozonation, both treatments intended to remediate bulking sludge. If the problem persists, since a future upgrade of the plant has already been planned, including selectors before the biological reactors can also be considered during the construction.

- Introducing an anaerobic digester has been proven in this study to be a successful way of energy recovery in Kabd WWTP. The economical estimations showed that the investment needed for the whole installation will hardly be competitive with the low prices of energy in Kuwait. Nevertheless, further research on the topic is recommended in order to consider the substitution of the aerobic digesters by anaerobic ones, not just from a merely economic point of view but in terms of sustainability and self-sufficiency of the plant.

- The model-based study showed to be a successful tool to evaluate the performance of the plant and identify the current issues on it. In order for future models to be more accurate in its predictions, a more detailed sampling campaign will be required, and a higher investment in material and staff is recommended for it, since obtaining more accurate results on the performance can be useful for plant operating. It is recommended that this model is further improved and a more accurate calibration is performed.

- Since generated sludge is currently being piled up without further treatment or disposal, it is recommended that research is carried out towards possible treatments and future reuse applications.

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Appendices

Appendix A. - Research Ethics Declaration Form

Part 1- Ethics Exemption letter

United Nations Educational, Scientific and Cultural Organization
IHE DELFT Institute for Water Education under the auspices of UNESCO

Research Ethics Committee
IHE Delft Institute for
Water Education
E ResearchEthicsCommitee@un-ihe.org

Date:	2020-03-16
To:	Carla Vazquez Gomara
MSc Programme:	Urban Water and Sanitation – Sanitary Engineering
Approval Number:	IHE-RECO 2020-009

Subject: Exemption for further ethical review

Dear Carla Vazquez Gomara,

Based on your application for Ethical Approval, your proposal "Modelling Wastewater treatment plant Kubd WWTP Kuwait" has been exempted from further revision by the Research Ethics Committee (RECO), IHE Delft. You need to notify the RECO of any modifications to your research protocol.

Please keep this letter for your records and include a copy in the final version of MSc. Thesis.

On behalf of the Research Ethics Committee, I wish you success in the completion of your research.

Yours sincerely,

Angeles Mendoza
Acting Ethics Coordinator

Copy to: Academic VP.
Copy to: Reviewer

Part 2- Personal statement

The development of this study has been subject to the main principles or virtues of the Netherlands Code of Conduct for Research Integrity 2018 (KNAW, et al., 2018). These principles are the following: honesty, scrupulousness, transparency, independence and responsibility. These are displayed throughout the entire development: from start to finish.

To organize this project, IHE Delft Water Institute contacted the University of Kuwait and in this way a link was created between the two countries. The person who opened this communication was Dr. Héctor García Hernández (IHE Delft), thanks to whom it was possible to work with Dr. Abdalrahman Alsulaili (University of Kuwait). The relationship between the two institutions requires a commitment to transparency, in terms of respect to the objectives and the means of the work. Thanks to the representative in the University of Kuwait with whom we worked, Dr. Alsuilaili, a third contact was established that demanded the same ethical requirements: the WWTP of Kabd (Kuwait), where the data field work was carried out, collecting the data that gives shape to this work. When several parties are involved in a project or, as it is the case, in an academic work (which serves as a practice not only for learning about wastewater treatment but also for learning about good work practices) it is essential that communication is fluid and transparent to avoid misunderstandings that, in a sector like this, can have consequences of a certain severity, since they interfere in the proper functioning of structures that are, ultimately, designed for society.

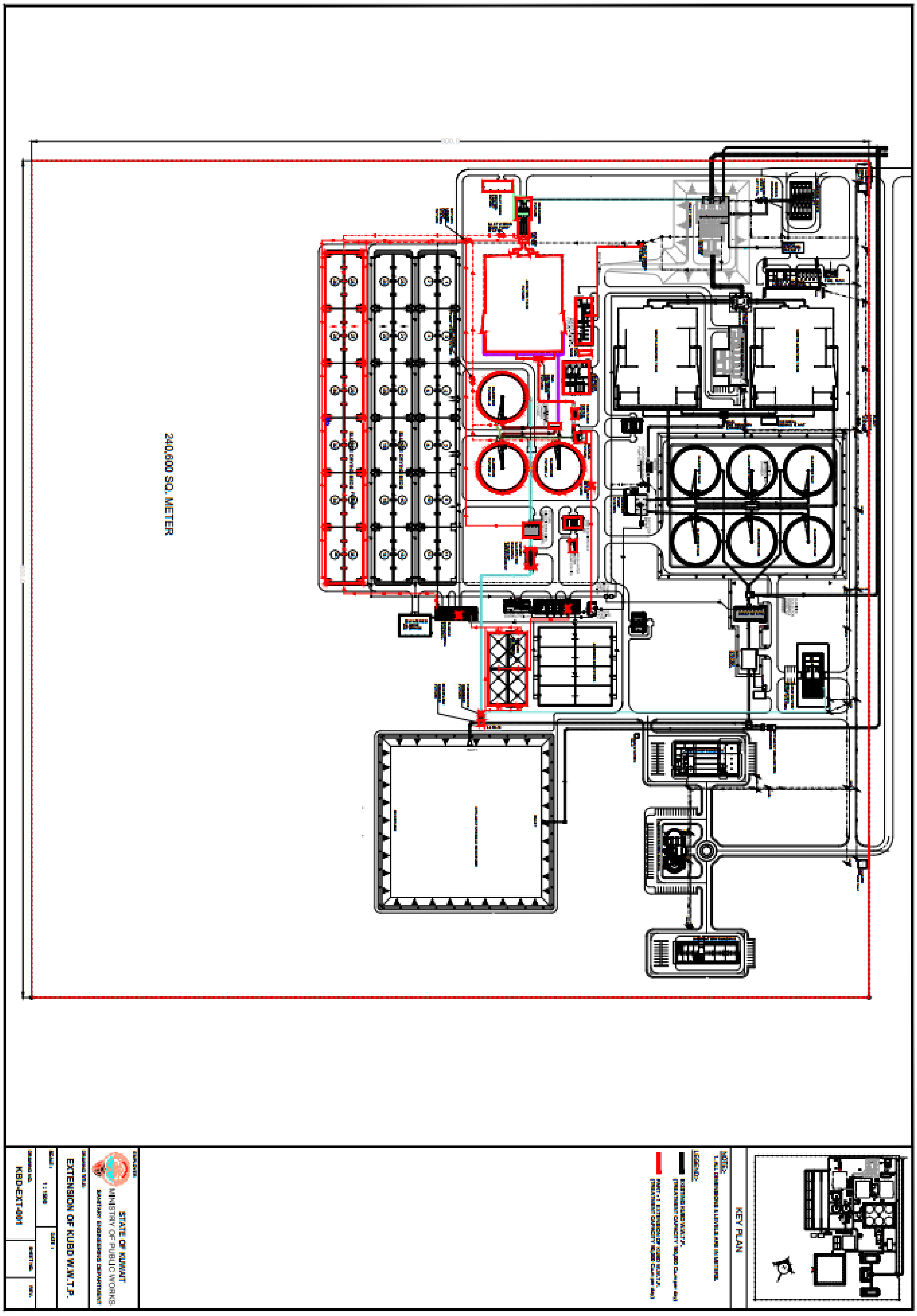
Thanks to the relationships established, the case study in the WWTP of Kabd could be started, and field work was carried out for 4 weeks in the months of November and December 2019. During this phase samples were taken in different locations of the treatment plant in addition to collect the information available in the plant's DCS (Data Control System) in charge of monitoring daily activity. The samples were taken with all possible scrupulousness in order to try to obtain the most reliable and honest results. The collected samples were transported daily to three different laboratories (two external of the plant and one of their own) that carried out the analyses. The treatment of the data collected from the monitoring system was carried out with all rigor and honesty.

