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Development and Evaluation of Strategies for Improving Norm Compliance for Nitrogen Compounds and Reducing Energy Consumption in Wastewater Treatment via Dynamic Simulation

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¡Muchísimas gracias a todos ustedes!

Gabriel, este libro es para ti.

Abstract

Nitrogen removal from wastewater is increasingly important to protect natural water sources and has proven a challenge for wastewater treatment plants in different countries. Strict discharge norms for nitrogen components and unfavourable wastewater quality are among the main challenges observed.

An example WWTP (450,000 $PE_{COD,120}$), representative of these challenges (i.e. strict discharge norm for NH_4-N and TN, partially unfavourable wastewater composition for upstream denitrification) was modelled with the software SIMBA. The model was calibrated, and validated, using different statistical parameters. The model was used for dynamic simulation to test different operational and automation strategies, to improve nitrogen removal.

The tested strategies considered the bypass of primary clarifiers, changes in the anaerobic, anoxic, and aerobic reactors configuration, changes in the aeration system (DO setpoint, the inclusion of online sensors and different control approaches in the aeration loop), the adjustment of the internal recirculation rate, the implementation of intermittent denitrification, among others.

The addition of an anaerobic digestion stage, considering the adjustment of the sludge age in the biological treatment and the treatment of the centrate (including nitrogen backload), was tested as well.

To evaluate the strategies' performance, an evaluation criteria chart was created to select the best strategies from an overall perspective, considering the improvements or deterioration in norm compliance, aeration requirements, pollutant emissions to the environment, and biogas production (if applicable).

The best overall results were obtained with strategies that aimed to improve the denitrification capacity (e.g. increase anoxic volume by reducing aerobic volume), adjusted the air requirements (e.g. inclusion of an NH_4-N online measurement in the aeration control loop), and provided flexibility (e.g. intermittent denitrification). With the right combination of strategies, the norm compliance was significantly improved e.g. reduced from 31 to 4 in a year, as well as the emissions to the environment.

The inclusion of an anaerobic digestion stage for sewage sludge treatment challenges the nitrogen removal even further, but similar optimisation strategies, based on the same approach were able to improve norm compliance.

However, none of the combinations, with or without anaerobic digestion, achieved total norm compliance. Therefore, a different technology than A2/O, an SBR treatment stage was designed, providing increased operational flexibility. The A2/O system in the computer model was replaced by an SBR process. This showed the best results, based on the criteria previously defined, with total norm compliance.

Based on the learnings of the design, redesign, and strategies tested, a guideline for an integral optimisation of nitrogen removal was developed, based on six pillars, considering a detailed WWTP operational analysis, the use of dynamic simulation as a tool, the testing of known and simple optimization approaches, the definition of clear and objective evaluation criteria, the consideration of anaerobic digestion (and the backload) and finally the re-evaluation of the type of technology for biological wastewater treatment.

Zusammenfassung

Die Entfernung von Stickstoff aus Abwasser wird immer wichtiger, um natürliche Wasserquellen zu schützen, und stellt für Kläranlagen in verschiedenen Ländern eine Herausforderung dar. Strenge Einleitungsnormen für Stickstoffkomponenten und eine ungünstige Abwasserqualität gehören zu den größten Herausforderungen.

Eine Beispielkläranlage (450.000 $EW_{CSB,120}$), die für diese Herausforderungen repräsentativ ist (d.h. strenge Einleitungsnorm für NH_4-N und TN, teilweise ungünstige Abwasserzusammensetzung für die vorgeschaltete Denitrifikation) wurde mit der Software SIMBA modelliert. Das Modell wurde anhand verschiedener statistischer Parameter kalibriert und validiert. Das Modell wurde für dynamische Simulationen verwendet, um verschiedene Betriebs- und Automatisierungsstrategien zur Verbesserung der Stickstoffentfernung zu testen.

Die getesteten Strategien umfassten u.a. die Umgehung der Vorklärbecken, Änderungen der Konfiguration der anaeroben, anoxischen und aeroben Reaktoren, Änderungen des Belüftungssystems (DO-Sollwert, Einbeziehung von Online-Sensoren und verschiedene Regelungsansätze im Belüftungskreislauf), die Anpassung der internen Rezirkulationsrate und die Einführung einer intermittierenden Denitrifikation. Die Hinzufügung einer anaeroben Faulungsstufe unter Berücksichtigung der Anpassung des Schlammalters in der biologischen Behandlung und der Behandlung des Zentrats (einschließlich Stickstoffrückbelastung) wurde ebenfalls getestet.

Um die Leistung der Strategien zu bewerten, wurde eine Tabelle mit Bewertungskriterien erstellt, um die besten Strategien aus einer Gesamtperspektive auszuwählen, wobei die Verbesserungen oder Verschlechterungen bei der Einhaltung der Normen, der Belüftungsanforderungen, der Stickstoffemissionen in die Umwelt und der Biogasproduktion (falls zutreffend) berücksichtigt wurden.

Die besten Gesamtergebnisse wurden mit Strategien erzielt, die darauf abzielten, die Denitrifikationskapazität zu verbessern (z. B. Erhöhung des anoxischen Volumens durch Verringerung des aeroben Volumens), den Belüftungsbedarf anzupassen (z. B. Einbeziehung einer NH_4-N-Online-Messung in den Belüftungsregelkreis) und Flexibilität zu bieten (z. B. intermittierende Denitrifikation). Mit der richtigen Kombination von Strategien konnte die Einhaltung der Normen erheblich verbessert werden, z. B. von 31 auf 4 pro Jahr, und auch die Emissionen in die Umwelt wurden deutlich reduziert.

Die Einbeziehung einer anaeroben Faulungsstufe für die Klärschlammbehandlung stellt eine weitere Herausforderung für die Stickstoffentfernung dar, aber ähnliche Optimierungsstrategien, die auf demselben Ansatz basieren, konnten die Einhaltung der Normen verbessern.

Keine der Kombinationen, ob mit oder ohne anaerobe Faulung, erreichte jedoch die vollständige Einhaltung der Norm. Daher wurde eine andere Technologie als vorgeschaltete Denitrifikation mit Bio-P Becken (A2/O), eine SBR-Behandlungsstufe, entwickelt, die eine größere betriebliche Flexibilität bietet. Das A2/O-System im Computermodell wurde durch ein SBR-Verfahren ersetzt. Dies zeigte die besten Ergebnisse, basierend auf den zuvor definierten Kriterien, bei vollständiger Einhaltung der Norm.

Basierend auf den Erkenntnissen aus der Planung, der Umgestaltung und den getesteten Strategien wurde ein Leitfaden für eine integrale Optimierung der Stickstoffentfernung entwickelt, der auf sechs Säulen beruht: einer detaillierten Betriebsanalyse der Kläranlage, dem Einsatz der dynamischen Simulation als Werkzeug, dem Testen bekannter und einfacher Optimierungsansätze, der Definition klarer und objektiver Bewertungskriterien, der Berücksichtigung der anaeroben Vergärung (und der Rückbelastung) und schließlich der Neubewertung der Art der Technologie für die biologische Abwasserreinigung.

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Abbreviations

Concepts

A2/O	Anaerobic, anoxic, aerobic, activated sludge system
ADM	Anaerobic Digestion Model
ASM	Activated Sludge Model
AT	Activated sludge tank
BOD	Biochemical Oxygen Demand
COD	Chemical Oxygen Demand
C/N	Carbon to nitrogen ratio
DO	Dissolved oxygen
DWA	German Association for Water Management, Wastewater and Waste, Registered Association (*Deutsche Vereinigung für Wasserwirtschaft, Abwasser und Abfall e.V.*)
HSG	*Hochschule Gruppe Simulation* (Simulation University Group)
HRT	Hydraulic Retention Time
ICA	Instrumentation Control and Automation
IWA	International Water Association
NH_4-N	Ammonium Nitrogen
NO_3-N	Nitrate Nitrogen
PAC	Polyaluminium chloride
PE	Population equivalents
PFS	Polyferric sulphate
SBR	Sequencing Batch Reactor
SC	Secondary clarifier
SRT	Solids retention time, sludge age
TP	Total Phosphorous
TN	Total Nitrogen
TKN	Total Kjeldahl Nitrogen
WWTP	Wastewater treatment plants

Compounds & Equations

A_{SC}	Surface of the secondary clarifier	m^2
$b_{H,T}$	temperature-dependent decay coeff. for heterotrophic biomass	1/d
C ($C_{COD,AT}$)	Total COD to biological treatment	g/m^3
DO	Dissolved oxygen in the aeration basin	mg/L
DO_{sp}	Dissolved oxygen in the aeration basin, set point	mg/L
dp	Change in the parameter value p	-
d_j	Index of agreement	-
dy	Change in the output y	-
e%	Error percentage	-
E_j	Model efficiency coefficient Nash–Sutcliffe	-
f_B	Fraction of non-volatile TSS	-
f_S	Fraction of inert soluble COD	-
f_A	Fraction of inert COD from particulate COD	-
F_{DS}	mass flow of dewatered sludge	Mg TS/d
F_{ES}	mass flow of excess sludge	Mg TS/d
F_{PS}	mass flow of primary sludge	Mg TS/d
F_{TerS}	mass flow of tertiary sludge	Mg TS/d
HRT	Hydraulic Retention time	h, d
$L_{COD,d}$	COD daily load	Mg/d
$L_{BOD,d}$	BOD_5 daily load	Mg/d
$L_{TN,d}$	TN daily load	Mg/d
$L_{TP,d}$	TP daily load	Mg/d
MLSS	Total suspended solids in the activated sludge	g/m^3
M_i	Modelled, estimated value	Unit
M_m	Mean modelled values	Unit
μ_{max}	Maximum growth rate	d^{-1}
O_i	Observed, measured values	Unit
O_m	Mean observed values	Unit
$PE_{COD,\ 120}$	Population equivalents, based on the COD value 120 g/(PE·d)	-
$PE_{BOD,\ 60}$	Population equivalents, based on the BOD value 60 g/(PE·d)	-
$PE_{TN,\ 11}$	Population equivalents, based on the TN value 11 g/(PE·d)	-
$PE_{TP,\ 1.8}$	Population equivalents, based on the TP value 1.8 g/(PE·d)	-

PF	Process factor	-
q_A	surface charge or surface feeding	m/h
Q_{air}	Air flow to the aerobic zone	Nm^3/d
Q_{biogas}	Biogas Production	Nm^3/d
Q_{ES}	Excess sludge flowrate	m^3/d
Q_{in}	Inlet flow	m^3/d
Q_{sludge}	Sludge flowrate after thickening	m^3/d
$Q_{sludge,thick}$	Sludge to thickening	m^3/d
Q_{RS}	Sludge recirculation flowrate	m^3/d
Q_{RZ}	Internal recirculation flowrate	m^3/d
Q_{out}	Outlet flow flowrate	m^3/d
R^2	Coefficient of determination	-
RMSE	Root mean square error	Unit
S	Sensitivity	-
S ($S_{COD,AT}$)	Soluble COD to biological treatment	g/m^3
S_I ($S_{COD,i,AT}$)	Inert COD to biological treatment	g/m^3
S_S ($S_{COD,biodeg,AT}$)	Easily biodegradable COD to biological treatment	g/m^3
SRT	Solids retention time, sludge age in the activated sludge tanks	d
SRT_{AD}	Sludge retention time in the anaerobic reactors	d
T	Wastewater temperature	°C
TS	Total solids	%
TS_{rem}	Total solids removal	%
TS_{Sludge}	Sludge concentration after thickening	g/l, %
TSS_{ES}	Total suspended solids in the excess sludge	g/L
T_W	Wastewater temperature	°C
T_{AD}	Temperature in the anaerobic digestion	°C
VS	Volatile solids feed sludge	%
V_{An}	Volume anaerobic tanks	m^3
V_{AT}	Volume activated sludge	m^3
V_{AT}	Volume of the activated sludge basins	m^3
V_D	Denitrification or anoxic volume	m^3
V_N	Nitrification or aerobic Volume	m^3
V_{PC}	Volume primary clarifiers	m^3
$V_{sludge,AD}$	Total sludge volume in the anaerobic reactor	m^3

$V_{reactor,AD}$	Total reactor volume	m^3
$V_{reactor,AD, corr}$	Corrected total volume of the anaerobic reactors	m^3
X ($X_{COD,AT}$)	Particulate COD to biological treatment	g/m^3
X_I ($X_{COD,i,AT}$)	Inert (slowly biodegradable) COD to biological treatment	g/m^3
X_S ($X_{COD,biodeg,AT}$)	Easily biodegradable COD to biological treatment	g/m^3

1 Introduction

1.1 Problem Definition

As countries grow in terms of population and industrialisation, the protection of water resources is increasingly critical, a situation that is and will be aggravated by the effects of climate change. Climate change modifies the hydrological cycle, altering the allocation of water, and modifying precipitation cycles and river flows (Unesco 2016); (EEA 2020). This creates water scarcity and droughts in some regions and floods in others, influencing water availability and quality, increasing the importance of water and wastewater treatment. Due to the necessary protection of drinking water sources and natural environments, especially when facing water scarcity and the effects of eutrophication, nitrogen removal regulations will continue to evolve towards stricter discharge values where they are not already stringent.

Nitrogen compounds, which contribute to the eutrophication of natural water bodies, are removed from the wastewater stream usually by biological processes in different configurations (Capodaglio et al. 2016). The main forms of nitrogen found in wastewater are ammonium/ammonia (NH_4^+ /NH_3), nitrate (NO_3^-) and organically bound nitrogen. Ammonium/Ammonia is highly oxygen-depleting, moreover, ammonia is toxic to aquatic species (Tchobanoglous op. 2014). Nitrate is also toxic in drinking water for infants and pregnant women (Sallenave 2017).

The removal of nitrogen is often among the most resource-intensive processes in wastewater treatment because resources such as electrical power for aeration and sludge and water pumping and/or recirculation are required. If the operating conditions are adverse, the costs and resource consumption (e.g. external sources of carbon) can be enormous. This is because biological nitrogen removal is usually carried out in activated sludge processes, via nitrification (aerobic process) and denitrification (anoxic process, which requires easily degradable C-sources). One of the most commonly applied strategies is upstream denitrification, where denitrification is placed before nitrification, requiring the recirculation of nitrate-rich wastewater.

An unfavourable C/N ratio for upstream denitrification is a common problem in WWTPs around the world, mainly due to long transport distances of wastewater in the sewer system and other adverse conditions (e.g. infiltration of N-rich extraneous water, pre-degradation of organic compounds e.g. in the sewer, pre-removal of organic substances e.g. use of septic tanks upstream from the sewer system). This, coupled with the fact that, especially, but not exclusively, in less industrialised countries, it is common to find a lack of instrumentation, control and automation (ICA), which does not allow the complexity of the system to be adequately managed, as several interdependent operational conditions must be met for biological nitrogen removal to occur, making it a system that must be closely monitored.

This is even more relevant today, as many advances are transforming the field of ICA. The development of cost-effective online sensors among others has made the technology available to WWTPs in less economically developed countries. The incorporation of ICA and the increase in available data can also be used beneficially, supporting and improving control systems and coordinating systems to improve energy efficiency and robustness to process disturbances (e.g. load variations, rain events, etc.). Modelling and simulation of WWTP is a useful tool in this regard, where diverse automation and control strategies can be tested cost and time-effectively.

Sewage sludge treatment and disposal are also relevant in this regard for different reasons. In China, for example, as the number of WWTP has multiplied in the last decades, and as sewage sludge is regarded separately from wastewater treatment, sustainable treatment and disposal pathways are required urgently (Liu and Han 2015)), (Lu et al. 2019), (Wei et al. 2020). Moreover, in an energy-hungry world, the inherent energy contained in sewage sludge should be exploited as a source of renewable energy.

WWTPs, which stabilise sludge aerobically (as is usual in China and other countries), could be transformed into plants that stabilise sludge anaerobically, thus covering part of their energy consumption and producing a safe product for disposal. The generated Nitrogen backload must, however, not be disregarded, and the feasibility of this shift must be carefully analysed. Further concerns about the feasibility of this technology should also be considered (e.g. economic viability, safety etc.)

The combination of stricter standards and unfavourable conditions for nitrogen removal is currently posing and will continue to pose enormous challenges to WWTPs in different parts of the world, such as increasing operational costs (chemicals, energy, etc.) and the consequent increase in wastewater treatment fees for the population. Because WWTPs are built as a long-term investment, most countries simply cannot afford to completely replace existing technologies with more efficient ones. Therefore, existing WWTPs will face increasing challenges. Fortunately, according to operational experiences in large-scale plants, e.g. in Germany, in many cases, it is possible to increase nitrogen removal sustainably, with the application of appropriate operation strategies and ICA. The application of modelling and simulation tools is useful to test optimisation strategies in WWTPs, both in operation and automation. Simulations allow this to be done at a low cost and without risk to plant operation.

This work addresses some challenges of nitrogen removal in an existing wastewater treatment plant, which can be representative of many other similar WWTP worldwide. This document will answer several questions such as:

- How would the WWTP behave under different operational conditions?
- Which operational and ICA modifications are necessary to increase norm compliance?
- How would the example WWTP behave from the perspective of different norms?
- When will it be necessary to add external carbon sources?
- What happens if the norms are sharpened?
- Which are the simplest modifications required to fulfil the norm compliance when dealing with the example WWTP?

The use of different operational and automation strategies will be tested to understand which are the simplest and easier to implement strategies that would lead to an improvement in norm compliance under different norms, looking always to increase the resource efficiency of the WWTP. The results of this work aim to help stakeholders make sustainable decisions on optimising existing plants or even building new plants.

1.2 Objectives of the Work

1.2.1 General Objective

The general objective of this work is to propose evidence-based solutions for more efficient and improved nitrogen removal in WWTP with upstream denitrification with challenges such as poor Carbon to Nitrogen ratio and strict nitrogen-compounds discharge limits, reducing at the same time the use of supplies and energy and evaluate the results from an integral perspective.

1.2.2 Specific Objectives

The specific objectives of this work are:

- Identify typical design and operational problems, especially with regard to nitrogen removal, of WWTP with activated sludge (and upstream denitrification), with the challenges mentioned above, based on an example WWTP.
- Develop strategies to increase nitrogen removal from wastewater, using fewer supplies (e.g. external carbon source and electrical energy) for WWTP with activated sludge (and upstream denitrification), based on the conditions of an example WWTP, and test them via modelling and simulation.
- Propose alternative biological treatment strategies for these wastewater characteristics (both qualitative and quantitative) to develop plans for efficient nitrogen removal, taking into account regulatory and local constraints.
- Propose a strategy to treat sludge anaerobically with energy production, improving energy efficiency and reducing the sludge dewatering and disposal effort.
- Extrapolate the obtained results by developing a brief guideline for optimizing biological nitrogen removal from municipal WWTP with the named challenges.

1.3 Research Hypothesis

For the present work, the following hypotheses are proposed:

- It is possible to overcome a large part of the challenges associated with nitrogen removal in a WWTP with simple operational and automation strategies, without the requirement of additional treatment steps (e.g. downstream denitrification, denitrification filter, etc.) or the expansion of the biological treatment step.
- The incorporation of an anaerobic sludge stabilisation stage (via anaerobic digestion of sewage sludge) in a large-scale WWTP, even when the WWTP faces challenges in nitrogen removal, is possible, and even positive in the overall WWTP operation and norm compliance.
- It is possible to improve the norm compliance in a WWTP and at the same time improve or at least not worsen the energy efficiency of the WWTP.
- The implementation of different types of treatment technologies (besides activated sludge with upstream denitrification) could be more adequate to treat wastewater under unfavourable conditions.

1.4 Methodology

The work focuses on suggesting concrete adaptation measurements for wastewater treatment plants with frequently poor C/N ratio and strict nitrogen-compounds (Total Nitrogen and ammonium-nitrogen) discharge limits, in order to improve the nitrogen compounds removal efficiency, reducing energy and resources consumption, based on the study of an example WWTP. The optimisation measurements will be supported by real data, simulation tools and modelling, but in the first stage, there will be no major modifications to the plant construction. The optimisation strategies will be based on operational and ICA strategies. The resource efficiency for nitrogen removal will be improved by considering three aspects:

- Reduce the discharge of nitrogen compounds in the treated wastewater
- Increase compliance with the local normative requirements, decreasing the number of times the discharge limits are surpassed
- Reduce the energy consumption of the biological step

After this, an alternative biological treatment strategy to improve operational flexibility, and the incorporation of anaerobic digestion as a measurement to improve the energy balance and the sludge disposal characteristics will be tested.

The procedure to be followed is described below and graphically in Figure 1. First, a literature review is conducted in order to establish the main traditional methods for nitrogen removal, and where and what are the main challenges regarding nitrogen removal in wastewater treatment plants worldwide using nitrification and denitrification. Modelling as a tool for optimisation of WWTP and the calibration and validation approach are also studied.

In the framework of the project PIRAT-Systems, there is data available from a WWTP in China, with problems in nitrogen removal, which will be used as an example. The example WWTP will be studied in detail, detecting the main operational challenges and design weak points, according to the available data. This will be done with a large set of data (three years of the full operational data set).

Based on the results of this analysis, different improvement strategies will be suggested and evaluated. Firstly, the focus will be on operational and ICA strategies to improve nitrogen removal and the resource savings potential. Secondly, a different type of technology for wastewater treatment will be tested (e.g. SBR, intermittent denitrification), for the existing wastewater.

This will allow to evaluate how the use of other technologies could improve the overall performance and efficiency of wastewater treatment. Moreover, this can provide valuable information for the technology selection for WWTP with challenging wastewater, from a nitrogen removal perspective. Moreover, the use of anaerobic digestion as a strategy to increase energy efficiency and sludge disposal characteristics will also be tested, considering that the generated backload can further worsen the C/N ratio for biological treatment. The complete analysis will be carried out as a relative comparison, comparing the results obtained in the baseline scenario (calibrated model results) with different hypothetical scenarios.

To evaluate the strategies and different technologies for wastewater treatment, modelling with the software SIMBA will be carried out, with dynamic simulations as a base. To do so, the first step is, after the mass and energy balance and subsequent operational data analysis, to use the WWTP data to calibrate the model and obtain a base model that is highly similar to the measured data of the example WWTP. A detailed plant scheme and a large amount of laboratory data, operational parameters, design information and information about the plant automation will be used.

When the models are calibrated and validated, the different optimization strategies for nitrogen removal can be tested and evaluated, under different scenarios. Then, a relative comparison can take place, by checking the plant performance and resource consumption for the treatment of the same amount and characteristics of wastewater. Another objective of the optimization is also the use of less energy to carry out the task of nutrient removal, therefore, energy consumption, in the form of air requirements, is also a parameter to be evaluated. The generated biogas and corresponding self-supply of electricity, when applying anaerobic sludge stabilisation, will also be evaluated.

The new biological treatment step, as well as the anaerobic digestion step, will be designed according to literature values and the mass balance and operational data analysis. Then models for the new biological treatment and anaerobic digestion stages are to be created, evaluated and compared with the baseline scenario. Here as well, operational and ICA strategies will be tested to improve the plant performance.

With the information gathered in the previous stages, a brief guideline will be developed, with general approaches and advice to improve nitrogen removal and consume less energy in the biological treatment in WWTP with upstream denitrification, and the option to use different technology, for wastewater with frequent low C/N ratio and strict discharge nitrogen compounds values.

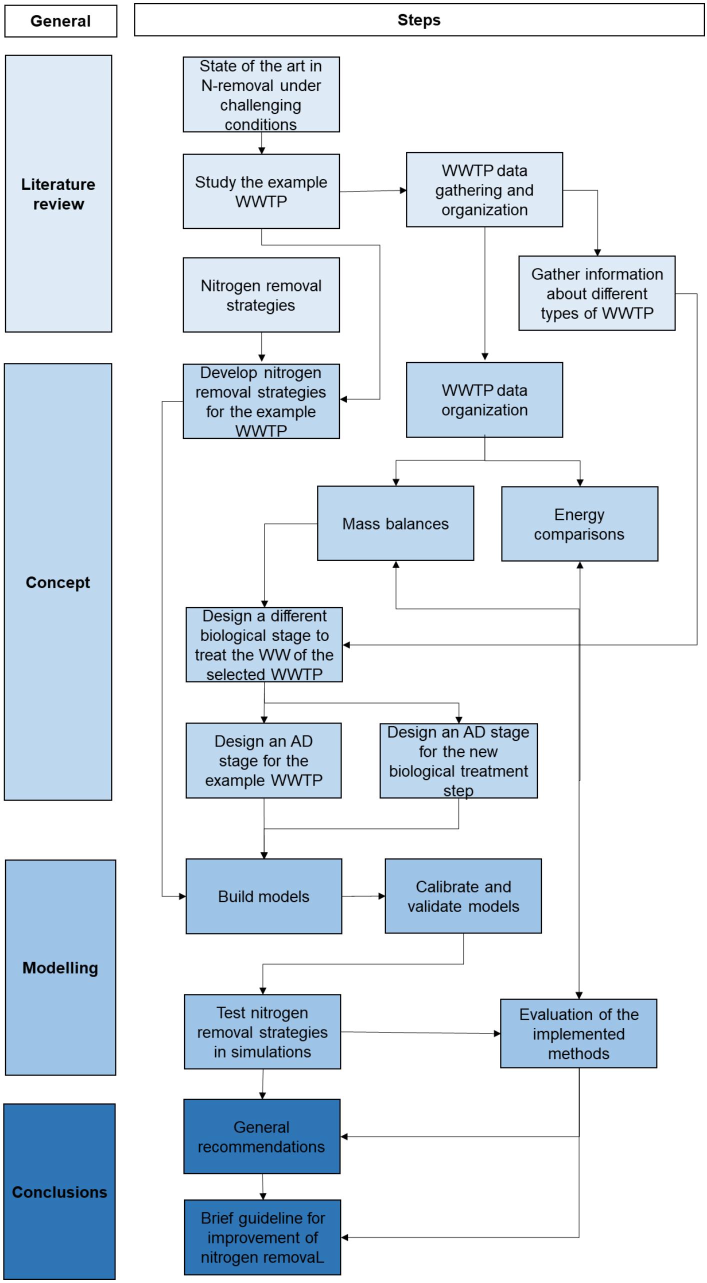

Figure 1. Methodology and procedure of the work

1.5 Publications Based on the Dissertation

Based on the work described in this dissertation, two peer-reviewed papers were published:

- Vergara-Araya, M.; Hilgenfeldt, V.; Peng, D.; Steinmetz, H.; Wiese, J. Modelling to Lower Energy Consumption in a Large WWTP in China While Optimising Nitrogen Removal. *Energies* **2021**, *14*, 5826. https://doi.org/10.3390/en14185826
- Vergara-Araya, M.; Hilgenfeldt, V.; Steinmetz, H.; Wiese, J. Combining Shift to Biogas Production in a Large WWTP in China with Optimisation of Nitrogen Removal. *Energies* **2022**, *15*, 2710. https://doi.org/10.3390/en15082710

Part of the results was also presented on several occasions in the last years:

- Oral presentation at the Shanghai IE-expo 2021, Symposium of the Sino-German Major Water Program Cooperation Theme block. "Innovative Technologies and Application on Water & Sludge Treatment & Management".
    - Title: "Dynamic modelling of wastewater treatment plants as a tool to improve operation and energy consumption".
    - Date: 21 April 2021, online.
- Oral presentation at the 27. SIMBA Treffen; Organized by ifak e.V
    - Title. „*Einsatz der Simulation zur Optimierung der Stickstoffelimination auf einem chinesischen Großklärwerk*".
    - Date: 04 May 2021, online.
- Oral presentation at the event series: „*Wasserwirtschaft im Dialog*" of the Hochschule Magdeburg-Stendal, Theme: „*Angewandte Forschung in der Abwasser-und Reststoffbehandlung*".
    - Title: „*Optimierung der Stickstoffelimination auf chinesischen Großklärwerken*".
    - Date: 23 June 2021, hybrid event.
- Poster at the IWA World Water Congress 2022.
    - Title: "Switching from Aerobic to Anaerobic Sludge Stabilization on a Chinese WWTP and Optimization of Nitrogen Removal" (Poster).
    - Date: 11-15 September 2022, Copenhagen, Denmark
    - Note: presented by MSc. Verena Hilgenfeldt (TUK)

2 Literature Research

2.1 Nitrogen and Nutrients in Wastewater

Nutrients are essential for the growth of microorganisms, plants and animals and are also required for biological wastewater treatment. Nutrients are normally limiting growth factors in aquatic ecosystems, Nitrogen is typically limiting in estuarine and marine systems and Phosphorus in freshwater systems (EPA 2009). However, wastewater contains usually high concentrations of both nutrients, contributing to an excess of nutrients discharge in the receiving water body, leading to accelerated eutrophication, i.e. anthropogenic nutrient enrichment in aquatic ecosystems. The undesired consequences of an increased nutrient loading include excessive growth of phytoplankton and macroalgae, sun blocking to submerged aquatic vegetation, decomposition of dead algae and phytoplankton with the consequence of low to no dissolved oxygen concentrations in deeper layers, loss of aquatic vegetation and fish and invertebrate kills (EPA 2009). Due to its negative effects on water bodies, nutrient removal is desirable and, in most cases, mandatory before discharging wastewater in water bodies after wastewater treatment processes worldwide.

2.1.1 The Nitrogen Cycle

The nitrogen cycle operates in the biosphere, in addition to the global carbon and oxygen cycles. These cycles, which ultimately involve all species, depend on a proper balance between the activities of the producers (autotrophs) and consumers (heterotrophs) in the biosphere (Boyle 2005). Even though nitrogen as dinitrogen gas (N_2), represents around 79% of the atmosphere, it is inaccessible in this form to most organisms, making nitrogen a scarce resource in soil and water, acting as a limiting factor in many ecosystems. Only when nitrogen is converted from dinitrogen gas into ammonia (NH_3) it becomes available to primary producers (Bernhard 2010). The nitrogen cycle describes the interactions between the different nitrogenous compound forms in nature (see Figure 2).

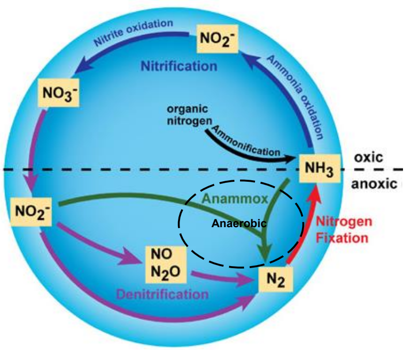

Figure 2. Nitrogen cycle (Bernhard 2010)

Aerobic, nitrifying bacteria and archaea can oxidize ammonium when released to the environment, either to nitrite (NO_2^-) or nitrate (NO_3^-). Under anoxic conditions, nitrate and nitrite can be reduced to dinitrogen gas through denitrification. Nitrite can also be combined with ammonium under anaerobic conditions to produce nitrogen gas in the anammox reaction (Kartal et al. 2010).

2.1.1.1 Nitrification and Denitrification

Nitrification is the sequential oxidation of ammonia to nitrate with intermediate nitrite formation (Chamy 2008). The process occurs in presence of oxygen and with the consumption of oxygen and is carried out by bacteria of the family *Nitrobacteriaceae* mainly by ammonium (AOB) and nitrite-oxidizing bacteria (NOB). The chemical equations describing this process are:

- Nitritation (Ammonia oxidation): $NH_4^+ + 1.5 \cdot O_2 \rightarrow 2 \cdot H^+ + H_2O + NO_2^-$
- Nitratation (Nitrite oxidation): $NO_2^- + 0.5 \cdot O_2 \rightarrow NO_3^-$

The oxygen required for the complete oxidation of ammonia to nitrate is 4.57 g O_2/ g N, but when cell synthesis is considered, the amount is reduced because oxygen is also obtained from the fixation of CO_2 into cell mass. Additionally, a large amount of alkalinity is required: 7.14 g of alkalinity (as $CaCO_3$) per gram of ammonia nitrogen converted (Tchobanoglous et al. 2003).

Nitrification is carried out by autotrophic bacteria i.e. use inorganic carbon for their synthesis processes and lithotrophs, as they obtain energy from inorganic compounds (Chamy 2008). Because nitrifying bacteria grow much slower than heterotrophic bacteria, systems designed for nitrification require much longer hydraulic and solids retention times than those systems designed only for organic carbon removal (Tchobanoglous et al. 2003).

Temperature is one of the most important factors determining nitrification, with optimal nitrification temperatures in the range of 25–28 °C. Temperatures below 8–10 °C mainly inhibit the second phase of nitrification, because *Nitrobacter sp*. is more sensitive to environmental conditions, resulting in the accumulation of nitrites in the system (Rodziewicz et al. 2019). Nitrifying bacteria are sensitive to several other environmental factors. Optimal nitrification rates are obtained at pH values between 7.5 and 8.0. At pH values below 6.8 nitrification rates decline significantly (Tchobanoglous et al. 2003).

Nitrifiers are also sensitive to a wide range of organic and inorganic substances. This makes it difficult to identify the source in case a nitrification inhibition is suspected and intensive sampling and tests are required (Tchobanoglous et al. 2003).

Denitrification is a process carried out by the respiration of heterotrophic microorganisms, which, under anoxic conditions, substitute oxygen with nitrate as an electron acceptor for the oxidation of organic matter (Chamy 2008). It is also referred to as nitrogen-oxide gasification. If we consider the term $C_{10}H_{19}O_3N$, which is often used to represent the biodegradable organic matter in wastewater, the chemical equation describing this process is (Tchobanoglous et al. 2003):

- Denitrification: $C_{10}H_{19}O_3N + 10\ NO_3^- \rightarrow 10\ CO_2 + 3\ H_2O + NH_3 + 10\ OH^- + 5\ N_2\uparrow$

The reduction of nitrate to molecular nitrogen is carried out in consecutive steps, with the production of several undesirable sub-products such as nitrite (NO_2^-), and nitric oxide (NO), which can inhibit the denitrification and nitrous oxide (N_2O), which has a high global warming potential. To transform nitrate to nitrogen gas, an equivalent of 2.86 g O_2/g NO_3-N is required, considering an equivalence of 0.25 moles of oxygen to 0.2 moles of nitrate for electron transfer in oxidation-reduction (Tchobanoglous et al. 2003).

In the heterotrophic denitrification reaction, one equivalent of alkalinity is produced per equivalent of NO_3^-N reduced i.e. 3.57 g of $CaCO_3$. This means, only about half of the alkalinity consumed in nitrification can be recovered in denitrification (Tchobanoglous et al. 2003). Wastewater has typically enough buffer capacity, but if it is low, the addition of substances to re-establish alkalinity e.g. lime is required. Most denitrifiers are facultative aerobic organisms i.e. they can use oxygen as well as nitrate or nitrite or oxygen, and some can also carry out fermentation under anaerobic conditions, in the absence of nitrate or oxygen (Tchobanoglous et al. 2003).

A basic requirement for denitrification is the availability of carbon sources because the bacteria carrying out the process are heterotrophs e.g. require organic carbon for their synthesis processes and metabolism. Therefore, in wastewater treatment technologies, by feeding raw wastewater into denitrification, the already available substrate will be used, avoiding the use of external carbon sources that can be very cost-intensive. However, if due to characteristics of the wastewater, or even due to poor plant operation, there is an unfavourable C/N ratio (COD/TN < 10:1), external C-sources may be required.

For nitrate to be used as the final electrons' acceptor, anoxic conditions must be provided, i.e. denitrification can be inhibited by the presence of dissolved oxygen.

2.1.1.2 Other strategies for nitrogen removal

Besides the typical nitrification/denitrification process, there are also other biological paths for nitrogen removal, which have gained importance in wastewater treatment in the last decade. Due to their complexity or special required conditions, most of them are only used to manage rich N side-streams or partial treatment of the mainstream.

Deammonification based-systems (Anammox)

Deammonification is the partial nitritation and anaerobic ammonia oxidation (Anammox) to form nitrogen gas. The anammox process is the anaerobic ammonium oxidation or coupled nitrification–denitrification, carried out by the Anammox bacteria. Its application in wastewater treatment has shown higher nitrogen removal and lower energy requirements than in conventional nitrogen removal (i.e. nitrification/denitrification) (Cho et al. 2020).

The application of anammox-based processes is usually adequate for treating warm wastewater with high ammonium content (Kartal et al. 2010); (Cho et al. 2020); (Ronan et al. 2021). This is one of the most widespread types of side-stream treatment for process water in WWTP with anaerobic digestion (Bachmann 2015).

The process is used in several full-scale WWTP worldwide (Lackner et al. 2014, DBU 2004), proving to be a good alternative to reduce the effects of the backload after anaerobic sludge stabilisation. Nitrogen removal rates between 46% and 94% are informed in the literature depending mostly on the nitrogen loading rate, temperature, DO and carbon content (Cho et al. 2020). However, the most common rates are between 50% and 80%.

Since it is an anaerobic process, the growth rate of the anammox bacteria is slow and the process is sensitive to temperature, therefore the start-up and operation of the process can be challenging.

The process can also be applied for the treatment of the mainstream in a WWTP, without forgetting that the process requires high ammonia concentrations and that at 35 °C, the growth rate is 12 to 15 days. This application has shown several challenges, mostly related to longer start-up periods, and inconsistent (variable) loading rates (e.g. due to low nitrogen concentrations), which can make the process inflexible and the effluent concentrations unstable (Cho et al. 2020).

The application of anammox in the side-stream will be briefly considered later (see Chapter 6.4.2.4), but its application for the mainstream is out of the scope of this work.

Nitritation/denitritation based-systems

With the incomplete oxidation of ammonia to nitrite (nitritation), avoiding the formation of nitrate, the direct reduction of nitrite to nitrogen gas (denitritation), nitrogen removal is possible. The application of this kind of treatment system is also limited to ammonium-rich side streams.

2.1.2 Nitrogen Removal in WWTP

The most common forms of nitrogen in wastewater treatment are organic nitrogen, ammonia (NH_3), ammonium (NH_4), nitrite (NO_2^-), nitrate (NO_3^-) and nitrogen gas (N_2) (Tchobanoglous et al. 2003). Nitrogen removal from wastewater is a biological process that usually involves nitrification and denitrification.

Besides the typical nitrification/denitrification process, there are other process conditions and strategies to achieve nitrogen removal in wastewater. Due to their complexity or special required conditions (e.g. requirement of pure cultures of specific bacteria, slow-growing bacteria, difficulties in the start-up of plants, etc.), most of them are only used to manage rich N side-streams, including Anammox systems (anaerobic ammonium oxidation), or Nitritation/denitritation based-systems (see Chapter 2.1.1.2). However, these technologies are out of the scope of this work.

2.1.2.1 Configurations for Nitrogen Removal

Biological nitrogen removal can be achieved with different treatment technologies and in different combinations. Single-sludge biological nitrogen removal processes are grouped according to whether the anoxic zone is located before, after or within the aerobic nitrification zone (Tchobanoglous et al. 2003). The main configurations can be observed in Figure 3 and are ((DWA 2016):

- Upstream denitrification: anoxic tank followed by the aeration tank
- Cascade denitrification: Two or more aeration tanks, whereas with pre-denitrification or simultaneous denitrification, are operated in series, where the outlet flow of the first set of tanks is the inlet flow of the second set (see Figure 3 (b)).
- Simultaneous denitrification: nitrification and denitrification is occurring in a single basin, and the water flows through denitrification and nitrification zones in the tank (see Figure 3 (c))

- Alternating denitrification: Two intermittently aerated tanks are fed in alternated succession, whereby water is fed from one non-aerated tank into the aerated tank
- Intermittent denitrification: In a single basin, the nitrification and denitrification phases alternate in time. Each phase duration can be set with a timer or by control strategies e.g. measuring nitrate or ammonium content, redox potential or oxygen consumption.
- Downstream denitrification: the denitrification tank is downstream of the nitrification tank (see Figure 3 (d))

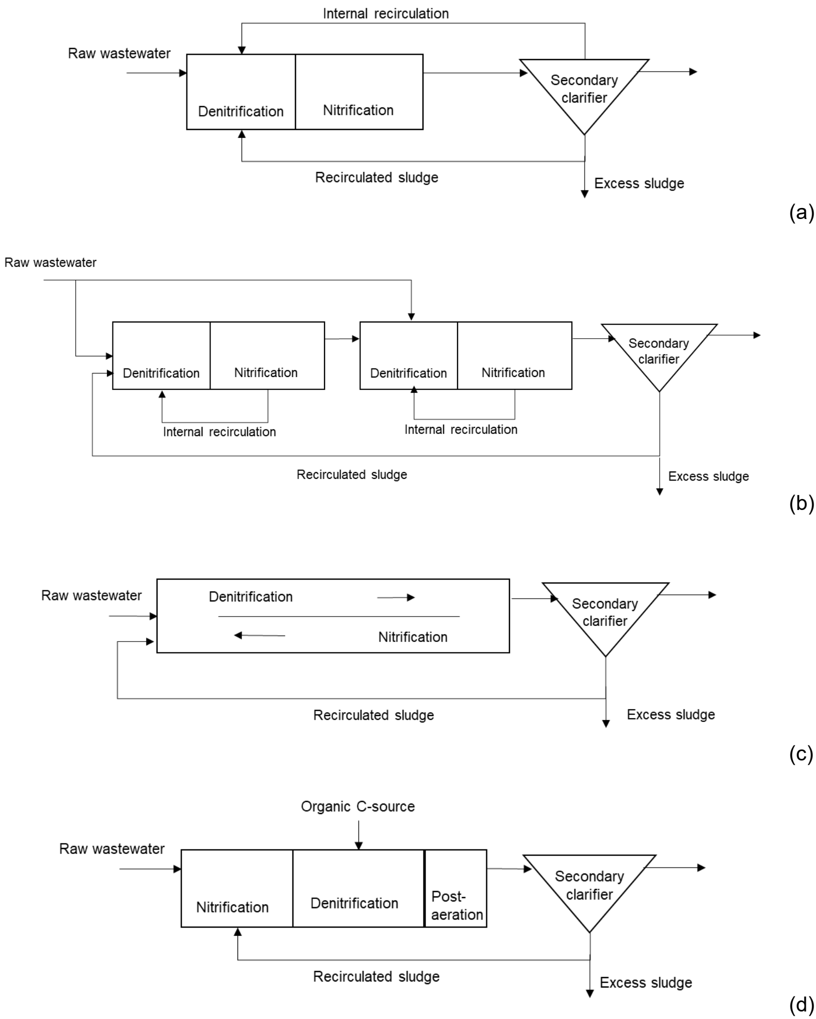

Figure 3. Scheme of different configurations for nitrogen removal in single sludge treatment systems. (a) Pre-denitrification; (b) Cascade denitrification; (c) Simultaneous denitrification; (d) Post-denitrification (own elaboration based on (DWA 2016))

For this work, only upstream denitrification and intermittent denitrification will be detailed, as they will be applied and discussed in the following chapters.

2.1.2.1.1 Upstream-Denitrification

The process consists of an anoxic tank followed by an aeration tank where nitrification takes place (see Figure 3 (a)). Nitrate produced in the aeration tank is recycled back to the anoxic tank, a process called internal recirculation, whilst the organic substrate in the influent wastewater provides the electron donor for oxidation-reduction reactions using nitrate (Tchobanoglous et al. 2003). The process is also named substrate denitrification or pre-anoxic denitrification and the internal recirculation must be limited only to the necessary amount to minimise the impairment of the denitrification by high loads of dissolved oxygen (DWA 2016). The disadvantage of this process is that complete denitrification is never possible because the nitrate concentration in the effluent is always approximately the same as in the recirculated wastewater (Gujer 1993).