During the fieldwork phase, conversations were held with plant managers and operators to try to learn first-hand about the current situation and the challenges they frequently face. The conversations held during this period served to better understand the operative perspective of the plant and the means available for them, as well as the way they work towards all the challenges that daily operations pose to them. This knowledge provided by operators and managers enriches and expands the work, and deserves to be treated with all responsibility.

After the field work stage, the rest of the work was carried out in IHE Delft. To put in order and make sense of the information collected, all the required technical literature was used, specifying in each case the origin of the source used. The research was carried out independently and rigorously, proposing ideas supported by previous studies (having referenced all of these). This phase was successfully completed thanks to the support and supervision of Dr. Héctor García and Dr. Tineke Hooijmans, who corrected and guided the work in a thoughtful and thorough way.

As can be seen after all this, the ethical values of the Netherlands Code of Conduct for Research Integrity 2018 (KNAW, et al., 2018) have been taken into account both in the approach and in the development of this study, giving rise to a truthful and honest job. This may be made available to all parties involved, so that they can benefit from the results received thanks to the cooperative relationship that has been established.

Appendix B. - Layout of the WWTP



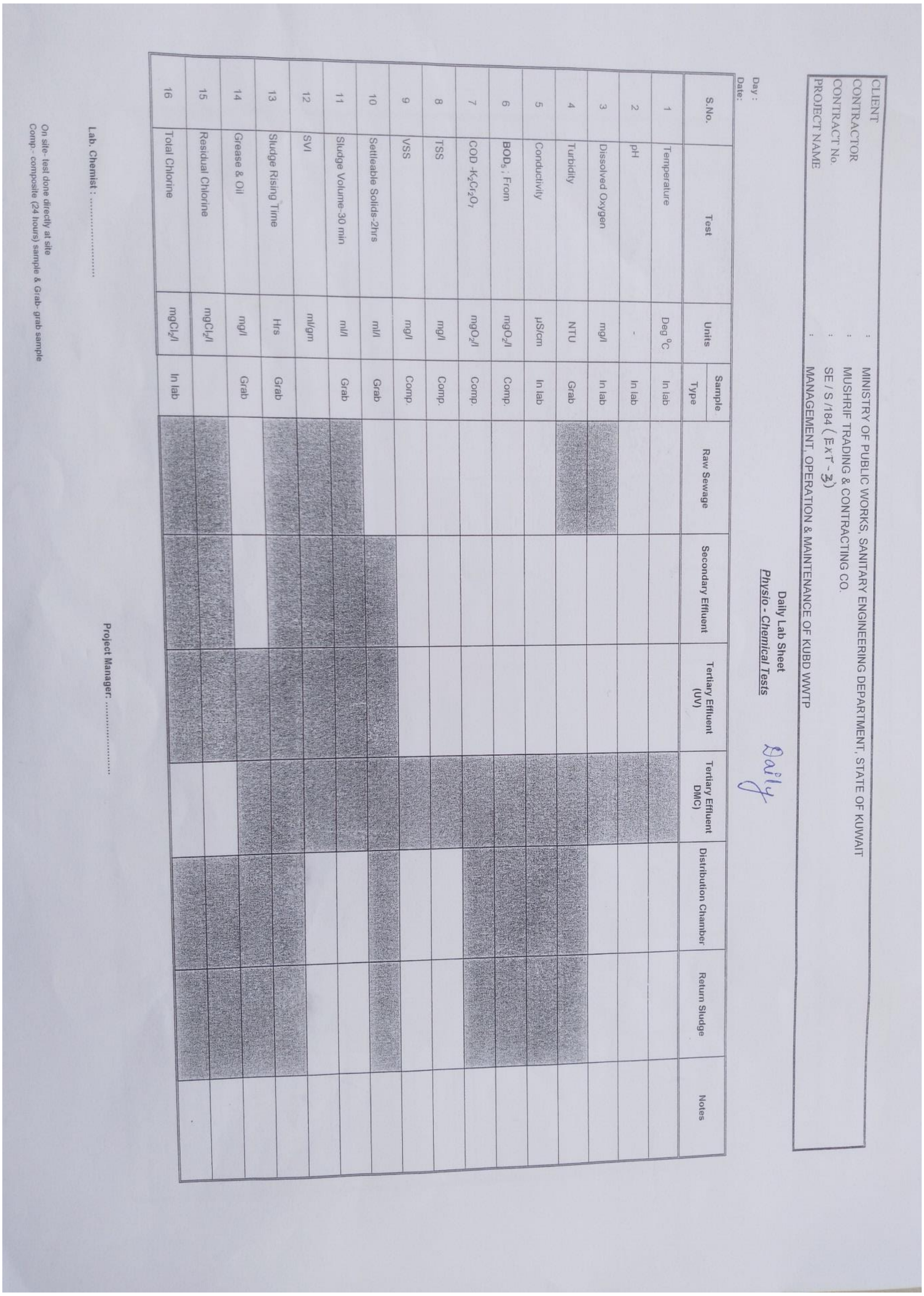
Appendix C. - WWTP Design data

DESIGN DATA					
Parameter	**Value**	**Unit**	**Description**		**Comments**
1. Influent (before preliminary treatment)					
Average flow		m3/day	Design average inflow		
Peak flow	270,000	m3/day	Design peak inflow		
TSS	300	mg/L	Total suspended solids		
ISS		mg/L	Inorganic suspended solids		
BOD5	300	mgBOD/L	Total Carbonaceous BOD5		
TCOD		mgCOD/L	Total COD		
TKN	38.3	mgN/L	Total Kjeldahl nitrogen		
TP	15	mgP/L	Total phophorus		
2. Activated sludge process	**(TWO AERATION UNITS , EACH UNIT HAS FOUR VLR BASINS)**				
VLR1					
Volume	31104	m3	Volume of the reactor		
Depth	6	m	Reactor depth	**1) No of basins - 8 Nos**	
Width	72	m	Reactor width	**2)Volume of each Basin - 3750 m3**	
Surface	5184	m2	Reactor surface	**3) Liquid depth - 6 meters**	
Aeration type		-	Mechanical disc aerators	**4) Width - 9 meters**	
Aeration set-point		mgO2/L	Aeration control set-point	**5) Length - 72 meters**	
Installed capacity	18.5	kW/h	Installed power input surface aeration	18.5	
Surface aeration efficiency		kgO2/kW/h	Factory design oxygen production efficiency		
Design DO concentration	2	mgO2/L	DO concentration for design	2	
Fine bubble reactor	**(TWO AERATION UNITS , EACH UNIT HAS TWO FINE BUBBLE TANKS WITH TWO CELLS IN EACH TANK)**				
Volume	30240	m3	Volume of the reactor	72*17.5*6*4	
Depth	6	m	Reactor depth	**1) No of Tanks - 4 Nos(two cells in each tank)**	
Width	70	m	Reactor width	**2)Volume of each Basin - 7560 m3**	
Surface	5040	m2	Reactor surface	**3) Liquid depth - 6 meters**	
Aeration type		-	Fine bubble diffusers	**4) Width - 17.5 meters**	
Aeration set-point		mgO2/L	Aeration control set-point	**5) Length - 72 meters**	
Installed capacity	1350	Nm3/h	Installed blower air input		
Bubble aeration efficiency		gO2/Nm3.m	Factory design oxygen production efficiency		
Aeration depth	6	m	Bubble rise height		
SRT	8.3	d	Sludge retention time	8.3	
DO concentration	3	mgO2/L	DO concentration for design	3	

RAS aeration tank	**(TWO AERATION UNITS , EACH UNIT HAS TWO RAS TANKS)**				
Volume	15036	m3	Volume of the reactor	**NOTE**	
Depth	6	m	Reactor depth	**1) No of RAS Tanks - 4 Nos(two RAS tankS in each aeration unit)**	
Width	70	m	Reactor width	**2)Volume of each tank - 3759 m3**	
Surface	2506	m2	Reactor surface	**3) Liquid depth - 6 meters**	
Aeration type		-	Coarse bubble diffusers	**4) Width - 70 meters**	
Aeration set-point		mgO2/L	Aeration control set-point	**5) Length - 35.8 meters**	
Installed capacity	1350	Nm3/h	Installed blower air input		
Bubble aeration efficiency		gO2/Nm3.m	Factory design oxygen production efficiency	**6) MLSS concentration at fine bubble tank-3750 mg/L**	
Aeration depth	6	m	Bubble rise height	**7) Solids concentration of return activated sludge,RAS-7500 mg/L**	
Design DO concentration	0.2	mgO2/L	DO concentration for design		
3. Secondary settling tanks	**(SIX CLARIFIERS , EACH CLARIFIER DIAMETER- 46 MTS, DEPTH - 5 MTS,)**				
Type	Round				
Maximum hydraulic loading	270,000	m3/day	Maximum flow for bypass conditions	180000	**180000 - RAS FLOW IN M3 PER DAY**
Volume	49856	m3	Total volume including cone		
Surface	9971.2	m2	Total surface for loading calculation	9971.2	
Depth	5	m	Side-wall depth		**Average water depth - 5 mts**
Diameter	46	m			
Slope settling cone		-	Meter decline per meter diameter	**RIM FLO , TWO BRO CLARIFIER FROM SIEMENS**	
Designed solids loading	7.051	kg/m2.h	Maximum design TSS loading	7.4	
Designed SVI	150	mL/g	Designed sludge volume index	150	
Type of chemicals dosed	NA	-	Type of chemicals dosed, if applicable	NA	
Quantity of chemicals dosed	NA	kg/d	Pure product	NA	
Waste activated sludge (WAS)					
Type of pump	Submersible pumps			2 pumps, 1 duty/1 standby	
Type of pump control	MANUAL CONTROL	-	e.g influent flow proportional		
Installed pump capacity	398	m3/h			
Return Activated sludge (RAS)					
Type of pump	Submersible pumps			4 pumps, 3 duty/1 standby	
Type of pump control	MANUAL CONTROL	-	e.g influent flow proportional		
Installed pump capacity	2500	m3/h			

4. Sludge treatment					
Secondary sludge thickening				**(three thickeners , two duty and one standby)**	
Thickening unit type	Gravity				
Unit capacity	8460	m3/d	Maximal processing capacity of sludge	**actual - 2400 m3/day from two thickening units**	
Digestion	**(DIGESTOR DIMENSION- LENGTH - 34 mts , WIDTH - 18 mts , DEPTH - 3.5 mts)**				
Volume	17136	m3	Volume digester	**8 DIGESTORS TOTALLY, 2142 M3 VOLUME PER DIGESTOR**	
SRT	17	d	Sludge retention time in digester		
Digested sludge dewatering	**(THREE DEWATERING UNITS , TWO DUTY AND ONE STANDBY)**				
Dewatering unit type	Belt press filter				
Capacity	1830	m3/day	Maximal processing capacity of sludge	**actual - 800 m3/day from two dewatering units**	

Appendix D. - Routine analysis charts



CLIENT	:	MINISTRY OF PUBLIC WORKS, SANITARY ENGINEERING DEPARTMENT, STATE OF KUWAIT
CONTRACTOR	:	MUSHRIF TRADING & CONTRACTING CO.
CONTRACT No.	:	SE / S /184 (EXT - 3)
PROJECT NAME	:	MANAGEMENT, OPERATION & MAINTENANCE OF KUBD WWTP

Daily Lab Sheet

Physio - Chemical Tests Daily

Day :

Date:

S.No.	Test	Units	Sample Type	Raw Sewage	Secondary Effluent	Tertiary Effluent (UV)	Tertiary Effluent DMC)	Distribution Chamber	Return Sludge	Notes
1	Temperature	Deg ^{0}C	In lab							
2	pH	-	In lab							
3	Dissolved Oxygen	mg/l	In lab							
4	Turbidity	NTU	Grab							
5	Conductivity	µS/cm	In lab							
6	BOD_5 ; From	mgO_2/l	Comp.							
7	COD -$K_2Cr_2O_7$	mgO_2/l	Comp.							
8	TSS	mg/l	Comp.							
9	VSS	mg/l	Comp.							
10	Settleable Solids-2hrs	ml/l	Grab							
11	Sludge Volume-30 min	ml/l	Grab							
12	SVI	ml/gm								
13	Sludge Rising Time	Hrs	Grab							
14	Grease & Oil	mg/l	Grab							
15	Residual Chlorine	$mgCl_2$/l								
16	Total Chlorine	$mgCl_2$/l	In lab							

Lab. Chemist : ....