This process configuration is the most commonly used for biological nitrogen removal in WWTP (McCarty 2018) due to the availability of easily degradable carbon sources in raw wastewater, the relatively simple retrofitting of existing plants and the production of alkalinity before the nitrification step (Tchobanoglous et al. 2003).

To increase operational flexibility, the last sections of the denitrification tank can also have aeration elements (DWA 2016).

2.1.2.1.2 Intermittent Denitrification

In a single basin, the nitrification and denitrification phases alternate in time. Each phase duration can be set with a timer or by control strategies e.g. measuring nitrate or ammonium content, redox potential or oxygen consumption. The process can be considered a completely mixed reactor. (DWA 2016).

This type of strategy has been applied successfully in large-scale WWTP in Germany (e.g. WWTP Hillersleben (Saxony-Anhalt), WWTP Florstadt (Hessen), WWTP Obere-Lutter (Nordrhein-Westfalia), among others).

2.1.2.2 Biological Treatment Stage Configuration

Different configurations for nitrogen removal can be applied to different biological treatment stage configurations. Since the possibilities are multiple, the focus will be set on two types of configurations A2O and SBR, which are the ones applied in the following chapters.

2.1.2.2.1 A2/O

The anaerobic-anoxic-aerobic (A2/O) configuration consists of three tanks. The first one is anaerobic and will serve to provide conditions for biological phosphorous removal. The second tank is anoxic, and as it is upstream from the aerated tank, it provides conditions for a pre-denitrification type nitrogen removal. This configuration has the internal recirculation of nitrate-rich activated sludge mixture from the aerobic to the anoxic tank, and the return of sludge to the anaerobic tank.

2.1.2.2.2 Sequencing Batch Reactor (SBR)

The principle of the Sequencing Batch Reactor process is based on the fact that all steps of the wastewater treatment process in a reactor are carried out in a certain chronological order, one after the other. As a rule, the raw wastewater is fed to the reactor discontinuously and the treated wastewater is withdrawn discontinuously (Wiese 2014). This means that, unlike continuous flow systems, in SBR the filling rate is a variable of the process and the time period assigned to each part of the process determines the biochemical reactions that occur, within the physical limitations of the reactor (Silverstein and Schroeder 1983).

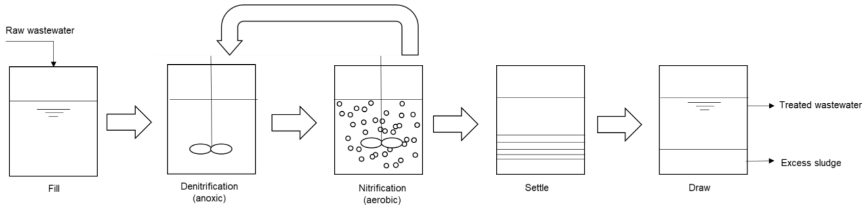

Figure 4. Scheme of the SBR process (own elaboration)

The process starts with the filling of the reactor. The reactor contains suspended biomass to treat the wastewater. Then comes the reaction phase, which can include nitrification, nitrification and denitrification or other. Depending on the operational strategy, the reactor can be continuously fed, up to the settling stage or repeat the reaction (denitrification-nitrification) cycle. After the sludge has settled, the treated wastewater is retrieved from the tank as well as the excess sludge. After the drawing stage, a pause can follow to re-start the cycle, by filling wastewater.

SBR operation can be defined by the duration time of each stage. To carry out denitrification, there are several operational strategies to follow. In the case of continuous feeding, two or more nitrification and denitrification phases can be planned (similar to simultaneous denitrification in oxidation ditches). If two (or more) tanks are alternately fed, the denitrification phase should be set at the beginning of the feeding (similar to intermittent denitrification). Upstream denitrification corresponds to systems with batch feeding, if the nitrogen elimination is not sufficient, is necessary to feed two (or more) times.

To assure sufficient nitrification, enough aeration must be assured between the last loading and the beginning of the sedimentation phase. In case of insufficient N-elimination caused by a temporarily unfavourable C/N ratio, it is always necessary to have the possibility to dose external C-sources, just as in conventional activated sludge systems (DWA 2009).

SBR have the advantage of a lower area footprint, because the mixed liquor remains in the reactor, eliminating the requirement of a separated secondary clarifier stage and sludge recirculation (Tchobanoglous op. 2014). SBR have also the option of a much more flexible operation due to its single-tank configuration. In comparison to CAS, the SBR process allows for the following adjustments to be made: total cycle duration, duration of each phase within the process cycle, the pattern of inflow, dissolved oxygen profile during aeration, operating top water level, and operating bottom water level (IWA 2014). This gives the WWTP more operational flexibility, which can be advantageous in the case of the influent wastewater in the example WWTP.

The SBR process is usually preceded by a typical pre-treatment system, involving screens and sand traps. To accommodate the continuous inflow of wastewater, the SBR system generally comprises either a storage/equalization tank and a single SBR tank or a minimum of two SBR tanks (IWA 2014). In the case of mixed sewage systems, it is also usual to have an equalisation tank.

SBR-based systems can be very complex, especially with a large number of reactors, and an adequate automation and control strategy is key for the proper functioning of the system. There are basically two approaches to automate different aspects of an SBR biological treatment step:

- **Fixed time-based sequential control (TSC**). Each stage of the plant operation occurs in a fixed time sequence, regardless of the operational conditions and sensor information. One of the main advantages of SBR is its flexibility, however, there are SBR plants that use fixed time-based sequential control (TSC), which cannot react flexibly (Steinmetz and Wiese 2006). For example, most SBR-based WWTP installed in China in the decade 2000 to 2010 used a time-based approach (Yang et al. 2010).
- **Real-time-control (RTC):** SBR have shown great success in implementing real-time control systems for nitrogen removal (Zanetti et al. 2012). With the help of modern supervisory control and data acquisition (SCADA) systems and dynamic Real-time control, it is possible to operate SBR plants more effectively (Steinmetz and Wiese 2006). Real-time control has proved to be an efficient way to increase process performances since it allows for improvement of the effluent quality, decreases energy consumption and increases the specific amount of wastewater treated (Zanetti et al. 2012); (Piotrowski et al. 2019).

2.1.3 Operational Conditions and Parameters for Nitrogen Removal

To remove nitrogen from wastewater via nitrification and denitrification, several operational conditions must be fulfilled. The rate of denitrification is affected mainly by the readily biodegradable COD concentration in the influent wastewater, the available biomass in the activated sludge system and temperature (Tchobanoglous et al. 2003). The main conditions to achieve conventional biological nitrogen removal are well known (Daigger and Littleton 2014) and are listed here and detailed in the next subsections.

- Adequate Carbon to Nitrogen (C/N) ratio and sufficient readily biodegradable carbon in the influent.
- An aerobic zone with a sufficiently long SRT and other environmental conditions sufficient to allow the growth of nitrifying bacteria (e.g. alkalinity, temperature).
- Recirculation of nitrate-rich water from the aerobic to the anoxic zone.

2.1.3.1 C to N Ratio and Readily Biodegradable COD in the Influent

Since heterotrophic bacteria are responsible for denitrification (see Chapter 2.1.1.1), organic matter is required as an electron donor for denitrification. The adequate C:N:P ratio for aerobic wastewater treatment, or microbial demand for nutrients, should be between 100:10:1 and 100:5:1 (Winkler 2012) (Permatasari et al. 2018); (Wang et al. 2020). This is the ratio required for healthy microbial growth in the activated sludge system, avoiding nutrient limitations. Different authors (see Table 1) inform values for the required COD/TN ratio from 4:1 to 8:1 as a minimum for denitrification.

Table 1. Required C/N ratio for denitrification according to different authors

C/N for denitrification	Notes	Source
3 – 7 (5 ideal)	Isolated strain ZY04 of *Acinetobacter johnsoni*	(Zhang et al. 2019)
4 – 8	Used in practice for the addition of external C-sources	(Tchobanoglous op. 2014)
4.6 – 6.7	COD/NO_3-N	(Yuan et al. 2017)
5.0 – 6.6	BOD/NO_3-N = 2.3 and COD/BOD = 2.17 – 2.85	(Narkis et al. 1979)
5.25	CH_3OH/NO_3-N = 3.5, at pH 7	(Timmermans and van Haute 1983)
6.8	COD /N = 6.8 and DO = 0.5 mg/L	(Zielinska et al. 2012)
7.06	Stoichiometric ratio	(Kim et al. 1997)
7.6	Use of different carbon sources: glucose, sodium acetate and methanol	(Sobieszuk and Szewczyk 2006)

Several operational measurements can be taken to counteract this problem, which will be tested and described in the following chapters. If these measurements fail to improve denitrification, external C-sources may be required, such as methanol, ethanol and acetic acid (DWA 2016), industrial residues from industries (e.g. food and beverages industry), or even hydrolysed or acidified primary sludge (Winkler 2012). However, the use of external carbon sources can be very cost-intensive and should be therefore avoided, reduced to a minimum or strongly optimized.

2.1.3.2 Sludge retention time (SRT) and hydraulic retention time (HRT)

Nitrifying bacteria grow slowly compared to heterotrophic microorganisms and are highly dependent on temperature. Therefore, their growth is used as a reference to control the sludge age, and the temperature is used for its calculation, as described in Table 2.

Table 2. Sludge age calculation according to DWA-A 131 (DWA 2016)

Sludge age required for WWTP with	Temperature	Value or Formula	Equation
Aerobic sludge stabilisation, with nitrification	< 12 °C	20 d	-
Aerobic sludge stabilisation, with denitrification	> 12 °C	25 d	-
Aerobic sludge stabilisation	> 12 C°	$\mathrm{SRT} = 25 * 1.072^{(12-\mathrm{T})}$	Equation 1
Anaerobic sludge stabilisation	-	$\mathrm{SRT} = \mathrm{PF} * 3.4 * 1.103^{(15-\mathrm{T})} * \frac{1}{1-\left(\frac{\mathrm{V_D}}{\mathrm{V_{AT}}}\right)}$	Equation 2

Where:

SRT = Sludge age, solids retention time, d ; T = Wastewater temperature, °C

PF = Process factor –; ; V_D = Denitrification volume, m^3

V_{AT} = Activated sludge volume, m^3

However, if the sludge will later undergo an anaerobic stabilization process, the sludge age must be maintained low to avoid further organic matter consumption, favouring biogas production. This is called dynamic sludge age adjustment. An adequate sludge age also assures sufficient biomass concentrations of nitrifying and denitrifying. Biomass enrichment is achieved by retaining biomass or by intensive sludge recirculation.

Internationally, it is also recognized that the sludge age is temperature dependent, for example in Figure 5, according to Suez.

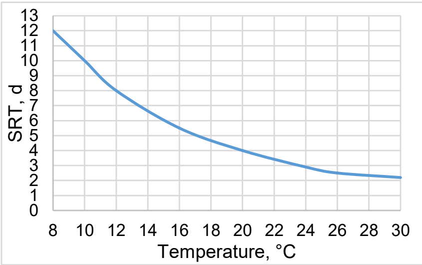

Figure 5. Suggested aerated sludge age required for nitrification according to Suez (based on (SUEZ 2007))

However, the information provided in the graph does not consider the size or proportions of the nitrification tanks, nor the type of sludge stabilisation. Other literature sources, simply provide ranges for the adequate SRT, depending on the activated sludge variation used (Tchobanoglous op. 2014), as summarized in Table 3.

Table 3. Suggested sludge age (SRT) and hydraulic retention time (HRT) according to Metcalf & Eddy (Tchobanoglous op. 2014)

Process	SRT d	HRT h		
		Total	Anoxic zone	Aerobic zone
Upstream denitrification (modified Ludzack-Ettinger process)	7 – 20	5 – 15	1 – 3	4 – 12
SBR	10 – 30	20 – 30	Variable	Variable
Oxidation ditch	20 – 30	18 – 30	Variable	Variable

The hydraulic retention time for denitrification must be sufficient to avoid incomplete denitrification, which would lead to high effluent nitrate concentrations. In suspended growth systems, anoxic-HRT of around 1–4 hours is typically used (Tchobanoglous et al. 2003). However, if the denitrification volume is too large, e.g. larger than 50% of the total volume, problems with sludge settling may arise, as the sludge settleability is worsened (Henze 2000).

2.1.3.3 Alkalinity

Alkalinity is the sum of hydroxides (OH^-), carbonates ($CO^{2-}{}_3$), and bicarbonates (HCO_3^-), representing all substances that can counteract hydrogen protons in a solution

(Tchobanoglous et al. 2003) measured in units of $CaCO_{3,\ eq}$ /L. Therefore, alkalinity can be defined as the capacity of a solution to neutralize hydrogen ions,

As mentioned in previous chapters, nitrification and denitrification processes require neutral pH to function optimally (Chapter 2.1.1.1), and the biological nitrogen removal nitrification and the denitrification-based process will have a balance tending to the accumulation of Hydrogen protons (H^+), because only about half of the alkalinity consumed in nitrification can be recovered in denitrification (Tchobanoglous et al. 2003). This can lead to an acidification of the solution.

That is why sufficient alkalinity, is of sum importance for the biological nitrogen removal process. If the alkalinity is consumed, the pH of the water solution will shift rapidly, moving away from the required neutrality. The residual alkalinity required to maintain a pH close to neutral, is around 70 – 80 $CaCO_{3,\ eq}$ /L (Tchobanoglous et al. 2003).

WWTP usually can dose substances to increment the alkalinity of the biological system, dosing for example lime. Regular measurement of alkalinity is therefore recommended.

2.1.3.4 Internal recycle rate and return sludge flow

Since denitrification requires nitrate as an electron acceptor, in the case of upstream denitrification, internal recirculation provides the required nitrate. Rates around 100 and 400% as a percentage of the average influent flow rate are used in suspended growth systems (Tchobanoglous et al. 2003). If the internal recirculation rate is too low, there is a risk of incomplete denitrification, if it is too high, the biological treatment could be destabilised, as the hydraulic retention time is shortened, load peak surges can emerge; moreover, unnecessary energy is used.

The return sludge flow controls the sludge concentration in the activated sludge tanks and assures the required biomass concentration and sludge age. Typically, are rates around 50 – 75% of the average design flow rate (Tchobanoglous et al. 2003).

2.1.3.5 Adequate aerobic and anoxic conditions

For denitrification, anoxic conditions are required. Dissolved oxygen (DO) inhibits denitrification, even at very low concentrations such as 0.2 mg O_2/l (Oh and Silverstein 1999). However, it must be taken into account that, anoxic regions can exist in an activated sludge floc and these micro-environments allow simultaneous nitrification in the outer region in contact with bulk water DO and denitrification in an inner anoxic region (Oh and Silverstein 1999). In denitrification, nitrate act as an electron acceptor providing chemically bounded oxygen. Newer studies have shown that the presence of higher concentrations of DO is not as critical, and that denitrification can still occur (Zhang and Zhang 2018); (Ji, et al 2015).

For nitrification, sufficient DO, at least 0.3 mg O_2/L and up to 4 mg O_2/L (Stenstrom and Poduska 1980) can be present, but typical operational values are around 1.5 – 3 mg O_2/L. The most economical operational values are however between 1.5 and 2 mg O_2/L (Barfüßer 2018). Higher concentrations may be used, but values above 4 mg O_2/L have not shown improvement in operation (Tchobanoglous et al. 2003). Nevertheless, DO concentrations below 1 or 0.8 mg/L, during nitrification can lead to emissions of N_2O, a gas with a huge

Greenhouse emissions potential, around 300-fold higher than CO_2 (Kampschreur et al. 2009); (Pinnekamp et al. 2017).

Air is artificially provided, with blower and aeration elements, and DO in the nitrification tank is usually controlled. Air control is one of the most relevant control parameters in an activated sludge system. The use of different online sensors i.e. Ammonia, Nitrate, redox potential, and others can also be added to control the nitrification performance. The incorporation of ammonia (and also nitrate) sensors can contribute to providing air only when necessary, indicating e.g. when all the ammonia has been oxidized or when the nitrate concentration increases, and vice versa. The use of redox sensors is a good indicator of the required aerobic and/or anoxic conditions. Different sensors can be used in combination, which is recommended, as different sensors will complement the information provided.

2.1.3.6 Temperature on Biological Activity and Nitrogen Removal

The temperature of domestic wastewater is variable and depends on the region and the season. In Germany, typical temperatures fluctuate between 8 and 20 °C, with clear differences between winter and summer. In warmer and/or tropical regions, temperatures between 20 and up to 35 °C can be found (e.g. Iran, Mexico, United Arab Emirates). In colder regions, the temperatures can reach a few degrees up to 16 °C (e.g. Norway) (DWA 2017). Typical design guidelines and activated sludge models have valid with temperatures between 8 and 20 °C (e.g. DWA-A 131, ASM1 to ASM3).

Temperature, as it influences the rates of biological reactions, affects the performance of biological systems. The maximum operating temperature for typical activated sludge systems is limited to about 35° to 40 °C, the maximum temperature for the growth of mesophilic organisms. Since the thermal inactivation of mesophilic bacteria occurs rapidly, short-term temperature variations above this spectrum must be avoided (Grady et al. 1999). Temperatures above 35 °C in aeration basins cause often dispersed growth of floc-forming and filamentous organisms (Jenkins et al. 2004).

Aerobic fermentation and nitrification stop with temperatures above 50 °C (Metcalf et al. 2004). But the floc formation is negatively influenced already by 32 °C. Due to the increased biological activity, there is an accumulation of insoluble secretions (e.g. oils and lipids), that result in entrapped in the sludge flocs. These secretions can also entrap air bubbles that make the sludge settling difficult (Gerardi 2003). In the lower range, the autotrophic-nitrifying bacteria practically cease functioning with temperatures below 5 °C and at even lower temperatures (from 2 °C), the carbon oxidizing bacteria become essentially dormant (Metcalf et al. 2004).

As mentioned before, the temperature is one of the most important factors determining nitrification, with optimal nitrification temperatures in the range of 25–28 °C. Temperatures below 8–10 °C mainly inhibit the second phase of nitrification (Rodziewicz et al. 2019). The influence of the temperature decreases with the increment of the sludge age, and its influence in systems with high sludge ages is almost negligible. Additionally, at high temperatures, the adaptation of the microorganisms to rapid temperature changes seems to be more slowly (Sperling 2007).

2.1.3.7 Sludge stabilisation

Sewage sludge is stabilised to reduce pathogens, reduce odours and reduce potential putrefaction, and it is achieved by reducing the organic fraction of the sludge (Volatile solids, VS) (Tchobanoglous op. 2014). Sewage sludge stabilisation is usually a legal requirement for its disposal and can take several forms, depending on the regulations: chemical, aerobic, or anaerobic stabilisation.

Chemical or alkaline sludge stabilisation is carried out by the addition of lime, rising the pH to 12 or higher accompanied by a temperature raise due to the exothermic reaction. The sludge is therefore unsuitable for the growth of microorganisms and pathogens. Due to the high chemical dosing, required, this type of sludge stabilisation is carried out usually in small WWTP, or as post-treatment.

Simultaneous aerobic sludge stabilisation is carried out in WWTP with activated sludge systems, by maintaining an adequate (temperature-dependant) sludge age (or SRT)) that allows the organic matter to be degraded inside the activated sludge basins. In Germany, for temperatures below 12 °C, the SRT must be >25 d. For higher temperatures, lower SRT can apply (DWA 2016). However, depending on local regulations in different countries, aerobic digestion may apply, and target values for organic matter degradation may apply or different retention times (e.g. 40 d at 20 °C and 60 d at 15 °C in the USA) (Tchobanoglous op. 2014).

Anaerobic sludge stabilisation is carried out in a separate process. The sewage sludge is thickened to >3% TS (usually 5 to 6%) and then feed to one or more mesophilic or thermophilic anaerobic reactors, for an HRT of at least 10 days (Tchobanoglous op. 2014). In practice, HRT is generally between 12 and 28 days depending on the configuration of the reactors (parallel, series, etc. (DWA 2014). The anaerobic fermentation produces biogas and stabilised sludge. The biogas, rich in methane, can be used to produce heat and power. Moreover, the sludge volume and mass are reduced.

However, when a WWTP has an anaerobic digestion stage, the system must be able to handle the generated nutrients backload coming from the mixed sludge liquor. In anaerobic fermentation, there is a release of ammonia-nitrogen and partly phosphorous (especially when P is removed biologically) due to the degradation of organic matter under reductive conditions. The COD backload from sludge liquor is negligible, as the anaerobic digestion process degrades organic matter intensively, and the remnant is mostly inert matter (Fimml 2010).

According to the DWA-A131 (DWA 2016), the proportion of nitrogen released as NH_4-N during digestion can be approximately estimated as 50% of the nitrogen incorporated in the biomass. Another source indicates that this backload can be estimated as 1.5 g N/(PE·d) (Fimml 2010).

Aerobic and anaerobic sludge stabilisation are the most common type of sludge stabilisation used and here, the plant size is one of the determining factors. For example, in Germany, it is common that WWTP below 20,000 PE, are designed for aerobic sludge stabilisation and those above 30.000 PE for anaerobic sludge stabilisation (see Figure 6).

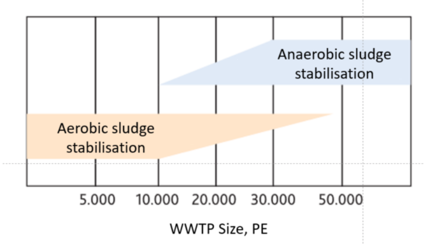

Figure 6. Typical use of aerobic and anaerobic sludge stabilisation in WWTP in Germany (from (Gretzschel et al. 2014)

Due to the favourable current conditions, e.g. availability of suitable technologies, high energy costs, high sludge disposal costs, strict norms for sludge disposal, etc., anaerobic digestion can be plausible even from a plant size of 10,000 PE (Gretzschel et al. 2014). Between 10,000 PE and 50,000 PE, a material and energy balance, as well as an economic evaluation must be considered to decide the adequate technology. For plants larger than 50,000 PE, anaerobic sludge stabilisation is used exclusively.

This has not been the case in China. The amount of sewage sludge produced in China has almost doubled since 2007, with still a large proportion being either dumped or landfilled without stabilisation (Smith et al. 2018). Anaerobic digestion is applied mostly in large-scale WWTPs, normally in mesophilic temperature ranges. But the ratio of WWTPs using anaerobic digestion is still low in China (Yang et al. 2015), being used in only around 100 plants in 2013 (Jin et al. 2014).

This has many explanations, such as several preconceived ideas against the use of anaerobic digestion, which have been partially documented in (Vergara Araya and Hilgenfeldt 2022) and in a series of interviews carried out by project partners in the framework of the project PIRAT-Systems, as documented by (Zimmermann et al. 2022). The main reasons are briefly described and complemented with literature here:

- Chinese WWTP operators and local authorities conceive wastewater and sewage sludge treatment as two separate processes, therefore the financing and fees for wastewater treatment do not always contemplate the processes for sludge stabilisation and disposal (Liu and Han 2015); (Zimmermann et al. 2022).
- Due to the low investment in sludge treatment in China, many WWTP are unable to construct and operate anaerobic digestion facilities (Yang et al. 2015). Low water prices also introduce perverse incentives for treatment facilities: the cost of sludge treatment is often not included in wastewater fees or charged at an insignificant rate, making it impossible for many plants to afford the costs (Liu and Han 2015).
- The low organic content of the wastewater and sewage sludge and the associated low biogas production (Jin et al. 2014); (Lu et al. 2019); (Wei et al. 2020); (Zimmermann et al. 2022). Although it might be true that the influent wastewater in WWTP in China has lower COD and BOD concentration on average than in other countries, it is known, that the operating sludge age of an activated sludge system controls to a great extent the amount of organic content in MLSS. Moreover, the amount of sand contained in the sludge can be reduced with a proper pre-treatment (e.g. aerated grit trap).

- The standards for sludge agricultural application are very strict, especially regarding heavy metal content. Most of the digested sludge does not meet these standards, reducing indirectly the incentive to use anaerobic digestion (Yang et al. 2015).
- Fear of explosions in biogas facilities is also a reason to avoid the installation of anaerobic reactors in WWTP. However, China has a vast experience in the application of biogas plants in households, agricultural and industrial applications (Giwa et al. 2020), which could be transferred to sewage sludge treatment. The use of ATEX equipment and adequate safety protocols are also important measures when managing biogas facilities.
- The backload of the sludge liquor can be detrimental to the overall nutrient removal process, especially in WWTP which already have problems fulfilling the normative requirements for nutrient removal, without anaerobic digestion. This is a problem that can be explored by using computer modelling and will be addressed in detail in this chapter.

China has a big share of large WWTP; 60% of the WWTP in China are between 50,000 and 250,000 PE[1] (Zhang et al. 2016). Therefore, if some of these regulatory and financial barriers are removed, many WWTP in China could benefit from the use of anaerobic sludge stabilisation.

2.2 Challenges in Nitrogen Removal in WWTP Worldwide

Meanwhile, COD removal does not represent a major challenge in technified WWTP worldwide, the removal of nutrients, especially nitrogen, remains to be challenging in many countries.

2.2.1 Current Situation in Different Countries

2.2.1.1 China

For example, approximately 50% of WWTPs in China do not meet the nitrogen discharge standard, and around 90% of WWTPs have problems with nutrient removal, especially nitrogen (Zhang et al. 2016). In North-eastern China, the COD/TN ratios in wastewater are very low, a situation that, due to the low temperatures, is especially critical for targeted nitrogen removal with denitrification/nitrification (Li et al. 2015). Values of COD/ TN ≤ 4–6 are common, making it extremely difficult to achieve efficient and stable nitrogen removal in the conventional biological treatment processes (Cao et al. 2013).

In recent years, China has strengthened its standards for wastewater treatment and plans to upgrade wastewater treatment plants (WWTPs) in areas where sensitive water bodies are located and or have poor water quality to meet the Grade I- A discharge standard (Niu et al. 2019) or even stricter discharge norms, especially in cities surrounding the Taihu lake catchment, which is the source for drinking water for one of the most populated regions in the country. It is also common to have plants designed for a different standard than the current discharge requirements (Zhang et al. 2021). This leads without a doubt to challenges in nutrient removal, as the plants must cope with fulfilling stricter standards, without the

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[1] Calculated with 200 L/(PE·d)

necessary infrastructure. WWTP design is sometimes based on little, poor quality or incomplete data about the wastewater quality and quantity, leading also to problems in the dimensioning and gaps between the actual requirements and the designed parameters.

Another problem is that the volatile solids present in the MLSS in activated sludge systems are between 25 to 50%, instead of the usual value of around 75% (Zhang et al. 2021). This indicates a highly mineralized sludge with low biomass content, which is possibly not enough to degrade the required organic matter and nutrients in the system.

The presence of ageing and disrepair of equipment (Zhang et al. 2021), due to a chronic lack of adequate funding, also contributes to creating problems in norm compliance.

2.2.1.2 Germany

Several WWTP in Germany also face challenges in this regard. According to the Municipal Wastewater Report 2019, in 2018 six WWTP larger than 10.000 PE (from a total of 352 WWTP) did not comply with the nitrogen discharge norms in the state of Baden-Württemberg (Umweltministerium BW 2019). In North Rhine-Westphalia, although all WWTP comply with the discharge limits, 48 WWTP larger than 10.000 PE do not comply with the load based-approach, removing less than 75% of the influent nitrogen (Umweltministerium NRW 2019). Reasons given for this, are operational problems, long sewer networks, dilution of the wastewater caused by infiltration water in the sewerage system and plant expansion. Long sewer networks are a known problem (see Chapter 2.2.2) that leads to smaller C/N ratios in the influent. The infiltration of water, i.e. unintentional inputs of groundwater, surface water, stratum water or springs to the sewer, causes a dilution of the wastewater, an increment in the influent flow. Potentially, undesired substances can also be contained in the infiltration water. The concept of operational problems is a very general term that includes failures in pumping, pre-treatment stages, failures in the automation systems, etc., which could lead to problems in norm compliance.

Several WWTP in the country require the dosing of external carbon sources to comply with the nitrogen discharge norms (see chapter 2.2.3).

2.2.1.3 Other Countries

A study at the WWTP Tehran, Iran (2.1 Mio PE) in 2011–2012 (Nourmohammadi et al. 2013), showed a stable removal of BOD_5 and unstable removal efficiency of total nitrogen, which is critical due to the progressively more stringent effluent requirements. Here, a variable COD/TN inlet ratio is also present, with an average of 9.6 (BOD_5/TN = 5.4) (Wichern et al. 2017). This WWTP operates at low sludge ages of around 5 d, which is slightly lower than the calculated required value (the wastewater temperature is relatively high all year round reaching between 22 and 28 °C). However, the main probable cause for the observed problems in nitrogen removal is related to the small proportion of anoxic tanks (V_D/V_{AT}) of only 13% (Wichern et al. 2017).

The recently finished project EXPOVAL shows an unfavourable C/N ratio for targeted nitrogen removal in WWTP in different countries e.g. WWTP Aguas Blancas (Acapulco, Mexico), WWTP Haikou (Haikou, China), WWTP Bekkelaget (Oslo, Norway) and WWTP Batumi (Batumi, Georgia) (DWA 2017). Large-scale WWTP in central Chile also shows variable C/N ratios, often unfavourable for nitrogen removal, however, the Chilean norm does not require a targeted nitrogen removal for discharges to fluvial water bodies (max. TN =50 mg/L) (MINSEGPRES 2000). This is also true for many other countries, which still lack strict regulations for nutrient discharge (see Table 4).

2.2.2 Causes of Low or Fluctuating C/N Ratio in the Influent

For wastewater with low COD content or with high nitrogen loads, denitrification can be problematic due to the unfavourable C/N ratio. Moreover, strong fluctuations in wastewater quality can also be problematic. Usually, domestic wastewater has enough easily biodegradable carbon sources for denitrification, but the following causes can generate wastewater with a low C to N ratio:

- **Long distance between wastewater collection and wastewater treatment location:** Degradation of COD and BOD in the process of wastewater transportation in sewers can be significant if the distance from the collection to the treatment point is long. This contributes to a pre-degradation of the organic sources, reducing the organic content in the wastewater previous to its treatment (Ashley et al. 2002). This is a problem commonly observed in WWTP in China (Liao et al. 2015), but in other countries such as Germany, it is a known problem.
- **Over dimensioning of the sewerage network:** the over-dimensioning of the sewer network contributes, as the previous point, to a pre-degradation of the organic compounds in the sewer due to the decrease in flow velocity. This situation has been reported as a common problem in China (Zhang et al. 2021), and in Germany as well, especially in regions with negative population development (TMLFUN 2012).
- **Use of upstream septic tanks**: in some countries, such as China and India, it is common to have septic tanks installed below buildings, where the wastewater is pre-treated before discharge to the sewers system, which avoids the discharge of the whole organic content into the urban sewage pipe network (Yang et al. 2015) (Zhang et al. 2016).
- **Separate toilet paper collection**: it is common use in most countries (e.g. Latin America, Africa, Asia) to dispose of toilet paper as solid waste, instead of its disposal in the toilet as is common in North America and North-western Europe (Iris Veldwijk 2017). This is relevant because toilet paper mainly contains cellulose and lignin i.e. medium degradable carbohydrates and slowly degradable organic substances, which have a COD of 1.2 g/(g·dry matter). Toilet paper contributes approximately 17.7 g/(PE·d) particulate biodegradable COD (Jönsson et al. 2005). Around 45% of toilet paper fibres are removed in fine screens (Li et al. 2020) and around 20% of the cellulose fibres can be found in primary sludge (Gupta et al. 2018). Around 40% of the remaining COD can be degraded biologically in secondary treatment, depending on the sludge age of the system (Li et al. 2020), contributing to the organic carbon load in the influent of the biological treatment.

2.2.3 Dosing of External Carbon Sources

When the C/N ratio in the influent of the biological treatment is unfavourable for upstream denitrification, the dosing of external carbon sources is a common strategy to provide easily degradable carbon sources to be oxidized with the oxygen contained in nitrate and remove it as nitrogen gas. Targeted dosing of external C-sources can contribute to maintaining norm compliance, even under unfavourable conditions. However, C-sources can be very expensive (e.g. ca. 212 €/ton in China[2]), therefore its use should be reduced to a minimum.

Due to the strict discharge normative, for example, due to an unfavourable C/N ratio in the wastewater, at the WWTP Magdeburg-Gerwisch (426.000 PE) in Saxony-Anhalt, external carbon sources have to be added to the activated sludge cascade from time to time (Ahlers 2020). The WWTP Halle-Nord (ca. 300.000 PE) in Saxony-Anhalt is currently carrying out studies to decrease the dosing of external C-Sources. The WWTP Höxter (30.000 PE) in North Rhine-Westphalia, showed also an unfavourable influent C/N-Ratio in an operational data study between the years 2010 and 2013, making necessary the dosing of an external C-source[3] (Kaub and Biebersdorf 2014).

2.2.4 Comparison of Discharge Values for Nitrogen

The poor nitrogen-removal performance turns especially critical when the wastewater is discharged in sensible water bodies and/or the discharge concentration values are set extremely low. Moreover, in the case of sensitive water bodies, in many countries, the local authority can set much stricter limit values. The current standards for wastewater discharge in different countries are summarized in Table 4.

There it can be seen that the discharge limits in the Tai Hu Lake area are among the strictest in terms of total nitrogen and ammonium. Only Switzerland, Luxembourg and Dubai come close, with very similar limits. In all four countries, the objective is to preserve valuable water bodies for drinking water supply. This is also the case for the standards associated with wastewater discharges into Lake Constance in the tri-border area of Germany, Austria and Switzerland. The discharge limits in most of the more industrialised European countries follow at least the parameters of the European Council Directive (European Commission 1991) UWWDTD, which is a desired, but voluntary, adherence.

Many Latin American countries (as an example here Chile and Mexico), have water discharge standards that are tailored to the receiving water body, and usually do not require targeted nutrient removal for the discharge in rivers and to the sea. However, wastewater collection and treatment rates are very heterogeneous across the region. In the case of Cuba, despite strict discharge standards, most wastewater, both domestic and industrial, is discharged untreated or partially treated into watersheds and the coastal zone (Díaz Duque 2018). In other words, a strict standard does not automatically mean compliance, as it is dependent on enforcement structures as well as available infrastructure.

According to the WHO (Schellenberg et al. 2020) (see Figure 7) most countries set limits for the discharge of BOD or COD, but more than 20% of countries simply lack standards for nitrogen discharge, and only ca. 20% have standards for ammonium or nitrate discharge.

[2] Information provided by a plant operator in the project PIRAT-Systems

[3] for a design temperature of 12 °C

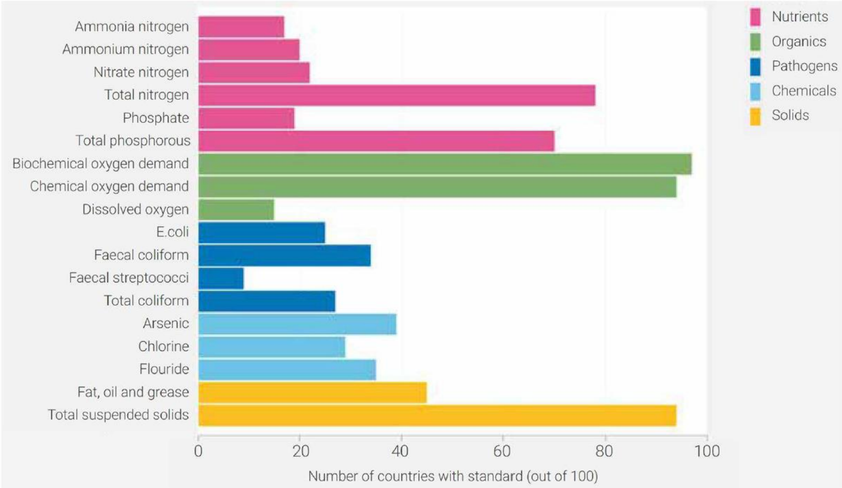

Figure 7. National discharge standards in different countries (WHO 2017 in (Schellenberg et al. 2020))

The values in Table 4 describe in detail some differences in discharge norms for wastewater in several countries. In there it is possible to see that different countries and regions have different approaches to the standards considering the following aspects, among others:

- **Measurement temperature**: the discharge standard can be independent of the wastewater temperature, or dependent, e.g. allowing lower standards for nitrogen discharge values in the winter months or under certain temperatures, as it is known that nitrification is sensitive to low temperatures.
- **Monitoring modality**: local authorities can decide if the monitoring will be carried out in grab (random) samples or composite (mixed) samples at different time intervals, typically 2 of 24 hours. Monitoring with random samples means that the WWTP does not have any chance to buffer possible peaks during operation, increasing significantly the stakes and changing the way the plant must be operated to comply with the norm every minute of operation. A mixed sample of 2 hours is still very strict, but it provides a short buffer time in case of peaks. However, in two hours the operational changes that can be carried out to improve the norm compliance are few. A more forgiving form of monitoring is the 24-hour composite sample, as peaks can be significantly dimmed in that period, allowing for more flexibility in norm compliance.
- **Monitoring periods**: WWTP can be monitored also using daily, monthly or yearly averages, by random inspections several times per year, etc.

Due to the accelerated changes that climate change and accelerated industrial development in developing countries are being confronted with, it is expected that stricter nutrient discharge standards will become increasingly common in the coming decades to protect drinking water sources and fragile natural ecosystems. This may confront many countries and regions with new challenges in their wastewater treatment, where adequate adaptation and coping mechanisms will be key to assure both environmental and economical sustainability.

Table 4: Discharge standard of pollutants for municipal wastewater treatment plants in different countries and requirements

Country	Requirement	Discharge limits, mg/L							Comment	Source
		COD	BOD_5	NH_4-N	NO_3-N, NO_2-N	TN	TP	$PO_4{}^{3}$-P		
China	Grade II-B	60	20	8 (15)	-	20	1	-	24 h-sample	GB 18918-2002 (Zhou et al. 2018)
	Grade I-A	50	10	5 (8)	-	15	0.5	-		
	Taihu Lake catchment	30	-	1.5 (3)	-	10	0.3	-		WWTP Operator
Germany	PE >10.000-100.000	90	20	10*	-	18*	2	-	2 h sample	AbwV (Bundesamt für Justiz 2020)
	PE >100.000	75	15	10*	-	13*	1	-		
	Bodensee lake catchment PE <40.000	60	15	-	-	18 [13]	0.3		24-h sample	(IGKB 2006)
Luxembourg	WWTP Beggen	50	10	1 ((2))	-	8	1 «0.5»		24 h-sample; NH_4-N 2-h sample	AGE[4] ((Thys et al. 2022)
United Arab Emirates	Harbour Area Dubai	100	50	2	40	10**	-	2		(Preisner et al. 2020) and
Denmark	-	75	10	-	-	8	0.4	-		
Sweden	**PE >10.000-100.000**	125 ˣ	25ˇ	-	-	15¨	2ʁ	-	UWWTD	(European Commission 1991)
	PE >100.000	125 ˣ	25ˇ	-	-	10¨	1ʁ	-	UWWTD	
Switzerland	**PE>100.000**	45	15	2	0.3	-	-	0.8		(Preisner et al. 2020) and (European Commission 1991)
USA	areas sensitive to eutrophication	-	30			3 – 5	0.1 – 1			(Preisner et al. 2020) and

[4] Luxembourg's supervisory water authority

Table 4 (continued) Discharge standard of pollutants for municipal wastewater treatment plants in different countries and requirements

Country	Requirement	Discharge limits, mg/L							Comment	Source
		COD	BOD_5	NH_4-N	NO_3-N, NO_2-N	TN	TP	$PO_4{}^{3}$-P		
European Union	10.000 > PE >100.000	125 ˣ	25ˇ	-	-	15¨	2 ʁ			
	PE >100.000	125 ˣ	25ˇ	-	-	10¨	1 ʁ			
United Kingdom	PE >10.000-100.000	125	25	-	-	15	2	-		
Mexico	Rivers A (agricultural use)	-	200 {150}	-	-	60 {40}	30 {20}	-	24-h sample	(CONAGUA 1996)
	Rivers B (urban or use)	-	150 {75}	-	-	60 {40}	30 {20}	-	24-h sample	
	Rivers C (water protection area)	-	60 {30}			25 {15}	10 {5}	-	24-h sample	
	Coastal water A (fishing, boating and other uses)	-	200 {150}	-	-	-	-	-	24-h sample	
	Coastal water B (Recreation)	-	150 {75}	-	-	-	-	-	24-h sample	
	Coastal water B (Estuaries)	-	150 {75}	-	-	25 {15}	10 {5}	-	24-h sample	
Cuba	Rivers and reservoirs, Type B (ground)	90	40	-	-	10**	4	-		NC 27:2012 (NC 2012)
	Rivers and reservoirs, Type A (surface)	70	30	-	-	5**	2			
Chile	Sea (within the coastal protection area)	-	60	-	-	50**	5	-	Grab sample★	(MINSEGPRES 2000)
	Rivers	-	35	-	-	50**	10	-	Grab sample★	
	Lakes	-	35	-	-	10	2	-	Grab sample★	

Values in ( ) brackets are for temperatures below 12 °C
Values in (( )) brackets are for temperatures below 8 °C
Values in [ ] brackets are for plants larger than 100.000 PE
Values in {} are for the monthly average
Values in « » are for the yearly average
* For temperatures above 12 °C
**TKN

*** for discharge to sensitive areas
ˇ plus a 70-90% load reduction
ˣ plus a 75% load reduction
¨ plus a 70-80 % load reduction
ʁ plus an 80 % load reduction
★or the average of three grab samples for short-time discharges

As reported in Table 4, the norms with the highest requirements for nitrogen removal can be found in China, Luxembourg, Switzerland and Dubai. However, the main difference with China is the sampling period of 24 hours, in comparison with the 2 hours required in European countries.

An important aspect is which technologies are used to achieve the treatment goals described in Table 4. Wastewater treatment technologies used in China are mainly A2/O and oxidation ditch, which have been adopted in over 50% of the WWTPs, treating 46% of the wastewater volume. One-quarter of the wastewater is treated by traditional activated sludge and SBR, and 28% is treated by other processes (e.g. AO, biofilm, chemical or physiochemical processes) (Zhang et al. 2016).

In Germany, the large majority (>93%) of the wastewater is treated with activated sludge technologies with targeted nutrient removal (DESTATIS 2018) in the 9,105 WWTP in the country (DWA 2021). Intermittent denitrification is used most frequently, especially for smaller WWTP, up to 10,000 PE. For larger plants, upstream denitrification is used more frequently, also in combination with intermittent denitrification, especially in plants up to 100,000 PE (DWA 2021).