Project Manager: ....

On site- test done directly at site

Comp.- composite (24 hours) sample & Grab- grab sample

CLIENT	:	MINISTRY OF PUBLIC WORKS, SANITARY ENGINEERING DEPARTMENT, STATE OF KUWAIT
CONTRACTOR	:	MUSHRIF TRADING & CONTRACTING CO.
CONTRACT No.	:	SE / S /184 (EXT – 2)
PROJECT NAME	:	MANAGEMENT, OPERATION & MAINTENANCE OF KUBD WWTP

Periodical Chemical Test Report

Date :

Day : Saturday & Tuesday

S.No	Tests	Units	Raw Sewage (Kubd)	Secondary Effluent	Tertiary Effluent	Aeration	Return Sludge	Aerobic Digestor	Remarks
1	Residue on Evaporation	mg/l							
2	Total Volatile Residue	mg/l							
3	Total Dissolved Solids	mg/l							
4	Total Solids	mg/g							
5	Alkalinity	mg/l							
6	Chloride	mg/l							
7	Chlorine Demand	mg/l							
8	Ammonia-Nitrogen	mg/l							
9	Nitrate-Nitrogen	mg/l							
10	Nitrite-Nitrogen	mg/l							
11	Organic-Nitrogen	mg/l							
12	Total Kjeldhal Nitrogen	mg/l							
13	Phosphate as PO4	mg/l							
14	Sulphate	mg/l							
15	H2S - Free	mg/l							
16	-Total	mg/l							
17	Heavy Metals - Aluminium	mg/l							
	- Copper	mg/l							
	- Boron	mg/l							
	- Lead	mg/l							
	- Cadmium	mg/l							
	- Mercury	mg/l							
	- Zinc	mg/l							
	- Iron	mg/l							
	- Calcium	mg/l							
	- Magnesium	mg/l							
	- Potassium	mg/l							
	- Sodium	mg/l							

Lab. Chemist : .... Project Manager: ....

CLIENT	:	MINISTRY OF PUBLIC WORKS, SANITARY ENGINEERING DEPARTMENT, STATE OF KUWAIT
CONTRACTOR	:	MUSHRIF TRADING & CONTRACTING CO.
CONTRACT No.	:	SE / S /184 (EXT - 2)
PROJECT NAME	:	MANAGEMENT, OPERATION & MAINTENANCE OF KUBD WWTP

Periodical Chemical Test Report

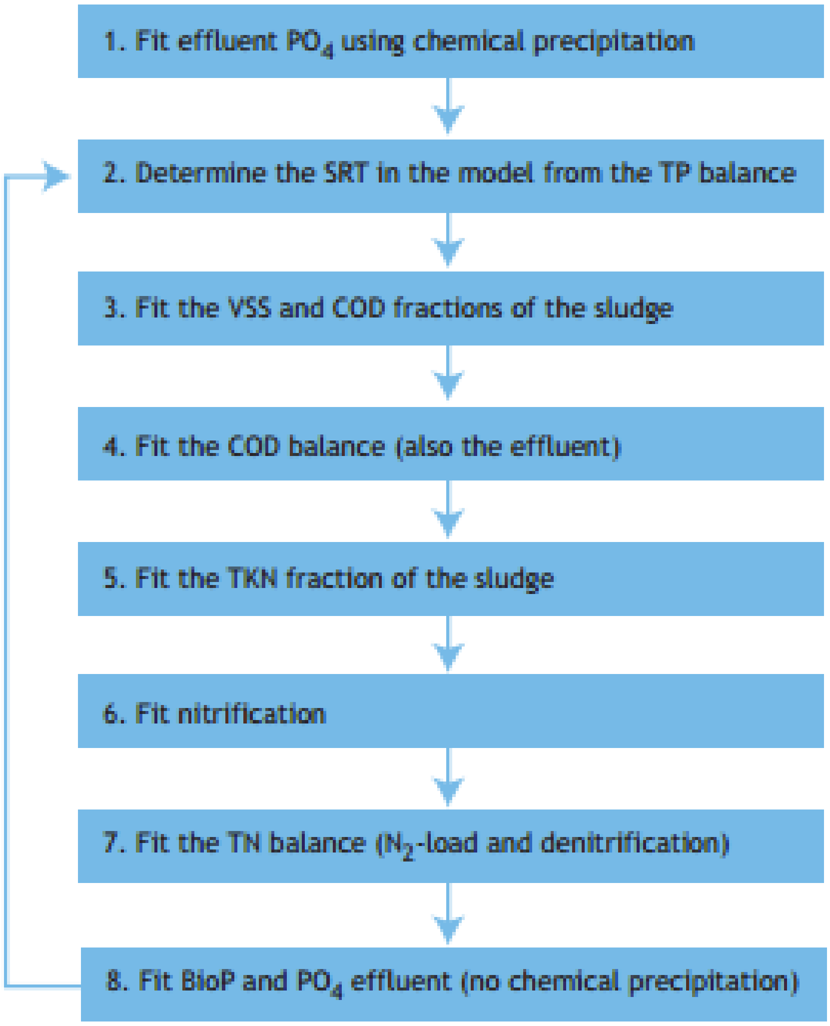
Date :

Day : Sunday & Wednesday

S.No	Tests	Units	Raw Sewage	Secondary Effluent	Tertiary Effluent	Aeration	Return Sludge	Aerobic Digestor	Remarks
1	Residue on Evaporation	mg/l							
2	Total Volatile Residue	mg/l							
3	Total Dissolved Solids	mg/l							
4	Total Solids	mg/l							
5	Alkalinity	mg/l							
6	Chloride	mg/l							
7	Chlorine Demand	mg/l							
8	Ammonia-Nitrogen	mg/l							
9	Nitrate-Nitrogen	mg/l							
10	Nitrite-Nitrogen	mg/l							
11	Organic-Nitrogen	mg/l							
12	Total Kjeldhal Nitrogen	mg/l							
13	Phosphate as PO4	mg/l							
14	Sulphate	mg/l							
15	H2S - Free	mg/l							
16	-Total	mg/l							
17	Heavy Metals - Aluminium	mg/l							
	- Copper	mg/l							
	- Boron	mg/l							
	- Lead	mg/l							
	- Cadmium	mg/l							
	- Mercury	mg/l							
	- Zinc	mg/l							
	- Iron	mg/l							
	- Calcium	mg/l							
	- Magnesium	mg/l							
	- Potassium	mg/l							
	- Sodium	mg/l							

Lab. Chemist : .... **Project Manager:** ....

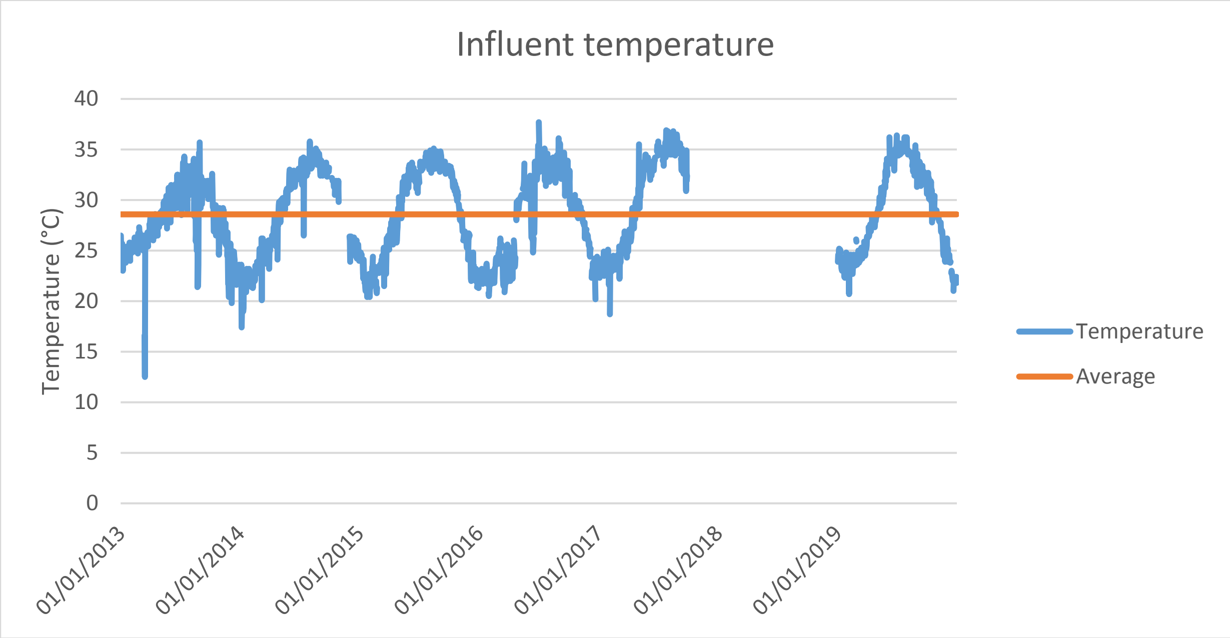
Appendix E. - Calibration procedure

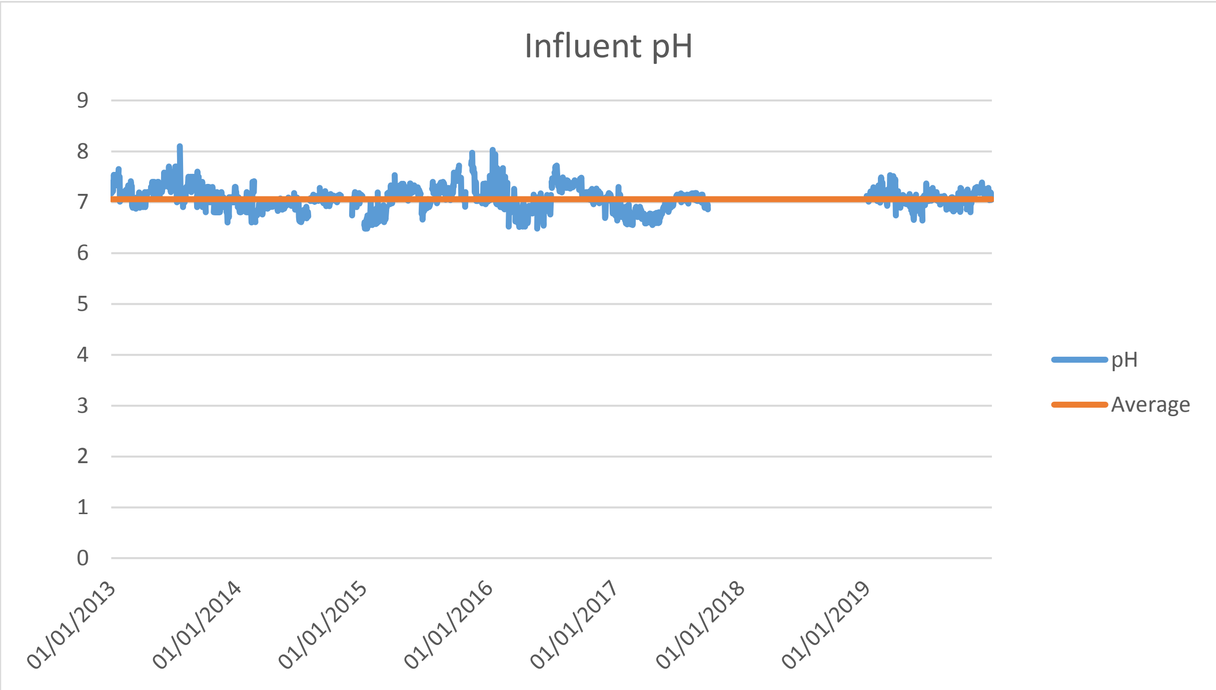


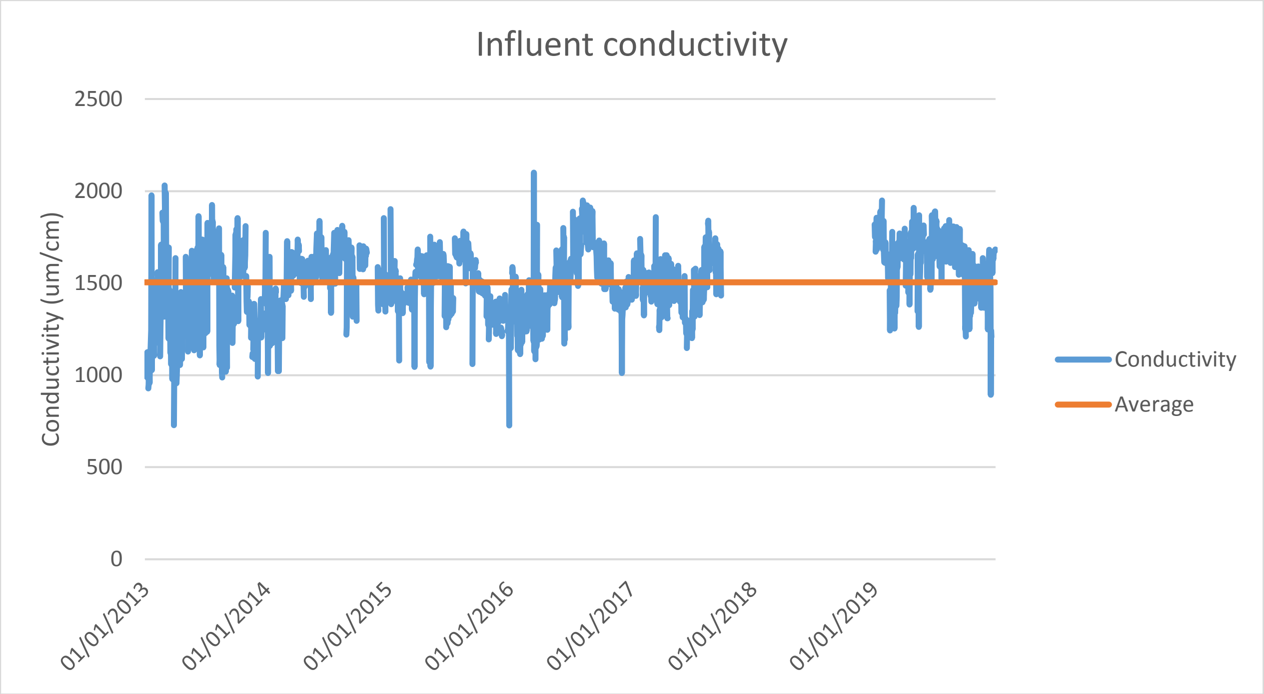
Appendix F. - Return Liquor Pump running hours

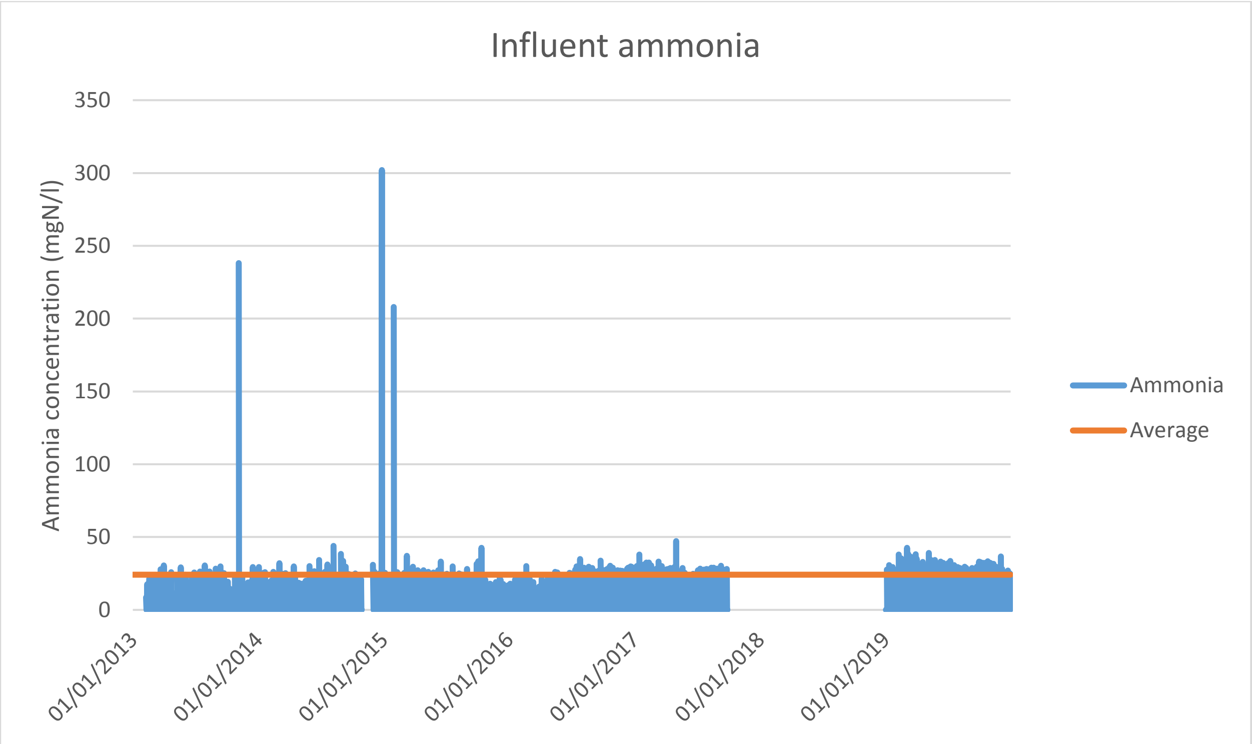
DATE	RUNNING HOURS	CAPACITY	EFFICIENCY	TOTAL
	h/d	m3/h	-	**m3/d**
19/Nov	17.8	230	70%	**2865.8**
20/Nov	16.3	230	70%	**2624.3**
21/Nov	15.6	230	70%	**2511.6**
22/Nov	16.1	230	70%	**2592.1**
23/Nov	13.4	230	70%	**2157.4**
24/Nov	16.6	230	70%	**2672.6**
25/Nov	16.7	230	70%	**2688.7**
26/Nov	16.6	230	70%	**2672.6**
27/Nov	16.5	230	70%	**2656.5**
28/Nov	17.9	230	70%	**2881.9**
29/Nov	19.2	230	70%	**3091.2**
30/Nov	17.3	230	70%	**2785.3**
01/Dec	18.1	230	70%	**2914.1**
02/Dec	16.8	230	70%	**2704.8**
03/Dec	17.9	230	70%	**2881.9**
04/Dec	18	230	70%	**2898**
05/Dec	16.9	230	70%	**2720.9**