Switzerland has ca. 800 WWTP (August 2017) (BAFU 2021) and the majority has a mechanical-biological treatment with targeted P-elimination (Suess et al. 2020). In the Canton Zurich for example, from the 61 WWTP > 500 PE, 48 WWTP use classic activated sludge, five use biofilm (fixed bed and fluidized bed), there are four SBR and two MBR, four use a combination (Amt für Abfall, Wasser, Energie und Luft 2021). In Sweden, wastewater treatment also takes place with mechanical biological treatment, most commonly activated sludge, with targeted nitrogen removal via nitrification and denitrification, and chemical phosphorous removal (Swedish EPA 2018).

Chile, for example, has 301 WWTP nationally in 2022, and ca. 49% of them (147) use the activated sludge technology, 12.6% lagoons (38) and 10.9% marine outfalls. The rest is divided into primary treatment, SBR and oxidation ditch (SISS 2022). Dubai has the WWTP Jebel Ali, the largest in the country (for 3.35 million inhabitants) (BESIX 2020) and uses activated sludge and produces treated wastewater for irrigation (AECOM 2020).

In the USA, advanced and conventional activated sludge are the mainly used technologies, representing ca. 59% of the WWTP, followed by oxidation ditch, with ca. 15% and attached growth with 14.6%. According to a study by the US EPA between 2019 and 2021, of ca. 1,032 WWTP, only 44% deliver TN values below 8 mg/L (US EPA 2022).

It results clear from the studied data, that activated sludge technologies are by far the most widespread technology for biological wastewater treatment, including nitrogen removal.

2.3 Modelling WWTP

Modelling of activated sludge processes became a common part of the design and operation of wastewater treatment plants already in early 2000 (Henze 2000). Models and simulation can be used as cost-effective tools to support decision-making, sustained with data and analysis, backing up the first steps for implementing changes and optimisation strategies. Dynamic simulation of wastewater treatment plants has been used as an instrument to increase the knowledge of the process and system behaviour, for optimisation studies, for training and teaching, and for model-based process control (Langergraber et al. 2004). The application of mathematical models for design, with a focus on the extension of existing

WWTP, has become common practice (Kroiss et al. 2021). Modelling of WWTP is based on physical models (e.g. settling, filtration), chemical models (e.g. precipitation) and biological models, which are the most complex ones. The most widely used models for activated sludge are the IWA Activated sludge models (ASM) and for anaerobic digestion, the IWA Anaerobic digestion models (ADM).

The impact of dynamic modelling in the wastewater treatment field, with the appearance of the ASM and ADM models, highlighted data and information gaps present and, most likely, drove the development of online measurements and the use of ICA strategies (Kroiss et al. 2021).

2.3.1 Activated Sludge Model No. 3 (ASM3)

The IWA task group on Mathematical Modelling for the Design and Operation of Biological Wastewater Treatment has developed since the 80s the activated sludge models (ASM), based on oxygen consumption, sludge production, nitrification and denitrification in activated sludge systems treating wastewater of primarily domestic origin (Henze 2000). The main goals of developing the ASM were to review existing models and to reach a consensus concerning the simplest mathematical model, capable of realistically predicting the performance of single-stage activated sludge systems, involving organic matter and nitrogen removal (Jeppsson 1996).

Activated sludge systems entail multiple, complex biochemical interactions. With the development of the ASM models, the scientific community could agree on a common nomenclature and way of presenting modelling results, contributing to a better understanding of activated sludge systems. The matrix organization of the equations (Peterson matrix), made it easier for researchers to work with the model, and follow changes and modifications. The models are based on kinetic equations, stoichiometric relations and mass balances. COD is the parameter defining carbonaceous material, and mass balances are based on it, meanwhile, nitrogenous material is based on measurements of TKN (Jeppsson 1996).

The first model was ASM1, which included nitrogen removal processes. ASM1 set the basis for activated sludge modelling, starting a productive discussion as to how such models can be improved and what are its limitations. From ASM1, several other models were developed and changed. With the inclusion of biological phosphorous removal processes, models ASM2 and ASM2d were created. Its latest version, ASM3 delivered in the year 2000, was designed to be the core of many different models and has corrected several shortcomings from ASM1, according to the IAWQ (Henze 2000).

ASM3 includes only the microbiological transformation processes, and chemical precipitation is not included. Modules for biological phosphorus removal, chemical precipitation, growth of filamentous organisms or pH calculations are not part of ASM3 but can be connected as add-on modules (Henze 2000). In Germany, the use of ASM3 has been complemented with the worksheet DWA-A 131 “Dimensioning of single-stage activated sludge plants” and is implemented in the simulation software SIMBA for WWTP modelling and optimization (Alex et al. 2015), (Ahnert et al. 2015).

2.3.2 Model Calibration and Validation

The usefulness of a model to be applied in a plant, whether laboratory or large-scale, is given by its ability to predict the behaviour of key parameters in the operation of such a plant. In order to do this, good calibration and subsequent validation are required. Calibration of a model is the adjustment of the model parameters to fit a set of data. The validation of the model is carried out by testing the model with a different set of data that the used for calibration, ideally under different conditions.

The calibration can be carried out as simply or as complexly as wished and also at different levels, depending on the objectives of the study. A model can be calibrated for a specific plant or set of plants, or even for a type of wastewater e.g. domestic wastewater in a city or country.

The calibration and validation process is far from trivial and therefore a systematic approach is required to avoid mistakes and properly document the calibration process for it to be reproducible. There are several systematic approaches to calibrating wastewater treatment systems using Activated Sludge Models. Some of the most relevant in literature are BIOMATH, HSG, STOWA and WERF (Sin et al. 2005).

In general, these methods suggest first setting the objective of the study. Then obtain a large amount of data from the studied WWTP, preferably with a detailed dynamic measurement campaign data. After that, check the data correctness and plausibility with mass balances and hydraulic models (checking HRT and SRT), and correct it if necessary. Only then, the model is built and is calibrated. A sensitivity analysis can be also carried out to further verify the soundness of the calibrated model. The measuring campaign should also consider the results obtained by the sensitivity analysis and verify them, if possible.

The calibration should be done by changing one parameter at a time and documenting all changes. Here, the priority for changing each parameter may be given or not, depending on the protocol. The most important parameters to be adjusted are usually the denitrification capacity and excess sludge production, but the model can be fitted to other parameters as well. The curve fit for each parameter can be checked visually or by using mathematical methods. After calibration, the results must be validated by testing the calibrated model with a new set of data, obtained under different conditions than the data used for calibration.

The HSG-Sim approach (Langergraber et al. 2004) was selected (for more detail see Annex 12.1), due to its relation with the author and its institution and due to its focus on the entire study process. The HSG group, based on the ATV-A 131 (2000) guideline – the previous version of the current DWA-A 131, from 2016 – and other available models for WWTP treatment stages, identified several kinetic and stoichiometric wastewater parameters for the calibration of the ASM model for domestic wastewater in Germany.

2.3.3 Model Fit

The evaluation of the model fit, i.e. when a simulation is good enough to make accurate predictions about the modelled system, can be done initially, by a visual evaluation. However, the use of mathematical tools is of great advantage to assure an objective evaluation of model fit.

The most basic evaluation method is the difference between the average values, according to the percentage error, as defined below. Moreover, the mathematical model fit quality dimensions to evaluate the fit of the model suggested by (Ahnert 2007) are described in the subsequent subsections. This same or a similar principle has been used in different simulation studies since then, even when using other simulation approaches (Hvala et al. 2018) (Młyński et al. 2019) (Lotfi et al. 2020), (Abba et al. 2021). However, there is no standardized procedure for model calibration and validation, although some approaches have been proposed (Seco et al. 2020).

2.3.3.1 Percentage Error (% e)

The percentage error registers as a percentage of the difference between an approximate value and an exact or known value. In this case, the known value corresponds to the observed, measured data and the approximate value to the modelled data.

$$\%\,\mathrm{e} = \frac{|\mathrm{O_i} - \mathrm{M_i}|}{\mathrm{O_I}} * 100$$

Where:

% e : percentage error

M_i : estimated, modelled, predicted value

O_i : observed, measured values

2.3.3.2 Coefficient of Determination (R^2)

The coefficient of determination, R^2, is used to analyse how differences in one variable can be explained by a difference in a second variable. The value of R^2 lies between 0 and 1, with 1 indicating a perfect match between the data set and the modelled data and 0 indicating no relation at all. The higher the value of R^2, the better will be the prediction and strength of the model.

$$R^2 = \left(\frac{\sum_{\mathrm{i}=1}^{\mathrm{n}}[(\mathrm{O_i} - \mathrm{O_m}) * (\mathrm{M_i} - \mathrm{M_m})]}{\sqrt{\sum_{\mathrm{i}=1}^{\mathrm{n}}(\mathrm{O_i} - \mathrm{O_m})^2 * \sum_{\mathrm{i}=1}^{\mathrm{n}}(\mathrm{M_i} - \mathrm{M_m})^2}}\right)^2$$

R^2 : coefficient of determination

M_i : modelled, estimated value

M_m : Mean modelled values

O_i : observed, measured values

O_m : average observed values

2.3.3.3 Root Mean Squared Error (RMSE)

Root Mean Square Error (RMSE) measures how much error there is between two data sets. It compares a predicted value (model) and an observed or known value. The smaller an RMSE value, the closer the predicted and observed values are.

$$\mathrm{RMSE} = \sqrt{\frac{\sum_{\mathrm{i}=1}^{\mathrm{n}}(\mathrm{M_i} - \mathrm{O_i})^2}{\mathrm{n}}}$$

Where:

RMSE : Root mean square error

M_i : estimated, modelled, predicted value

O_i : observed, measured values

n : number of values in the data set

This value has evaluative power, only when compared to different model fits for the same parameter (e.g. evaluating different model fits for the parameter COD), but the value itself does not provide information to directly evaluate the model fit. Therefore, the value is calculated, but will not be used to evaluate the model fit.

2.3.3.4 Nash-Sutcliffe Model Efficiency Coefficient (E_j)

The Nash-Sutcliffe model efficiency coefficient is widely used to assess the predictive power of hydrological models. The efficiency is defined as one minus the sum of the absolute squared (j=2) differences between the predicted and observed values normalized by the variance of the observed values during the period under investigation (Krause 2005).

$$E_j = 1 - \frac{\sum_{i=1}^{n} |M_i - O_i|^j}{\sum_{i=1}^{n} |M_i - O_m|^j}$$

Where:

E_j: Nash-Sutcliffe model efficiency coefficient

O_m: mean of observed discharges

M_i: modelled discharge

O_i: observed discharge at time t

An efficiency of 1 corresponds to a perfect match of modelled discharge to the observed data. An efficiency of 0 indicates that the model predictions are as accurate as the mean of the observed data, whereas an efficiency of less than zero occurs when the observed mean is a better predictor than the model.

Ahnert et al. (Ahnert 2007), recommends the use of j=1, as the use of j=2 delivers high values even with mediocre modelling results. In this evaluation, both approaches (j=1 and j=2) will be considered.

2.3.3.5 Index of Agreement (d_j)

The index of agreement (d) is a standardized measure of the degree of model prediction error. The value varies between 0 and 1, where 1 indicates a perfect match and 0 indicates no agreement at all. Here, equivalent to the logic described for E_j, j=1 and j=2 will be considered for the evaluation.

$$d_j = 1 - \frac{\sum_{i=1}^{n} (O_i - M_i)^j}{\sum_{i=1}^{n} (|M_i - O_m| + |O_i - O_m|)^j}$$

Where:

d_j: index of agreement

O_m: mean of observed discharges

M_i: estimated, modelled discharge

O_i: observed discharge at time t

2.3.4 SIMBA Software

There are several software's for water systems simulation, particularly for activated sludge in WWTP: e.g. AQUASIM, ASIM, WEST, and SIMBA, among others (Schütze et al. 2002). The Magdeburg-Stendal University of Applied Sciences acquired in last years licenses to use SIMBA® due to its experience in simulation with this tool and to the close relationship with ifak e V., the developers.

SIMBA® is a simulation system that allows the holistic consideration of sewer systems, wastewater treatment plants, sludge treatment and rivers. SIMBA can be applied for a large variety of tasks in engineering practice and research and education, from plant and process design to analysis and operational optimisation of operation of urban wastewater systems (ifak 2019). The software has been widely used, especially in the German-speaking community for water systems modelling e.g. in (HSGSim 2008), (Ahnert et al. 2015) and (Torregrossa and Hansen 2018), for example, to demonstrate adequate performance, for studies to improve the operation (costs, critical situations), as well as for planning and dimensioning of WWTP (ifak 2018).

For wastewater treatment, there are several models available, for biological treatment based on the IWA ASM models. The model *asm3h* is one of the default models in SIMBA®, using ASM3 with modifications and parameters following the HSG 89 approach, as described in (Böhnke 1989) and (Dohmann 1993). Moreover, this model will calculate simulation results in accordance with the German design guideline DWA-A 131 (ifak 2018). For a detailed description of the different processes available in SIMBA, see Annex 12.2.

2.4 ICA Strategies for Aeration and Nitrogen Removal

Due to the increasing complexity and increasingly strict requirement for wastewater discharge, WWTP tends to rely heavily on ICA strategies. The main strategies relevant to this work are related to aeration and nitrogen removal, with a focus on upstream denitrification.

The supply of air is a key factor in the biological treatment of wastewater, as oxygen is required for the oxidation of organic matter and the nitrification process (Åmand et al. 2013). Since aeration is an energy-intensive process, the biological treatment usually represents the largest proportion of the energy requirements in a WWTP, close to or above 50%, a trend that is observed in different countries and types of processes (Vergara-Araya et al. 2021).

Therefore, adequate control of the aeration process is usually an obvious starting point for the optimization of the energy consumption in a WWTP. The most basic approaches to control air in activated sludge systems are:

2.4.1 Dissolved Oxygen (DO) Set Point Feedback Control

The most classical approach to control aeration in the aeration tanks, is a set-point-based control, where a target DO concentration is defined, usually between 1.5 and 2 mg O_2/L (Åmand et al. 2013). One or multiple online DO sensors, strategically installed in the aeration basin, show the real DO concentration of the tank at the current time (Controlled variable), and the controller compares this value with the target (set-point) value (DWA 2006). Therefore, a feedback control modifies the aeration intensity, either by changing the valve opening or turning on and off the air compressors/blowers (manipulated variable), to reach the set-point DO concentration.

This approach is basic but effective i.e. under the right conditions the treatment goals can usually be reached, however, it tends to deliver more air than necessary for the oxidation of organic matter and ammonium, incurring in inefficiencies.

2.4.2 Ammonium Feedback Control

A common improvement of the DO set-point approach, is the incorporation of an ammonium measurement, in an ammonium feedback control. Here, the measurement of ammonium, either at the end of the aeration tank or the effluent of the biological treatment indicates if more or less air is required to oxidize the ammonium present: if no ammonium is present, the aeration can be reduced if not, the aeration can be increased.

Many modifications can be carried out here, for example, define a curve to determine how much air is required according to the measured ammonium concentration. If there is a facultative aerobic/anoxic zone, it can be aerated or not depending on the treatment requirements, based on ammonium measurement as well (DWA 2006).

2.4.3 Nitrate Feedback Control

The incorporation of a nitrate measurement, can also be an indicator of the air requirements or provide information to control the internal recirculation of nitrate-rich activated sludge mixture or to control the dosing of external carbon sources (DWA 2006).

2.5 Overview of Simulation Studies for WWTP Optimization and Nitrogen Removal

Due to the complex nature of wastewater treatment and the challenges imposed by the removal of nitrogen, simulation studies have been and still are a useful tool to make improvements and test different strategies.

The creation of models and the testing of automation strategies were and are common applications for aeration control (Åmand et al. 2013). Simulation studies can also incorporate, for example, the DWA-A 131 approach, as showed by Alex et al. (Alex et al. 2015) and tested practically by Ahnert et al. (Ahnert et al. 2015).

The calibration process of simulation studies is not always straight–forward process, as sometimes many variables are sensitive, as found by Liwarska-Bizukojc et al. (Liwarska-Bizukojc et al. 2011). As the ASM models are COD-based, the COD fractionation is key in calibrating a model that is representative of the process that is being modelled (Ahnert et al. 2021), as shown by Muserere et al. (Muserere et al. 2014).

The use of modelling for activated sludge systems boomed in the mid-1990s, and the field continues to be a relevant research topic, with a stable share of published articles in the last decades (Ahnert and Krebs 2021). A strong increase has been observed in the use of keywords such as Anaerobic Digestion, Co-Digestion and Biogas, as well as in Adsorption (related to activated carbon) and Sewage Sludge, among others. At the same time, a decrease in the keywords Nitrification, Denitrification and Phosphorous Removal, among others, has been observed (Ahnert and Krebs 2021). This shows the research interest in the modelling of activated sludge systems has shifted in the last decades, moving away from the basics of ASM and ADM, and moving forward to the modelling of advanced technologies.

The optimization of WWTP has been continuously a relevant topic in the last decades (Kroiss et al. 2021). However, plant-wide modelling using real full-scale data has not been systematically researched (Hvala et al. 2018). However, several studies using this approach were found. For example, the last upgrade of the WWTP Vienna, relied on long-term data analysis, pilot-scale experiments, mass balances and also dynamic simulation. The model, built in 2013 and modified since then, allows to predict future demand and production of energy under different scenarios and load conditions (Kroiss and Klager 2018).

Another example is the one of a WWTP (435,000 PE) with carbon removal and nitrification, which was upgraded to comply with the required TN and TP discharge standards, with the use of dynamic simulation (Hvala et al. 2018). Here, the denitrification capacity was limited, and different approaches, such as A2/O and intermittent A2/O and AO were tested, but still showed challenges in achieving the required 10 mg/L TN in the effluent, making necessary a side-stream treatment for reject water from the sludge line.

Similar findings were obtained in a study of the WWTP Slupsk (Poland). With the use of dynamic simulation, aeration savings of up to 36%, together with an increase in energy production could be achieved (Zaborowska et al. 2017). This was achieved by improving the performance of primary clarifiers, and the implementation of advanced nitrogen removal processes, including anammox. The biological nutrient removal model (BNRM), developed at the Valencia University, which is similar to the ASM2d, has been applied in several experiences, from WWTP design, upgrading of WWTP, development of control strategies, among others (Seco et al. 2020). In their upgrading study for the WWTP Denia (Spain), the plant, originally designed for COD removal and nitrification, was upgraded to comply with the European norm for TN and TP discharge, among others, by transforming primary settlers to anoxic tanks.

Based on a general overview of different publications, the challenges faced by different studies, are related to the required complexity of the models, and the time-consuming process of building the models and calibrating them. Moreover, the inclusion of what we could call non-standard processes is usually also a challenge (Seco et al. 2020). Additionally, although WWTP data is usually available, there is an underuse of it, even if the data quality may be questionable (Seco et al. 2020), as the precise characterization of the influent data is one of the utmost key aspects for modelling (Hvala et al. 2018).

2.6 Summary of Chapter 2

Nitrogen is a nutrient, which is removed from wastewater in WWTP in biological processes, to avoid eutrophication of natural water bodies. The classic biological paths are nitrification and denitrification and can occur under different configurations, most commonly as upstream denitrification (or pre-denitrification).

These biological processes, require several operational conditions such as a suitable C/N ratio, sufficient sludge age, alkalinity, recycle rates, adequate aerobic and anoxic conditions, temperature and degree of sludge stabilisation.

Nitrogen removal in WWTP has proven to be a challenge in many regions worldwide, especially considering the sharpening of the discharge norms for treated wastewater, which are a consequence of water scarcity and the pollution of sensitive water bodies, among others, reaching values as low as 1 mg/L NH_4-N. Challenges such as an unfavourable influent C/N ratio can be solved by adding external carbon sources, however, this is an undesirable supply cost.

One way to approach the optimization of nitrogen removal is the application of computer modelling and simulation, which is a powerful tool and application to test operational and automation strategies in a safe and cost-time effective manner. Several studies have shown the potential of simulation as a valuable tool to plan and optimize the operation of WWTP.

This work will use the software SIMBA and the models will be based on the ASM3 models, applying the HSG Sim approach.

3 Example WWTP

In order to address some of the challenges related to nitrogen removal observed in the literature research, a concrete example of WWTP will be studied. This plant will serve as an example, and as a base to define which strategies are useful here and could therefore be useful in other similar WWTP.

The study has the objective to test optimization strategies for nitrogen removal in a real WWTP, using dynamic mathematic modelling as a tool. The objective is to improve norm compliance, reduce emissions to the environment, and at the same time decrease or maintain the energy consumption of the WWTP.

The first step in that direction is to describe the WWTP in detail and carry out an operational data analysis, which is carried on in this chapter.

3.1 WWTP Description

The example wastewater treatment plant is located close to the central stretch of China's coastline and discharges its wastewater in the Tai Hu catchment area. The WWTP is one of the largest in its district (ca. 450.000 PE). The WWTP treats mostly municipal wastewater, but around 10-20% of the treated wastewater comes from the food industry. The plant is a traditional mechanical-biological treatment plant with aerobic sludge stabilization. It has a mechanical pre-treatment with screens, an aerated grit chamber, and primary settling. Its biological step is activated sludge type A^2/O (anaerobic, anoxic, aerobic) and carries out additional chemical Phosphorous elimination. An aerial photo and scheme of the example WWTP is shown in Figure 8.

The treated wastewater is filtrated and disinfected before discharge. The sewage sludge is thickened, dewatered and transported for incineration in a thermal power plant or disposed of in a landfill. The example WWTP possesses only a few online measurements and must rely heavily on manual measurements and the operators' experience.

This WWTP, is representative of many WWTP in China, as A2/O is the mainly used technology for biological wastewater treatment (Zhang et al. 2016), and as observed in the literature research, upstream denitrification is also the main technology used in WWTP in the world. In a study of WWTP worldwide, surveying information of more than 47,300 WWTP, ca. 39.2% of the WWTP in the world, carry out advanced treatment, i.e. removal of nutrients (Ehalt Macedo et al. 2021).

Moreover, as approximately 75% of the WWTP in China correspond to medium size plants, treating between 1,000 – 10,000 tons of wastewater per day (Jin et al. 2014), the example WWTP is on the average size in the country. According to the study previously named, the average size of the surveyed WWTPs worldwide is around 50,000 PE (Ehalt Macedo et al. 2021), this is smaller than the example WWTP.

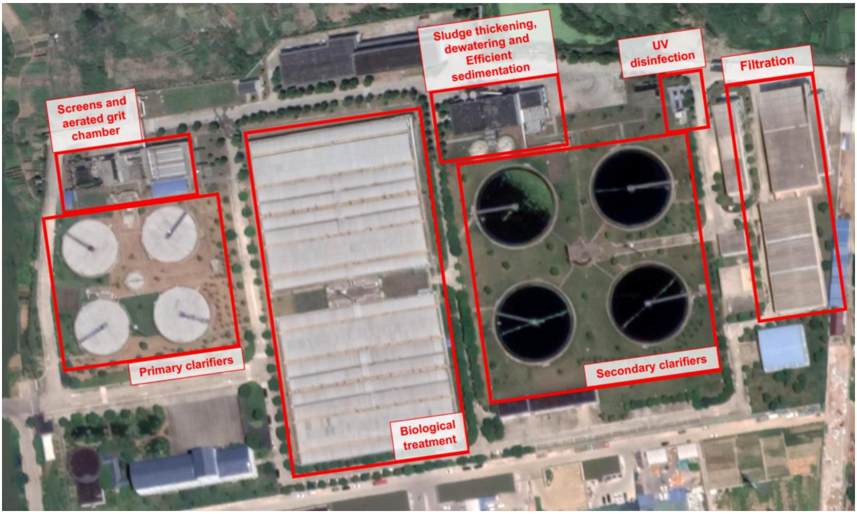

Figure 8. Aerial view of the example WWTP (Google Maps, modified)

3.2 Design and Operational Parameters

To evaluate the performance and operation of the example plant, the data between 2017 and 2019 is assessed. As there are no known specific guidelines for the design and evaluation of WWTP in China, the evaluation is carried out according to the German guidelines provided by the ATV DVWK-A 198 (ATV-DVWK 2003), DWA-A1 31 (DWA 2016) and DWA-A 216 (DWA 2015). The DWA-A 131 and EXPOVAL, its more recent international derivation for cold and warm climates (DWA 2017), have been used internationally in different studies (Ahnert et al. 2021). These are standardized methods to evaluate WWTP operation and performance, which can be applied to many different types of WWTP, considering the specificities of each location (e.g. discharge norms, temperature, weather, type of sewer, etc.).

The example WWTP was built in two phases. The first one corresponded to a 75,000 m^3/d inflow and was completed in 2009. Phase II and upgrading incorporated another 75,000 m^3/d inlet flow in 2015. The plant was designed to comply with the Grade 1-A standard (GB18918-2002) effluent parameters (see Table 7).

However, in recent years, over-urbanization and industrialization have seriously compromised the water quality in the Tai Hu Basin area, reaching a state of extremely serious water pollution (Wang et al. 2016). Therefore, according to the national authorities, the Tai Hu basin, as a sensitive water body, has to achieve quality level III according to the Environmental Quality Standards for Surface Water (GB3838-2002) (see Table 7) (Wang et al. 2016). This has led to a tightening of the regulations in the catchment area, enforcing provincial and city regulatory standards, stricter than the national regulations. For the studied WWTP, the new regulation “City Assessment Standard” was enforced as of 2021. In order to comply with the new, stricter norms, the example WWTP started in 2020 with upgrading measurements, including an additional internal recirculation and a downstream denitrification filter.

The design flow of the plant is 150,000 m^3/d, a value that is often surpassed during peak periods, even for weeks, especially in summer. This can lead to hydraulic overload, which is already observed in the secondary clarifiers with surface loading values above the recommended 1.6 m/h (see Table 8). The inlet flow and its composition appear to be highly dependent on the precipitations in the region. In fact, in the year 2019, a significant increase in the pollutants load was observed i.e. 34% more COD on average respect to 2018, related to heavy rain periods in March and by the end of summer 2019. This is possibly due to surges and dragging of deposits in the sewer. The main influent parameters are summarized in Table 5.

Table 5. Relevant influent parameters of the example WWTP

Parameter (WWTP influent)	**Symbol**	**85% -Quantile**					**Unit**
		2017	**2018**	**2019**	**2017 - 2019**	**2017 - 2018**	
Plant size	$PE_{COD,\ 120}$	437,719	460,734	619,620	506,024	**449,227**	PE
	$PE_{BOD,\ 60}$	338,067	351,852	445,435	378,451	344,959	PE
	$PE_{TN,\ 11}$	341,936	351,810	432,597	375,447	346,973	PE
	$PE_{TP,\ 1.8}$	308,982	357,265	409,167	358,471	333,124	PE
Daily influent flowrate	$Q_{in,d}$	145,469	143,653	153,213	149,588 (121,445)	144,560 (120,586)	m^3/d
COD load	$L_{COD,d}$	52.5	55.2	60.7	60.7	53.9	Mg/d
BOD_5 load	$L_{BOD,d}$	20.3	21.1	21.4	23.1	21.5	Mg/d
TN load	$L_{TN,d}$	4.8	5.0	6.6	5.4	4.9	Mg/d
TP load	$L_{TP,d}$	0.56	0.64	0.73	0.64	0.6	Mg/d
Inhabitant-specific loads**	$L_{COD,PE,d}$	116.7	112.8	164.9	142.0	119.7	g/(PE·d)
	$L_{BOD,PE,d}$	45.1	46.9	59.4	51.7	46.0	g/(PE·d)
	$L_{TN,PE,d}$	10.6	11.1	14.6	12.1	10.9	g/(PE·d)
	$L_{TP,PE,d}$	1.2	1.4	1.6	1.4	1.3	g/(PE·d)
COD/TN ratio	COD/TN	12.8 (10.4)	12.6 (10.5)	17.3 (12.5)	14.3 (11.2)	12.7 (10.4)	-
BOD/TN ratio	BOD/TN	5.2 (4.0)	5.0 (4.1)	6.1 (4.4)	5.4 (4.2)	5.1 (4.0)	-
COD/BOD_5 ratio	COD/BOD_5	2.8	2.7	2.8	3.4 (2.8)	3.5 (2.7)	-

* Values in brackets () are for the average values
** Calculated with 450.000 PE

The COD/BOD ratio variates mostly between 2 and 4 (85% of the COD/BOD values are between 2 and 4), an indication of a moderately biodegradable influent. This is a trend observed in many Chinese WWTP (see Chapter 2.2). Especially during rainy periods, the COD/BOD ratio reaches values well above 4, which is an indicator of slowly or non-biodegradable solids carryover to the plant.

The composition of nitrogen in the influent and effluent of the WWTP is summarized in Table 6.

Table 6. Nitrogen composition in the influent and effluent of the example WWTP

	Parameter	Symbol	2017 - 2019		Unit	% of TN (average)
			85% Quantile	Average		
Influent	Total nitrogen	TN	46.1	37.4	mg/L	
	Ammonium nitrogen	NH_4-N	37.4	29.3	mg/L	78.3%
	Nitrate nitrogen**	NO_3-N	1.2	0.9	mg/L	2.4%
	Organic nitrogen*	N_{org}	14.3	7.2	mg/L	19.3%
Effluent	Total nitrogen	TN	9.98	8.39	mg/L	-
	Ammonium nitrogen	NH_4-N	0.52	0.33	mg/L	3.9%
	Nitrate nitrogen**	NO_3-N	9.09	7.41	mg/L	88.3%
	Organic nitrogen*	N_{org}	1.25	0.65	mg/L	7.7%

*Calculated
**Data available from 2018

The nitrogen content in the influent of the example WWTP is mostly ammonium, and only low concentrations of nitrate in the inflow are observed (<1 mg N-NO_3/L), characteristic of dominantly domestic wastewater. The inflow COD/TN ratio is relatively variable, with values often below the desired minimum ratio of 100:10 (Winkler 2012) (Permatasari et al. 2018); (Wang et al. 2020) half of the time. Due to the relatively low COD/BOD_5 ratio, the BOD_5/TN ratio is even lower, reaching values around 4 on average. There are only occasional longer periods (from one to maximum two weeks) with constantly unfavourable C/N ratios, which could justify an external C-source dosing.

The influent pH value is often under 7.0, which is in the lower range for domestic wastewater (Henze 2011). Moreover, it dropped visibly in the last half of 2019, from ca. 7.2 to 6.8 on average. It is not clear if this is due to a change in the influent characteristics or to a change in the measurement method. However, possible inhibitions in biological nitrogen removal have been, in principle, discarded, as there is no relation between the nitrogen concentration in the effluent and the influent pH value. As mentioned in the literature research, inhibitions in nitrification can be observed from pH < 6.8 (Tchobanoglous et al. 2003). Moreover, it is important remembering that alkalinity is not measured in the example WWTP.

The example WWTP fulfils the current norms for COD removal, but, as for many WWTP in China, they could have problems fulfilling the 2021 norm for nitrogen (Total Nitrogen and ammonium nitrogen), as effluent values in previous years would have exceeded the current norm. The outflow ammonium concentration is normally very low (0.52 < NH_4-N mg/L), with periodical punctual peaks (see Annex 12.3.3). These peaks could be due to poor mixing, due to large nitrification volume, outdated aeration diffusers, and a low degree of instrumentation and automation (see next Chapter 3.3).

Moreover, the denitrification basin seems to be too small in proportion to the total activated sludge volume, as it is below the recommended 20% described in the DWA-A 131. The requirements for the proportion of denitrification are not commonly mentioned in the international literature (e.g. Metcalf & Eddy). Some international studies outside Germany consider this parameter for the design of upstream denitrification processes, but they are derivative from the German standards, e.g. (Insel et al. 2015), (Insel et al. 2019).

This, together with a too long HRT in the primary clarification stage (HRT ≥ 2 h), and an often unfavourable C/N ratio in the influent, which is worsened due to the large primary clarifiers, are probably the main aspects affecting negatively nitrogen removal at the example WWTP.

The low phosphorous concentrations in the effluent (0.1 mg/L TP on average) are due to an overdosing of precipitant agents (e.g. PFS) in the efficient sedimentation treatment stage and therefore it is not known to what extent the biological phosphorous elimination is effective.

Graphs with the influent flow, influent and effluent concentrations for COD, TN and NH_4-N, cumulative distributions and relevant ratios in time during the studied period, can be found in Annex 12.3.

Table 7. Monitoring values (24 h composite sample) of the example WWTP compared to the requirements of the national norm Grade I-A and the City Assessment Standard for the Tai Hu Basin

Parameter	Concentration, mg/L					Reference
	COD	BOD	NH_4-N	TN	TP	
Average	17.3	5.7	0.3	8.2	0.1	From WWTP data
Quantile 85%	20.4	7.5	0.5	10.0	0.2	
(GB18918-2002) grade I-A standard	50	10	5 (8)	15	0.5 [1]	(State Environ. Protection Admin. Peoples Republic of China 2002)
(GB18918-2002) grade I-B standard	60	20	8 (15)	20	1 [1.5]	
(GB18918-2002) grade II standard	100	30	25 (30)	-	3	
(GB18918-2002) grade III standard	120	60	-	-	5	
Quality III (Water body quality to reach in the Taihu basin)	6	-	1	1	0.05	(Tang et al. 2012)
City Assessment Standard (CS)	30	10	1.5 (3)	10	0.3	WWTP operator

Numbers in round brackets ( ) are for wastewater temperatures below 12 °C
Numbers in square brackets [ ] are for plants built before 2006

The norm GB18918-2002 indicates that the sampling and monitoring are carried out at the end of the treatment process outfall of the WWTP. The effluent should be equipped with an automatic proportional sampler device or online measurements. The sampling frequency must be at least once every 2-h, to take 24-h mixed samples, to deliver the daily average value. The standard 1-A is the basic requirement for the effluent from urban WWTP to be used as reuse water. The standard 1-A is applied when the effluent from the WWTP is introduced into rivers and lakes with low dilution capacity for uses such as urban landscape water and general water reuse[5] (State Environmental Protection Administration Peoples Republic of China 2002).

[5] Translated from Chinese

Table 8. Relevant dimensions of the example WWTP

Parameter	Symbol	Value	Unit	Information
Primary clarifiers (PC)	V_{PC}	4 · 3,200	m³	PC volume, four units
	HRT_{PC}	*3.1 (2.6)	h	-
Temperature	T	10 – 24	°C	Wastewater temperature
Activated sludge basins volume	V_{AT}	4 · 24,000	m³	Total volume
		213.3 ˣ	L/PE	Specific total volume
	V_N	4 · 16,000	m³	Aerobic volume (nitrification)
		142.2 ˣ	L/PE	Specific nitrification volume
	V_D	4 · 4,000	m³	Anoxic volume (denitrification)
		35.6 ˣ	L/PE	Specific denitrification volume
	V_{An}	4 · 4,000	m³	Anaerobic volume
		35.6 ˣ	L/PE	Specific anaerobic volume
	V_D/V_{AT}	16.7	%	Denitrification volume ratio
	V_D/V_{AT}	20 – 60	%	Recommended value (DWA 2016)
Sludge age (activated sludge)	SRT_{design}	15 - 20	d	Design value
	SRT_{calc}	*29.8 (22.6)	d	Calculated average (2017-2019)
Secondary clarifier (SC)	A_{SC}	4 · 1,600	m²	Surface of the SC
	$q_{A,SC}$	*0.98 (0.8)	m/h	Surface feeding
	$q_{A,SC,max}$	≤ 1.6	m/h	Recommended value for horizontal-flow SC (DWA 2016)

* Values with * correspond to the 85% quantile in the period 2017-2019
Values in brackets () are for the average
ˣ Specific volumes calculated with 450,000 PE
SRT = Sludge age, solids retention time

3.3 Processes Description and Analysis

For the optimisation of nitrogen removal at the example WWTP, the individual components and processes of the WWTP plant are of great importance, as each stage plays an important role and can be depicted in the model of the plant for testing different optimisation strategies. A general and a detailed scheme of the example WWTP are shown in Figure 9 and Figure 10.

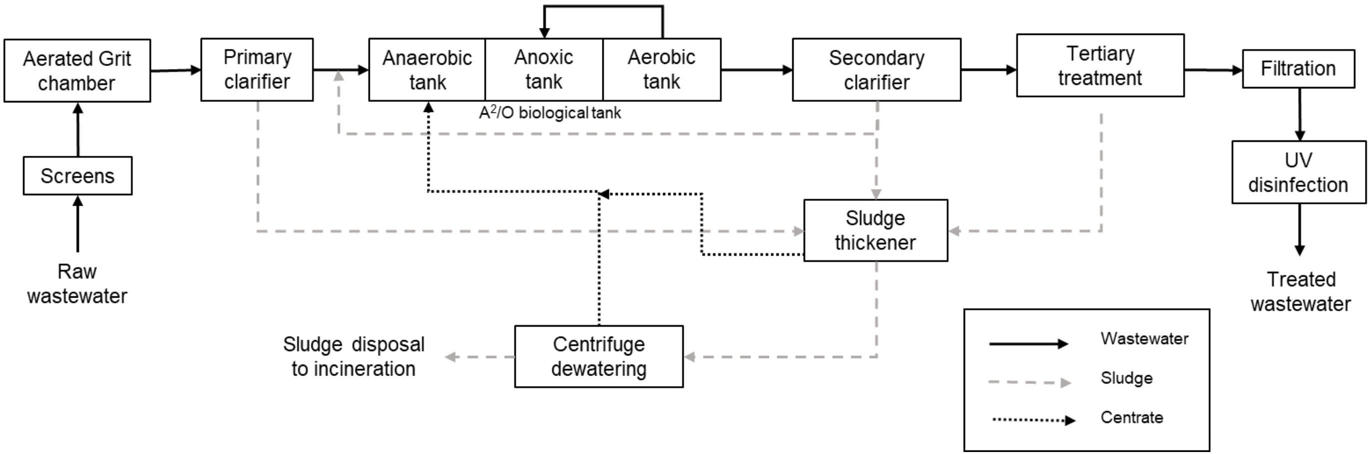

Figure 9. General flow scheme of the example WWTP (Vergara-Araya et al. 2021)

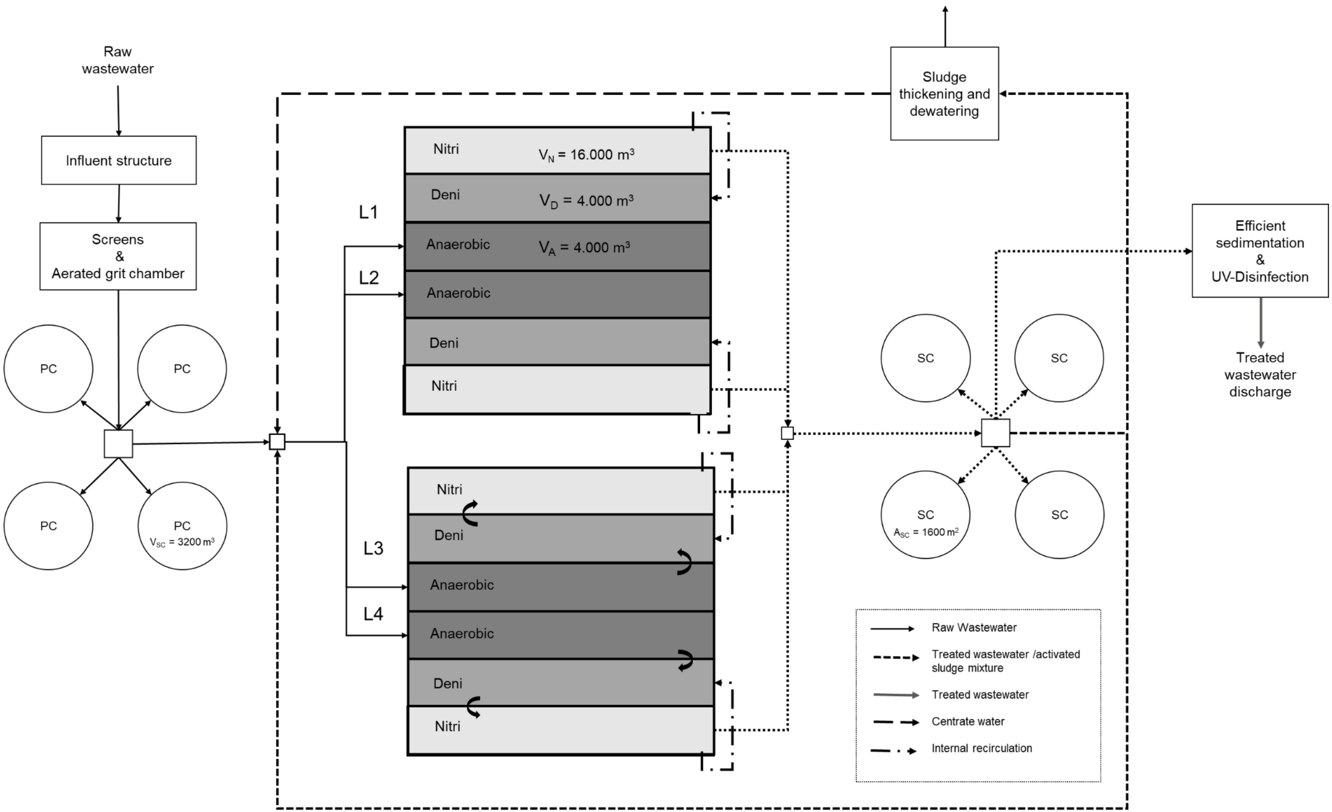

Figure 10. Detailed flow scheme of the example WWTP

3.3.1 Mechanical Treatment (Pre-treatment and Primary Clarification)

The influent wastewater is lifted in a pumping station with five submersible pumps plus a reserve pump. The pre-treatment consists of coarse and fine screens for the removal of large and coarse solids. Then the water flows to a two-lane aerated sand trap of 23 by 8 meters. Fat removal in this stage is not informed. After the pre-treatment, the wastewater goes to primary clarification to remove settleable organic solids. The plant has four primary clarifiers, each with a diameter of 30 m and 4.5 m in depth. The designed Hydraulic retention time is 2 hours, but in reality, the average is ca. 2.6 h. The collected primary sludge has an average concentration of 26 g/L, but it fluctuates heavily from day to day, from 10 to 50 g/L.

3.3.2 Activated Sludge System

The wastewater after primary clarification is feed parallel to each treatment line. Each line has a volume of 16,000 m^3 (total activated sludge volume, V_{AT} = 96,000 m^3) divided into three main sub-areas: Anaerobic (A), Anoxic for Denitrification (D) and Aerobic for Nitrification (N). The volumes are distributed as $V_A : V_D : V_N = 1 : 1 : 4$. This results in a Denitrification volume of 16.7% with respect to the total biological treatment volume. The volume of activated sludge tanks per inhabitant equivalent is 213 L/$PE_{COD,120}$. This value is comparable with similar size plants in Germany e.g. Magdeburg-Gerwisch WWTP (426.000 $PE_{COD,\ 120}$) with 203 L/PE; this plant has a cascade denitrification setup and a much larger denitrification volume with a V_D/V_{AT} of ca. 43%.

Each line has its own internal recirculation for nitrate-rich wastewater and the mixing in the anaerobic and anoxic zones is provided by 7 submerged stirrers. Water flows along each section of the reactor, which is more than 80 metres long. The nitrification tank is divided into 3 sections, all of equal length and air injection takes place at four points along each section of the tank.

The sludge concentration in the biological tanks (Mixed liquor suspended solids, MLSS) fluctuates between 4 and 8 g/L, reaching its lowest values in late summer, consistent with the variation in nitrification activity associated with temperature fluctuations. Typical MLSS concentrations in activated sludge tanks are around 2 and 5 g/L (Tchobanoglous op. 2014) (PDEP 2014) because too high MLSS concentrations can limit oxygen transfer to the sludge flocs (Krampe and Krauth 2003). MLSS values above 6 g/L are observed during the winter months in 2018 and 2019. The organic content of the activated sludge reaches an average of 59% (62% in the 85%-Quantile) reasonable for a plant with aerobic sludge stabilization.

Regarding the sludge age, contradictory information has been collected. On one hand, according to the WWTP design, the target SRT is 29.4 d, calculated with 3,500 g/m^3 MLSS, however, the MLSS concentration is usually way above that value, making the real sludge age even higher. On the other hand, the plant operator has informed a target SRT between 15 and 21 d, which is often too low to reach sludge stabilisation (≥ 25 d according to the DWA-A 131). Even when considering the effect of the wastewater temperature, i.e. when T< 19 °C the target SRT for aerobic sludge stabilisation is > 15 d; when T< 14 °C the target SRT is > 21 d, according to Equation 4.

The excess sludge amount is not collected by the plant operators, which is an indicator that the SRT is probably a parameter which is not closely monitored. Therefore, the operational sludge age is estimated as the difference between the daily dried sludge produced and the estimated primary and tertiary sludge production, according to the plant operator. An overview of the sludge amounts is provided in Figure 11, where the difference between dewatered sludge and primary and tertiary sludge represents the excess sludge amount.

$$F_{ES} = F_{DS} - (F_{PS} + F_{TerS}) \qquad \text{Equation 3}$$

Where:

F_{ES} = mass flow of excess sludge, Mg TS/d

F_{DS} = mass flow of dewatered sludge (measured), Mg TS/d

F_{PS} = mass flow of primary sludge (estimated as 60% of the influent settlable solids), Mg TS/d

F_{TerS} = mass flow of tertiary sludge (estimated based on the consumption of PFS), Mg TS/d

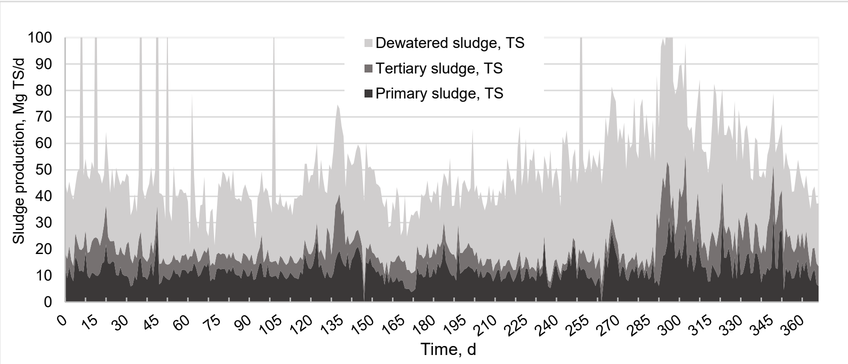

Figure 11. Estimation of the amount of primary and tertiary sludge, based on the dewatered sludge amount.

The SRT is calculated using Equation 4, with the measured MLSS concentration and the excess sludge concentrations. The total volume is considered constant, as 96,000 m^3.

$$\mathrm{SRT} = \frac{\mathrm{MLSS} * \mathrm{V_{AT}}}{\mathrm{TS_{ES}} * \mathrm{Q_{ES}}} \qquad \text{Equation 4}$$

Where:

SRT = sludge age, sludge retention time, d

MLSS = Mixed liquor suspended solids, g/m^3

V_{AT} = activated sludge basins volume, m^3

TS_{ES} = total solids concentration of the excess sludge, g/m^3

Q_{ES} = mass flow of the excess sludge, m^3/d

The required sludge age for aerobic sludge stabilisation according to the DWA-A131 is calculated in Equation 5,

$$\mathrm{SRT} = 25 * 1.072^{(12-\mathrm{T})} \qquad \text{Equation 5}$$

Where: T = wastewater temperature, °C

The average is 38 days, but it fluctuates heavily, as can be seen in Figure 12. It is worth remembering that aerobic sludge stabilization is not recommended for WWTP this size (or with COD loads this high) in Germany.

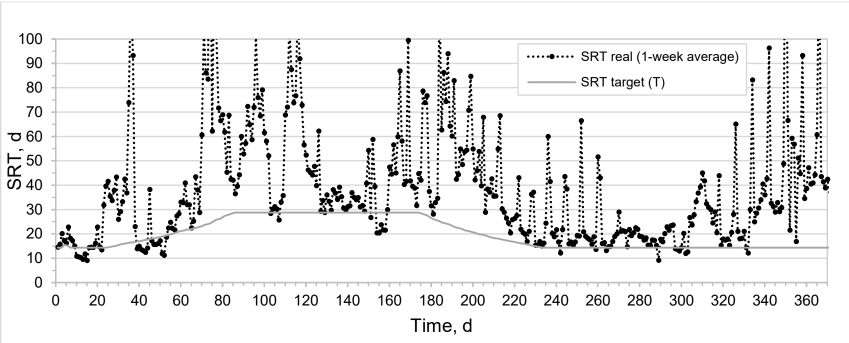

Figure 12. Estimated sludge age in the example WWTP in the period September 2017 and September 2018, as weekly average (black dashed line) and target sludge age (grey line) based on the temperature according to the DWA-A 131 for aerobic sludge stabilisation

In Figure 12, it results clear that the current sludge age fluctuates heavily, the peaks come from the estimation based on the sludge amount produced, as observed in Figure 11 and the fluctuations in MLSS concentration. The fluctuating SRT is detrimental to the process stability and could be the reason for the fluctuating ammonium concentrations in the effluent. In most of the studied period, the real SRT is much higher than the minimum required (up to 1100% higher than the target SRT in the studied period), leading to higher aeration requirements. In some short periods, the sludge age is smaller than the minimum required according to the temperature, up to 42% below in the studied period.

This can be detrimental to nitrification, e.g. leading to ammonium peaks, as can be observed in Figure 13, e.g. during the periods between days 40 and 70, 100 and 120 and 300 and 320, which all come after a period where the average sludge age has shifted abruptly or has been low for a longer period of time.

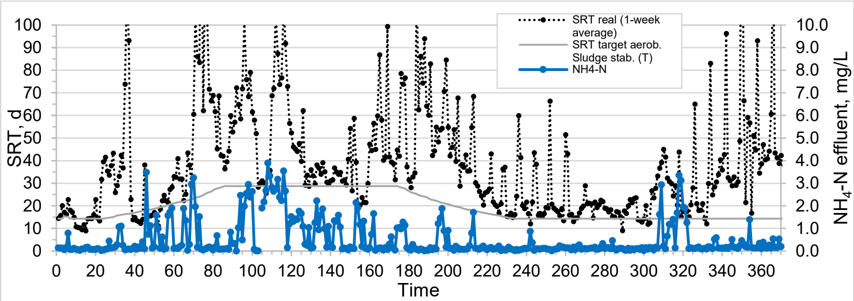

Figure 13. Estimated sludge age in the example WWTP in the period September 2017 and September 2018, as weekly average (black dashed line) and target sludge age (grey line) based on the temperature according to the DWA-A 131 for aerobic sludge stabilisation and effluent ammonium concentration in blue.

According to the maximum q_A value, for horizontal-flow secondary clarifiers in the example WWTP appears to be hydraulically underloaded with an average of 0.8 m/h (0.98 m/h, 85% quantile).

3.3.3 Efficient Sedimentation Tank and Fibre Filter Tank

The high-density sedimentation tank (efficient sedimentation) is a combination of a coagulation zone with dosing of Ferric salts ($FeCl_3$ or polyferric sulphate (PFS)), a flocculation zone with the addition of Polymer (PAM), and a lamellar sedimentation tank with a static thickener. This tank has a higher performance than a traditional sedimentation tank, separating organic substances and SS in a smaller volume and area. To assure low concentrations of Phosphorous at the outflow of the plant, there is an overdosage of ferric salts (1.7 g dosed Fe per g influent TP on average). From May 2019, the dosing of Ferric salts was replaced by polyaluminium chloride (PAC)[6]. The fibre filter tank acts as a filter for particulate solids that might be still present in the wastewater outlet flow after the secondary clarification and efficient sedimentation process.

3.3.4 Sludge Treatment

The sludge produced in the activated sludge process is aerobically (partially) stabilized and it is thickened together with primary and tertiary sludge. The sludge is mechanically thickened and then dewatered in a decanter centrifuge. The supernatant and centrate water are returned to the process for treatment in the water line. The dewatered sludge is disposed of via incineration outside of the WWTP.

The lack of importance of the sludge line evidenced by the lack of measurements is not a coincidence. In China, the cost of sludge treatment is often not included in wastewater fees or charged at an insignificant rate, making it impossible for many plants to afford the costs (Liu and Han 2015). Besides, the WWTP usually have no incidence in the disposal pathway, which is decided by the local authorities.

[6] The WWTP returned to PFS at the end of 2020

3.3.5 Energy Consumption and Operational Values

The total power consumption and power consumption ratio have increased steadily in the last three years. This goes hand in hand with an intensification of the wastewater discharge norms. The example WWTP had an average power consumption ratio of 0.29 kWh/m^3 in 2017 and reached 0.36 kWh/m^3 in 2019. The power consumption ratio in kWh per m^3 treated wastewater is somewhat related to the variation in TN concentration in the influent (see Figure 14), indicating a dominating energy consumption in the activated sludge stage.

The energy consumption of WWTP with conventional activated sludge is estimated in a range of between 0.27 – 1.89 kWh/m^3, depending on the country (Gu et al. 2017) locating the example WWTP in the lowest range. Factors that influence this value are geographical (e.g. location of the WWTP, topography), plant size (PE, influent load), types of the treatment process, type of equipment used, degree of self-sufficiency, age of the WWTP, the experience of the managers, etc. (Gu et al. 2017), (Niu et al. 2019).

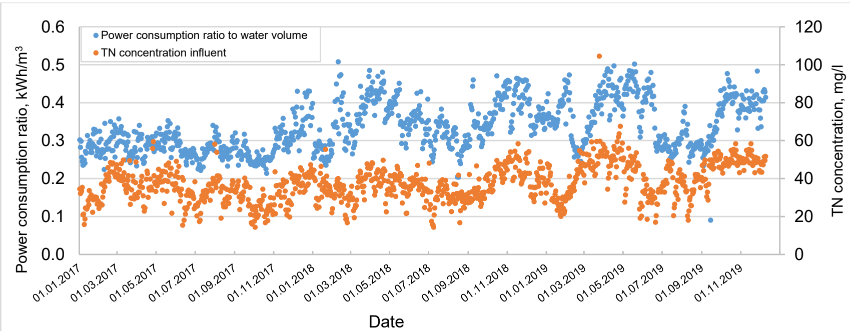

Figure 14. Power consumption ratio and Total nitrogen (TN) concentration in the influent of the example WWTP between 2017 and 2019

There are several international guidelines, especially in Europe, to evaluate the energy efficiency of WWTP such as the CEN/TR 17614 European Standard Method for Assessing and Improving the Energy Efficiency of Waste Water Treatment Plants (CEN 2021), the Austrian Benchmarking of Wastewater Treatment Plants (ÖWAV 2018) and the German Worksheet for Energy check and Energy analysis DWA-A 216 (DWA 2015).

One of the most complete and detailed ones is the German DWA-A 216, therefore it is used for evaluation and comparison. When calculating the energy consumption ratio in relation to the BOD-based plant size[7] (390,000 PE$_{BOD,60}$), the energy consumption is on average 36.2 kWh/(PE·a). This, according to the energy consumption observed in plants of similar size in Germany (i.e. size class 5) and treatment technology (activated sludge with aerobic sludge stabilisation) in (DWA 2015), situates the example WWTP in the lowest 46% (see Figure Figure 15). It is worth remembering that in Germany, WWTP this size do not have aerobic sludge stabilisation.

[7] According to DWA A-216

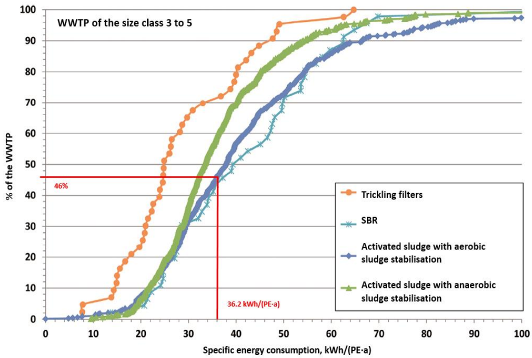

Figure 15. Specific total electricity consumption depending on the treatment process according to the DWA-A 216 (DWA 2015)

The example WWTP is therefore relatively energy efficient, but three main aspects should be remembered before making a direct comparison. First, WWTP in China are on average much larger than in Germany. 60% of the WWTP in China are between 50,000 and 250,000 PE[8] (Zhang et al. 2016). The average size in Germany is 13,800 PE[9] (BMU 2017), especially considering the WWTP that stabilizes sludge aerobically (normally < 30,000 PE) and there is an aspect of the economy of scale that makes larger WWTP more efficient in terms of specific energy consumption.

Second, there is a difference in the norms to comply with. Not only stricter norms apply for decades now in Germany (in China only recent changes), but also the sampling for control is stricter (2-hour composite sample vs 24-hour composite sample in China). A third aspect is that sludge treatment is not comparable between both countries. Meanwhile, in Germany, sludge undergoes a complex treatment with strict regulations (stabilisation requirements, required dryness, costs of transport and disposal), usually, including anaerobic sludge stabilisation in larger plants, China does not always consider sludge treatment as part of wastewater treatment. Chinese WWTP have usually fewer processes for sludge treatment and lower requirements for transport and disposal, and the stabilisation degree is not established.

3.4 Design Check WWTP

The WWTP size and dimensions were checked, to see if the current plant size and dimensions can fulfil the new discharge values requirements i.e. norm CS (see Table 7). There are no known general guidelines for the design of WWTP in China, at least in English. There are published papers (e.g. (Palmer and Fritz 2004), (Foerster et al. 2021)) which provide some design information but are not detailed.

Therefore, the required volume and dimensions were calculated according to the guidelines provided by the DWA-A 131 and Metcalf & Eddy (Tchobanoglous op. 2014), and the treated

[8] Calculated with 200 L/(PE·d), for influent flowrates between 10,000 and 50,000 m^3/d,

[9] Calculated with 200 L/(PE·d), for 10.000 WWTP, treating 10.7 Billion m^3/a.

wastewater amount and characteristics. The main assumptions of the design are summarized in Table 9 and more details can be found in Annexes 12.4.2 and 12.4.3.

Table 9. Assumptions for the re-design of an A2/O stage according to the DWA-A 131 and Metcalf & Eddy

Parameter		Example WWTP	Unit	Comment
Wastewater temperature	T	12	°C	Recommended design temperature
Sludge age	SRT	25 (calculated)*	d	Required for aerobic sludge stabilisation
Mixed Liquor Suspended Solids	MLSS	3.5	g/L	Design concentration according to the plant operator
Influent flow	$Q_{in,\ 85\%\ quantile}$	149,588	m^3/d	85% quantile, recommended for design
COD load to activated sludge system	$L_{COD,in,\ 85\%\ quantile}$	55.4	Mg/d	85% quantile, recommended for design
Specific COD load to activated sludge system	$L_{spec,COD,in,\ 85\%\ quantile}$	123.1	g/(PE·d)	Calculated with 450,000 PE
Effluent NH_4-N conc. norm Grade I-A	$C_{NH4\text{-}N,\ out,\ I\text{-}A}$	2	mg/L	Target to comply with the discharge norm
Effluent NH_4-N concentration, norm CS	$C_{NH4\text{-}N,\ out,CS}$	0.5	mg/L	Target to comply with the discharge norm
Effluent NO_3-N concentration, norm Grade I-A	$C_{NO3\text{-}N,\ out,\ I\text{-}A}$	10	mg/L	Target to comply with the discharge norm
Effluent NO_3-N concentration, norm CS	$C_{NO3\text{-}N,\ out,\ CS}$	8	mg/L	Target to comply with the discharge norm

* Metcalf & Eddy

The comparison between the real WWTP dimensions and the required size according to the DWA-A 131, is summarized in Table 10.

Table 10. Comparison between the example WWTP and the re-designed parameters

Parameter	Example WWTP	Norm Grade I-A		Norm CS		Unit
		DWA-A 131	M&E	DWA-A 131	M&E	
V_{AT}	96,000	151,800	55.600	151,800	113,400	m^3
Specific V_{AT}	213.3	337.8	123.5	337.3	252.0	L/PE
V_D/V_{AT}	0.17	0.42	0.10	0.47	0.24	-

The comparison shows that the overall volume (V_{AT}) of the example plant according to DWA-A 131 is too small to comply with the effluent requirements of the norm CS, and the denitrification proportion (V_D/V_{AT}) is also too small. The German approach focuses as well on having enough biomass for nitrification, based on the design temperature.

In the calculation procedure in M&E, the total volume depends highly on the effluent NH_4-N concentration and the concentration of NO_3-N in the recirculated sludge. Finally, the anoxic tank volume depends on the desired HRT in the anoxic tanks, and the capacity to reduce the recirculated nitrate. According to these results, the current volume is large enough to comply with the norm Grade I-A, but it is slightly too small to comply with the norm CS. In this case, the denitrification volume proportion is also too small.

This has several probable causes, and some of them can be simultaneously true:

- The plant was designed with different parameters and different assumptions.
- The plant was designed to comply with the requirements of the norm Grade I-A, and the current requirements (norm CS) are stricter, therefore the required volume is larger.
- There was no reliable information about the wastewater characteristics and quantity at the moment of the design, and more favourable wastewater characteristics or less quantity were considered.
- The plant was designed for different wastewater characteristics, e.g. higher C/N ratio
- The design did not consider the rainwater and the maximum flow or was designed for average values instead of 85%-percentile.
- The design and operational sludge age is too high (See Chapter 3.3.2) and it is probably a parameter which not closely monitored.
- The real operational MLSS concentration is higher than the value proposed by the operator for the design

Some of these problems have been recently documented in the paper from Zhang *et al.* (Zhang et al. 2021), which lists several problems, among them a mismatch between the designed WWTP and the actual wastewater quality, insufficient facilities and problems in the design, low efficient facilities, insufficient equipment, etc.

However, the static dimensioning does not take into account possible reserves in everyday operation, and therefore the parameters and procedure followed in the DWA-A 131, can lead to over-dimensioning of the activated sludge stage. Moreover, the design according to the DWA-A 131 is COD based, and the COD/BOD ratio of the example WWTP is relatively high, 2.4 on average.

It is important remembering, especially for subsequent subchapters, that inflexible or undersized designs of WWTP cannot be fully solved by ICA alone (Olsson et al. 2014), however, improvements can be obtained.

For the secondary clarifier, the information provided in Table 11 was used. The redesigned secondary clarifier according to the DWA-A 131 (A = 4,167 m^2) is much smaller than the current one (A = 6,362 m^2). Therefore, the current activated sludge system (biological tanks and secondary clarifier) appears to be under-dimensioned considering the German standard. This could bring problems in the excess sludge concentration and extraction, as it favours the extraction of diluted sludge or sludge with fluctuating concentration, and in the energy efficiency of the plant, because there is an unnecessary pumping of water and increased requirement of the thickening and dewatering treatment stages.

However, when considering the design based on Metcalf and Eddy; the size of the plant appears adequate. In fact, the design of the secondary clarifiers in the example WWTP could be based on a similar methodology to the one suggested by Metcalf & Eddy.

Table 11. Parameters for the re-dimensioning of the secondary clarifier (SC)

Parameter		Example WWTP	DWA-A 131	M&E	Unit	Information
Surface loading	$q_{A,max}$	0.98*	1.5 **	-	m/h	
Design Influent flow for rainy weather	Q_M	150,000*	150,000*	150,000*	m^3/d	85%-quantile value (no information about rainy or dry weather is available)
Area SC	$A_{SC,\ total}$	6400	4,167	6,233	m^2	Calculated as: $A_{SC} = Q_M/q_A$
Area single SC	$A_{SC,\ unit}$	1600	1040	1560		
Number of SC	n_{SC}	4	4	4	-	
Diameter single SC	$D_{SC,unit}$	45	36.4	44.5	m	
Height to Radio ratio	$H_{SC}:R_{SC}$	0.23	0.23	0.23	-	
Height SC Estimated	$H_{SC,unit}$	5.1	4.2	4	m	Estimated
Total volume of SC	$V_{SC,\ total}$	32,450	17,500	-	m^3	
Solids loading	L_{solids}	1.0*	-	5.95	kg MLSS/ $(m^2 \cdot h)$	Between 4 and 6 kg MLSS/$(m^2 \cdot h)$. Calculated as $(1+R) \cdot Q \cdot MLSS/A$
*85% Value; **Maximum value						

This design check shows that it is very hard to evaluate a WWTP statically, therefore the use of mathematical models to do it, considering the dynamics of the influent, can be useful to understand the shortcomings of the design and the option for improvement and optimization.

3.5 Summary of Chapter 3

The example WWTP (450,000 $PE_{COD,120}$) is representative of WWTP in China and other locations worldwide, based on activated sludge with upstream denitrification, variable C/N influent ratio and strict effluent values. The WWTP has a classic pre-treatment, including primary clarifiers, although it does not stabilize sludge anaerobically, which is typical in China, but not in Germany for a plant that size.

The sludge age in the activated sludge stage is very variable, so it does not appear to be a controlled variable. The SRT is usually much higher than the minimum required, but there are some periods where it is below, which can cause some problems in nitrogen removal and process stability.

The WWTP is relatively energy efficient, when compared with similar plants in Germany, however, due to the lack of anaerobic sludge stabilisation, the comparison is carried out with a smaller WWTP.

There are no known Chinese standards for the design of WWTP, but a comparison between the standard DWA-A 131 (Germany) and the design proposal by Metcalf and Eddy (USA) show certain differences in the activated sludge volume and secondary clarifiers. The German standard indicates the WWTP is under-dimensioned, and M&E does not. The German standard is COD based and puts more emphasis on denitrification capacity.

The design check showed some of the shortcomings of a static design approach, therefore in the next chapter, the dynamic modelling approach will be used to check the operation of the example WWTP and suggest realistic optimization and improvement measurements.

4 Model of the WWTP

Although the static design is still a widely used method for designing and testing the performance of WWTPs, it has several limitations, as noted in previous chapters. Most of these limitations can be overcome with dynamic modelling, using software tools. Dynamic modelling of WWTPs takes into account the variable nature of the influent wastewater and allows testing of dynamic operational and automation strategies, taking into account the complexity and level of challenges faced by modern WWTPs.

In this chapter, the example WWTP is modelled, based on a standard calibration and validation procedure, to provide in the end a useful model for operational testing to improve plant performance.

The example plant is modelled as a classical WWTP with mechanical biological treatment, including an activated sludge stage with upstream denitrification and biological and chemical P-removal. The filtration stage was modelled as an ideal secondary clarifier, to remove all suspended solids.

The WWTP was modelled with the information provided by the operator and observations carried out during plant visits in 2019 in the framework of the PIRAT-Systems project (https://www.bauing.uni-kl.de/pirat-systems). The example WWTP was built with the model *asm3h,* default in SIMBA, including phosphorous precipitation by the addition of ferric salts. However, the phosphorus concentrations and phosphorous removal were not modelled, as these parameters are not considered in this work. The *asm3h* includes the IWA Activated Sludge Model Nr. 3 (ASM3) with modifications and parameters following the recommendations by the researchers' group HSG (http://hsgsim.org) as described in (Dohmann 1993). This model will calculate simulation results in accordance with the German design guideline DWA-A 131 (DWA 2016). The model was built initially in SIMBA Version 3.2.26, but the software has been updated several times since, the last used is 4.3.4 (March 2021).

To carry out the modelling, the guidelines provided by the HSG group (Langergraber et al. 2004) were followed, as described in Section 2.3.2.

4.1 Pre-simulation

In order to test preliminarily the plausibility of modelling the selected WWTP in SIMBA, to build the basic model structure and choose the corresponding blocks, a model with average values (i.e. steady-state model) was built. This also allows to carry out sensitivity analysis and identify the parameters with higher relevance for the study objectives.

After this first test, the inlet data flow block “TG_HSG” was incorporated. This block creates a variable inflow based on the HSG procedure, which calculates a typical dry weather influent pattern based on a method developed by the HSG group and published in (Langergraber et al. 2007) and (ifak e.V. 2018). The block was completed with average data from September 2017 and September 2018 for the example WWTP. Here, several parameters, such as tank volume, types of aerators, sludge indexes, etc. were adjusted.

The results of the steady-state model calibration are presented in Figure 16. The static model shows a very good fit for COD and a good fit for ammonium and nitrogen values in the effluent, as the difference between the measured average and modelled values is less than 5% and

15%, respectively. The main parameters are summarized in Table 12. The difference between the average values is less than 15% for all parameters.

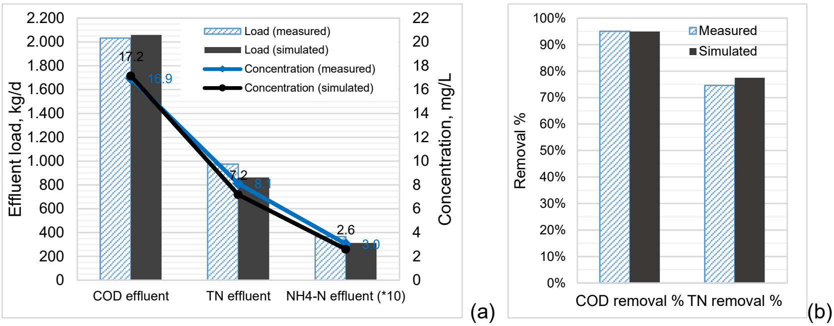

Figure 16. (a) Effluent load and concentrations comparison; (b) COD and TN removal comparison. Measured (blue) and simulated values (black) (Modified from (Vergara-Araya et al. 2021))

Table 12. COD and nitrogen balance and further data from the preliminary model

Parameters	Unit	From WWTP data	Simulated	Difference %
SRT	d	20.9*	20.9	0%
Excess sludge	m^3/d	2,300*	2,000	13.0%
Primary sludge concentration	mg/L	26.4	26.4	0%
Primary sludge flowrate	m^3/d	572	544	4.9%

* Not measured, estimated by the plant operator

There are slight differences between the reports/measured data and the simulated data. It must be considered that here the pre-simulation is carried out with average values, which are not necessarily representative of all the conditions that take place during a year in the WWTP. Moreover, the sludge age is an estimated value by the plant operator, but it is a controlled variable in the model. However, this can be the cause of the differences in the effluent values.

The primary sludge concentration is measured in the example WWTP and can be set as a parameter in the model. However, there is an evident difference in the excess sludge production between the WWTP data and the simulated value. It is, however, important to highlight, that the sludge production is not measured in the example WWTP, therefore the used value is an estimate provided by the plant operator during a plant visit in 2019. This might be the main cause of the difference between both values.

4.1.1 Sensitivity Analysis

The robustness of the model was tested with a sensitivity analysis. A sensitivity analysis is a tool to assess the effect of changes in input parameters value on the output value of a simulation model. A sensitivity analysis consists of the generation of response curves by modifying the input data of a model (Torregrossa and Hansen 2018). Here, a sensitivity analysis will help to identify which parameters have a larger influence on the nitrogen-compounds concentration in the effluent of the WWTP, plus COD concentration in the effluent and sludge production, since these are the most relevant parameters for the objectives of this work.

To evaluate the sensitivity of the model, the method proposed by van Veldhuizen et al. (van Veldhuizen et al. 1999) is followed. The sensitivity (S) of the parameters (p) with respect to y (effluent ammonium, nitrate and COD concentration and sludge production) is a dimensionless number calculated by:

$$S = \frac{\mathrm{dy}\ (p)}{\mathrm{dp}(y)} \qquad \text{Equation 6}$$

Where:

dp = change in the parameter value p ; dy = change in the output y

The parameters where S>1, are considered sensitive. The sensitivity was calculated for the following parameters was analysed based on a 10% change of the standard values (see Table 13):

- Distribution of COD in the influent (COD fractionation) over respectively:
  - Fraction TSS to COD
  - Fraction of non-volatile TSS (f_B)
  - Fraction of inert soluble COD (f_S)
  - Fraction of inert COD from particulate COD (f_A) (X_I/X_S)
- Internal flows of sludge and mixed liquor
- Air flow to the aerobic zone
- Oxygen set point for the aeration controller
- Sludge retention time
- And a modification of the reactors hydraulic: from one reactor per zone (anaerobic, anoxic, aerobic) to three per zone, maintaining the total volume.

For these tests, one parameter was modified at a time. The MLSS concentration in the activated sludge basins was maintained constant at 3,500 mg/L.

The effluent nitrate concentration is sensitive towards changes in the fraction of inert soluble COD (S ≈ 5.7), as it represents a reduction in the available COD for denitrification. The effluent COD is also highly sensitive to this parameter as well (S ≈ 340) since almost all COD in the effluent should be inert and soluble, especially after a filtration stage. The sludge production is also sensitive to all changes in COD fractionation (-11.34 < S < 20.1), influencing both excess and primary sludge production. The sensitivity for all other parameters is less than 1, i.e. not sensitive. The complete results of the sensitivity analysis can be found in Annex 12.6.

Based on these results, for the model calibration, it would be advisable to carry out a measuring campaign for the soluble and particulate inert fractions of COD. Other reasons to measure the wastewater characteristics are:

- An increased pre-degradation of the COD – which is to be expected due to the existing long sewer networks typical in China;
- Higher wastewater temperatures – which is the case for short periods at the example WWTP (measurements for the mineral TS and the inert particulate COD ($X_{COD,I}$) in the inflow are explicitly recommended at T > 25 °C (DWA 2016)).

Table 13. Data for the sensitivity analysis

Influent COD fractionation and parameters	Symbol	Units	Formula (DWA 131)	Selected value	10% increase
Fraction TSS to COD	TSS/COD	-		0.475	0.5225
Fraction of non-volatile TSS	f_B	-	$\frac{X_{inorgTS,IAT}}{X_{TS,IAT}}$	0.3	0.33
Fraction of inert soluble COD	f_S	-	$\frac{S_{COD,inert,IAT}}{C_{COD,IAT}}$	0.05	0.055
Fraction of inert COD from particulate COD	f_A	-	$\frac{X_{COD,inert,IAT}}{X_{COD,IAT}}$	0.3	0.33
Internal sludge recirculation	Q_{RS}	m^3/d	100% Q_{in}	120,000	132,000
Internal water recirculation	Q_{RZ}	m^3/d	200% Q_{in}	240,000	264,000
Air flow to the aerobic zone	Q_{air}	Nm^3/d	-	$9.09 \cdot 10^5$	$1.00 \cdot 10^6$
Dissolved oxygen in the aeration basin	DO_{sp}	mg/L	-	3	3.3
Sludge age (SRT)	SRT	d	-	27.75	30.5
Set up of hydraulic model	-		-	1 reactor	3 reactors

Due to the limitations imposed by the pandemic in 2020 and 2021, a measuring campaign *in situ* was not possible. Therefore, the default values, advised by the DWA-A 131 were used for calibration (see Chapter 4.2.1).

4.1.2 Other Tests

Due to the model characteristics and the objectives of this work, some further tests were carried out with the pre-simulation model and are described below.

4.1.2.1 Dissolved Oxygen Set Point

The dissolved oxygen set point in the aeration controlled of the aerated tanks was modified, maintaining the MLSS concentration at 3,500 mg/L and approximately constant sludge age. The average effluent nitrogen compounds concentrations are compared in Figure 17.

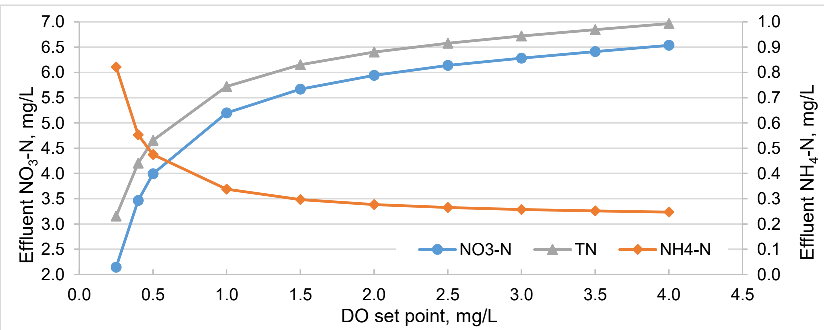

Figure 17. TN, NO_3-N and NH_4-N effluent concentrations by modifying the DO set point in the steady state model of the example WWTP

It results clear that increasing the DO set point is counterproductive for the effluent concentrations and that there is no reason to operate at high DO set points (e.g. 3 mg O_2/L). The results, however, must not be over interpreted, simply assuming that a decrease in the oxygen set point is the solution for improved nitrogen removal. It must be noted, that oxygen contents of less than 0.8 mg/L should generally be avoided, as this increases the risk of the formation of bulking and floating sludge and even the formation of nitrous oxide (Pinnekamp et al. 2017).

Unfortunately, the formation of N_2O, with ca. 300 times greater global warming potential than CO_2, is not considered in the standard models, and its quantification is out of the scope of this work. The modelling of N_2O emissions in wastewater treatment has been modelled by several authors, as reported by (Mannina et al. 2016) and even an extension of ASM3 for N_2O modelling has been proposed by (Blomberg et al. 2018). The ifak e.V., together with other project partners, has also integrated the emissions of N_2O in SIMBA in the framework of the project NoNitriNox (ifak e.V. 2016).

These results can be interpreted as an indicator of a limited denitrification capacity, since, by decreasing the oxygen set point, the conditions are closer to anoxic. Based on this information, and to avoid the previously named problems, the minimum DO set point value of 0.8 mg/L will be taken into account for the following tests.

4.1.2.2 Denitrification Volume

The operational plant analysis and the previous simulation results show that the aerobic part of the plant works very effectively, reducing COD and ammonium nitrogen to values that are usually well below the norm. Regarding nitrogen removal, the problems seem to be in the denitrification stage. To modify the denitrification capacity of the plant, the denitrification volume (V_D) can be increased. For example, the Worksheet DWA-A 131 recommends denitrification volumes between 20% and 60% of the total activated sludge volume (V_{AT}) (DWA 2016). As described in Chapter 3.4, other design approaches, such as the one proposed by Metcalf & Eddy, do not suggest a specific proportion of denitrification volume. The denitrification volume is determined by the HRT and the amount of nitrate to denitrify, coming from the recirculation rate.

The example WWTP has a denitrification volume equivalent to 16.7% of the total activated sludge volume (20% when considering only the anoxic and aerated tanks V_{N+D}). Therefore, tests increasing this volume (V_D) by reducing the nitrification volume (V_N) were carried out, as described in Table 14. The total volume (96,000 m^3) and the anaerobic tank volume (16,000 m^3) are maintained.

Table 14. Denitrification and nitrification volumes and their respective ratios

V_D / V_{AT}	V_D m^3	V_N m^3
0.17	16,000	64,000
0.2	19,200	60,800
0.3	28,800	51,200
0.4	38,400	41,600
0.5	48,000	32,000
0.6	57,600	22,400

The results, shown in Figure 18, show clearly that by increasing the denitrification volume (by reducing the aerated fraction of the tank), the final nitrate concentration can be reduced up to 15.6%, from 6.28 to 5.30 mg/L. Meanwhile, the ammonium concentration increases, however, its final concentration remains still well below 0.5 mg/L. For COD outlet concentrations, only small variations of less than 0.15% are observed (results not shown).

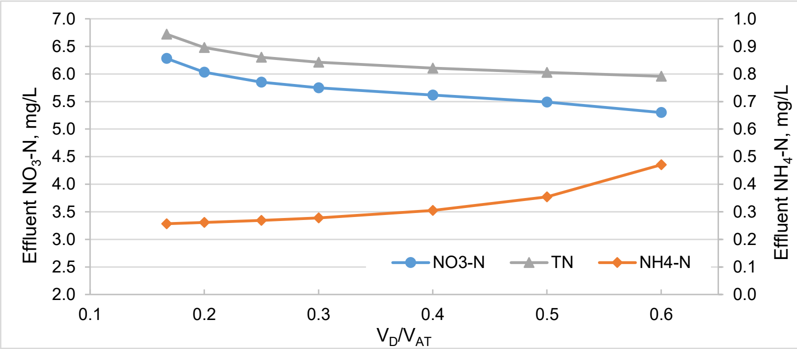

Figure 18. TN, NO_3-N and NH_4-N effluent concentrations by increasing V_D/V_{AT} in the steady state model of the example WWTP

This is an important result for the example plant, since aerating a smaller volume (i.e. lower energy consumption), better nitrogen removal rates can be achieved. The anoxic zone still will require mixing, so electricity will still be required there, with additional stirrers or by pulse aeration (i.e. short aeration pulses of a few seconds with the function of mixing and not aerating). It must be checked in detail to which extent this can be achieved and which technical modifications are required to stop or significantly reduce aeration in a proportion of the nitrification zone, transforming it permanently into an anoxic zone.

4.1.2.3 By-pass (Decommissioning) of Primary Clarifiers

The total or partial by-pass of primary clarifiers (PC) can contribute to improving the C/N ratio and therefore improve denitrification. By-passing in this context refers to taking a certain portion of the primary clarifiers out of service (decommissioning) and passing the influent through a smaller total volume and HRT. The corresponding volumes with partial or total by-pass are shown in Table 15.

Table 15. Primary clarifiers volume with partial and total bypass

PC volume, m^3	as % of the total volume	N° of PC in operation
12,720	100%	4
9,540	75%	3
6,360	50%	2
3,180	25%	1
0	0%	0

As observed in Figure 19, the by-pass of primary clarifiers by itself does not influence significantly the effluent values for nitrogen compounds, reducing a maximum of 3.1% TN concentration in the effluent. This is probably because the denitrification capacity is too small. However, this strategy can bring advantages in combination with others, such as the variation in the denitrification proportion V_D/V_{AT} which is tested later in Chapter 5.1.2.

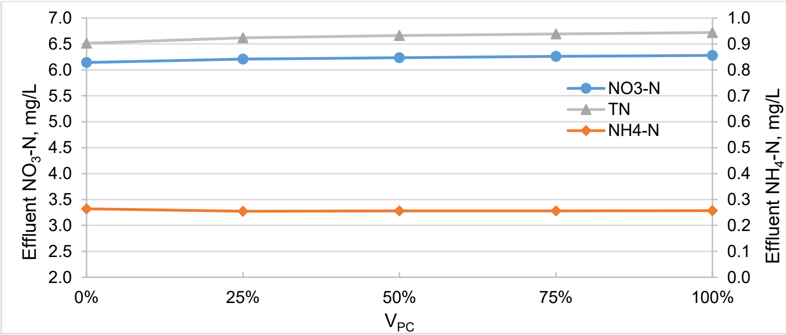

Figure 19. TN, NO_3-N and NH_4-N effluent concentrations by bypassing total or partially the primary clarifiers in the steady state model of the example WWTP

4.2 Model Building and Calibration

Based on the model for the pre-simulation (steady state model), a full model was built. Meanwhile, the pre-simulation model used only average values (i.e. static model), and the full model uses a dynamic data set (i.e. dynamic modelling). Due to the data availability and first simulation results, data between September 2017 and September 2018 (from now on "Calibration period") was used for the model calibration and data from years 2018 and 2019 for the model validation. The data from the first half of 2017 will not be considered due to the inconsistency in laboratory analysis and lack of relevant measurements as nitrate measurements both in influent and effluent, DO in nitrification tanks, etc.

The main aspects that were modified and that define the model, i.e. those with relevance for the calibration, are listed in the next subchapters.

4.2.1 Adjustment of the Influent Characteristics

The measured influent flow rate, COD, TKN[10] and TP concentrations are the basis for the influent data. The conversion block from influent data to *asm3h* considers the fractionation of COD and alkalinity. Most default values were maintained (see Table 16), except for the fraction TSS to COD, which is known.

[10] TKN calculated as TN – NO_3-N.

Table 16. Conversion block influent parameters

Mean dry weather values	Symbol	Selected values
Fraction TSS to COD	TSS/COD	190/400
Fraction of non-volatile TSS	f_B	0.3
Fraction of inert soluble COD	f_S	0.05
Fraction of inert COD from particulate COD	f_A	0.3
Fraction of SS from biodegradable COD	f_{COD}	0.2
Alkalinity	S_{alk}	8

The alkalinity values, which are not measured in the example WWTP, were reduced from 10 mg/L (default value in the software) to 8 mg/L to improve the fit for the ammonium nitrogen values. This is also based on the observed pH fluctuations in the operational data analysis and the almost inexistent dosing of a form of buffer in the example WWTP.

4.2.2 Primary Clarifiers

The total tank volume informed by the operator was used, divided into four tanks, and the sludge extraction is controlled by the calculated primary sludge (PS) flowrate production. As described in the plant operational analysis, the TSS concentration fluctuates sharply between days, but this approach provided a good fit in terms of primary sludge production load (see Figure 20). The criteria to evaluate the model fit is described in 4.3, as well as the corresponding values for primary sludge production.

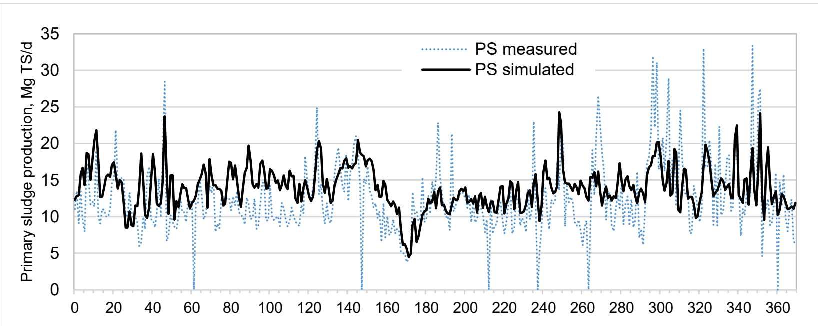

Figure 20. Primary sludge production comparison between calculated and modelled

4.2.3 Type of Reactors (hydraulic behaviour) in Activated Sludge Tanks

Since the real tanks are long and narrow, it is assumed that the tanks behave as Plug flow reactors (PFR). Therefore, each section of the tanks (anaerobic, anoxic and aerobic) is modelled as three CSTR in series, each with equal volume.

4.2.4 Dissolved Oxygen Control and Air Distribution in the tanks

The set point for the DO concentrations in the aerated tanks is adjusted according to the average measured DO concentration informed by the plant operators and controlled in a PI-type controller. In reality, there are online, and manual DO measurements in the aeration tanks, and the aeration is adjusted accordingly to reach the desired DO concentration (3 or 2 mg/L), therefore the computer model uses a variable set-point to represent the real operational condition. According to the plant operator, there is a single DO online measurement per nitrification tank, followed by periodic manual measurements and there are no other online measurements that influence the aeration automation. Based on this information, and the fact that in these large PFRs of approx. 10 x 90 m per tank, it is expected to have a non-uniform DO distribution.