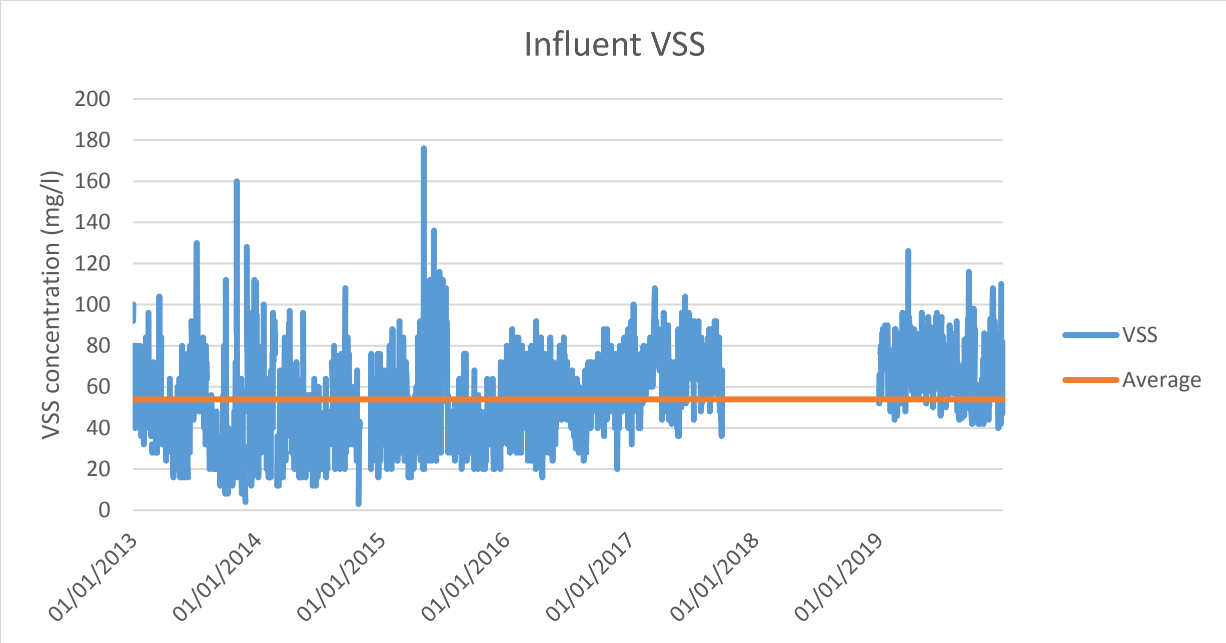
Appendix G. - Historical data






Influent conductivity
2500
2000
1500
1000
500
0
Conductivity (um/cm)
01/01/2013
01/01/2014
01/01/2015
01/01/2016
01/01/2017
01/01/2018
01/01/2019
Conductivity
Average


Influent ammonia
350
300
250
200
150
100
50
0
Ammonia concentration (mgN/l)
01/01/2013
01/01/2014
01/01/2015
01/01/2016
01/01/2017
01/01/2018
01/01/2019
Ammonia
Average



Historical data on thickener supernatant

2017		
MONTH	**WAS FLOW TO THICKENER**	**THICKENED SLUDGE TO AD**
….	**m3**	**m3**
JANUARY	67168	18773
FEBRUARY	53927	17531
MARCH	48217	17531
APRIL	67782	25453
MAY	52804	17837
JUNE	38735	12445
JULY	34	0
AUGUST	23162	5743
SEPTEMBER	39858	11000
OCTOBER	55371	18790
NOVEMBER	47432	19454
DECEMBER	59115	14095
TOTAL	**46133.75**	**14887.67**
INF/EFF RATIO	**0.323**	
SUPERNATANT	**0.677**	

2018		
MONTH	**WAS FLOW TO THICKENER**	**THICKENED SLUDGE TO AD**
….	**m3**	**m3**
JANUARY	54716	21784
FEBRUARY	50154	22036
MARCH	74392	50154
APRIL	40091	16394
MAY	41186	16394
JUNE	20238	8484
JULY	36344	14899
AUGUST	55659	23204
SEPTEMBER	36562	15840
OCTOBER	68355	18254
NOVEMBER	49350	24239
DECEMBER	50802	26592
TOTAL	**48154.08**	**21522.83**
INF/EFF RATIO	**0.45**	
SUPERNATANT	**0.55**	

2019		
MONTH	**WAS FLOW TO THICKENER**	**THICKENED SLUDGE TO AD**
….	**m3**	**m3**
JANUARY	28429	13849
FEBRUARY	39639	18581
MARCH	40611	18586
APRIL	23142	8324
MAY	42464	13647
JUNE	44894	1654
JULY	35889	968
AUGUST	35313	0
SEPTEMBER	36257	0
OCTOBER		
NOVEMBER		
DECEMBER		
TOTAL	**36293.11**	**8401.00**
INF/EFF RATIO	**0.23**	
SUPERNATANT	**0.77**	

Appendix H. - Results of the calibrated model

Elements	Flow [m3/d]	Total COD [mg/L]	BOD [mg/L]	VSS [mg/L]	TSS [mg/L]	TKN [mgN/L]	Ammonia [mgN/L]	Nitrate [mgN/L]	Total P [mgP/L]	Phosphate [mgP/L]
Influent	44,903.25	531.00	317.82	118.23	242.23	36.00	29.02	10	4	3.6
Aerated tank	75,311.88	2,817.72	985.47	1,950.81	4,048.77	229.27	17.32	0.01	68.85	0.11
RAS tank	29,695.00	6,728.66	2,326.05	4,679.31	9,785.59	529.43	21.06	0.00	167.45	0.11
Effluent	45,241.88	150.12	46.57	90.93	188.71	28.42	17.32	0.01	3.31	0.11
Ret Liquor	713.63	498.45	200.24	256.56	467.19	47.32	21.01	3.24	46.31	39.74
Thick. Supernatant	228.75	712.03	244.34	482.7	1,001.80	70.73	17.32	0.01	17.12	0.11
Anaerobic Digester	146.25	9,361.09	738.92	6,442.03	18,005.40	751.85	4.07	17.58	402.59	209.92
Dewatering supernatant	131.63	123.11	2.92	21.47	60.02	12.85	4.07	17.58	210.57	209.92
Filter backwash influent	353.25	500	245.21	197.71	272.71	45	29.7	0	4	2