To realistically model the DO distribution in the aerated tanks, an air distribution profile was used: 60% for the front section of the tank, 25% for the middle section and 15% for the rear section. Additionally, the maximum capacity of the existent 8 blowers (135 m^3/min each) is considered and limited to 40%, since outdated diffusers are present in the plant, which are there, with poor to zero maintenance since the plant inauguration in 2009 and 2015, respectively. Moreover, this configuration delivered a better fit for NH_4-N effluent concentrations. However, the ammonium peaks observed in the effluent may be caused by mixing problems in the aerated tanks, combined with aeration problems, effects that cannot be modelled in SIMBA#. For this purpose, computational fluid dynamics (CFD) modelling would be more adequate.

4.2.5 Inclusion of the Temperature Influence in Nitrification Tanks

Wastewater temperature is an essential parameter for modelling, since nitrification is highly sensitive to temperature changes, as described in Chapter 2.1.1.1. Moreover, the observed ammonium peaks could be temperature-related. Based on the first measurements of wastewater temperature at the example WWTP, which started on July 2019, a peak was observed in August. This is consistent with the average temperatures in the region, which are very predictable on a yearly basis, as observed in Figure 21**,** with temperatures up to. 4 °C in winter and ca. 30 °C in summer.

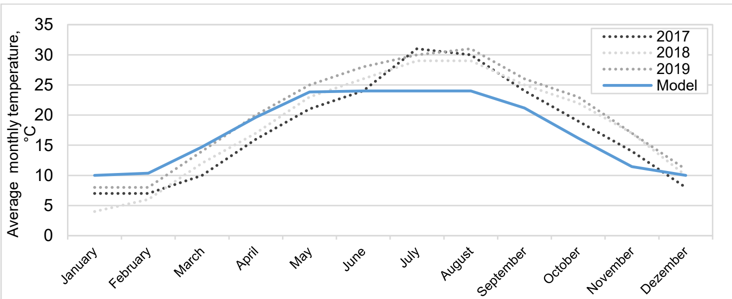

Figure 21. Average air temperature in the region in the years 2017 to 2019 (grey dashed lines) and wastewater temperature used in the model (blue)

To estimate the wastewater temperature in the calibration period (September 2017 and September 2018), the air temperature in the region was used as a reference. The lower and higher temperatures were cut off between 10 °C and 24 °C for the modelling for two reasons:

(1) The operator informed the wastewater temperature is rarely under 12 °C (relevant because it is the cut-off value for the norm)

(2) Water has a high specific heat capacity, meaning that the amount of heat required to change its temperature is high as well, therefore, water will not change temperature as fast as air. As shown by (Golzar et al. 2020) the wastewater temperature at the influent of a WWTP usually differs by a few degrees from air temperature, especially in the extreme temperature ranges[11]. In winter, the wastewater temperature is slightly higher and in summer it is slightly lower than air, also because, as wastewater is transported, it is partially isolated in the underground sewerage.

The DWA guidelines for designing activated sludge systems (DWA-A 131) are limited to temperatures between 8 and 20 °C and the ASM Models have been tested in the range of 8-23 °C. For lower or higher temperatures, it is recommended to carry out pilot tests or use findings from previous large-scale operation of comparable plants (DWA 2016).

However, studies for the treatment of wastewater in cold or warm regions have shown that the parameters presented by DWA-A 131 can be valid also for temperatures between 5 and 30 °C (DWA 2017). It is argued that, as the equations to estimate the sludge age (SRT) include the temperature-dependent decay coefficient for the heterotrophic biomass ($b_{H,T}$), the equation can be valid for temperatures above the initially suggested range. The study suggests as well, that for lower temperatures, the design sludge age for stabilisation should be calculated for an SRT > 30 d.

4.2.6 Adjustment of the Sludge Age

To calibrate the model, the first parameter to adjust is the sludge age, which in this case is done by adjusting the only related parameter that is known, the sludge concentration in the activated sludge tanks. The sludge production is not measured in the example WWTP. An estimation of the sludge production estimation was carried out, based on the methodology suggested by Metcalf and Eddy (see Annex 12.7). However, since the methodology requires the assumption of several parameters, it was decided to continue the calibration according to the known parameter, MLSS.

By knowing the target sludge concentration in activated sludge tanks, the excess sludge extraction is controlled using a PI controller, measuring the TSS after the last aerated tank. The measured solids concentration in the tanks (MLSS) and the obtained sludge age are shown in Figure 22. In certain periods, the model does not reach the measured MLSS concentrations, which are above 8 g/L. MLSS concentrations above 6 g/L seem too high for the system and could lead to limitations in oxygen transfer. It should be checked if the concentrations are measured in a representative manner.

4.2.7 Limits for Different Equipment

According to the plant description data, several pumps and equipment were described with their corresponding limitations in the model:

- Pump internal recirculation: 2,355 $m^3/h \cdot 12$ units,
- Internal recirculation calculated as 200% of the influent flow rate

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[11] In the study of the Henriksdal WWTP (Stockholm), even with extreme air temperatures of -20.7 °C or 31.8 °C, the wastewater temperature reached only 2.08 °C and 23.7 °C, respectively.

- Sludge recirculation calculated as 100% of the influent flow rate
- Pumps sludge recirculation: 2,093 $m^3/h \cdot 4$ units
- Aeration: 135 $Nm^3/min \cdot 8$ units = 194,400 Nm^3/d

The results obtained after the calibration adjustments are shown in Figure 22. After adjusting the MLSS concentration in the activated sludge tanks, the model shows a very good fit for COD, NO_3-N and TN values, as discussed in the following section. The fit of ammonium nitrogen was more challenging, requiring all the adjustments mentioned above. It is worth noticing that before day 120, there are no laboratory measurements for nitrate.

4.3 Model Fit

A key factor to evaluate a model is the model fit, in other words, when a simulation is good enough to make accurate predictions about the modelled system. Initially, the model fit was carried out by a visual evaluation. The comparison between the measured and modelled values can be found in Figure 22.

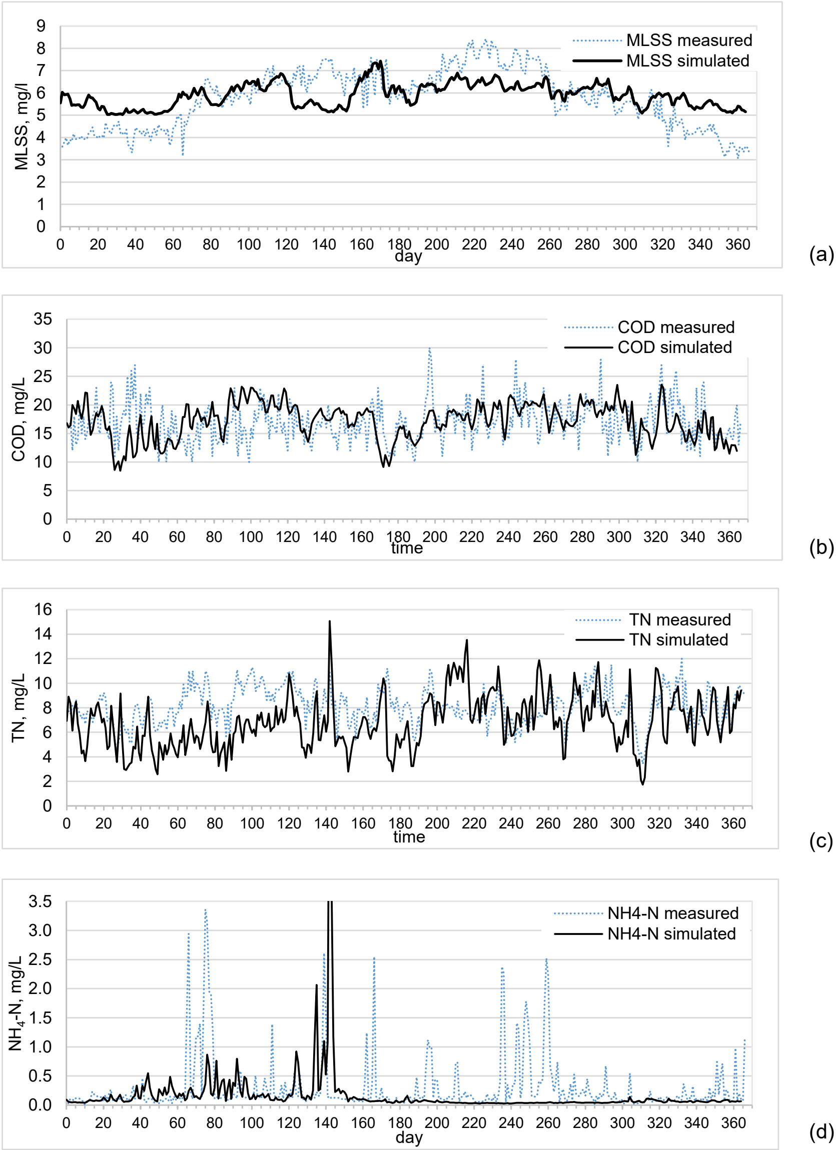


Figure 22. Model calibration results: (a) MLSS concentration; (b) Effluent concentration of COD; (c) Effluent concentration of TN; (d) Effluent concentration of NH_4-N. Measured in laboratory (dashed blue), simulated values (black).

Afterwards, the mathematical tools described in Section 2.3.3 were used:

- Statistical evaluation factors (SEF), as percentage error (e%):
    - Mean,
    - Median,
    - 85% quantile and
    - Standard deviation (SD).
- Coefficient of determination (R^2)
- Model efficiency coefficient Nash-Sutcliffe (E_1, E_2)
- Index of agreement (d_1, d_2)

As there are not generally accepted and easy-to-use criteria in the field of modelling of WWTP (Langergraber et al. 2004), a combination of the previously named parameters will be used. The evaluation power regarding the model fit of the statistical evaluation factors is limited, their weight in the overall evaluation is lower, and are evaluated as a single parameter (SEF). The remaining model fit quality dimensions will be weighted as equal for the overall evaluation.

The parameters to be evaluated are:

- Primary sludge (PS) production (as described in 4.2.2)
- Concentrations of COD, TN, NO_3- N, NH_4-N in the effluent and
- MLSS in the activated sludge tanks.

The ranges indicating which values indicate the goodness of the model fit are described in Table 17. Additionally, points are assigned according to the model fit, where 5 points indicate a perfect model fit and < 1 no fit.

Table 17. Keys to evaluate the model fit according to the selected parameters

Colour code					
Fit	**Very good fit**	**Good fit**	**Medium fit**	**Low fit**	**No fit**
e%	0% - 5%	5% - 25%	25% - 40%	40% - 50%	> 50%
R^2	1 – 0.8	0.8 – 0.6	0.6 – 0.4	0.4 – 0.2	< 0.2
E_j	1 – 0.7	0.75 – 0.5	0.5 – 0.2	0.25 – 0	< 0
d_j	1 – 0.8	0.8 – 0.6	0.6 – 0.4	0.4 – 0.2	< 0.2
Points (p)	**[5 ; 4]**	**] 4 ; 3]**	**] 3 ; 2]**	**] 2 ; 1]**	**] 1 ; 0]**

First, the evaluation of the statistical values is carried out. Here, the modelled values (M) are compared with the laboratory or observed values (O) in Table 50 (Annex 12.5), and the percentage error (e%) is calculated for the mean, median, 85%-quantile and standard deviation (SD). Moreover, points are assigned for each calculated value in Table 18. According to this evaluation, the fit for COD, nitrate and MLSS is very good. For total nitrogen the fit is good and for ammonium the fit is low, confirming the findings of the visual evaluation.

Table 18. Evaluation of the fit of the SEF values based on e%

Parameter	| e% | value						Points					
	COD	TN	NO_3-N	NH_4-N	MLSS	PS	COD	TN	NO_3-N	NH_4-N	MLSS	PS
Mean	1.1%	15.5%	0.2%	45.8%	2.5%	13.1%	4.9	3.7	5.0	1.4	4.8	3.9
Median	4.4%	16.5%	2.3%	47.3%	0.5%	19.3%	4.6	3.7	4.8	1.2	4.9	3.5
85%-Quantile	1.6%	6.2%	13.3%	52.5%	10.5%	1.3%	4.8	4.5	3.9	0.8	4.1	4.9
SD	12.4%	42.3%	63.0%	7.8%	58.8%	38.8%	4.0	1.6	0.0	4.4	0.3	1.9
Average SEF							**4.6**	**3.4**	**3.4**	**1.9**	**3.5**	**3.5**

The model fit factors dimensions R^2, E_1, E_2, d_1, and d_2 are calculated as well and points are assigned. The values and points summary are presented in Table 19. After the evaluation of the SEF and remaining model fit factor dimensions, each evaluated parameter is weighted as equal and an average of the obtained points per parameter is calculated. The points awarded per parameter, indicate an overall good to medium model fit (3.25 points on average).

Table 19. Model fit according to different evaluation methods for different parameters

Parameter	Value						Points					
	COD	TN	NO_3-N	NH_4-N	MLSS	PS	COD	TN	NO_3-N	NH_4-N	MLSS	PS
SEF	4.9%	20.1%	19.7%	38.4%	18.1%	23.3%	4.6	3.4	3.4	1.9	3.5	3.5
R2	0.17	0.36	0.36	0.04	0.61	0.23	0.9	1.8	1.8	0.2	3.1	1.2
E1	0.00	0.22	0.22	0.00	0.98	0.88	1.0	1.9	1.9	1.0	4.9	4.5
E2	0.00	0.40	0.40	0.00	1.00	0.99	1.0	2.6	2.6	1.0	5.0	4.9
d1	1.04	0.74	0.74	0.73	1.01	1.06	5.0	3.8	3.8	3.7	5.0	5.0
d2	1.00	0.93	0.93	0.93	1.00	1.00	5.0	4.7	4.7	4.7	5.0	5.0
Average							**3.0**	**3.0**	**3.0**	**2.1**	**4.4**	**4.0**

4.4 Model Validation

For the model validation, the time after the calibration period was used (September 2018 to December 2019). The measured DO concentration and MLSS in that time-period were used, and the water recirculation rate was also increased to 300%, as informed by the plant operator. Small adjustments in the air distribution were made in order to improve the fit.

The fit for this time period is medium, with large deviations from time to time in COD and nitrate nitrogen (and TN) values, as can be observed in Figure 23 (left column). The increase in the concentration of nitrate in the model is directly correlated to a drop in the DO in the aerated tanks. The model shows a very good fit during the time period between July and December 2019 (see Figure 23, right column), which involves summer operation and a temperature decrease.

The same parameters calculated in the previous chapter were calculated for the validation period and are summarized in Table 20. The obtained values indicate a good to a medium fit one again. Based on the visual evaluation and the points assigned, a medium to a good fit of the model for the studied parameters is observed. Therefore, the model is considered validated.

Table 20. Model fit according to different evaluation methods for different parameters in the validation period

Parameter	Value					Points				
	COD	TN	NO_3-N	NH_4-N	MLSS	COD	TN	NO_3-N	NH_4-N	MLSS
SEF	44.3%	38.3%	52.7%	135.1%	35.8%	1.5	3.0	2.7	1.5	2.1
R2	0.05	0.77	0.77	0.07	0.45	0.3	3.9	3.9	0.4	2.3
E1	0.00	0.45	0.45	0.00	1.33	1.0	2.8	2.8	1.0	6.3
E2	0.00	0.69	0.69	0.00	0.89	1.0	3.8	3.8	1.0	4.5
d1	1.00	0.85	0.85	1.00	0.83	5.0	4.3	4.3	5.0	4.2
d2	0.76	0.98	0.98	0.90	0.97	3.9	5.0	5.0	4.6	5.0
Average						**2.1**	**3.8**	**3.7**	**2.2**	**4.1**

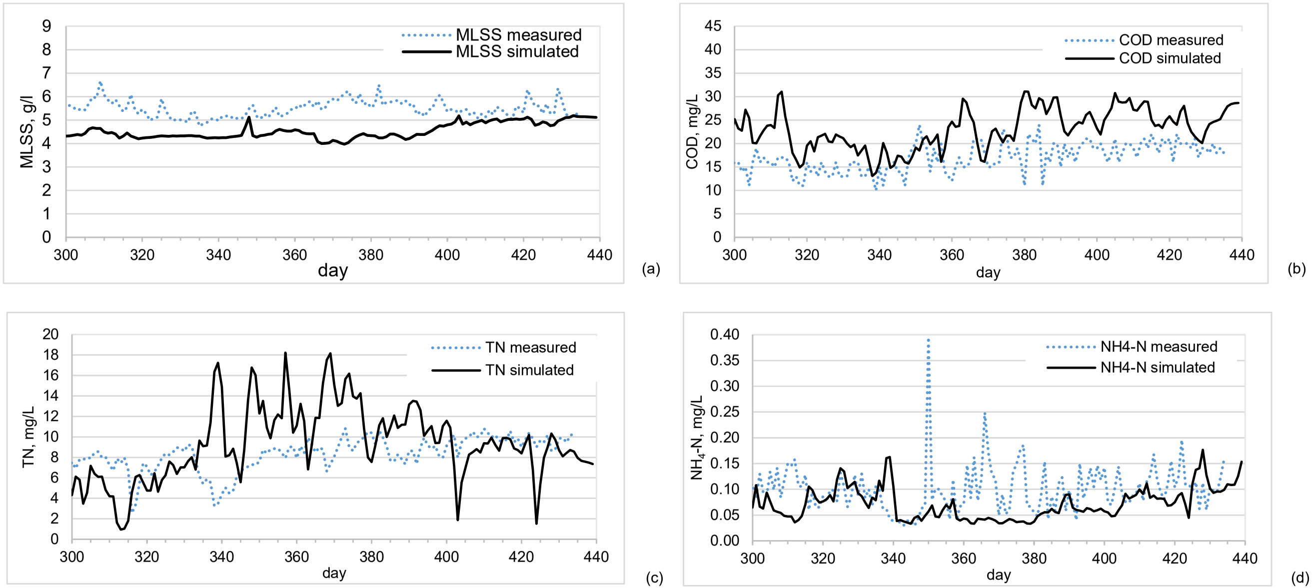


Figure 23. Model validation values (a) MLSS concentration; Effluent concentration (b) COD; (c) TN and (d) NH_4-N. Measured in laboratory (blue, dashed), simulated values (black)

4.5 Summary of Chapter 4

Chapter 4 presents the different parameters that were adjusted to obtain the required model fit. The calibration parameters were:

- Primary sludge production
- Effluent concentration of COD, TN, NH_4-N and
- MLSS concentration in the activated sludge tanks.

The obtained model shows a “very good” or “good” fit for the studied parameters, first according to a visual comparison, and then, based on several mathematical parameters as suggested by other simulation studies:

- Statistical evaluation factors (SEF), as percentage error (e%):
    - Mean,
    - Median,
    - 85% quantile and
    - Standard deviation (SD).
- Coefficient of determination (R^2)
- Model efficiency coefficient Nash-Sutcliffe (E_1, E_2)
- Index of agreement (d_1, d_2)

The obtained model fit is enough to test optimisation strategies for nitrogen removal, which is carried out in the following chapter.

5 Tests of Operational and ICA Strategies with the Model

Once a model that reliably represents the real plant has been developed, the testing phase can begin, where various optimization strategies can be tested and adjusted. In this section, a detailed description of the main tests carried out is described.

The first parameter of comparison will be compliance with the standard. The analysis will start with the norm compliance for the current standard that the WWTP must meet (i.e. CS standard). The CS standard considers a non-compliance, each daily average value that is above the required value as described in Chapter 3.1, Table 7

The goal of the performed tests is to minimize the total number of norm non-compliances for TN and NH_4-N in a year, to the minimum, ideally, to zero, if possible. There is no hierarchy to prioritize the type of norm non-compliances, so norm non-compliances for COD, TN and NH_4-N, are considered the same.

The approach described below was followed:

1. The first set of tests consists of operational strategies, i.e. the variation of the way the plant is operated. This includes, for example, varying the number of lines in operation in the activated sludge system.
    - The first test performed was to reduce the total volume of the activated sludge system, as this is necessary for the maintenance of the activated sludge tanks.
    - Then a reduction of the residence time in the primary clarifiers was tested, to recover more COD for biological treatment, by consecutively taking out of operation each of the 4 ponds currently in operation.
    - Finally, a ratio change in the anoxic volume was tested, either by ceasing aeration of a section of the aerobic tanks or by using free volume from other tanks as extra anoxic volume.
2. The second set of tests corresponds to tests with the change of automation system.
    - Here we started with tests based on the inclusion of ammonium and nitrate sensors for aeration control.
    - Then the implementation of intermittent aeration was tested, also based on nitrate and ammonium sensors.
    - Then the adjustment of the sludge age of the system was tested, as this parameter was found to have high variability in the actual operation of the plant.
3. In the third stage, the combination of the most successful individual strategies was tested.

Once the results were obtained for the norm CS, the best scenario results were compared for three other different standards selected from Table 4. The first standard is the current standard (CS). The second is a laxer standard, such as the standard to be met at the EU level, according to the currently valid water directive. Then the German standard for WWTP class 5 (> 100,000 PE), was tested. Finally, the results were compared with the standard currently valid in Luxembourg, which is the strictest standard found in the literature research. The requirements for the different tested norms are described and discussed in Chapter 5.4.

5.1 Conventional Regulation and Operational Strategies

5.1.1 Activated Sludge Volume Reduction

WWTP must decommission part of the activated sludge tanks for maintenance purposes from time to time. To test the plausibility of reducing the activated sludge tanks volume in a dynamic simulation, two tests were carried out by simulating the emptying of one of the four operating lines:

T1: One line out of service ($V_{AT} = V_{AT} - (V_{N,i} + V_{D,i} + V_{An,i}) = 96{,}000 - (16{,}000 + 4{,}000 + 4{,}000)\ m^3 = 72{,}000\ m^3$)

T2: One line out of service ($V_{AT} = 72{,}000\ m^3$) and $V_D/V_{AT} = 0.3$

Figure 24 shows that the emptying of an operating line in the activated sludge process negatively influences compliance with the total nitrogen standard ("City Standard" norm, CS). Increasing the denitrification volume to $V_D/V_{AT} = 0.3$ allows to counteract this problem, eliminating part of the negative effects in nitrogen removal, even when compared with the baseline scenario (i.e. four lines in operation).

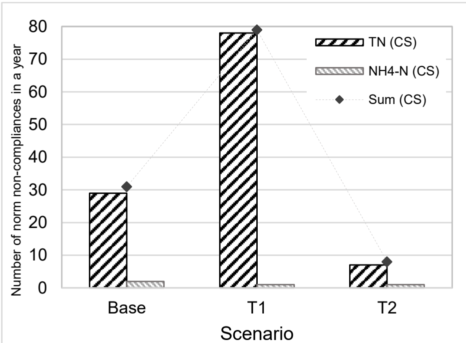


Figure 24. Number non-compliances of the norm CS in a year with a reduction of the V_{AT}. Base is the baseline scenario, T1 represents putting out of service one line of the activated sludge reactors and T2 represents putting out of service one line of the activated sludge reactors, but increasing the denitrification proportion

These results indicate that an activated sludge system with a more favourable distribution of anoxic and aerobic volumes can be operated with a lower volume, contributing to energy savings by reducing the air requirements, reducing friction loss - increasing the pressure in the air line-, and reducing the stirring required. Based on this result, with the change in the V_D/V_{AT} proportion, the theoretical treatment capacity of the biological treatment stage could be augmented by 25% i.e. reaching a total of 562,500 $PE_{COD,120}$. This could, however, challenge the hydraulic capacity of the secondary clarifiers and the capacity of further treatment stages.

5.1.2 Partial or Total By-pass of Primary Clarifiers

By bypassing the primary clarification, totally or partially – reducing the volume and therefore the HRT–, the COD pre-removal is reduced, contributing to improving the C/N ratio for denitrification. Several tests were carried out to reduce the primary clarification volume, putting out of order one tank at a time. With the bypassing of the whole primary clarification, the average TN-effluent concentration can be reduced only by 6.1% (maintaining the V_D proportion). This strategy will be tested, but in combination with dynamic use of the remaining volume for denitrification, as described in the following section.

5.1.3 Increase of the Denitrification Volume

The plant analysis and the preliminary simulation results showed that the denitrification volume proportion is too small (see Chapter 3.2). To modify the denitrification capacity of the plant, the denitrification volume (V_D) can be increased.

5.1.3.1 Change in Denitrification Volume Proportion

First, tests increasing this volume (V_D) by reducing the nitrification volume (V_N) were carried out, increasing it to 50% as described in Table 21. The total volume (96,000 m^3) and the anaerobic tank volume (16,000 m^3) are maintained (see Figure 25). In each scenario, V_{AT} is constant at 96,000 m³ (total activated sludge volume including the anaerobic tank volume).

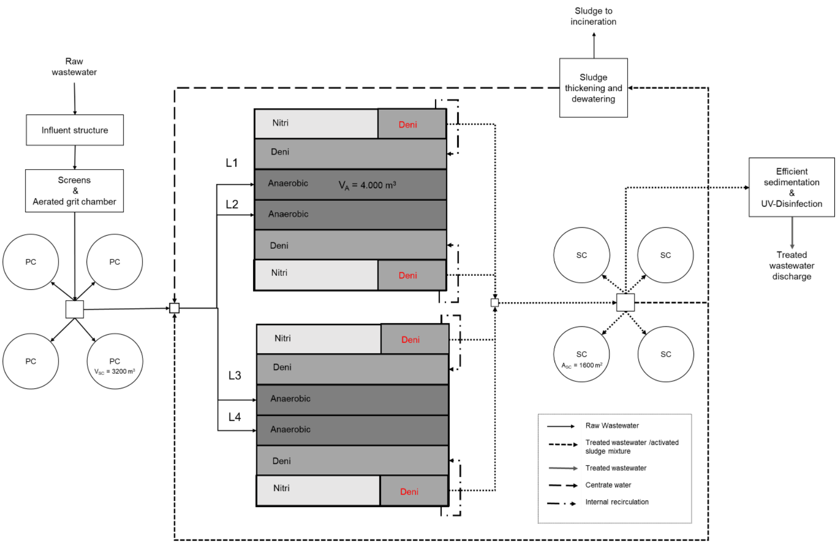


Figure 25. Scheme of the WWTP, with the use increase in denitrification volume proportion, by decreasing the nitrification tanks volume. The extension of the V_D is marked in red.

Table 21. Denitrification and nitrification volumes and their respective ratios

Scenario	V_D / V_{AT}	V_D m^3	V_N m^3
Base	0.17	16,000	64,000
T3	0.3	28,800	51,200
T4	0.4	38,400	41,600
T5	0.5	48,000	32,000

An increase in the denitrification volume effectively improves the nitrogen effluent values. According to the tests, a proportion of 0.3 V_D/V_{AT} is the best option, improving the Total Nitrogen removal (up to 12.5 %), and reducing the number of times the current and future norm is not fulfilled (from 31 to 8 nom non-compliances in a year) (See Figure 26).

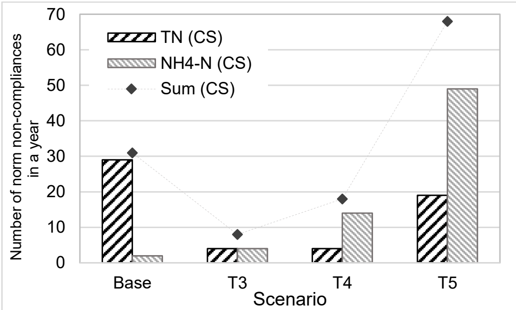


Figure 26. Number non-compliances of the norm CS in a year in scenarios T3 to T5 (change in the denitrification volume proportion) Base represents the baseline scenario (V_D/V_{AT} =0.17), T3 scenario with V_D/V_{AT} =0.3; T4 scenario with V_D/V_{AT} =0.4 and T5 scenario with V_D/V_{AT} =0.5.

From the obtained outcome, it results clear that the system can profit greatly from an increase in the denitrification volume proportion. However, an increase to 40% (T4) is less beneficial, as it maintains the norm non-compliances for TN, but it increases the ones related to ammonium nitrogen. As studied in previous chapters (see Chapter 3.4), it appears the total activated sludge volume is, on certain periods, too small, therefore, this reduction of the nitrification volume is detrimental to the nitrification process.

A similar, but more pronounced effect, is observed in T5, where the V_D/V_{AT} = 50% seems to propel a misbalance in the nitrification/denitrification process: the nitrification capacity is too small, therefore not enough nitrate is available for the denitrification process, leading to 68 norm non-compliances in a year.

5.1.3.2 Use of primary clarification volume as denitrification tanks

As has been observed so far in this plant analysis and modelling, the denitrification capacity of the plant is limited. Therefore, when bypassing primary clarifiers, this empty volume could be used as a denitrification tank. For this, stirring of the primary clarifiers (to avoid sludge settling) and an additional internal recirculation would be required (see Figure 27).

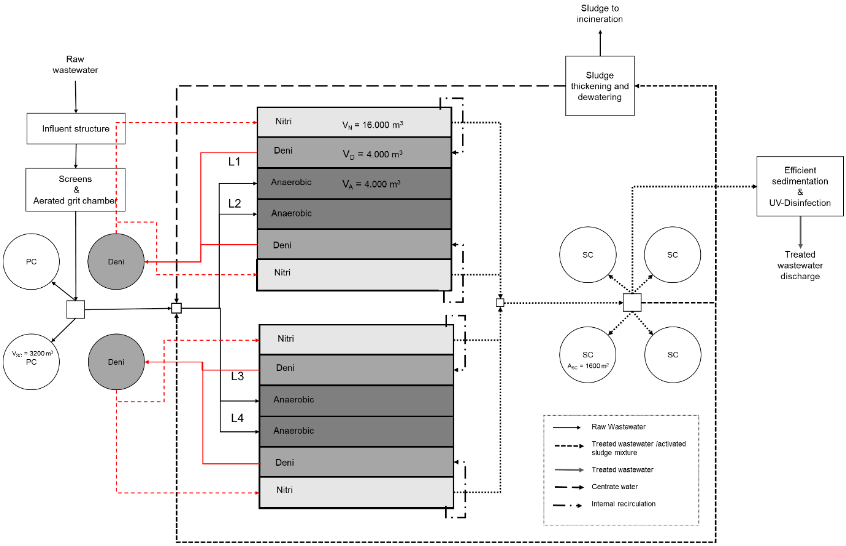


Figure 27. Scheme of the WWTP, with the use of 50% of the primary clarifiers as denitrification volume. The red lines represent the new pipelines to the decommissioned PC and the dashed red lines represent the pipelines from the decommissioned PC back to the nitrification tanks.

Based on this approach, tests were carried out, and the modelled proportions are described in Table 22. For example, with the bypass of 50% of the primary clarifiers, the denitrification volume can be increased by 20%, reaching a total V_D/V_{AT} of 21.8%.

Table 22. Denitrification volume change with the use of bypassed primary clarifiers as denitrification tanks

Scenario	Used PC Volume (V_{PC})	Bypassed V_{PC}	New V_D ($V_{D,initial}$ + bypassed V_{PC})	Increase in V_D	V_D/V_{AT}
	%	m³	m³	%	-
Base	100% (4 PC)	0	16,000	-	0.167
T6	50% (2 PC)	6,360	22,360	40%	0.218
T7	25% (1 PC)	9,540	25,540	60%	0.242

With an increase in the denitrification volume, by using the bypassed primary clarifiers, the number of times the norm is surpassed can be reduced significantly, for example from 29 to only 4 times per year for TN for the CS norm (see Figure 29). Here again, some problems with ammonia nitrogen are observed, with an increase in the norm surpassing, going from 2 to 4 times per year in T7 with a 75% bypass of primary clarifiers and denitrification volume replacement. There are almost no changes in COD values, and therefore no problems with norm compliance.

5.1.3.3 Use of Primary Clarification Volume as Denitrification Tanks – Dynamic Adjustment Based on C/N Influent Values

Depending on the C/N ratio (COD/TN) in the influent, a different control strategy can be tested. A bypass of primary clarifiers and its use as denitrification tanks only when the C/N ratio is below a minimum value. As can be seen in Figure 28, the C/N ratio is extremely variable, from one day to the next: As a trend, it is lower, on average, between February and April, the dryer months, but no other trends could be identified.

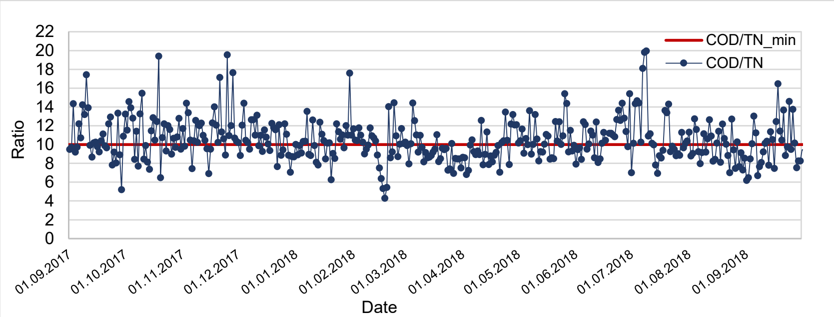


Figure 28. COD/TN ratio between September 2017 and September 2018. The red line represents the proportion C:N = 100:10.

To successfully implement this strategy, daily averages will be used. This means that, when detecting a low C/N ratio value, one or more PC tanks will be bypassed avoiding primary sludge removal. The free PC volume is used for denitrification, therefore stirring will take place and the internal recirculation for a nitrate-rich flowrate must be turned on. The switch from primary clarification to Denitrification tank will take place according to the following conditions:

(1) When C/N >10, use the full volume of primary clarification;

(2) If C/N <10, bypass a portion of the PC volume and use this volume as a denitrification tank (without scaling).

This will define the scenarios:

- T8: same proportions as T6 (bypass of 50% of the V_{PC}, and use as additional V_D, V_D/V_{AT} = 21.8%) with a dynamic shift of PC and anoxic tanks based on the influent C/N ratio.
- T9: same proportions as T7 (bypass of 75% of the V_{PC}, and use as additional V_D V_D/V_{AT} = 24.2%) with a dynamic shift of PC and anoxic tanks based on the influent C/N ratio.

It must be discussed, how realistic would be to implement the scenarios represented by T8 and T9 in a WWTP. This will depend on how rapidly the switch from primary clarifier to denitrification tank takes place. In the tested scenarios T8 and T9, this occurs every 12 hours, and this can be challenging to apply in a real WWTP, but it can be automated.

As in previous cases, there are no changes in the effluent COD concentration (see Annex 12.8). In the case of nitrogen, a reduction in its concentration is observed and simultaneously, the number of times the norm is not fulfilled (see Figure 29) is reduced with an increase in V_D and bypass of PC.

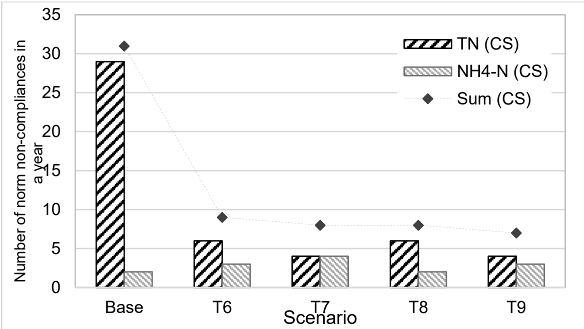


Figure 29. Number non-compliances of the norm CS in a year with scenarios T6 to T9 (change in the denitrification volume by bypass of primary clarifiers). T6 scenario with by-pass of 50% V_{PC} and V_D/V_{AT} = 21.8%; T7 scenario with by-pass of 75% V_{PC} and V_D/V_{AT} = 24.2%; T8 scenario with by-pass of 50% V_{PC} (V_D/V_{AT} = 21.8%) and shift of PC and anoxic tanks based on the influent C/N ratio; T9 scenario with by-pass of 25% V_{PC} (V_D/V_{AT} = 24.2%) and shift of PC and anoxic tanks based on the influent C/N ratio.

In comparison with the non-dynamic PC volume replacement with denitrification, some ammonium peaks can be avoided, reducing at least one time per year the surpassing of the discharge standard. For TN, the number of times the standard is exceeded does not change. For this strategy to be tested in praxis, it must be studied how easy it is to switch from primary clarification to denitrification e.g. stirrers, recirculation of nitrate-rich wastewater (addition of an internal recirculation).

5.2 Conventional Control Strategies

5.2.1 NH_4-based Aeration Control

The principle of an ammonium-based control of aeration is that the amount of air depends on the effluent NH_4-N values. A low concentration of ammonium indicates when most of it has been oxidized, and therefore, aeration can be reduced or even stopped.

The first tests were carried out by setting a maximum and a minimum ammonium nitrogen concentration (T10 to T13 in Table 23). It must be highlighted that, despite some very low values for NH_4-N being tested in simulations, currently only values of NH_4-N ≥1 mg/L can be realistically measured online. Note that the set point limit values for the NH_4–N concentration are based on the high requirements of the City Assessment Standard (CS).

By limiting the ammonium concentration to a maximum of 1.35 mg/L (90% of the maximum value according to the CS norm), the TN effluent concentrations are significantly reduced, mainly due to a reduction of the aeration, which leads to increased nitrate removal. This leads also to an important decrease in the number of times the discharge standards are not fulfilled (see Figure 31). The very low DO concentrations obtained, provide denitrification-like conditions in the aeration tanks, contributing to increased denitrification.

Table 23. Parameters for ammonium nitrogen and DO set point in scenarios T10 to T15

Scenario	Parameters		
	Min NH_4-N mg/L	Max NH_4-N mg/L	DO set point mg/L
T10	0.3	0.5	0 – 2
T11	0.3	0.75	0 – 2
T12	0.5	0.9	0 – 2
T13	0.5	1.35	0 – 2
T14	0.3	1.05	0.8 – 2 (curve in Figure 30)
T15	0.3	1.05	1 – 2 (curve in Figure 30)

However, this strategy leads to very low DO average concentrations in the nitrification tanks i.e. below 0.5 mg O_2/L, which is not adequate for the operation, as mentioned in the pre-simulations tests (Chapter 4.1.2). To avoid the above-named problems, a limitation for the minimum and maximum DO concentrations (0.8 to 2 mg O_2/L) in nitrification tanks was included in a program block in SCL[12]. Additionally, an existing block for the variation of the DO set point, based on an NH_4-N curve (see Figure 30) was also used in strategies T14 and T15.

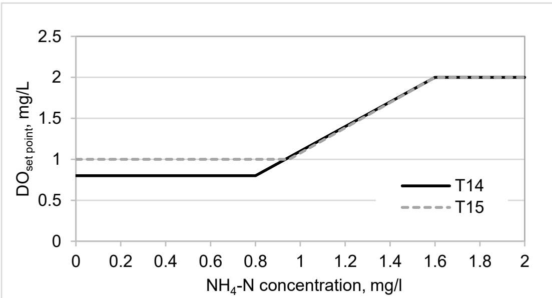


Figure 30. DO set point vs NH4-N in the effluent (Scenarios T14 and T15)

From the effects shown in Figure 31, it results clear that limiting the DO concentration in nitrification can contribute to an improvement in the nitrogen removal results, considering the requirements to comply with the City Assessment Standard, but an increase in the DO limitations applied in T14 and T15 show worse results, with the number of times the norm is not fulfilled is increased three times.

The air amount injected into the nitrification tanks is reduced by using this ammonium-based control strategy, but the DO concentration in the aeration tanks is on average 0.93 mg/L. With this DO concentration, the possibility to have problems with sludge settling and emission of nitrous oxide is reduced, but to confirm or dispute this, an evaluation of the sludge settleability must be carried out in experiments with real sludge in a laboratory.

[12] Siemens SCL (Structured Control Language)

The TN effluent concentrations are slightly reduced, and the number of times the discharge standards are not fulfilled is also reduced, but still high. The main advantage of this strategy is the possibility to save energy due to the reduction in air use. The amount of required air is reduced by 7.3% in a year.

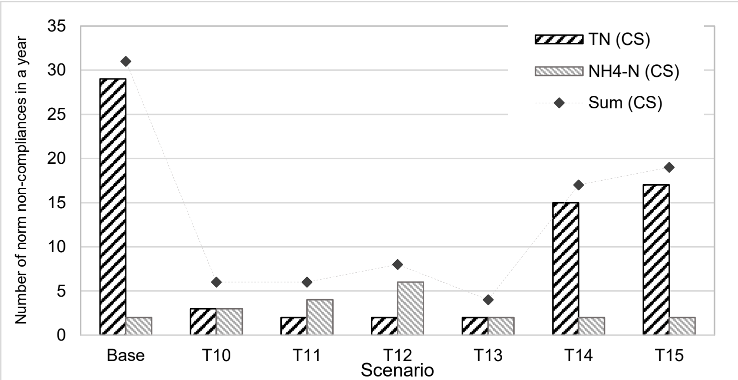


Figure 31. Number non-compliances of the norm CS in a year with scenarios T10 to T15 (NH_4-based aeration control). T10 scenario with 0.3 ≥ NH_4-N ≥ 0.5 mg/L and 0 ≥ DO ≥ 2 mg/L; T11 scenario with 0.3 ≥NH_4-N ≥ 0.75 mg/L and 0 ≥ DO ≥ 2 mg/L; T12 scenario with 0.5 ≥NH_4-N ≥ 0.9 mg/L and 0 ≥ DO ≥ 2 mg/L; T13 scenario with 0.5 ≥NH_4-N ≥ 1.35 mg/L and 0 ≥ DO ≥ 2 mg/L; T14 scenario with NH_4-N Curve and 0.8 ≥ DO ≥ 2 mg/L; T15 scenario with NH_4-N Curve and 1 ≥ DO ≥ 2 mg/L

It results evident that an NH_4-N-based DO control strategy can contribute to comply with both regulations, but by itself is not enough. The very low DO concentrations obtained in the first tests without DO limitation simply provided better conditions for denitrification, a capacity which, according to the WWTP analysis and model tests, is too low in the example WWTP. However, the potential problems with sludge quality and N_2O emissions have to be addressed, as the aim of the optimisation strategies is to improve the overall performance of the WWTP – or at least not worsen it.

5.2.2 NO_3-based Control

The principle of a nitrate-based control of aeration is that low NO_3-N values indicate when most of the nitrate has been consumed. At that point, aeration should be increased to support the activity of the nitrifying bacteria and enable them to produce more nitrate (Barfüßer 2018). The control of aeration based on nitrate is carried out identically as for ammonium, but with the measurements of a nitrate sensor before nitrification and different target values. The used parameters are summarized in Table 24.

Table 24. Parameters for nitrate nitrogen and DO set point in scenarios T16 and T17

Scenario	Parameters	
	Min NO_3-N mg/L	Max NO_3-N mg/L
T16	0.3	5.0
T17	3.0	8.0

A nitrate-based control does not result in a significant reduction of the nitrogen components concentration in the effluent, nor a reduction of the number of norm non-compliances (see Figure 32). Due to the difference in the concentrations selected for the aeration control, the results are the opposite: with very low NO_3-N concentrations (T16), the ammonium non-compliances increase. With higher low NO_3-N concentrations (T17), the TN and NH_4-N norm non-compliances are practically the same as in the base scenario.

According to the literature, in a NO_3-N-based control strategy, there can be problems when very low NO_3-N levels occur when NH_4-N levels are also low. Then, aeration has no effect because there is hardly any ammonium available to be nitrified (Barfüßer 2018). To have low nitrate and simultaneously low ammonium concentrations would be the ideal case to comply with the discharge standards, but from the automation perspective, this could lead to unnecessary aeration. Based on these results, nitrate should not be used as a single parameter controlling the aeration, but in combination with ammonium nitrogen measurements.

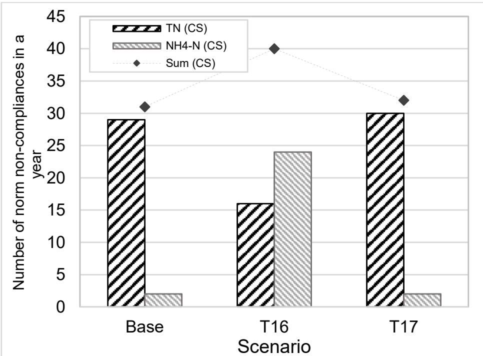


Figure 32. Number of non-compliances of the norm CS in a year with scenarios T16 and T17 (NO_3-based aeration control). T16 scenario with NO_3-N based aeration control, with 0.3 ≥ NO_3-N ≥ 0.5 mg/L; T17 scenario with NO_3-N based aeration control, with 3 ≥ NO_3-N ≥ 8 mg/L

5.2.3 Intermittent Aeration

Intermittent aeration is the alternation of aerated and anoxic phases in a single tank. When most NH_4-N has been oxidized, aeration can be stopped to favour denitrification and enable the system to reduce nitrate. As indicated by the DWA-A 131 (2016), the denitrification phase duration can be set with a timer or adjusted by a control strategy, whereas by the nitrate content, the ammonium content, the change of the redox potential or the oxygen consumption (DWA 2016). Here, the duration of the aeration was tested based on: Time, ammonium-nitrogen concentration in the effluent of the activated sludge, ammonium-nitrogen and nitrate-nitrogen concentration in the effluent of the activated sludge. The tested strategies are described in detail below and the used values are in Table 25.

Intermittent denitrification based on time

In this model, a pulse block allows to switch the aeration from zero to the desired set point and the fluctuation time can be set. The fluctuation time was varied as described in Table 25.

Intermittent aeration based on NH_4-N concentration (V_N)

The aeration is turned on based on the ammonium-nitrogen concentration. If the NH_4-N concentration in the effluent of the activated sludge tanks is larger than the set point, then the aeration is turned on, with a set point of 2 mg O_2/L. If the ammonium nitrogen concentration is lower, the aeration is turned off, to reach anoxic conditions for denitrification. The intermittent aeration is carried out in the aeration (nitrification) basin.

Intermittent aeration based on NH_4-N concentration (V_{AT})

Identical to the last strategy, but in the total activated sludge basin except for the anaerobic tanks (V_D + V_N).

Intermittent aeration based on NH_4-N and NO_3-N concentration (V_{AT})

The aeration is turned on and off based on the ammonium nitrogen and nitrate-nitrogen concentrations, if the NH_4-N concentration in the effluent of the activated sludge tanks is larger than the set point, then the aeration is turned on, with a set point of 2 mg O_2/L. If the nitrate-nitrogen concentration is higher than the set point, the aeration is turned off, to reach anoxic conditions for denitrification. The intermittent aeration is carried out in the total activated sludge (V_D + V_N) basin.

Table 25. Parameters for nitrate-nitrogen and DO set point in scenarios T18 to T26

Strategy		Aeration interval*, min	Aeration duration**, min
Time based, V_N	T18	30	2.1 (7%)
	T19	30	3.0 (10%
	T20	30	9.0 (30%)
	T21	60	30.0 (50%)
NH_4 –N based, V_N		Max NH_4 –N in the AT basins	
	T22	0.50	
	T23	1	
NH_4 –N based, V_{AT}		Max NH_4-N in the AT basins	
	T24	1	
NH_4 –N & NO_3-N based, V_{AT}		Max NH_4-N in the AT basins	Max NO_3-N in the AT basins
	T25	1	8
	T26	1	5

* The aeration interval is the frequency between aeration periods, i.e. how often aeration takes place. The remaining time is anoxic, favouring denitrification.

** Aeration duration is the length of time that will be aerated during an aeration interval

.

The aeration times in strategies T18 to T20 are relatively short (7%, 10 and 30%, aeration time, respectively) which could lead to increased wear of the aeration structure (i.e. blowers, diffusers, etc.). The remaining, non-aerated time, would be used for denitrification. As can be observed from the results in Figure 33, the best strategies are the ones that tend to improve the denitrification capacity, here by aerating intermittently only the nitrification basin, therefore maintaining a constant denitrification volume and increasing it when the aeration basin is not aerated (T22 and T23). Regarding the rest of the tested strategies, although they show improvement with respect to the baseline scenario, with between 12 and 20 norm non-compliances in a year, the results in T22 and T23 are better.

This is an indicator that intermittent aeration by itself cannot solve the problems in the tanks' proportion and distribution (low V_D/V_{AT}), which affect the denitrification capacity. However, the ammonium-based control shows promising results.

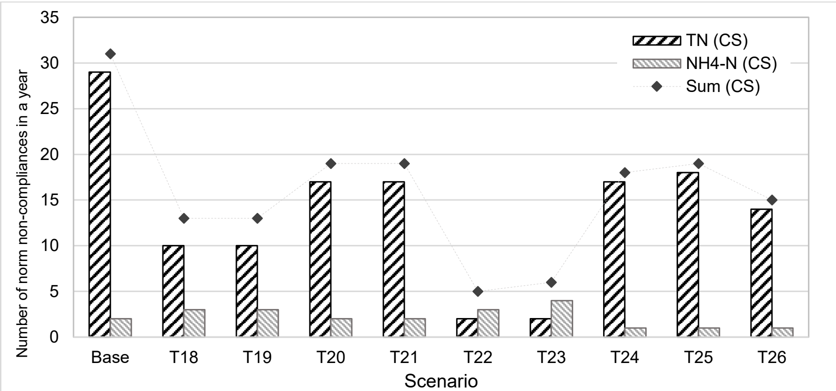


Figure 33. Number of non-compliances of the norm CS in a year with scenarios T18 to T26 (intermittent aeration). Scenarios T10 to T21 are time based; T22 and T23 NH_4-N based, in V_N; T24 NH_4-N based, in V_{AT}; T25 and T26 NH_4-N & NO_3-N based

5.2.4 Adjustment of the Sludge Age (SRT)

As discussed in Chapter 3.3.2, the real SRT fluctuates heavily, and it is apparent it is not a control parameter in the example WWTP. This can be detrimental to the plant's operation, leading to process instability. The required SRT for aerobic sludge stabilisation is temperature dependent and can be calculated according to Equation 5 as described in Chapter 3.3.2.

A dynamic adjustment of the sludge age according to temperature was tested. For the example WWTP, the minimum required SRT was calculated for different wastewater temperatures. The target sludge ages in time were compared in Figure 12.

The target sludge age, based on the temperature, according to the DWA-A 131, was tested in the model and the results are presented in Figure 34. Improving the SRT stability by itself can contribute to improve the effluent values and at the same time to save energy, as the biomass present in the activated sludge system will be adjusted to the required effect, reducing the aeration requirements. This aspect will be discussed later, in Chapter 5.3. It is recommended to implement an SRT-based control in the plant, to contribute to the operational stability and plant reliability.

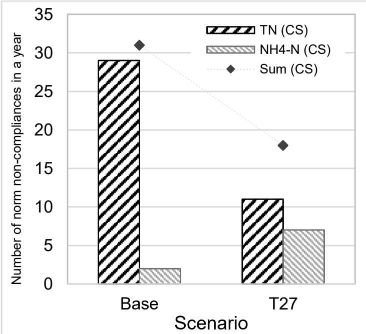


Figure 34. Number of non-compliances with the norm "City Assessment Standard" in a year with scenario T27 (SRT adjustment)

5.2.5 Summary of the Conventional Regulation Strategies

A comparison of all the individual tested strategies is presented in Figure 35. The goal of the different strategies is first to reduce the number of norm non-compliances. Secondly, to identify the best strategies to do it, in order to test in the following chapter, the best combinations.

As can be seen in Figure 35, none of the tested strategies can deliver 100% of norm compliance in a year period. The single strategies by themselves cannot completely solve the challenges posed by the sharpening of the discharge norm (CS). This is not an uncommon problem, as discussed in Chapter 2.5. The same challenge was shown in the study of (Hvala et al. 2018), where despite testing different strategies to improve nitrogen removal in a WWTP via computer modelling, a side stream treatment to comply with the discharge norm of TN < 10 mg/L could not be avoided. Other studies (Zaborowska et al. 2017) rely also on the use of side-stream treatment (e.g. Anammox) to fulfil the norm requirements for nitrogen removal.

The here tested strategies can, however, reduce them significantly from 31 in the base scenario to only 4 or 5, which represents a reduction of up to 87%. The graph below shows that scenarios T10 to T13 (i.e. scenarios with aeration control based on NH_4-N concentration in the effluent without DO limits) show some of the best results, but as discussed in Chapter 5.2.1, they are probably not applicable due to the high possibility of N_2O emissions due to the too low DO concentrations obtained in the aeration tanks.

Other strategies that show very good results are T22 and T23, which show the effects of the implementation of ammonium-based intermittent aeration.

The transformation of primary clarifiers to denitrification tanks is also a very interesting strategy, as observed in scenarios T6 to T9, with a significant minimization of the number of norm non- compliances. A similar approach was also tested successfully by other authors, for the upgrading of the WWTP Denia (Spain), to comply with the European norm for TN and TP discharge (Seco et al. 2020).

In general, it can be concluded that the strategies oriented to increment the denitrification capacity are more successful.

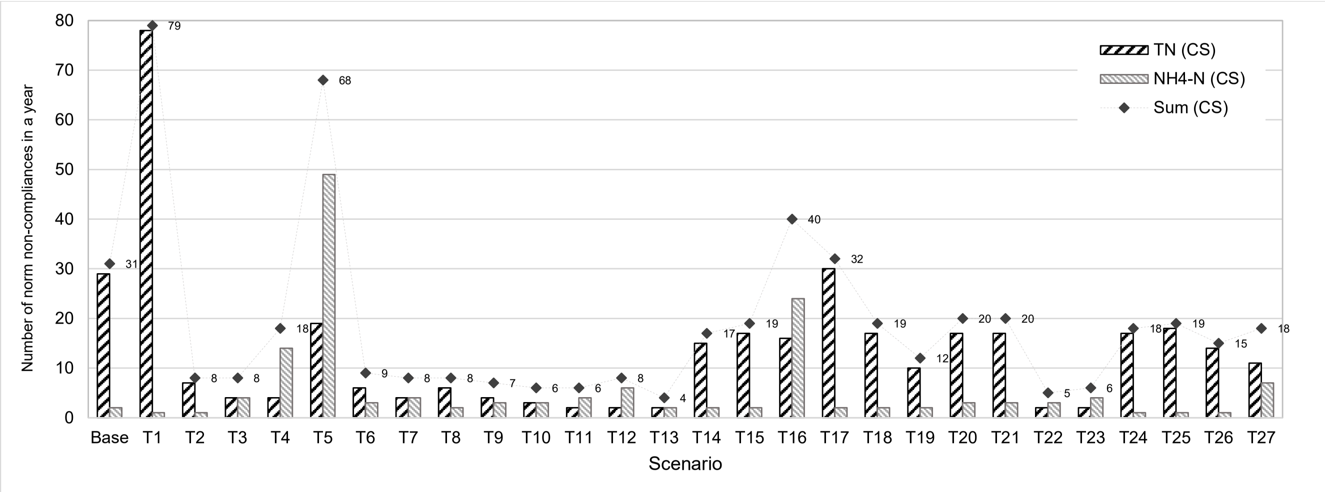


Figure 35. Number of non-compliances of the norm CS in a year with scenarios T1 to T27

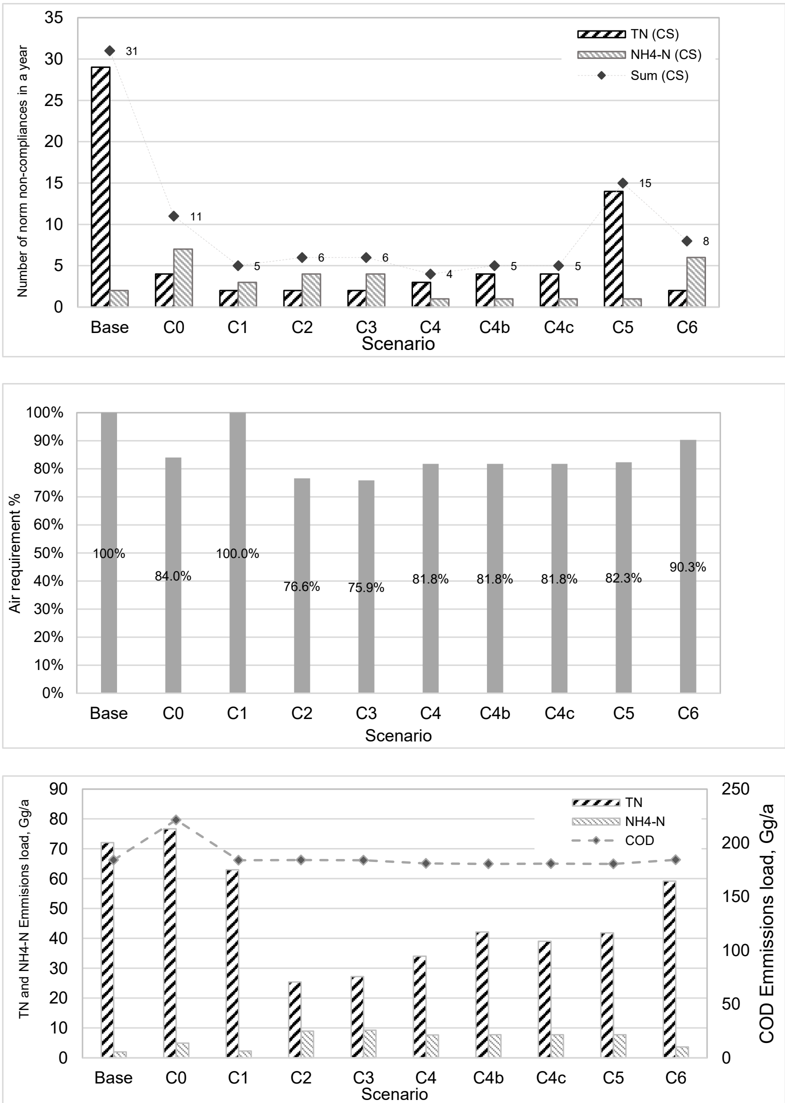
5.3 Combination of Strategies

Based on the best results obtained with the individual strategies, different combinations of strategies were tested, combining the best results and creating synergies. The combinations are based on some of the best results obtained in the tests T1 to T27. The description of the tested combinations is shown in Table 26.

Table 26. Description of the tested Combinations of strategies

Combination	Description
Combi 0	• DO set point decrease to 2 mg O_2/L • V_D/V_{AT} =30% (by reduction of the aerated volume to expand the denitrification volume from 16,000 m^3 to 24,000 m^3)
Combi 1	• Bypass of 50% of the PC volume • V_D/V_{AT} =30% (by reduction of the aerated volume to expand the denitrification volume from 16,000 m^3 to 24,000 m^3)
Combi 2	• Bypass of 50% of the PC volume • V_D/V_{AT} =19% (by the use of the empty PC volume as denitrification volume) • Intermittent aeration, based on NH_4-N concentration (V_N): o If NH_4-N > 1 mg/L, then DO_{sp} =2 mg/L, o else, DO_{sp} =0.01 mg/L
Combi 3	• Bypass of 50% of the PC volume • V_D/V_{AT} =30% (by reduction of the aerated volume to expand the denitrification volume from 16,000 m^3 to 24,000 m^3) • Intermittent aeration, based on NH_4-N concentration (V_N): o If NH_4-N > 1 mg/L, then DO_{sp} =2 mg/L, o else, DO_{sp} =0.01 mg/L
Combi 4a	• Bypass of 50% of the PC volume • V_D/V_{AT} =19% (by the use of the empty PC volume as denitrification volume) • Intermittent aeration, based on NH_4-N and NO_3-N concentrations (V_{AT}): o If NH_4-N ≥ 1 mg/L, then DO_{sp} =2 mg/L, o If NO_3-N ≥ 5 mg/L, then DO_{sp} =0.01 mg/L
Combi 4b	• Bypass of 50% of the PC volume • V_D/V_{AT} =19% (by the use of the empty PC volume as denitrification volume) • Intermittent aeration, based on NH_4-N and NO_3-N concentrations (V_{AT}): o If NH_4-N ≥ 1 mg/L, then DO_{sp} =2 mg/L, o If NO_3-N ≥ 5 mg/L, then DO_{sp} =0.01 mg/L • NO_3-N based internal recirculation (RZ): o If NO_3-N > 5 mg/L then RZ = 200% Q_{in} o else RZ = 10% Q_{in}
Combi 4c	• Bypass of 50% of the PC volume • V_D/V_{AT} =19% (by the use of the empty PC volume as denitrification volume) • Intermittent aeration, based on NH_4-N and NO_3-N concentrations (V_{AT}): o If NH_4-N ≥ 1 mg/L, then DO_{sp} =2 mg/L, o If NO_3-N ≥ 5 mg/L, then DO_{sp} =0.01 mg/L • NO_3-N based internal recirculation (RZ): o if NO_3-N <1 mg/L then RZ = 10% Q_{in} o if 3 < NO_3-N < 5 mg/L then RZ = 50% Q_{in} o if 5 < NO_3-N < 7 mg/L then RZ = 150% Q_{in} o if NO_3-N > 7 then RZ = 200% Q_{in}
Combi 5	• Bypass of 50% of the PC volume • V_D/V_{AT} =30% (by reduction of the aerated volume to expand the denitrification volume from 16,000 m^3 to 24,000 m^3) • Intermittent aeration, based on NH_4-N and NO_3-N concentrations (V_{AT}): o If NH_4-N ≥ 1 mg/L, then DO_{sp} =2 mg/L, o If NO_3-N ≥ 5 mg/L, then DO_{sp} =0.01 mg/L
Combi 6	• Bypass of 50% of the PC volume • V_D/V_{AT} =33% (by an increase of the denitrification volume by replacing the anaerobic volume with anoxic (and changing the water recirculation point)).

To evaluate the best combinations, three criteria are monitored and compared in Figure 36: Norm compliance: comparison of the number of norm non-compliances in a year; Aeration requirements: the amount of air in Nm^3/d, in comparison with the base line scenario; Pollutants emissions: the amount of TN, NH_4-N and COD emissions (as mass) in a year.



(a)

(b)

(c)

Figure 36. (a) Number of non-compliances of the norm CS in a year; (b) Air requirements; (c) Pollutants load, with scenarios C0 to C6

The increase in the denitrification proportion to $V_D/V_{AT} = 0.3$ is helpful to improve the TN effluent values, as shown in T2 and T3. By maintaining the DO_{sp} in 3 mg/L (T3), only 3.5% less air is required in a year. In Combination 0, when decreasing the DO_{sp} to 2 mg/L, 16% less air is required. Combination 0 contributes to improve the current norm compliance, reducing the number of times the norm is not fulfilled in a year. However, there is an increase in the total pollutants emission, as there is less oxygen available for the oxidation of ammonium and COD.

Combination 1 increases significantly the norm compliance, but it does not save energy for aeration. In contrast, Combinations 2, 3 and 4 increase significantly the norm compliance, and save significant energy for aeration. An increase in the denitrification capacity together with the flexibility offered by the intermittent aeration is a combination of strategies that show close to optimal results in terms of norm compliance, pollutants emissions and energy savings in aeration.

In combinations 4b and 4c, just as in Combi 4, an increase in norm compliance and energy savings is observed. However, the incorporation of a nitrate-based control, and control for the recirculation, either simple (variation C4b) or complex (variation C4c) flow does not represent an improvement in the effluent values, nor in the air requirements. Savings in recirculation are not significant either (0.2% and 0.1% respectively are estimated). The recirculation flowrate changes, but the peaks counteract the lows at the end (see Annex 12.9). As these strategies show higher pollutants emissions and are more complex, variant 4 is selected as better.

Combination 5 presents higher non-compliances for TN in a year, indicating that the process benefits from having a separate denitrification volume. The pollutants emissions are similar to C4b but worse than C2, C3 and C4 because in C5 the nitrification volume is reduced.

Combination 6 improves the norm compliance with a simple approach: eliminating the anaerobic tank for biological phosphorus removal to increase the denitrification capacity. This could be possible because the plant already has a targeted chemical P elimination process. This strategy reduces the air required by almost 10%, but not in stirring, as they remain the same. The pollutants emission is reduced when compared with the base line scenario, but the reduction is less than in strategies C2 to C5.

The results show that the poor configuration for denitrification of the WWTP (too low anoxic tank proportion) can be counteracted by reducing the aerated volume because the plant has an activated sludge volume that is large enough. This has consistently proven to be the most effective – and probably the easier to apply– strategy.

Moreover, intermittent denitrification can be a viable option for the, sometimes, unfavourable conditions of the treated wastewater. The total or partial by-pass of the primary clarifiers is of secondary importance in the overall denitrification capacity.

With the change in denitrification proportion, as a base, automation strategies and sensors add to the plant performance and an adequate combination of several strategies can help to improve the current condition considerably.

Even with simple changes, as the incorporation of an ammonium sensor to the aeration control loop can be advantageous, saving energy and improving the overall plant performance. However, none of the tested strategies can avoid completely norm exceedances, showing that additions such as the use of external carbon sources or the inclusion of a post-treatment (e.g. post denitrification) should be evaluated. Despite this, even when a post-treatment or external chemicals may be required, the incorporation of the tested strategies can contribute to saving resources such as electricity or external C-sources which would be required less often.

5.3.1 Evaluation of the Combination Scenarios

To better understand the results obtained with the different combination scenarios, selected criteria will be evaluated using the symbology described in Table 27. The selected criteria and results are shown in Table 28.

Table 27. Scenarios Evaluation symbology in relation to the base scenario

Evaluation Criteria	Symbol	Value
Strong positive impact (≥ 40% better)	+++	+3
Moderate positive impact (≥ 20% better)	++	+2
Slight positive impact	+	+1
Neutral impact	0	0
Slightly negative impact	-	-1
Moderate negative impact (≥ 20% worse)	- -	- 2
Strong negative impact (≥ 40% worse)	- - -	-3

A "positive" evaluation indicates changes in results, in terms of e.g. lower emissions, lower air consumption, fewer norms non-compliances. The "negative" impact evaluation indicates the opposite. The 20% or 40% criteria to decide if the value is moderate or strongly positive or negative, applies for each numeric value, with the units described in Figure 36.

Table 28. Scenarios evaluation comparison

Criteria	Scenario								
	C0	**C1**	**C2**	**C3**	**C4**	**C4b**	**C4c**	**C5**	**C6**
Sum, CS norm compliance	+3	+3	+3	+3	+3	+3	+3	+3	+3
TN, CS norm compliance	+3	+3	+3	+3	+3	+3	+3	+3	+3
NH_4-N, CS norm compliance	-3	-3	-3	-3	+3	+3	+3	+3	-3
TN, emissions	-1	-1	+3	+3	+3	+3	+3	+3	+1
NH_4-N, emissions	-3	-1	-3	-3	-3	-3	-3	-3	-3
COD, emissions	-2	0	0	0	0	0	0	0	0
Air consumption	+1	0	+2	+2	+1	+1	+1	+1	+1
Average	**-0.3**	**0.1**	**0.7**	**0.7**	**1.4**	**1.4**	**1.4**	**1.4**	**0.3**
Evaluation	**-**	**+**	**+**	**+**	**++**	**++**	**++**	**++**	**+**

According to this evaluation criteria, considering all the previously evaluated aspects (i.e. norm-compliance, emissions, and air consumption), scenarios C4 to C5 show the best overall results: they increase significantly the norm compliance and reduce enormously at the same time the emissions to the environment, simultaneously decreasing the air requirements.

These results are obtained in the scenarios with intermittent denitrification (NH_4-N based) and denitrification volume increase. This is an expected result, based on the results obtained in Chapter 5.2, in the single-changes scenarios.

5.3.2 Summary of the Combination of Strategies

It can be observed that the base scenario is the least favourable and all tested combination scenarios show improvements in terms of norm compliance. Meanwhile, in all combination scenarios, the TN norm compliance is significantly increased.

However, in many of them, at the same time, the norm compliance for ammonium nitrogen is increased. The same trend can be observed for the emissions because all strategies point to an improvement in the denitrification capacity and more efficient use of the air. There is an undeniable balance and compromise between the oxidized ammonium and the removed nitrate. As can be observed in Figure 36 (a), a significant reduction in norm non-compliances for TN (associated mostly to nitrate nitrogen) can be achieved, but the norm non-compliances for ammonium nitrogen increase slightly when compared with the base scenario.

5.4 Tests with Other Discharge Norms for the Treated Wastewater

In a rapidly changing world, wastewater treatment plants must be able to adapt to new discharge standards. To better understand the potential of the previously tested strategies, it is interesting to test them under different conditions, as other discharge norms. Therefore, the best scenario results obtained for the norm CS, according to Table 28 (i.e. Combi 3, Combi 4 and Combi 5) and the Base scenario, are compared to three other different standards selected from Table 4 and summarized in Table 29.

The first standard is the usual standard applied for the discharge of Wastewater in China, Grade I-A, followed by the current standard applied to the example WWTP (CS). The third is a laxer standard, the standard to be met at the EU level (here called “Norm EU”), according to the currently valid water directive. Then the German standard for WWTP size class 5 (> 100,000 PE), here called “Norm GER”, was tested. Finally, the results were compared with a fifth standard (“Norm LUX”), the standard applied in Luxembourg, for the WWTP Beggen, which is the strictest standard found in the literature research.

Table 29. Selected Norms for comparison

Standard /Norm			COD	TN	NH_4 -N	Sample frequency
Country	**Information**	**Name**	**mg/L**	**mg/L**	**mg/L**	
China	Grade I-A	Grade I-A	50	15	5 (8)	24-h
China	City standard, Taihu Basin	CS	30	10	1.5 (3)	24-h
European Union	European Water Directive	Norm EU	125	10	-	24-h
Germany	WWTP Size Class 5	Norm GER	75	13* (-)	10* (-)	2-h
Luxemburg	WWTP Beggen	Norm LUX	50	8	1 ((2))	24-h 2-h for NH_4-N

The values in round brackets ( ) are for wastewater temperatures below 12 °C
The values in double brackets (( )) are for temperatures below 8 °C
* The German norm AbwV, indicates that between 01/05 and 31/10 a concentration of up to 25 mg/L for TN and NH_4-N is permitted if the reduction of TN is at least 70% (in a 24-h average)

The German norm also contemplates a “4 out of 5 rule”, which indicates that if in 5 measurements of the preceding state inspections (in a period up to 3 years), only one does not comply, and the value does not exceed the norm in more than 100%, the norm is considered fulfilled (Bundesamt für Justiz 2020).

Another interesting point is the planned update to the EU Water directive ("EU rules on urban wastewater treatment") (European Commission 2021), which plans to sharpen nutrient removal and recovery from WWTP in the next years (European Commission 2019). The process should have its first results, after a review of the public consultation, in October 2022.

Although the concentration values between the Norm_LUX and CS norms are relatively similar for NH_4-N, the norm in Luxembourg requires a control of the values every 2 hours. Therefore, the standard to be met is much stricter. Something similar occurs with the German norm which also requires a 2-h composite sample, but for higher discharge values.

Even with a 2-h sampling strategy, it is not expected that there are multiple controls in one day. Therefore the maximum theoretical norm non-compliances, considering each 2-hour composite sample possible in a day (see values in brackets in Table 30), from now on named "Maximum", is compared with an estimation of a realistic control, of only one 2-hour composite sample per day (see Figure 37 (a) (b) and (c)), from now on named "Realistic".

The "4 out of 5 rule" from the German standard will not be considered, as there is no realistic way to estimate how often the authorities will control.

How strict the last two standards (Germany and Luxembourg) are, can be observed in Figure 37 (c), where the results that represent less than 5 norm non-compliances per year with the norms Grade I-A, CS and Norm EU in Scenarios C3, C4 and C5, show a significant increase the in the number of non-compliances in the last two norms, both in the maximum and the realistic approach.

In total nitrogen and ammonium nitrogen, this difference is large for the Luxembourgian norm. For example, in the case of the base scenario, the number of norm non-compliances for TN with the Luxembourg norm (realistic) is more than twice as high as with the CS norm.

The limit value for ammonium nitrogen in the German norm (10 mg/L) is much higher than in the CS norm (1.5 mg/L) and in Luxemburg (1 mg/L), therefore the number of non-compliances there is lower in all scenarios.

Since the strategies tested in scenarios C2 C3 and C4 were oriented to improve denitrification (see Chapter 5.3), and they tend to increase slightly the ammonium effluent values, the number of norm non-compliances for ammonium nitrogen is increased in the combination scenarios for almost all norms.

Based on the obtained results in the best scenarios, the addition of an external C-source would not contribute significantly to improving the total norm compliance. It is possible to see in Figure 37 that, based on the tested strategies, the WWTP does not have problems with the removal of nitrate, but there are limitations in the oxidation of ammonium. As the operational and automation strategies significantly improve the denitrification capacity (e.g. with only two TN norm non-compliances in a year in scenario C3 for all norms), the dosing of C-source could be required, but only on very few occasions in a year.

Therefore, to further improve norm compliance, the optimization strategies should be oriented to improve ammonium removal, for example, improve aeration capacity (e.g. increasing the capacity of the blower, upgrading or cleaning the aeration elements or diffusors, remove sand from the aeration tanks, etc.), improve mixing and avoid short-circuiting or dead zones in the nitrification tanks, etc. Most of these strategies, however, are either out of the scope of this work, or cannot be tested in SIMBA, and other tools (e.g. Computational Fluid Dynamics (CFD) simulation) would be necessary.

In the case of the Luxembourgian norm, it is possible that the dosing of an external C-source in Scenarios C4 and C5 improves norm compliance, or that even a downstream denitrification stage is necessary. However, here again, the nitrification presents more problems to comply with the extremely strict standard of 1 mg/L (2-h composite sample).

From these results, it can be concluded that the strategies designed and deemed effective for compliance with the CS standard are somewhat less effective in meeting the requirements of a standard such as the Luxembourg standard, with very low discharge values on a 2-hour composite sample basis. However, it is clear that the tested strategies, especially scenario C3 can significantly improve norm compliance under all tested norms. A more detailed discussion of the conditions to comply with this discharge norm is out of the scope of this work.

In the case of the German norm, as the TN and specially NH_4-N discharge concentrations are higher, the 2-h composite sample does not have a dramatic effect on the number of norm non-compliances per year, in general even lower than for the CS norm. Therefore, the strategies here tested could be useful as well in German WWTP, even of the largest size class of WWTP.

Table 30. Summary of the number of norm non-compliances with different norms for scenarios C3, C4 and C5

Norm	TN				NH4-N				Total			
	Base	**C3**	**C4**	**C5**	**Base**	**C3**	**C4**	**C5**	**Base**	**C3**	**C4**	**C5**
Grade I-A	1	0	1	1	0	1	0	0	1	1	1	1
CS	29	2	3	14	2	4	1	1	31	6	4	15
Norm EU	29	2	3	14	-	-	-	-	29	2	3	14
Norm GER	10 (64)	2 (17)	2 (16)	2 (16)	0	1 (8)	1 (8)	0	10 (64)	2 (29)	3 (24)	3 (16)
Norm LUX	89	2	11	29	3 (21)	12 (106)	10 (91)	17 (124)	92 (110)	14 (108)	21 (102)	46 (124)

The values in brackets () are for the maximum number of norm non- compliances, considering all possible 2-h based non-compliances in a day

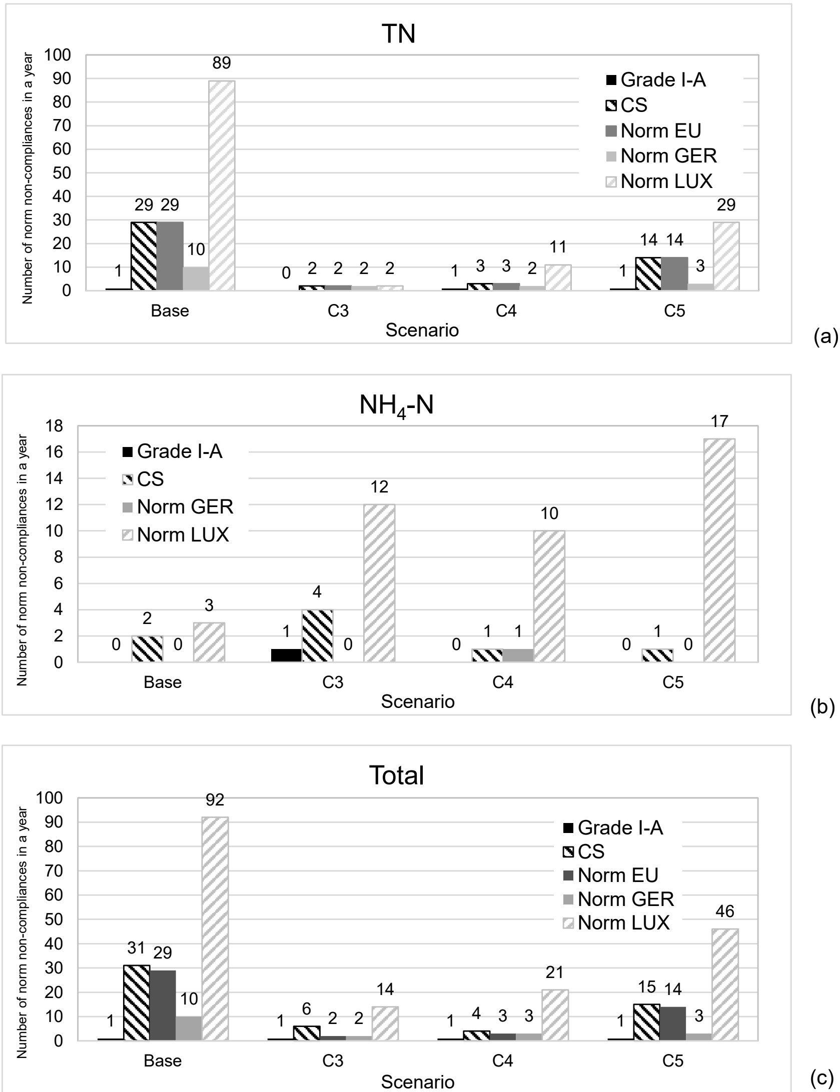


Figure 37. Number of non-compliances in a year for different norms, based on the "realistic" assumption for (a) TN; (b) for NH_4-N (Note: the EU Norm does not have a limit for NH_4-N); (c) Total; Scenarios C3 (Bypass of 50% of V_{PC}, V_D/V_{AT} =30% (reduction of the aerated volume), intermittent aeration based on NH_4-N); C4 (Bypass of 50% of V_{PC}, V_D/V_{AT} =19 (use of the empty V_{PC} as V_D), intermittent aeration, based on NH_4-N and NO_3-N concentrations); C5 (Bypass of 50% of V_{PC}, V_D/V_{AT} =30% (reduction of the aerated volume), intermittent aeration based on NH_4-N and NO_3-N concentrations)

5.5 Summary of Chapter 5

Several strategies to improve the removal of nitrogen compounds, oriented to increase the norm compliance for the CS norm were tested in the model built in Chapter 4.2. Twenty-seven different strategies (T1 to T27) were tested and the results were depicted and discussed. The strategies included operational strategies, such as the decommissioning of primary clarifiers, the increase in the denitrification volume, as well as automation strategies, based mostly on the incorporation of ammonium nitrogen online sensors, the application of intermittent aeration instead of the existing upstream denitrification, and the control of the sludge age.

These strategies were tested individually, and all of them show at least a small improvement in overall norm compliance, but some of them show very promising results. It was observed that the most effective are the ones oriented to increase the denitrification capacity of the example WWTP. The results show it is possible to increase norm compliance, with simple operational and automation strategies.

The most effective individual strategies in terms of norm non-compliances were selected, to be tested in nine different combinations (C0 to C6).

The combination of strategies such as the increase of denitrification volume by decreasing the aerated volume, and ammonium-based intermittent denitrification were the most successful strategies, reducing the total number of norm non-compliances from 31 in one year to 4-5 per year. None of the strategies could reach 100% of norm compliance.

However, these results show it is possible to significantly increase norm compliance and at the same time reduce the energy consumption associated with aeration. In many cases, the overall emissions to the environment are also reduced. The optimisation strategies tend to significantly improve the TN norm compliance, and at the same time maintaining or slightly increasing the norm non-compliances (and emissions) for ammonium nitrogen.

In Table 31 and Table 32, the main results are summarized for all tested strategies and combinations.

Afterwards, the obtained results, for the best combinations available (C3, C4 and C5) were evaluated with different norms, laxer and stricter ones: Grade I-A (China), EU-Water directive, and the German and Luxembourgian discharge norms.

The tested strategies are very effective for the first four mentioned norms, and less effective for the norm from Luxembourg, due to its extremely low discharge values on a 2-h composite sample basis for ammonium nitrogen. However, a clear improvement from the Base scenario can be noticed. In this case, further optimization strategies would be required, which are not discussed in detail in this work.

Because most norm non-compliances are related to NH_4-N and not to nitrate, it is concluded that the dosing of external C-sources would be necessary only on punctual occasions.

Once cleared the objective of improving the norm compliance for nitrogen removal, it arises the question, of whether it is possible to stabilize the sludge anaerobically, without compromising the plant performance. For a WWTP of the size of the example WWTP, it is estimated this would make a lot of sense energetically and from a sludge disposal perspective, however, the norm compliance for nitrogen compounds might be a challenge. This is the topic explored next chapter.

Table 31. Number of norm non-compliances in a year and air requirements with combinations of strategies T1 to T27

Variation	Strategy	Parameters		Number of norm non-compliances in a year				Aeration requirements
				TN		NH_4-N		
				I-A	CS	I-A	CS	Δ with base
Modifications in V_D/V_{AT}		**V_D/V_{AT}**	**V_{PC}**					
	Base	0.17	100%	1	29	0	2	Base
	T3	**0.30**	**100%**	**1**	**4**	**1**	**4**	**-3.4%**
	T4	0.40	100%	2	4	2	14	-
	T5	0.50	100%	2	19	4	49	-
Increase in V_D, decrease in V_{PC}		**V_D/V_{AT}**	**V_{PC}**					
	T6	0,218	50%	0	6	1	3	
	T7	0.242	25%	0	4	2	4	
Dynamic increase in V_D, decrease in V_{PC}		**V_D/V_{AT}**	**V_{PC}**					
	T8	0.218	50%	0	6	1	2	-0.7%
	T9	**0.242**	**25%**	**0**	**4**	**1**	**3**	
Max and min values for NH_4-N based aeration control		**Min NH_4.N, mg/L**	**Max NH_4.N, mg/L**					
	T10	0.3	0.5	0	3	1	3	-17.4%
	T11	0.3	0.75	1	2	1	4	
	T12	0.5	0.9	0	2	1	6	
	T13	**0.5**	**1.35**	**0**	**2**	**1**	**2**	**-13.1%**
Max and min values f–r NO_3 -N based aeration control		**Min NO_3.N, mg/L**	**Max NO_3.N, mg/L**					
	T16	0.5	5.0	1	16	2	24	
	T17	0.3	8.0	0	30	1	2	

Table 31 (continued). Number of norm non-compliances in a year and air requirements with combinations of strategies T1 to T27

Variation	Strategy	Parameters		Number of norm non-compliances in a year				Aeration requirements
				TN		NH_4-N		
				I-A	CS	I-A	CS	Δ with base
NH_4-N based aeration control w/ DO limits		**DO sp, mg/L**	**NH_4-N sp, mg/L**					
	T14	0.8 - 2	0.3 – 1.05 Curve	**0**	**15**	**1**	**2**	-
	T15	1 - 2	0.3 – 1.05 Curve	0	17	1	2	
Intermittent aeration, time based		**Aeration interval, min**	**Aeration duration, min**					
	T18	30	2,1	3	17	1	2	-17.7%
	T19	30	3,0	**2**	**10**	**1**	**2**	-
	T20	30	9,0	0	17	1	3	-1.4%
	T21	60	30,0	0	17	1	3	-
Intermittent aeration, NH_4-N based		**NH_4-N max, mg/L**						
	T22	0.5		**0**	**2**	**1**	**3**	**-21.9%**
	T23	1		0	2	1	4	-23.3%
Intermittent aeration, NH_4-N and NO_3-N based		**NH_4-N max, mg/L**	**NO_3-N max, mg/L**					
	T24			2	17	0	1	
	T25			2	18	0	1	
	T26			1	14	0	1	
T-based SRT		**T range, °C**	**SRT range, d**					
	T27	10- 24	28.7 – 14.3	1	11	2	7	
Norm values				**Norm discharge values, mg/L**				
				15	10	5 (8)	1.5 (3)	

- **Grade I-A:** GB18918-2002 Grade 1-A standard
- **City standard**: Special discharge limits, City Assessment Standard
- The values inside the brackets are the control values when the water temperature is ≤ 12 °C
- The values in **bold** are the best result for each strategy
- **Note**: in each strategy, COD effluent concentrations were not surpassed.
- sp = set point
- Curve (see Figure 30)

Table 32. Number of norm non-compliances in a year and air requirements with combinations of strategies C0 to C6

Combination	Parameters					Number of norm non-compliances in a year				Aeration requirements
						TN		NH_4-N		
						I-A	CS	I-A	CS	Δ with base
Base scenario	**V_D/V_{AT}**	**V_{PC}**								
	0.17	100%	-	-	-	1	29	0	2	Base scenario
	V_D/V_{AT}	**DO_{sp}**								
Combi 0	0.3	2	-	-	-	2	4	2	7	16.0%
Combi 1	**V_D/V_{AT}**	**V_{PC}**								
	0.3	50%	-	-	-	0	2	1	3	0%
Combi 2	0.218	50%	-	-	-	0	2	1	4	23,4%
Combi 3	**V_D/V_{AT}**	**V_{PC}**	**Int. Aeration NH_4-N sp, mg/L**							
	0.3	**50%**	1	-	-	**0**	**2**	**1**	**4**	**24,1%**
Combi 4	**V_D/V_{AT}**	**V_{PC}**	**Intermittent Aeration (total V_{AT})**		**RZ**					
			NH_4-N, mg/L	**NO_3-N, mg/L**						
	0.218	50%	1	5	200%· Q_{in}	1	3	0	1	18.2%
Combi 4b	0.218	50%	1	5	Scale 1*	1	4	0	1	18.2%
Combi 4c	0.218	50%	1	5	Scale 2**	1	4	0	1	18.2%
Combi 5	**V_D/V_{AT}**	**V_{PC}**	**Intermittent Aeration (total V_{AT})**							
			NH_4-N, mg/L	**NO_3-N, mg/L**						
	0.167	50%	1	5	-	1	14	0	1	17.7%
Combi 6	0.33	50%	-	-		0	2	2	6	9.7%
					Norm values	15	10	5 (8)	1.5 (3)	

- **I-A:** GB18918-2002 Grade 1-A standard
- **CS**: Special discharge limits, City Assessment Standard
- The values inside brackets are valid when the water temperature is ≤ 12 °C
- The values in **bold** are the best result for each strategy
- **Note**: in each strategy, COD effluent concentrations were not surpassed
- sp = set point

Scale 1
If NO_3-N >5 mg/L
then RZ = 200%· Q_{in}
Else RZ = 100%· Q_{in}

Scale 2
If NO_3-N < 1 mg/L Then RZ = 10%· Q_{in}
1 < NO_3-N < 3 mg/L Then RZ = 50 Q_{in}
38 3 < NO_3-N < 5 mg/L Then RZ = 100%· Q_{in}
5 < NO_3-N < 7 mg/L Then RZ = 150%· Q_{in}
NO_3-N > 7 mg/L Then RZ = 200%· Q_{in}

6 Addition of Anaerobic Digestion Stage

Anaerobic sludge stabilisation is a strategy widely used in WWTP worldwide, e.g. in Germany, that can contribute positively to the energetic and mass balance of the WWTP. As discussed in Chapter 2.1.3.7, anaerobic digestion of sewage sludge generates methane, an energy-rich gas, reducing the organic matter and mass of the sludge (Barreto Dillon 2015).