Appendix I. - Model configuration results

Aerated reactors

Elements	Total COD [mg/L]	BOD [mg/L]	TSS [mg/L]	VSS [mg/L]	TKN [mgN/L]	Ammonia [mgN/L]	Nitrate [mgN/L]
VLR1.Anx1	1,213.10	519.95	1,376.20	676.95	92.93	25.58	4.36
VLR1.Anx2	1,205.58	513.23	1,384.08	685.3	92.93	25.24	1.74
VLR1.Anx3	1,200.75	507.85	1,390.06	691.91	92.93	24.96	0.07
VLR1.Aer1	2,924.45	1,064.43	4,055.41	1,966.74	230.55	21.7	0.01
VLR1.Aer2	2,906.61	1,048.70	4,070.92	1,981.13	230.49	19.81	0
VLR1.Aer3	2,892.81	1,036.84	4,081.66	1,990.05	230.43	18.39	0
VLR2.Anx1	2,888.37	1,032.32	4,078.78	1,989.50	230.39	18.37	0
VLR2.Anx2	2,888.32	1,031.61	4,077.25	1,989.79	230.39	18.48	0
VLR2.Anx3	2,888.26	1,031.03	4,075.80	1,989.91	230.39	18.54	0
VLR2.Aer1	2,881.04	1,027.07	4,075.26	1,988.09	230.32	18.03	0
VLR2.Aer2	2,875.72	1,023.90	4,074.06	1,985.57	230.26	17.72	0
VLR2.Aer3	2,870.85	1,020.89	4,072.76	1,982.99	230.2	17.49	0
FB1	2,841.81	1,001.76	4,061.99	1,965.88	229.74	17.09	0
FB2	2,817.72	985.47	4,048.77	1,950.81	229.27	17.32	0.01

RAS tank

Elements	COD - Total [mg/L]	Total suspended solids [mg/L]	Volatile suspended solids [mg/L]	Ammonia [mgN/L]	Nitrate [mgN/L]
RAS tank influent	6,831.25	9,856.43	4,749.10	17.32	0.01
RAS tank effluent	6,728.66	9,785.59	4,679.31	21.06	0

Appendix J. - TP calibration approach

Flow balance

Code	Description	Measure/ Calculate	Equation	Value (m3/day)
Q1	Raw influent	Measure		179613
Q2	Influent to Line 1	Calculate	(Q1+Q21)/4	46629
Q3	Influent to Line 2	Calculate	(Q1+Q21)/4	46629
Q4	Influent to Line 3	Calculate	(Q1+Q21)/4	46629
Q5	Influent to Line 4	Calculate	(Q1+Q21)/4	46629
Q6	Effluent Line 1	Calculate	Q2+Q12	76501
Q7	Effluent Line 2	Calculate	Q3+Q13	76501
Q8	Effluent Line 3	Calculate	Q4+Q14	76501
Q9	Effluent Line 4	Calculate	Q5+Q15	76501
Q10	Clarifiers outlet	Calculate	Q6+Q7+Q8+Q9-Q11-Q16	181237
Q11	RAS flow	Measure		119487
Q12	RAS to RAS tank 1	Calculate	Q11/4	29872
Q13	RAS to RAS tank 2	Calculate	Q11/4	29872
Q14	RAS to RAS tank 3	Calculate	Q11/4	29872
Q15	RAS to RAS tank 4	Calculate	Q11/4	29872
Q16	WAS flow	Measure		5279
Q17	Thickener supernatant	Estimation	0.61*Q16	3220
Q18	Dewatering supernatant	Estimation	0.9*Sludge flow	1853
Q19	Sludge effluent	Calculate	Q16-Q17-Q18	206
Q20	Influent backwash	Calculate	Q21-Q17-Q18	1830
Q21	Return liquor	Estimation	70% pump efficiency	6903

TP mass balance

Description	Distribution chamber	Aeration 1	Aeration 2	Aeration 3	Aeration 4	Sludge line	Return liquor	Clarifiers	RAS distr. Chamber	Overall System
Raw influent	760									760
Influent to Line 1	-210	210								0
Influent to Line 2	-210		210							0
Influent to Line 3	-210			210						0
Influent to Line 4	-210				210					0
Effluent Line 1		-2831						2831		0
Effluent Line 2			-2831					2831		0
Effluent Line 3				-2831				2831		0
Effluent Line 4					-2831			2831		0
Clarifiers outlet								-408		-408
RAS flow								-10395	10395	0
RAS to RAS tank 1		2599							-2599	0
RAS to RAS tank 2			2599						-2599	0
RAS to RAS tank 3				2599					-2599	0
RAS to RAS tank 4					2599				-2599	0
WAS flow						459		-459		0
Thickener supernatant						-84	84			0
Dewatering supernatant						-1	1			0
Dewatered sludge						-371				-371
Influent backwash							5			5
Return liquor	90						-90			0
Balance	10	-22	-22	-22	-22	3	1	60	0	-13
Deviation	1.2%	-0.8%	-0.8%	-0.8%	-0.8%	0.8%	-1.1%	0.5%	0.0%	-1.7%

Calibrated values

		INFLUENT	FB2 tank	RAS TANK	EFFLUENT	RET LIQ	THICK	DEWAT
COD	Biowin	531	19	26	146	508	758	81
	Measurement	531	21	27	62	586	1259	113
	Error	0.0%	9.5%	3.7%	-135.5%	13.3%	39.8%	28.3%
BOD5	Biowin	281			40			
	Measurement	273			27			
	Error	-2.9%			-48.1%			
TSS	Biowin	253	2807	6696	151	498	878	70
	Measurement	241	3210	7698	145	485	851	74
	Error	-5.0%	12.6%	13.0%	-4.1%	-2.7%	-3.2%	5.4%
VSS	Biowin	129	1631	3883	88	297	510	35
	Measurement	117	1774	3976	90	255	445	28
	Error	-10.3%	8.1%	2.3%	2.2%	-16.5%	-14.6%	-25.0%
TKN	Biowin	40	197	447	26	54	73	27
	Measurement	36	231	552	28	68	101	17
	Error	-11.1%	14.7%	19.0%	7.1%	20.6%	27.7%	-58.8%
Ammonia	Biowin	29	15	17	15	21	15	21
	Measurement	29	16	18	17	20	19	9
	Error	0.0%	-6.3%	-5.6%	-11.8%	5.0%	-21.1%	133.3%
Nitrate	Biowin	10	0.01	0.01	0.01		0.01	
	Measurement	10	0.16	0.1	0.1		0.1	
	Error	0.0%	-93.8%	-90.0%	-90.0%		-90.0%	
TP	Biowin	4	39	94	3	28	12	79
	Measurement	4	37	86	4	10	24	1
	Error	0.0%	5.4%	9.3%	-25.0%	180.0%	-50.0%	7800.0%