China has almost doubled the amount of sewage sludge produced in the last decade, but the use of anaerobic sludge stabilisation is still very low. This has many explanations, such as several preconceived ideas against the use of anaerobic digestion and local barriers as discussed in detail in Chapter 2.1.3.7.

Chinese WWTP could profit from the application of the technology, therefore a technical evaluation of the effects, advantages, and disadvantages in the framework of this work are interesting. Due to the plant size of the example WWTP, an anaerobic sludge stabilization is recommended. When well designed and operated, this will contribute to the overall efficiency of the plant, by generating electricity and heat through the produced biogas.

Here, a relative comparison is aimed, estimating if the inclusion of an anaerobic digestion step for sludge stabilisation would be feasible. To do that, the first step is the design of an anaerobic sludge stabilisation stage, considering the dimensions of the reactors and associated facilities. This is followed by the adjustment of the sludge age, and then by the modelling of the dynamic biogas production, potential energy savings, and nitrogen backload and influence in the effluent values.

With the incorporation of an anaerobic digestion stage, the system must be able to handle the generated nutrients backload coming from the mixed sludge liquor. In anaerobic fermentation, there is a release of ammonia-nitrogen and phosphorous due to the degradation of organic matter under reductive conditions.

According to the DWA-A 131 (DWA 2016), the proportion of nitrogen released as NH_4-N during digestion can be approximately estimated as 50% of the nitrogen incorporated in the biomass. Other sources indicate that this backload can be estimated as 1.5 g N/(PE·d) (Fimml 2010). From both estimation methods, the last one is selected, since it gives a larger N concentration. For a plant size of 345,000 $PE_{BOD,60}$ approx. 4.9 mg N/l in the return sludge liquor is estimated, reaching a backload of approx. 14.7% (considering the average influent TN concentration during the modelled period). This increases the denitrification requirements of the plant, and the effect will be observed in the effluent values. It is expected, however, that with adequate automation strategies, as described in previous chapters, the plant can comply better with the discharge norms.

In order to add an anaerobic sludge stabilisation stage, the sludge age of the activated sludge process must be modified. After that, the anaerobic reactors will be dimensioned.

6.1 Sludge Age Modification

To incorporate an anaerobic digestion stage, the sludge age must be adjusted. The example plant operates at very high sludge ages on average, even higher than the required 25 days for cold temperatures, based on provided and calculated data.

The required sludge age for anaerobic sludge stabilisation is calculated according to DWA-A 131 in Equation 7.

$$\mathrm{SRT} = \mathrm{t_{TS}} = \mathrm{PF} * 3.4 * 1.103^{(15-\mathrm{T})} * \frac{1}{1-\left(\frac{\mathrm{V_D}}{\mathrm{V_{AT}}}\right)} \qquad \text{Equation 7}$$

Where:

SRT = sludge age

T = temperature in °C

V_{AT} = Activated sludge volume

V_D = denitrification volume

PF = Process factor = 1.5 (> 100.000 PE)

As can be seen in Figure 38, the inclusion of an anaerobic sludge stabilisation stage requires a drastic reduction of the sludge age. This will save energy for aeration since a lower amount of biomass in the activated sludge systems requires less oxygen and no additional oxygen is required for the stabilisation of the sewage sludge. At the same time, the sludge extraction will be increased and therefore the energy for pumping. Due to the biomass reduction in the system, the activated sludge tank volume could even be reduced.

However, the stability of the biological treatment process can be reduced, due to the decreased biomass retention in the system, an aspect that can be evaluated with the use of dynamic simulations.

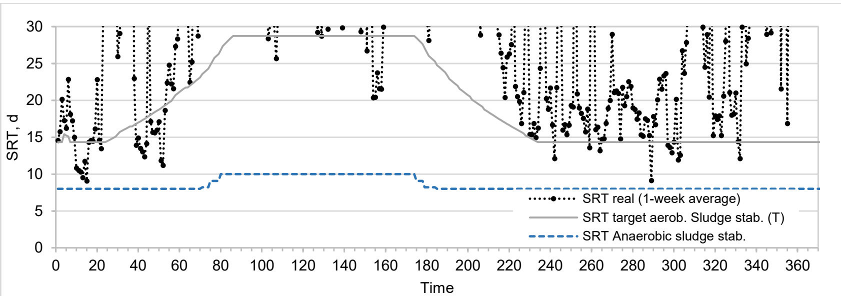


Figure 38. SRT: current weekly average (dashed black), required for aerobic sludge stabilisation (light grey), and required for anaerobic sludge stabilisation (dashed blue)[13]

In order to avoid stability problems that may arise due to the unfavourable conditions of the example WWTP (e.g. small size tank, too small denitrification proportion), the system will be operated in the model at a slightly higher sludge age than the minimum required according to the DWA-A 131, two days higher than the target SRT. This is a conservative approach towards the biological stage design, but it will influence biogas production negatively.

[13] For a better view of the real SRT, see Figure 12

6.2 Anaerobic Digestion Stage Dimensioning

The anaerobic sludge stabilisation stage is designed based on the calculated sludge production, according to DWA-A 131. For this design, a temperature of 12 °C was selected and the COD fractionation described in Figure 70 was applied. The fractionation of the COD in the influent wastewater is calculated based on the 85%-percentile of the inlet COD concentration between the years 2017 and 2019. According to the HRT in primary clarifiers (HRT_{PC} >2 h), the COD removal is estimated at 30%.

The soluble COD and the corresponding fraction are summarized in Table 33 and Annex 12.10.

Table 33. COD fractionation percentages

<table>
<tr><td colspan="4" align="center">C
100%</td></tr>
<tr><td colspan="2" align="center">S
40%</td><td colspan="2" align="center">X
60%</td></tr>
<tr><td align="center">S_S
36%</td><td align="center">S_I
4%</td><td align="center">X_S
54%</td><td align="center">X_I
6%</td></tr>
</table>

Where:

C ($C_{COD,AT}$) = Total COD to biological treatment

S ($S_{COD,AT}$) = Soluble COD to biological treatment

X ($X_{COD,AT}$) = Particulate COD to biological treatment

S_S ($S_{COD,biodeg,AT}$) = Easily biodegradable COD to biological treatment

S_I ($S_{COD,i,AT}$) = Inert COD to biological treatment

X_S ($X_{COD,biodeg,AT}$) = Easily biodegradable COD to biological treatment

X_I ($X_{COD,i,AT}$) = Inert (slowly biodegradable) COD to biological treatment

It is important to note that due to the restrictions imposed by the pandemic in 2020 and 2021, real samples of the wastewater at the example WWTP were not possible, and therefore the fractionation is estimated based on literature data. This represents some uncertainty in the base model and here, in the excess sludge production. However, since the inclusion of an anaerobic digestion stage implies changes in sludge age and MLSS, sludge production will change anyway.

The primary and tertiary sludge production were used as calculated in Section 3.3.2 and are not modified for the design of the anaerobic digestion stage. Primary sludge production is one of the most defining parameters for anaerobic digestion and biogas production since primary sludge is the richest in energy.

Annex 12.10 presents the static calculation of primary, excess, and tertiary sludge production at 12 °C. The estimated excess sludge production is 26,524 kg/d fresh mass, and the total sludge to thickening is 43,338 kg/d (15,254 kg/d primary sludge and 1,670 kg/d tertiary sludge).

The anaerobic digestion stage was designed according to the parameters presented in Table 34. The target sludge concentration after thickening is 50 g/l, therefore the average sludge flow to anaerobic digestion is ca. 870 m^3/d. For a target sludge age of 22 d, the required anaerobic digestion volume is 20,000 m^3.

Table 34. Calculation of the design parameters for the anaerobic sludge stabilization stage

Parameter		Value	Units	Information
Sludge to thickening	$Q_{sludge,thick}$	4,154	m^3/d	Calculation
Sludge concentration after thickening	TS_{Sludge}	50	g/l	Assumption
Sludge flowrate after thickening	Q_{sludge}	869	m^3/d	Calculation
Sludge retention time in the anaerobic reactors	SRT_{AD}	22	d	Assumption
Total sludge volume	$V_{sludge,AD}$	19,117	m^3	
Total reactor volume	$V_{reactor,AD}$	21,984	m^3	$V_{sludge,AD}$ + 15%
Corrected total volume	$V_{reactor,AD,\ corr}$	22,000	m^3	Value round off
Corrected sludge volume	$V_{sludge,AD}$	**20,000**	m^3	Value round off
Temperature	T_{AD}	37	°C	

The total digestion volume (considering an extra 15% volume as headspace for biogas) of 22,000 m^3 should be divided into smaller tanks to assure proper mixing, at least 2 reactors of 11,000 m^3 each. This provides also more operational flexibility. An egg-shaped reactor is preferred, due to the favourable area/volume ratio, providing fewer heat losses and better mixing. Moreover, this type of reactors has already been applied in China (e.g. WWTP Bailongang). For the design, a diameter of 26 m and a total height of 31 m was considered. The whole calculation can be found in Annex 12.10.1.

Moreover, a 2,000 m^3 tank for the mixing and equalisation of primary, secondary and tertiary sludge is installed previous to the digester. Additionally, a 3,800 m^3 tank for the dosing of centrate is planned (see Annex 12.10.1).

The required area for the anaerobic reactors and peripheral equipment is ca 1,024 m^2. Since there is empty land in the northern surrounding areas of the WWTP, it must be clarified if it could be used for this purpose. A proposal for the location of the anaerobic digestion stage is shown in Figure 39.

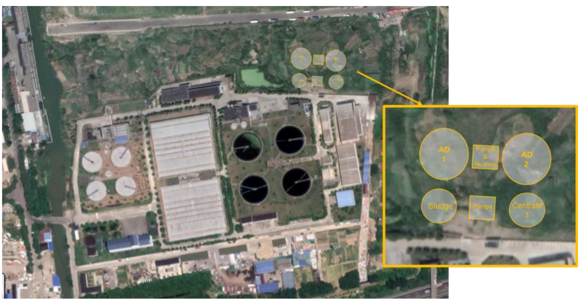


Figure 39. Scheme of the possible location of the anaerobic digestion stage[14]

[14] The dimensions shown in the scheme are for reference only.

The biogas production rate from sewage sludge can vary widely as can be observed in Table 35.

Table 35. Biogas production rate in NL/(PE·d) according to different authors

Location	Value, NL/(PE·d)	Information	Reference
Rhineland-Palatinate (Germany)	24.9	Average of 4 WWTP size class 5 in Rhineland-Palatinate	(Knerr et al. 2017)
Germany	18 - 23	WWTP with activated sludge and anaerobically stabilised sludge	(DWA 2014)
Germany	11	Remaining after sludge is stabilised aerobically	(DWA 2014)
WWTP Maidao (Qingdao, China)	30.4	Literature	(Lin et al. 2018)
	12.1 - 34.4 (27.5)	WWTP with Biofilter, 560,000 PE COD, 120	Operational Data 2020-2021 (CEW)
WWTP Bailongang (Shanghai, China)	4.25	Calculated with $5.47 \cdot 10^6$ $PE_{COD,120}$ and 7.5 m^3 biogas/m^3 sludge	Data from a visit in January 2019
Values in brackets ( ) are average values			

The biogas production will be estimated in the lowest range to have a conservative estimation of the biogas production, as according to the collected data presented in Table 35, the biogas production rate is lower than in other countries Moreover, several authors indicate that sewage sludge in China has a low VS/TS ratio, lower than 60% (Yang et al. 2015), (Liao and Li 2015), (Duan et al. 2016), (Xu et al. 2021). Typical methane concentrations in biogas from sewage sludge are between 60% and 65% (Tchobanoglous op. 2014) and the more conservative value was selected.

For the calculation of the potential Biogas produced, the information obtained in the mass balance and information summarized in Table 36 is used. It is for example assumed that the electrical efficiency of the gas engine is 40% and 45% for the thermal energy, an efficiency reachable by modern gas engines this size (Paschotta 2010).

It is calculated that the thermal energy is enough to supply the required heat to maintain the digester at 37 °C, even with air temperatures of 4 °C, which is the lowest in the winter season in the region. This calculation is detailed in Annex 12.10.2.

Table 36. Design parameters for biogas production in the anaerobic sludge stabilization stage

Parameter		Value	Units	Information
Plant size	P	345,000	$PE_{BOD,60}$	Calculated
Biogas production	Q_{biogas}	5,175	Nm^3/d	Calculated
Methane content	CH_4%	60	%	Assumption
Energy	E_{biogas}	32,603	kWh/d	Calculated
Electrical efficiency	η_{el}	40%		Assumption
Electricity	E_{el}	13,041	kWh_{el}/d	Calculated
Thermal efficiency	η_{th}	45%		Assumption
Heat	E_{th}	11,411	kWh_{th}/d	Calculated
Engine size	P	0,6	MW	Calculated

6.3 Model of the Example WWTP with Anaerobic Digestion

The model with anaerobic digestion is based on the first calibrated model (see Section 4.2). The model includes an anaerobic digestion stage, based on a mixture of the IWA Anaerobic digestion model (ADM) and the model Siegrist 2 (Siegrist et al. 2002b), *admsieg02d*, which is standard for this kind of application in SIMBA, and it is presented in Figure 40. To estimate the digestate and biogas characteristics in the computer model, a single digester is considered, with a total volume of 22,000 m^3 (max. sludge volume 20,000 m^3).

The biogas and sludge production in the model are compared with the theoretical values calculated in Chapter 6.3.1. Afterwards, in Chapter 6.3.2 the nitrogen backload obtained in the model is described and discussed.

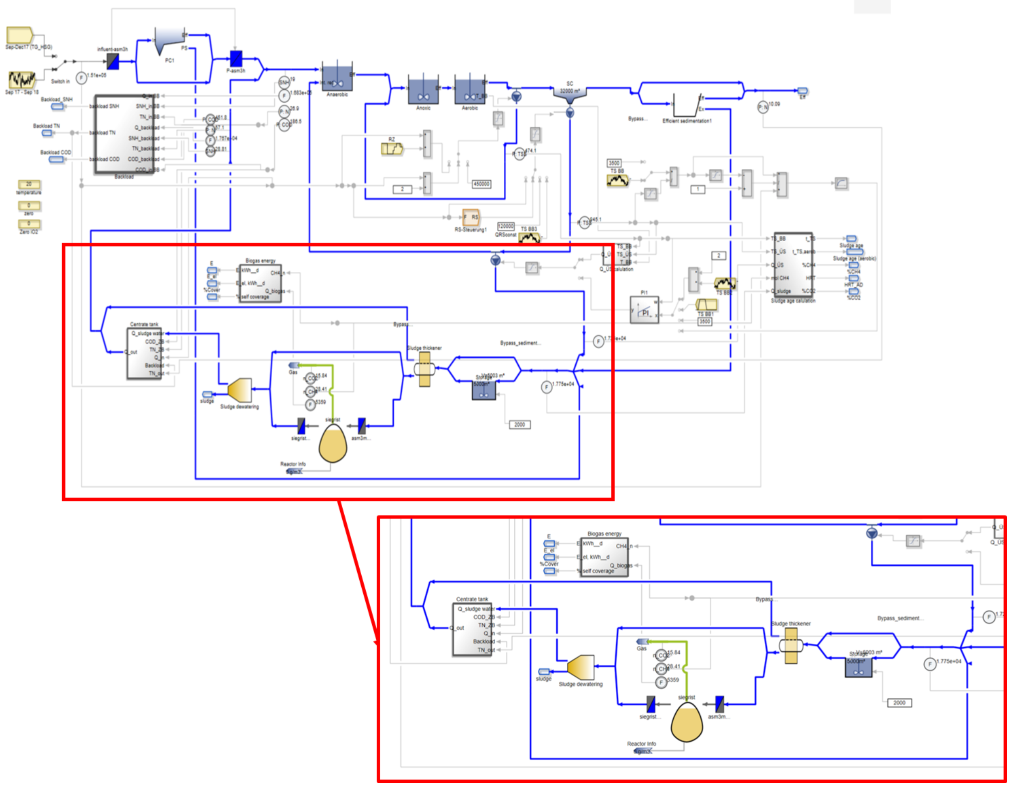


Figure 40. Model of the example WWTP with anaerobic digestion and close-up to the anaerobic digestion stage

6.3.1 Biogas and Sludge Production

The biogas production in the scenario base with anaerobic digestion (from now on called AD-0) is 5,558 m^3/d, equivalent to 16.1 $L_{biogas}/(PE_{BOD}\cdot d)$. This value is in the range of the collected data from several WWTP in Germany and China (see Table 35), but it is in the lower range, as the system is not operating at an ideal sludge age. The estimated electricity production from biogas, together with the calculated daily energy consumption is presented in Figure 41. The energy consumption in the base scenario with anaerobic digestion (AD-0), is based on the power consumption during the calibration period (real data [15]), minus the savings in aeration – due to the sludge age reduction and lower MLSS –, plus the increase in excess sludge pumping.

The biogas, transformed in a CHP with an assumed 40% electrical efficiency (η_{el}), generated energy to cover an average of 39% of the new energy demand as observed in Figure 42. The heat efficiency is assumed as 45%, as described in Chapter 6.2. It was calculated, that even under unfavourable conditions i.e. poor isolation in winter (see Annex 12.10.2), the heat required would suffice to heat two egg-shaped digesters (as dimensioned in Annex 12.10.1).

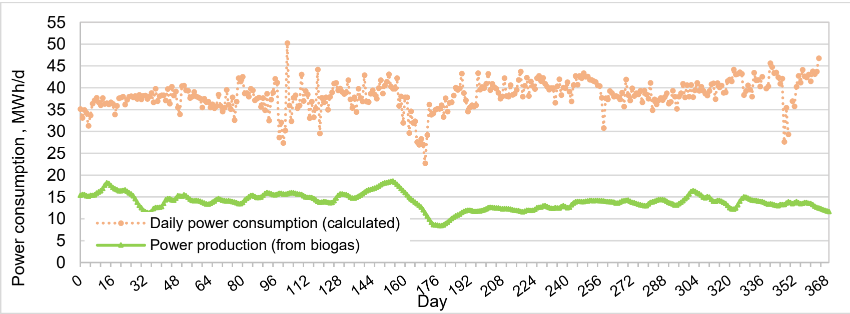


Figure 41. Energy consumption in the example WWTP and production from biogas

An additional benefit of the anaerobic sludge stabilisation is that the sludge to disposal is reduced on average by 21%, from ca. 108 to 85 m^3 per day (19.4 to 15.3 Mg TS/d). At an average cost of 277 CNY per ton of disposed sludge – 21 CNY/Mg for the transport to the closest incineration plant and 256 CNY/Mg for its incineration, according to the plant operator –, the total savings are estimated as ca. 2,3 Million CYN per year (302,776 EUR/a [16]).

The potential energy savings are estimated. According to the energy analysis carried out by (Vergara-Araya et al. 2021), aeration represents 29.9% of the power consumption in the plant, and total recirculation is 3.1%. In the example WWTP the lift pumps require a large portion of the total electricity (17.1%), as well as the odour control system with 13.5% and the advanced treatment with 12.3% of the total. The mixing of the activated sludge systems represents 5.6% of the total electricity consumption.

[15] Informed daily energy consumption by the plant operator in the studied period.

[16] 1 CNY = 0.13 EUR (04/07/2021)

It is assumed, that the excess sludge pumping is 1/3 of the total energy required for recirculation (i.e. 2/3 is used for the recirculation of water), equivalent to 1.03% of the total electricity requirements. The baseline scenario with aerobic digestion (AD-0) saves 10.7% of the air required. At the same time, 133% more excess sludge is pumped. Moreover, as 21 % less sludge must be treated, it is assumed that 21% less energy for sludge thickening and dewatering is required. The total balance gives a total of 0.77% fewer energy requirements, as summarized in Table 37, i.e. a very small change. This change is small due to several reasons:

- The required electricity reductions are only 0.46 GWh/a for aeration and 0.33 GWh/a for sludge dewatering.
- The required electricity increases by 0.04 GWh/d for sludge pumping.
- The AD stage, a new consumer, increases the electricity consumption by 0.62 GWh/d
- The WWTP already has a primary clarification stage, therefore there are no additional savings in aeration in the biological treatment stage due to the pre-removal of COD, nor is the load reduced.
- The electricity for wastewater lifting +odour control + advanced treatment (ca. 40% of the total) remains unchanged.

When the electricity from biogas is increased, the total balance is almost 37% less electricity consumption.

The electricity production from biogas is equivalent to 5.1 GWh in a year, giving a net power consumption of ca. 9 GWh/a, with a total reduction in the energy consumption of 37% in relation to the scenario without anaerobic digestion. This means, a change from 31.5 kWh/(PE·a) in the base scenario to 19.9 KWh/(PE·a) in AD-0. According to Zeng et al. in China, WWTP with more than 100,000 PE, the average energy consumption is about 16 kWh/(PE·a) (Zeng et al. 2017).

This shift would move the example WWTP from 35% to the 6% most efficient WWTP according to the DWA-A 131 (DWA 2015). The electrical and thermal energy balance is presented in Figure 42. A part of these results was presented in (Vergara-Araya et al. 2022).

Table 37. Changes in energy consumption between scenario Base and AD-0 (Vergara-Araya et al. 2022)

Electrical energy demand	**Base** GWh/a	**AD-0** GWh/a	**Change in AD-0 with respect to Base**
Aeration	4.24	3.78	-10.73 %
Excess sludge pumping	0.15	0.19	+133 %
Sludge dewatering	1.52	1.19	-21 %
Digester (pumping, mixing)	-	0.62	+4.37%
Total demand	14.17	14.07	-0.77 %
Total production	-	5.1	-
Total balance	14.17	8.97	-36.77%

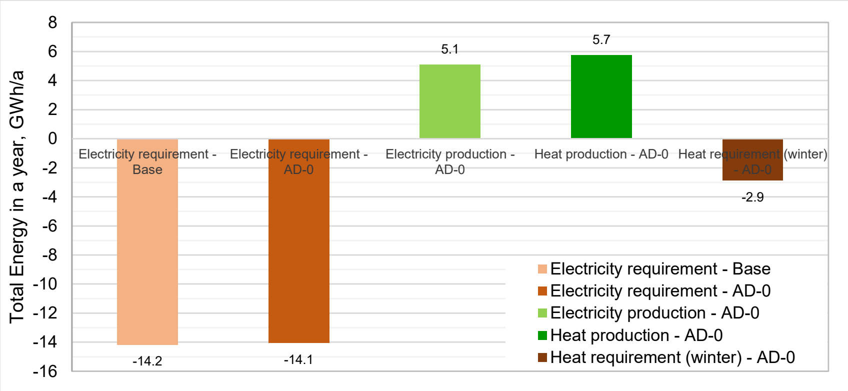


Figure 42. Energy consumption in the calibration period and potential energy (power and heat) production from biogas in the baseline scenario (AD-0) (Vergara-Araya et al. 2022)

6.3.2 Estimation of the Nitrogen Backload

When including an anaerobic stage for sludge treatment, there is a release of ammonia-nitrogen and phosphorous due to the degradation of organic matter under reductive conditions. This can lead to an important backload of nutrients to the biological treatment when the water from dewatering is recirculated, and a consequent increase in the effluent values.

This was demonstrated in the model with anaerobic digestion in SIMBA, obtaining a backload of total nitrogen between 17% and 39% of the influent TN load. The backload proportion values are sometimes higher than the values suggested by other authors, between 10 and 25% (Janus 1996) (DWA 2016). This partially higher percentage variation is due to the high variability of the influent nitrogen load (see Figure 43). The COD backload is 5% on average.

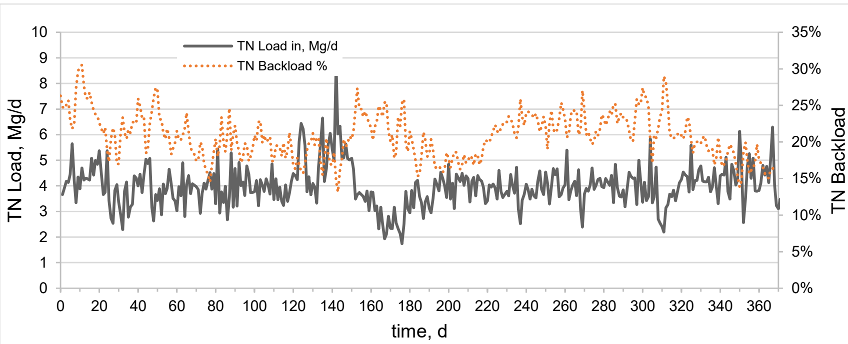


Figure 43. Comparison between TN in the effluent in scenarios Base and AD-0, and TN backload in AD-0

The effluent COD increases well, and in this case, does not lead to an increase in the number of norm non-compliances, but it comes closer to the discharge values much more often. This aspect was not concerning in the previously tested scenarios, but now it has to be controlled closely.

Due to the effect of the backload due to the inclusion of an anaerobic digestion stage, it is necessary to incorporate operational and automation strategies to avoid norm non-compliances. The number of norm non-compliances increased significantly, from 29 to 65 for TN and 2 to 12 for NH_4-N, in the CS norm as can be seen in Figure 44.

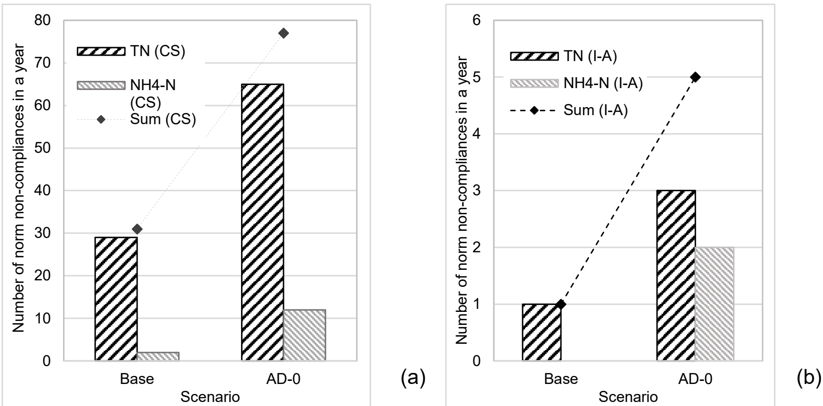
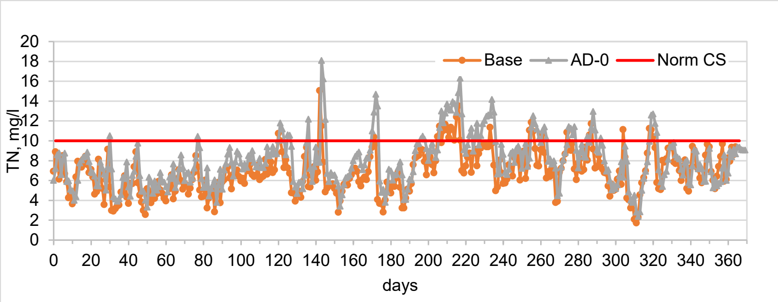


Figure 44. Comparison between the number of non-compliances in a year in the base scenario (without AD) and AD-0 (a) Norm CS; (b) Norm Grade I-A

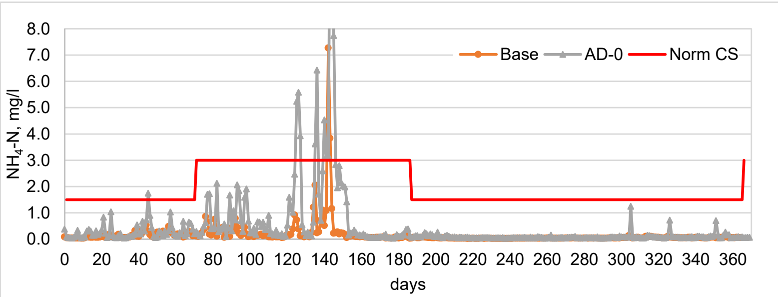
This difference can be observed in Figure 45, which compares the effluent concentrations for NH_4-N and TN in scenarios Base and AD-0. The AD-0 scenario shows higher ammonium nitrogen peaks between days 70 and 160 (see Figure 45 (b)), which are associated with a lower wastewater temperature (winter months) and also a high nutrient backload, between 10 to 20% NH_4-N. There are some periods when the effluent TN in AD-0 is higher than in the Base scenario (see Figure 45 (a)). Some of them are associated with the already named NH_4-N peaks (around day 140), and the rest to increased amounts of nitrate in the effluent.

When comparing the scenario with anaerobic digestion with the scenarios with aerobic sludge stabilisation (discussed in Chapter 5), and as can be seen in Figure 45, it results clear that the nutrients that are re-released into the anaerobic degradation of organic matter in the sewage sludge, are the main cause for the increase in the effluent concentrations and finally, in emissions to the environment.

It will be tested in the model if and which operational and automation strategies can serve to counteract this effect, and to which extent the norm compliance can be improved.



(a)



(b)

Figure 45. Comparison between the effluent concentrations for NH_4-N and TN in scenarios base and AD- 0

6.4 Tests of Operational and ICA Strategies with the AD Model

Based on the model presented and detailed in Chapter 6.3, several operational and automation strategies are tested. Several of them are based on the results already obtained in Chapter 5.

6.4.1 Centrate Dosing Strategy

Centrate is a stream highly concentrated in nitrogen compounds due to the biological degradation of biomass during the anaerobic digestion stage, and it can be dosed strategically, in low load periods to reduce the norm non-compliances. The centrate stream in the example WWTP model varies between 500 and 1,000 m^3/d and has a total nitrogen concentration of around 1,000 mg/L.

The centrate dosing strategy is based on a 3,800 m^3 centrate storage tank, installed after the sludge dewatering stage. It is important to mention that different tank sizes were tested, and this was the smallest tank size that influenced positively the effluent values. The centrate pumping to the biological treatment stage can be carried out based on several operational parameters. Several dosing strategies were tested, among them dosing based on the:

- Influent TN load to the biological tanks
- Backload as a percentage of the influent TN load
- Influent C/N ratio to the biological tanks
- Average TN-effluent concentration

The latter strategy was the most successful in reducing the TN effluent values (see Figure 47). The selected strategy is the dosing of centrate based on the 2-hour average effluent value for TN. Several linear relations were tested, relating the dosing flow of the sludge liquor (centrate) and the TN effluent values. The best results were obtained using the curve in Figure 46.

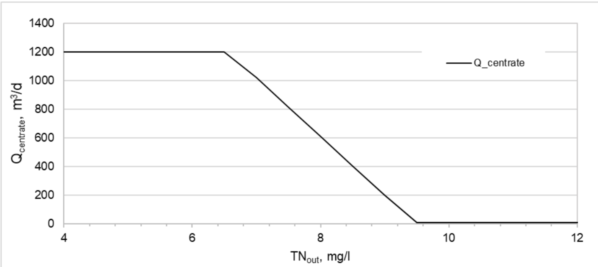
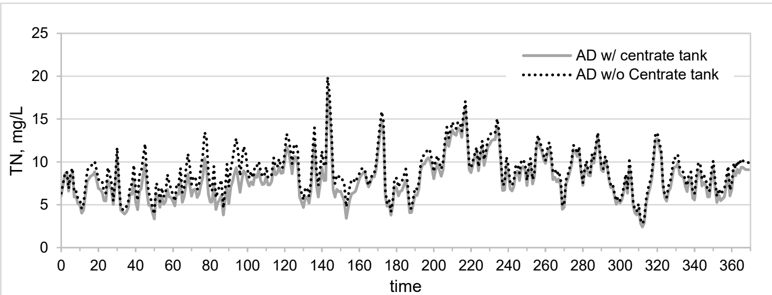
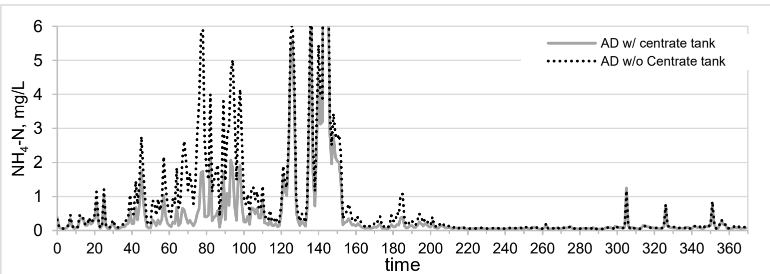


Figure 46. Centrate dosing according to TN concentration effluent values selected for the dosing of centrate (sludge liquor) to the biological treatment



(a)



(b)

Figure 47. Comparison between effluent TN and ammonium nitrogen values without centrate dosing strategy (dashed black) and with the selected dosing strategy (grey)

6.4.2 Conventional Regulation, Operational and Control Strategies

Some of the strategies tested in Chapter 4.5, plus some specific for side-stream treatment (e.g. Anammox process) were tested in the model with anaerobic digestion.

6.4.2.1 Bypass of Primary Clarifiers and Use as Centrate Storage Tank

In this variation, the by-passed primary clarifiers can be used for centrate storage. With the by-pass of 50% of the primary clarifiers volume, based on the decommissioning of 2 out of 4 PC tanks, the centrate storage can be increased by 267%, providing more operational flexibility.

6.4.2.2 Increase of the Denitrification Volume

As described in Section 5.1.3.1, the denitrification volume (V_D) can be increased by reducing the nitrification volume (V_N). A proportion of V_D / V_{AT} = 0.3 is used for the tests, as this is the proportion that previously showed the best results in terms of norm-compliance (see Chapter 5.1.3).

6.4.2.3 NH_4-based Aeration Control

As described in Section 5.2.1, a curve with the variation of the DO set point, based on an NH_4-N curve (see Figure 48) was used.

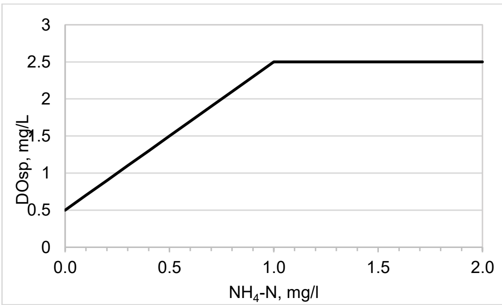


Figure 48. DO set point in aeration tank vs NH4-N in the effluent

6.4.2.4 Anammox-like Conditions

Ammonium-rich side streams can be treated in deammonification processes, as described in the literature research (see Chapter 2.1.1.2). As one of the most popular types of side-stream treatment, the process is used in several full-scale WWTP worldwide (Lackner et al. 2014, DBU 2004). Nitrogen removal with Anammox is highly dependent on temperature, but it will be assumed that this is not a limitation for this scenario. Literature informs nitrogen removal rates between 46% and ca. 94% (depending on the nitrogen loading rate, temperature, DO and carbon content) (Cho et al. 2020). However, the most common rates are between 50% and 80%. The modelling of an anammox stage is out of the scope of this work, however, a simplified version, based on the average nitrogen removal was tested. For this scenario, a constant nitrogen removal (TN and NH_4-N) of 70% (load base) was selected, based on the review from Cho et al (Cho et al. 2020).

Since Deammonification processes are inhibited due to high COD concentrations, Anammox cannot be used to treat pure centrate (2,850 mg COD/l on average), but the more diluted stream of total mixed sludge liquor (i.e. filtrate from pre-thickening (thickening pre-digestion) and centrate) is suitable with a concentration of approx. 500 mg COD/L on average.

6.4.3 Combinations

Based on the best results obtained with the individual strategies, different combinations of strategies were tested, combining the best results and creating synergies. The description of the tested combinations is shown in Table 38. The main results obtained in the simulated scenarios AD-0 to AD-9 are published in (Vergara-Araya et al. 2022).

Table 38. Description of the tested Combinations of strategies with AD

Combination	Description
AD-0	• WWTP with anaerobic digestion and SRT adjustment
AD-1	• WWTP with anaerobic digestion and SRT adjustment • TN-based centrate dosing $V_{centrate\ tank}$ = 3,800 m^3
AD-2	• WWTP with anaerobic digestion and SRT adjustment • TN-based centrate dosing $V_{centrate\ tank}$ = 10,160 m^3 (3,800 m^3/d + 50% V_{PC}) • Bypass of 50% of the Primary Clarifier (PC) volume (V_{PC})
AD-3	• WWTP with anaerobic digestion and SRT adjustment • TN-based centrate dosing $V_{centrate\ tank}$ = 10,160 m^3 (3,800 m^3/d + 50% V_{PC}) • Bypass of 50% of the PC volume (V_{PC}) • V_D/V_{AT} =30% (by reduction of the aerated volume to expand the denitrification volume from 16,000 m^3 to 24,000 m^3)
AD-4	• WWTP with anaerobic digestion and SRT adjustment • TN-based centrate dosing $V_{centrate\ tank}$ = 10,160 m^3 (3,800 m^3/d + 50% V_{PC}) • Bypass of 50% of the PC volume (V_{PC}) • V_D/V_{AT} =30% (by reduction of the aerated volume to expand the denitrification volume from 16,000 m^3 to 24,000 m^3) • NH_4-based aeration with DO vs NH_4-N curve (Figure 48)
AD-5	• WWTP with anaerobic digestion and SRT adjustment • TN-based centrate dosing $V_{centrate\ tank}$ = 10,160 m^3 (3,800 m^3/d + 50% V_{PC}) • Bypass of 50% of the PC volume (V_{PC}) • V_D/V_{AT} =33% (by increase of the denitrification volume by replacing the anaerobic volume with anoxic (and changing the water recirculation point)). • NH_4-based aeration with DO vs NH_4-N curve (Figure 48)
AD-6	• WWTP with anaerobic digestion and SRT adjustment • TN-based centrate dosing $V_{centrate\ tank}$ = 10,160 m^3 (3,800 m^3/d + 50% V_{PC}) • Bypass of 50% of the PC volume (V_{PC}) • V_D/V_{AT} =33% (by increase of the denitrification volume by replacing the anaerobic volume with anoxic (and changing the water recirculation point)). • Intermittent aeration, based on NH_4-N concentration (V_N): If NH_4-N > 1 mg/L, then DO_{sp} =2 mg/L, else, DO_{sp} =0.01 mg/L
AD-7	• WWTP with anaerobic digestion and SRT adjustment • TN-based centrate dosing $V_{centrate\ tank}$ = 10,160 m^3 (3,800 m^3/d + 50% V_{PC}) • Bypass of 50% of the PC volume (V_{PC}) • Anammox-like process, for the treatment of centrate, with 70% NH_4-N removal.
AD-8	• WWTP with anaerobic digestion and SRT adjustment • TN-based centrate dosing $V_{centrate\ tank}$ = 13,340 m^3 (3,800 m^3/d + 75% V_{PC}) • Bypass of 75% of the PC volume (V_{PC}) • NH_4-based aeration with a target NH_4-N of maximum of 0.7 mg/L in the effluent
AD-9	• WWTP with anaerobic digestion and SRT adjustment • TN-based centrate dosing $V_{centrate\ tank}$ = 13,340 m^3 (3,800 m^3/d + 75% V_{PC}) • Bypass of 75% of the PC volume (V_{PC}) • NH_4-based aeration with DO vs NH_4-N curve (Figure 48)

To evaluate the best combinations, four criteria are monitored and compared in Figure 49: Norm compliance: comparison of the number of norm non- compliances in a year; Aeration requirements: in comparison with the base line scenario; Pollutants emissions: amount of TN, NH_4-N and COD emissions (as mass) in a year; Biogas/electricity production per year. The criteria to evaluate the different scenarios are described in Chapter 5.3.1, detailed in Table 27, and will be carried out below, in Table 39.

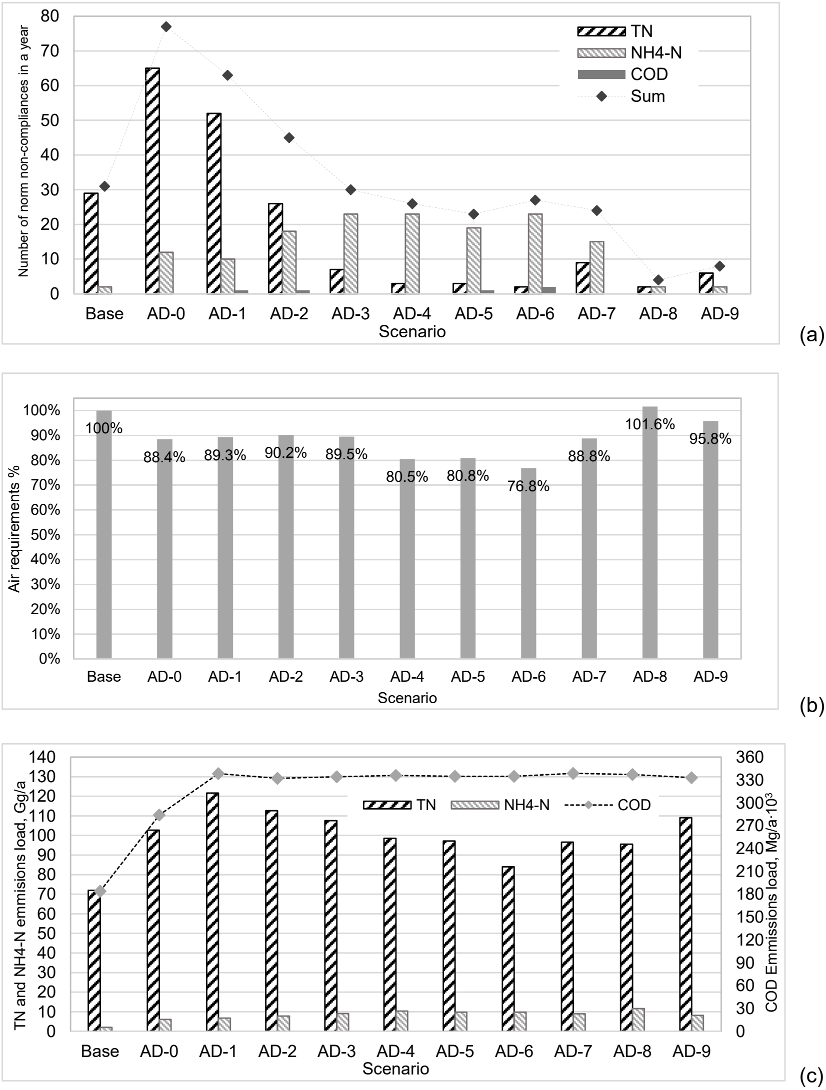


Figure 49. (a) Number of non-compliances of the norm CS in a year; (b) Air requirements; (c) Pollutants load, with scenarios Base and AD-0 to AD-9 (Vergara-Araya et al. 2022)

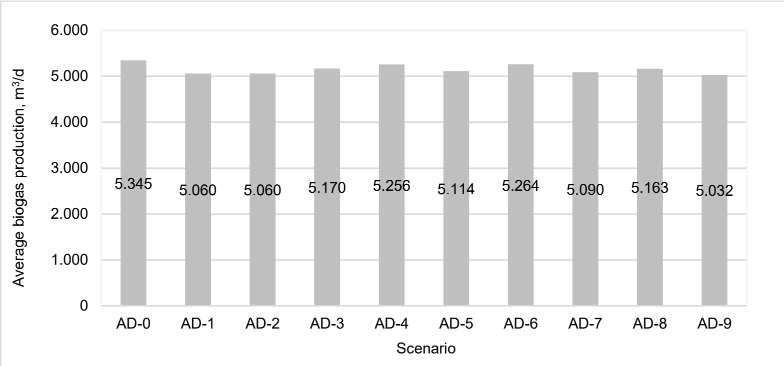


Figure 50. Biogas production, with scenarios AD-0 to AD-9

As described in Table 27 (Chapter 5.3.1), a “positive” or “negative” evaluation, indicates changes in results, in terms of pollutants emissions, air consumption, norms non-compliances, and in this section also biogas production. The combination scenarios will be compared with the base scenario with anaerobic digestion “AD-0” and the results are shown in Table 39.

Table 39. AD Scenarios evaluation comparison

Criteria	Scenario								
	AD-1	**AD-2**	**AD-3**	**AD-4**	**AD-5**	**AD-6**	**AD-7**	**AD-8**	**AD-9**
Sum, CS norm compliance	-3	-3	-3	-3	1	1	2	2	2
TN, CS norm compliance	3	3	3	3	3	3	3	3	3
NH_4-N, CS norm compliance	3	-3	-3	-3	-3	-3	-3	3	3
TN, emissions	1	1	1	-1	-1	-1	-1	-1	1
NH_4-N, emissions	1	2	3	3	3	3	3	3	2
COD, emissions	1	1	1	1	1	1	1	1	1
Air consumption	1	1	1	1	1	2	1	-1	1
Biogas production	1	1	1	1	1	1	1	1	1
Average	**1.0**	**0.3**	**0.4**	**0.1**	**0.7**	**0.9**	**0.9**	**1.4**	**1.9**
Evaluation	**+**	**0**	**0**	**0**	**+**	**+**	**+**	**+**	**++**

According to this evaluation criteria, considering all the evaluated aspects, the scenarios AD- 8 and AD-9 show the best overall results: they increase significantly the norm compliance and reduce at the same time emissions to the environment. AD-9 reduce simultaneously the air requirements.

In the baseline scenario with anaerobic digestion (AD-0), it is possible to observe a reduction in the required air. Due to the reduction of the SRT, there is less biomass in the system and the air requirements are lower to maintain the desired DO set point. This trend is observed in all tested scenarios with anaerobic digestion, except for the last two (AD-8 and AD-9).

The ammonium rich centrate water to the biological treatment causes an important gain in the backload concentrations and a rise in the non-compliances per year and corresponding pollutants emissions. By following the same approach as in previous chapters, combinations of strategies were tested successfully.

Even when the combinations in AD-1 can reduce the number of norm non-compliances, it does not contribute to reduce the pollutants emissions load. It seems as if the nitrogen emissions are better distributed in time, with the effect of a large centrate tank, rather than reduced. Augmenting the centrate tank volume due to the by-pass of 50 % of the primary clarifiers volume (AD-2) leads to a further reduction of the norm non-compliances. However, both scenarios show more norm non- compliances than the Base scenario, before the introduction of anaerobic digestion.

The reduction of the aerated volume to favour denitrification tested in AD-3 shows an important rise in the non-compliances for ammonium and a reduction in the TN non-compliances. Due to the intense backload, more nitrification volume (or biomass) is required.

AD-4 and AD-5 are the scenarios with a higher reduction of the number of norms non-compliances in a year. The ammonium-based aeration control shows, as in previous scenarios, that it is a powerful control strategy to reduce nitrogen emissions and save aerations costs. The number of norm non-compliances is, however, still high, with 23 and 26 per year respectively.

The obtained total number of norm non-compliances in a year is comparable with scenario AD-7, which integrates an Anammox process for the treatment of centrate water. However here, the non-compliances are increased for TN and decreased for Ammonium nitrogen. As the anammox process involves deammonification, more ammonium is transformed into other nitrogen forms (nitrite and nitrogen gas). In AD-7, the total effluent load is reduced, and biogas production is maintained.

AD-6 shows comparable results in terms of norm non- compliances with AD-4 and AD-5, but the TN emissions are significantly reduced, by 17% in comparison with AD-0. The omission of anoxic tanks improves the denitrification capacity, without reducing the nitrification volume and intermittent nitrification contributes to reducing nitrogen emissions. It is noticeable, though, that the scenarios with the lowest air consumption (AD-4 to AD-6), are also the ones with the highest amount of norm non-compliances for ammonium nitrogen. These savings, therefore, come at a non-negotiable cost.

The increase in the bypass of primary clarifiers from two to three, as tested in scenarios AD-8 and AD-9, has a double effect: less carbon is removed before the biological treatment, and the use of the bypassed primary clarifiers as centrate storage provides a significantly larger storage volume. This, together with an ammonium-based aeration control, are the strategies that provide the best results in terms of norm non-compliances per year, with 4 and 8 times, respectively. This is a reduction of 87% and 74% with respect to the Base scenario. The application of an ammonium-based control with a maximum effluent value for NH_4-N in AD-8 is best to reduce the overall norm non-compliances.

These last scenarios lead yet to a very small reduction or even to a small increase in air requirements (+1.6%), in comparison with the larger saving in scenarios AD-4 to A-6. This is still a positive consequence of the shift to anaerobic sludge stabilisation, because (almost) the same amount of air required for the Base scenario is enough to manage the ammonium in the influent and from the backload when the adequate operational and ICA strategies are applied. This represents overall, an improvement.

The biogas production presented in Figure 50 fluctuates between 11.2 and 11.9 L/(PE_{COD} · d)[17] in the different scenarios. This value is low compared with German WWTP, but it is in the range of values observed in China (see Table 35).

The difference in biogas production in the different scenarios changes by less than 6%, due to small differences in excess sludge production due to different operation modes, and primary sludge production. This is an indicator that the optimisation of nitrogen removal does not negatively influence the production of biogas.

The tests show that the introduction of an anaerobic sludge treatment stage can contribute not only to save energy with the production of biogas but also to save sludge disposal costs, without increasing the aeration requirements. Moreover, with adequate nitrogen removal strategies, it is possible to counteract almost completely the negative effects of the backload generated by anaerobic sludge treatment, when considering norm compliance.

Regarding pollutants emission, an increase in the overall emissions is observed in all scenarios with AD, compared with the scenarios without AD. This indicates on one hand, that better norm compliance is not necessarily an indicator of the final total emissions to water bodies. The example WWTP shows in the base scenario frequent peaks in TN and NH_4-N in the effluent, but the average concentrations are lower than in the scenarios AD-0 to AD-9.

As discussed in Chapter 6.3.2, and compared in Figure 45, the nutrients that are re-released during the anaerobic fermentation of organic matter in sewage sludge, are the main cause for the increase in emissions.

On the other hand, it must also be discussed if other environmental parameters, besides the direct pollutants emissions to water bodies, are relevant as well. For example, from a Greenhouse emission (GHG) perspective, aerobic sludge stabilisation can lead to more N_2O emissions (there is more aeration and longer SRT). Instead, in WWTP with anaerobic sludge stabilisation, a fraction of the organic matter will be degraded aerobically, and the rest anaerobically without the emission of nitrous oxide. Meanwhile, in WWTP with anaerobic digestion, the sludge line is also potentially an emitter of CH_4. Energy consumption is also an important source of indirect GHG emissions (Parravicini et al. 2016), which is why the production of electricity and heat from biogas offsets a large proportion of the required energy, reducing the related GHG emissions in the scenarios with AD.

Moreover, the storage of partially aerobically stabilised sludge can contribute to further GHG emissions, if not managed correctly, and its disposal in landfills leads to further pollution of air, water and soil. This can also occur in WWTP with anaerobic digestion if the sludge is not stabilised. However, there is an incentive to degrade most organic matter to produce biogas and obtain energy from it.

[17] 14.6 - 15.5 L/(PE_{BOD} · d)

6.5 Summary of Chapter 6

In order to understand how the incorporation of anaerobic digestion would affect the performance of the example WWTP, an anaerobic digestion stage was designed and incorporated into the full model (as described in Chapter 4). Additionally, the sludge age was adjusted to the required by the wastewater temperature, according to the DWA-A 131 (2016).

Due to an increase in the backload for TN and NH_4-N, the incorporation of anaerobic digestion (as shown in AD-0) increases significantly the number of norm non-compliances in a year.

Nevertheless, the sewage sludge production is reduced by ca. 21%, reducing the thickening and dewatering requirements. Moreover, the electricity production from biogas can cover ca. 37% of the energy demand in the studied period.

To reduce the negative effects of AD, several strategies were tested in the modified model, starting by automating the dosing of sludge liquor (centrate). This change already showed improvements in norm compliance.

After this, similar strategies to the ones tested in Chapter 5, including a scenario with a side-stream sludge liquor treatment (Anammox–like process) were tested successfully. The negative effects observed in AD-0 when compared to the Base scenario, associated with the nitrogen backload i.e. increase in norm non-compliances, increase in emissions, etc., can be counteracted or even improved by using adequate operational and automation strategies.

This can be done without negatively affecting the production of biogas, and without increasing the energy required for aeration in the WWTP.

The results so far show it is possible to improve the operation of the example WWTP with adequate operational and automation changes. However, the question remains as to whether the technology used in the example WWTP is the most appropriate to treat the collected wastewater to the desired level of treatment. In the next chapter, this will be tested, to see if a different technology can perform better from different perspectives.

7 New Biological Treatment–Stage – Sequencing Batch Reactor (SBR)

Given the results from previous Chapters (5 and 6) and the literature research (Chapter 2), the question arises: Is upstream denitrification the best technology to treat the collected wastewater in the example WWTP?

As shown previously, there are many possibilities to remove nitrogen biologically. One of the types of treatment that offers great operational flexibility, and which current studies propose as the best option to treat wastewater (sometimes with very strict discharge norms) is SBR (Hug and Wettstein 2018); (Alagha et al. 2020); (Thys et al. 2022). Moreover, this type of technology is widely used in Switzerland (e.g. WWTP Basel, WWTP Birs, WWTP Uster) (Thys et al. 2022), a country that has strict discharge values for nutrients (please refer to Table 4).

The sequencing batch reactor (SBR) is an alternative to classical activated sludge treatment (CAS), but still makes use of activated sludge for the treatment of wastewater. The SBR process is based on carrying out all steps of the wastewater treatment processes in a single reactor, in a certain sequential, chronological order. This technology was used in 10% of the domestic WWTP in China in 2015 (Zhang et al. 2016).

SBR use less space (Tchobanoglous op. 2014) and offers a much more flexible operation The SBR process allows for several adjustments e.g. total cycle duration, duration of the phases in the process cycle, inflow regime, DO profile during aeration, filling and emptying levels, etc. (IWA 2014). This flexibility can be advantageous in the case of the example WWTP. To accommodate the continuous inflow of wastewater, the SBR system generally comprises either a storage/equalization tank and a single SBR tank or a minimum of two SBR tanks (IWA 2014). In the case of mixed sewage systems, it is also usual to have an equalisation tank.

SBR-based systems can be very complex, especially with an increasing number of reactors, and require an adequate automation and control strategy. SBRs are usually based on two approaches: fixed time-based sequential control (TSC) or real-time control (RTC).

Numerous feedback control strategies can be implemented to adapt the duration of the different phases to the operating conditions, depending on the pollution load and the current situation (Steinmetz and Wiese 2006):

- NH_4 sensors control the duration of the aerated phase.
- NO_3 probes control the duration of the fill phase and denitrification.
- Sludge level and total suspended solids probes control the duration of the settling and draw sludge phases.

These types of control strategies for SBR are still valid today. In this chapter, an SBR treatment stage for treating the wastewater from the example WWTP will be designed and modelled in SIMBA, in order to test its behaviour, and possible optimisation strategies and compare the results with the ones obtained in Chapters 5 and 6.

The base scenario for SBR will be based on the simplest automation strategy (time-based cycle) and aerobic sludge stabilisation, to compare with the example WWTP in the Base scenario. The base-SBR design will start with a relatively long cycle duration, and a shorter cycle will also be tested. After that, an anaerobic sludge stabilisation stage will be included, to see how the systems react to the already-known challenges imposed by the nutrients backload.

Afterwards, the results obtained in the different scenarios will be compared based on the already defined criteria of norms compliance, air requirements, pollutants emissions and biogas production.

7.1 Design of an SBR Stage

To design an SBR biological treatment process for the example WWTP, the guidelines provided by the Leaflet DWA-M 210 (DWA 2009) and DWA-A 131 were used. Following these guidelines, the design is carried out in the steps described in Annex 12.11. Here, to start with the simplest approach, a fixed time-based sequential control (TSC) approach is followed.

The SBR system is designed for alternating, continuous feeding, with the treatment goals: carbon elimination, nitrification and denitrification. For sludge stabilization, aerobic sludge stabilisation is calculated. The addition of an anaerobic digestion stage will be discussed later, in Chapter 7.3.2. The calculation is carried out for the 85%-percentile conditions for the COD load and an 8-hour cycle (t_{cycle} = 8 h).

The base for calculating the biomass requirements is the DWA-A 131 (2016). The DWA-M 210 (2009) is based on the old ATV-DVWK-A 131 (2000). There, the biomass required in the activated sludge reactor is calculated based on the BOD, meanwhile, the new approach (2016), which is used in this work, is based on the COD. The V_D/V_{AT} ideal for the treatment of the example wastewater with aerobic sludge stabilization is 0.423. The sludge age for aerobic sludge stabilization and denitrification, at a design temperature of 12 °C, is 25 days.

Multiple tanks are necessary to have enough operational safety and flexibility and avoid too large reactor size where mixing and sludge extraction problems could arise. To comply with the usually applied sizes and heights, according to the DWA-M 210 (2009) eight reactors (n=8) were chosen. The reactors are fed offset, one hour each (t_F) until the 8 hours cycle is completed.

The process will have an equalization tank upstream of 20,000 m^3 of the biological treatment step. The downstream equalization tank is of the same size and will serve to equalize 4 h at the average flowrate. The equalisation tank is used to have more operational flexibility.

The most important parameters of the reactor design are summarized in Table 41. It is interesting to note that the required biological reactor volume for aerobic sludge stabilisation, is ca. 206,000 m^3 ($V_{R,single}$ = 25,700 m^3), which is slightly larger than the total activated sludge volume (V_{AT} + V_{SC}) of the example WWTP, and also when compared with the plant redesign calculated in Section 3.4 (see Table 40).

As a reference, in the design of an extension for the WWTP Beggen in Luxembourg (please refer to (Thys et al. 2022)), the design considered a smaller SBR volume (115,500 m^3) for a similar amount of COD load (ca 50 Mg/d). It must be remembered that this WWTP stabilizes the sludge anaerobically, which reduced the required volume.

Moreover, the authors found that not enough nitrifying bacteria are present to assure the required effluent ammonium-nitrogen values (<1 mg/L in 2-h composite samples) in a reliable manner. Therefore, a downstream residual nitrification and denitrification stage was included, using the existing infrastructure of a Biofilter (original biological treatment stage designed for 210,000 PE) and the dosing of methanol. This means, the total biological treatment volume is actually larger than the designed SBR volume, and it requires the dosing of external C-sources.

The here-designed SBR requires a much larger amount of biomass than the redesign of the WWTP Beggen ($M_{TS,SBR}$ 1.5 times larger), and consequently a larger volume, to stabilize the sludge aerobically, and also avoid the limitation in nitrification.

The cycle times are defined in Table 42. As observed in this table, two denitrification cycles are required in all scenarios to comply with the discharge norm for nitrogen.

Table 40. Comparison between the example WWTP, re-designed A2/O and SBR biological treatment stages

Parameter	Example WWTP	Redesign A2/O*	Redesign SBR*	Unit
Guideline **Biological treatment**	**Real plant** **A2/O**	**DWA-A 131** **A2/O**	**DWA-M 210** **SBR**	
V_D/V_{AT}	0.17	0.47	0.42	-
V_{AT}	96,000	151,800	-	m^3
V_{SC}	95,430**	49,900	-	m^3
V_{AT+SC} or V_{SBR}	191,430	201,700	206,000	m^3

*Designed to comply with norm CS, without anaerobic digestion
** Estimated with Height to Diameter H_{SC}:D_{SC}=0.33

Table 41. SBR design summary (Scenario SBR-0)

	Parameter	Symbol	Values	Unit
General parameters	Sludge stabilisation	-	Aerobic	-
	Temperature	T	12	°C
	Cycle length	t_Z	8	h
	Sludge age	SRT	25.0	d
	Sludge mass in the SBR	$M_{TS,SBR}$	780	Mg
	Denitrification proportion	V_D/V_{AT}	0.423	-
85%-Percentile conditions	Sludge volumetric index	SVI	124.6	mL/g
	Inlet flow (85%-value)	Q_{in}	6.233	m^3/h
	Number of reactors	n	8	
	Volume after completion of the clear water discharge	V_{min}	19,501	m^3
	Maximum feed volume discharged per cycle	ΔV_{max}	6,233	m^3
	Volume exchange ratio	f_{AA}	0.24	-
Reactor dimensions	Reactor volume	V_R	25,734	m^3
	Reactor volume total	$V_{R,total}$	205,871	m^3
Denitrification and oxygen	Effluent nitrate (z =2)	$S_{NO3,AN}$	4.6	ml/g
	Number of nitrification/denitrification phases during a cycle	z	2	-
	Excess sludge daily flow	$Q_{ES,d}$	270.6	m^3/d
	Total daily mass of excess sludge	$F_{ES,d}$	21,646	kg/d

Table 42. Steps duration in an 8-hour cycle, aerobic sludge stabilisation

	Sludge stabilisation	Symbol	Time
Phase duration in h	Cycle duration	t_{cycle}, h	8
	Duration of the sedimentation phase	t_{Sed}	1.00
	Duration of the clear water removal phase	t_{Ab}	0.45
	Total duration of the filling phase	$t_{,F}$	1.00
	Duration of the 1st filling phase	t_{F1}	0.65
	Duration of the 2nd filling phase	t_{F2}	0.35
	Duration of the idle time	t_{idle}	0.00
	Total duration of the denitrification phase	t_{D}	2.44
	Duration of the 1st denitrification phase	t_{D1}	1.59
	Duration of the 2nd denitrification phase	t_{D2}	0.85
	Total duration of the nitrification phase	t_{N}	3.11
	Duration of the 1st nitrification phase	t_{N1}	2.02
	Duration of the 2nd nitrification phase	t_{N2}	1.09
	Duration of the reaction phase	t_{R}	5.55

7.2 Model of an SBR

The WWTP with the SBR biological treatment was modelled in SIMBA and the general scheme can be found in Figure 51. Only the biological step treatment was modified, and the pre-treatment stages and sludge management stages were maintained.

The plant was modelled with one 2,500 m^3 upstream equalisation tank per reactor (total volume 20,000 m^3) and a single downstream equalisation tank of the same size. The multiple equalisation tanks are not usual in reality, this decision is made in order to simplify the modelling, but in reality, this would be replaced by a single tank.

The first filling phase is designed to reach 22.000 m^3 and the second filling phase achieves the maximum volume of 25.400 m^3. The reaction and filling times are fixed, according to Table 42. The sludge extraction stops when the reactor has reached the target MLSS concentration of 4500 mg/L. This is the reason why the cycles duration in the model is slightly longer. A one-day period can be seen in Figure 52.

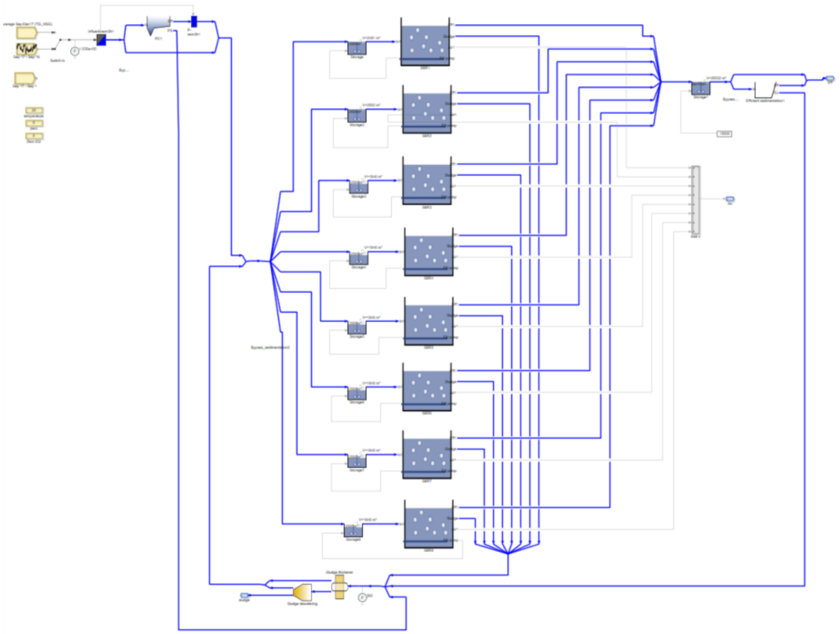


Figure 51. Model of the example plant with SBR as biological treatment

The modelled scenario with aerobic sludge stabilisation is denominated SBR-0, the base scenario for SBR. The norm compliance is 100% for all parameters, but it is dependent on the size of the equalisation tanks. A volume of 20,000 m^3 is the minimum required according to the system's dynamic behaviour because lower dimensions would lead to norm non-compliances.

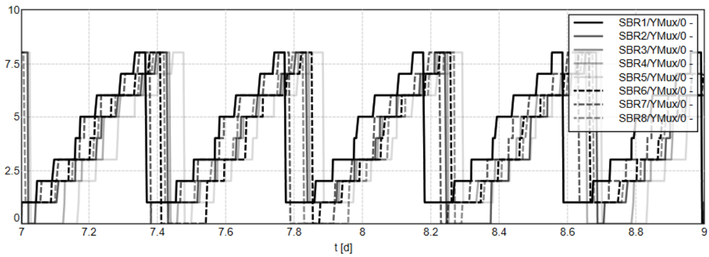


Figure 52. Phases in the SBR-0 cycle. (1) Fill + mix 1; (2) Mix 1; (3) Aerate + mix 1; (4) Fill + mix 2; (5) Mix 2; (6) Aerate+ mix 2; (7) Settle and remove sludge; (8) Decant and remove clear water

The sludge production in SBR-0 is 75.4 m^3/d on average (13.8 Mg TS/d), 30% lower than in the Base scenario. Due to the lower sludge age (25 days), compared with the base scenario in the example WWTP (38 days on average, heavily fluctuating as shown in Figure 38), an increase in the sludge production could be predicted, however, the amount of biomass in the system is almost 26% lower in the SBR system (when comparing the design SBR-0 and Base scenarios), and the target MLSS concentration is lower as well. This newly designed treatment stage operates at the target sludge age based on the wastewater temperature according to the DWA-A 131 (2016) of 25 days.

As could be checked in Chapter 3.4, the existing A2/O stage of the example WWTP is not designed according to the DWA-A 131 requirements, and it is operated with an erratic SRT therefore the volumes and proportions for the SBR-based WWTP are quite different. It is important to consider that the Base scenario represents a smaller activated sludge volume and a system with a too-high, and too-variable sludge age. Moreover, there is more organic matter in the effluent (see the COD emissions comparison in Figure 57).

7.3 Tests to Challenge Nitrogen Removal in the SBR System

As the designed system already fulfils the norm requirements, a shorter cycle and the addition of anaerobic digestion will be tested to study the norm compliance and emissions load.

7.3.1 Shorter SBR Cycle

The plant was modelled for a shorter cycle (t_{cycle}) of 6 hours. A shorter cycle can be advantageous because it increases the biological treatment capacity of the WWTP and makes it more flexible. This can be very interesting in China or other countries with similar framework conditions, with mixed sewage systems, high amounts of extraneous infiltration water, and where growing urban population and growing amounts of wastewater to treat can pose a challenge to the existing infrastructure. A shorter cycle can be applied for example, for rainy weather. This scenario is called SBR-1, and the cycle times are detailed in Table 44. More details about the design can be found in Annex 12.11.

A shortening of the SBR cycle does not influence negatively the norm compliance, which is also 100% for this scenario. The amount of sludge to disposal is almost identical to scenario SBR-0, with 75.8 m^3/d (13.7 Mg TS/d).

7.3.2 Including an Anaerobic Sludge Stabilization Stage

Due to the size plant, an anaerobic sludge stabilization is recommended, as described in previous chapters. However, as discussed in Chapter 6.3.2, when including an anaerobic stage for sludge treatment, the sludge liquor which returns to the biological treatment leads to an important backload of nutrients to the biological treatment.

The anaerobic stabilisation stage has the same dimensions as the one designed in Chapter 6.2 and was included in the model with SBR. This scenario is called SBR-2 AD, and the cycle times are detailed in Table 43 and Table 44. For the sake of comparison, the results from scenario SBR-0 are included again in both tables.

When considering anaerobic sludge stabilization, the denitrification proportion changes to 0.45 and the design sludge age is 12.4 days. The required volumes and biomass in the system are significantly reduced. More details about the design are in Annex 12.11.

In this model, the volume of the upstream and downstream tanks was doubled, to provide more flexibility to handle the nutrient backload. A scheme of the model is presented in Figure 53.

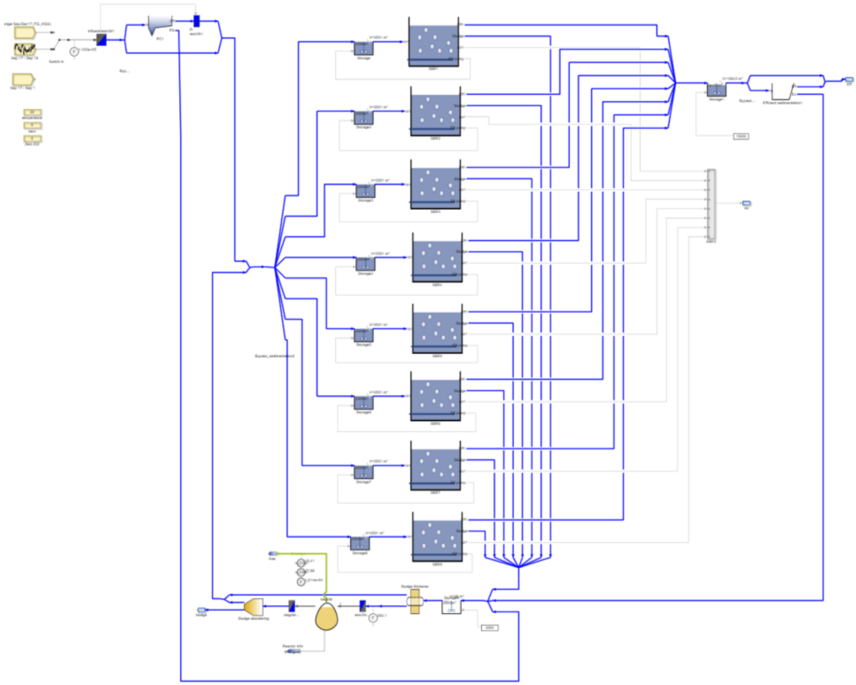


Figure 53. Model of the example plant with SBR as biological treatment and AD (Scenario SBR-2 AD)

The biogas production in SBR-2 AD is significantly higher than in scenario AD-0 (WWTP with A2/O and anaerobic sludge stabilisation) as can be seen in Figure 54. This is probably to the difference in the sludge age, as the SBR system is operated at the ideal STR according to the temperature (DWA-A 131), in comparison with the AD-0 scenario, which uses a slightly higher SRT (see Chapter 6.1), therefore the biogas production is maximized. Accordingly, this result should not be interpreted as an inherent advantage of the SBR technology, but rather as a more efficient distribution of the COD due to the more ideal configuration. Additionally, the SBR-2 AD system has twice more biomass in the system than AD- 0, as the treatment volume of the SBR system is much larger.

Due to the more favourable configuration, the larger treatment volume and the larger amount of biomass in the system in SBR-2 AD, the COD removal is increased. This can be observed in the lower COD emissions (see Figure 57). At the same time, this COD is transformed in the anaerobic phase into biogas.

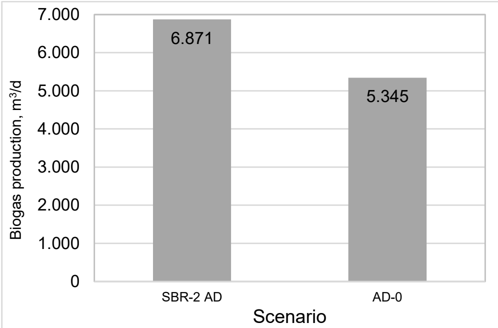


Figure 54. Comparison of the biogas production in scenarios SBR-2 AD and AD-0

The amount of sludge to disposal is 48% smaller than in the Base scenario and 25.6% lower than in scenario SBR-0, with 56.2 m^3/d (10.1 Mg TS/d) (see Figure 55). The effect of a smaller sludge age, more efficient COD removal and the expected reduction after the anaerobic sludge stabilisation stage can be clearly seen.

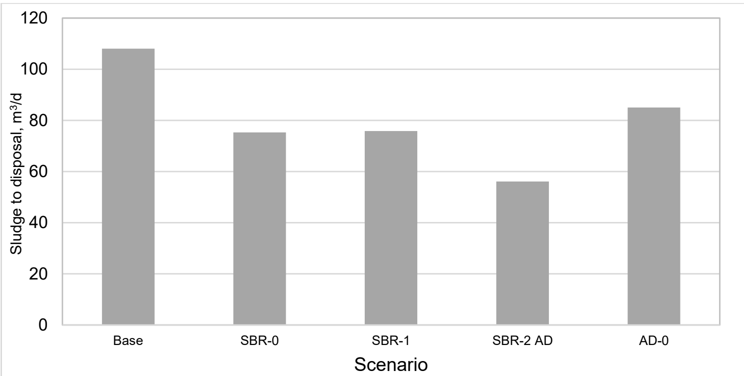


Figure 55. Comparison of the sludge to disposal in the SBR scenarios, Base and AD-0

Table 43. Design summary of the scenarios SBR-0, SBR-1 and SBR-2 AD

	Parameter	Symbol	Values SBR-0	SBR-1	SBR-2 AD	Unit
General parameters	Sludge stabilisation	-	Aerobic	Aerobic	Anaerobic	-
	Temperature	T	12	12	12	°C
	Cycle length	t_Z	8	6	8	h
	Sludge age	SRT	25.0	25.0	12.4	d
	Sludge mass in the SBR	$M_{TS,SBR}$	780	838.6	437.0	t
	Denitrification proportion	V_D/V_{AT}	0.423	0.44	0.45	-
85%-Percentile conditions	Inlet flow (85%-value)	Q_{in}	6.233	6,233	6,233	m³/h
	Number of reactors	N	8	8	8	
	Volume after completion of the clear water discharge	V_{min}	19,501	20.965	10.924	m³
	Maximum feed volume discharged per cycle	ΔV_{max}	6,233	4.675	6.233	m³
	Volume exchange ratio	f_{AA}	0.24	0.18	0.36	-
Reactor dimensions	Reactor volume	$\mathbf{V_R}$	25,734	25,640	17,157	m³
	Reactor volume total	$\mathbf{V_{R,total}}$	205,871	205,119	137,257	m³
Nitrification / Denitrification	Effluent nitrate (z =2)	$S_{NO3,AN}$	4.6	3.4	6.5	ml/g
	Number of nitrification/denitrification phases during a cycle	Z	2	2	2	-
	Excess sludge daily flow	$Q_{ES,d}$	270.6	265.6	306.2	m³/d
	Total daily mass of excess sludge	$F_{ES,d}$	21,646	21,245	24,496	kg/d
Sludge	Sludge flow to disposal	$F_{Sludge,disp}$	75.26	75.82	56.1	m³/d

Table 44. SBR steps duration in SBR-1 and SBR-2

	Sludge stabilisation / Scenario	Symbol	Aerobic SBR-0	Aerobic SBR-1	Anaerobic SBR-2 AD
Phase duration in h	Cycle duration	t_{cycle}, h	8	6	8
	Total duration of the filling phase	t_F	1.00	0.75	1.00
	Duration of the 1st filling phase	t_{F1}	0.65	0.45	0.60
	Duration of the 2nd filling phase	t_{F2}	0.35	0.30	0.40
	Duration of the idle time	t_{idle}	0.00	0.00	0.00
	Total duration of the denitrification phase	t_D	2.44	1.67	2.53
	Duration of the 1st denitrification phase	t_{D1}	1.59	1.09	1.20
	Duration of the 2nd denitrification phase	t_{D2}	0.85	0.58	1.33
	Total duration of the nitrification phase	t_N	3.11	2.13	3.02
	Duration of the 1st nitrification phase	t_{N1}	2.02	1.38	1.50
	Duration of the 2nd nitrification phase	t_{N2}	1.09	0.74	1.52
	Duration of the reaction phase	t_R	5.55	3.80	5.55
	Duration of the sedimentation phase	t_{Sed}	1.00	1.00	1.00
	Duration of the clear water removal phase	t_{Ab}	0.45	0.45	0.45

7.4 Comparison Between the Modelled Scenarios

The described scenarios are compared in terms of air requirements and effluent load. The compared scenarios are described in Table 45.

Table 45. SBR scenarios description

Scenario	Description
SBR-0	• t_{cycle} = 8 h • Time-based control of the different phases, • Phases duration according to the design according to DWA-M 210 • Aerobic sludge stabilisation
SBR-1	• t_{cycle} = 6 h • Time-based control of the different phases, • Phases duration according to the design according to DWA-M 210 • Aerobic sludge stabilisation
SBR-2 AD	• t_{cycle} = 8 h • Time-based control of the different phases, • Phases duration according to the design according to DWA-M 210 • Anaerobic sludge stabilisation

7.4.1 Air Comparison

The air requirement in each scenario is compared using as a base the “Base” scenario, with 100% as seen in Figure 56. The amount of air required in SBR-0 is 45.8% lower than in the Base scenario with A2/O on the example WWTP for several reasons. First, the sludge age in the SBR system is fixed at 25 days, much lower than the 38 days on average of the Base scenario (and less fluctuating as well).

Second, the designed denitrification proportion for the SBR is much larger (42.3% vs 16.7%) and therefore the nitrification requirements are lower (i.e. the aerated phases are shorter). Additionally, as more nitrate is degraded, a larger proportion of the COD is oxidized during denitrification, and therefore the air requirement for COD oxidation is lower. Moreover, the concentration of biomass in the tanks is maintained at ca. 4.5 g/L, lower than on the Base scenario with values above 6 g/L in winter.

When the scenario SBR-0 is compared with a scenario with a lower sludge age, such as T27 (please refer to Chapter 5.2.4), the required air is 89.6%, very similar to AD-0.

The amount of air required is very similar in the three tested scenarios for SBR, showing that here the role of the sludge age is less significant than in the A2/O system. Here instead, the aerated proportion (which is reflected as aerated time in the cycle) is much lower, changing from V_D/V_{AT} =0.17 in the Base scenario to V_D/V_{AT} =0.42 in SBR-0.

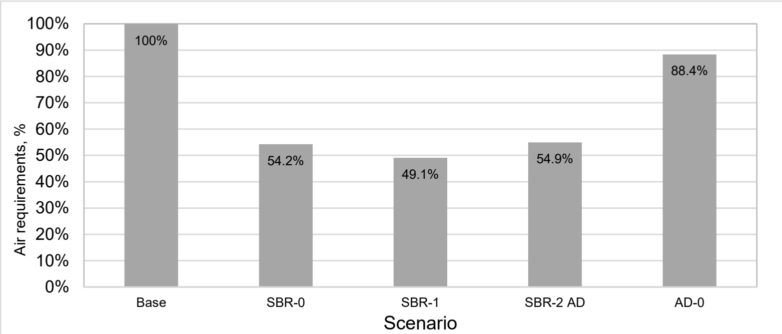


Figure 56. Air requirements comparison for the scenarios SBR-0, SBR-1, SBR-2, Base and AD-0

7.4.2 Pollutants Emission Load Comparison

The pollutants emissions load in SBR-0 is lower for TN but actually higher for COD and NH_4- N (see Figure 57), giving an overall emissions load 9.7% higher in comparison with the base scenario.

Therefore, it can be concluded that due to the tailored design of the tanks, the larger reactor volume plays a significant role to distribute the effluent pollutants, avoiding norm non-compliances. Moreover, as observed in the A2/O system, an increase in nitrate removal comes with a slight worsening in the ammonium concentration in the effluent.

The shorter cycle (t_{cycle} = 6 h) in SBR-1 is beneficial from the emissions perspective, as all the pollutants emissions in a year are reduced in comparison to SBR-0. The COD and ammonium, which depend on the aerated phase, are slightly higher than in the Base scenario, meanwhile, the TN removal is improved due to an improvement in denitrification. The shorter cycle seems to be more appropriate for the designed conditions.

The pollutants emission for TN and NH_4-N in SBR-2 AD is similar to SBR-0 because the system can cope better with the backload of nutrients. The COD emissions, however, are higher with anaerobic digestion. Possibly, the COD present in the backload is already mineralized and therefore goes through biological treatment.

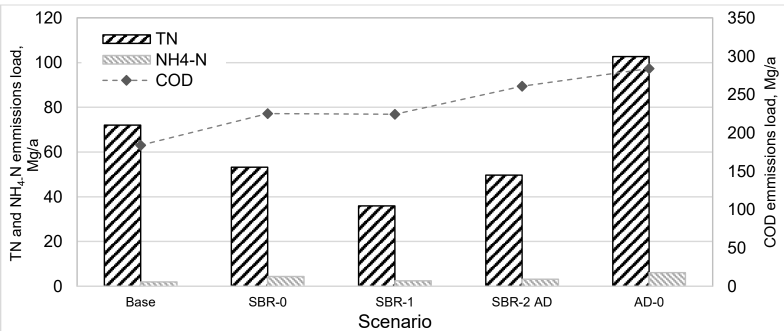


Figure 57. Pollutants emission comparison for the scenarios SBR-0, SBR-1, SBR-2 AD, Base and AD-0

7.5 Evaluation of the Scenarios with SBR

The modelled scenarios are compared based on the criteria defined in Chapter 5.3.1. The norm compliance, the pollutants emissions and the air requirements are compared in Table 46.

Table 46. SBR-based Scenarios evaluation comparison

Criteria	Scenario		
	SBR-0	**SBR-1**	**SBR 2AD**
Sum, CS norm compliance	3	3	3
TN, CS norm compliance	3	3	3
NH_4-N, CS norm compliance	3	3	3
TN, emissions	2	3	2
NH_4-N, emissions	-3	-2	-3
COD, emissions	-2	-2	-3
Air consumption	3	3	3
Average	**1.3**	**1.6**	**1.1**
Evaluation	**+**	**++**	**+**

The overall evaluation of the SBR scenarios shows that the SBR-1 is the best scenario, which uses a shorter treatment cycle. This scenario shows the most improvements in TN emissions. The three scenarios improve the norm compliance; therefore, the valuation shows no difference there. In all three scenarios, the ammonium emissions are worsened, but the norm is fulfilled, a trend that was previously observed in the scenarios tested in Chapters 5 and 6.

The here presented SBR systems have a very basic, time-based cycle operation strategy. The application of modern ICA technologies, e.g. ammonium-based aeration phase duration, nitrate-based denitrification phases, sludge level-based sedimentation phases, etc., the results could be further improved. This was demonstrated in the study of the WWTP Beggen, with the inclusion of a flexible nitrogen elimination strategy, based on online measurements of NH_4-N and NO_3-N (Thys et al. 2022).

7.6 Summary of Chapter 7

The SBR technology is selected as an alternative to A2/O to test the potential of a different configuration for the biological treatment for nitrogen removal in the example WWTP.

An SBR stage is designed for the example WWTP, based on the guidelines provided by the DWA-M 2010 (2009) with a very simple, time-based automation approach and an 8-hour cycle. The designed biological treatment consists of eight parallel reactors with upstream and downstream equalization tanks.

To test the technology, the A2/O stage was replaced by the designed SBR stage in the model in SIMBA (i.e. the influent wastewater, pre-treatment and sludge treatment processes remained unchanged) and the potential of this technology was evaluated under the same criteria as previously: norm compliance, air requirements, pollutants load.

The designed SBR stage achieved full norm compliance for the norm CS and produced less sludge, however, increased the emissions for ammonium nitrogen and COD. The designed SBR has a much larger treatment volume and a tailored denitrification proportion, which is why this scenario shows full norm compliance for the norm CS as well as a reduction of the required air.

Afterwards, in order to challenge the nitrogen removal, two further tests were conducted: the reduction of the cycle time to 6 hours (SBR-1), and the inclusion of an anaerobic sludge stabilization stage (SBR-2 AD), with the corresponding sludge age modification. Both scenarios show also full norm compliance, coping very well with the discharge norm. The adjustment of the denitrification proportion and the larger reaction volume, continue to be the defining factors. In the case of the anaerobic sludge stabilisation, due to the different sludge age used, the biogas production in the SBR system was also higher.

In order to draw more general conclusions about the results presented between Chapters 3 and 7, the obtained results must be discussed in depth, which is carried out in the following chapter.

8 Discussion

In this section, several questions and topics that have been addressed during the development of this thesis will be discussed in depth in order to broaden the perspectives and scope of this work. This discussion aims to interpret and explain the obtained results, to provide, a better understanding of them and a basis for the potential of extrapolation.

Therefore, from the results and this discussion, general recommendations for the optimisation and design of WWTP similar to the example WWTP will be addressed in Chapter 9.

The discussion is centred on four main topics:

- Technologies for biological nitrogen removal: it is necessary to address the technology's limitations and even question the technology applied in the example WWTP.
- Example WWTP and wastewater treatment in China: the operational data analysis and redesign of the WWTP, showed some shortcomings and characteristics that must be discussed for a better understanding.
- Dynamic models and simulation: what are the advantages but also the limitations of the application of dynamic modelling and simulation?
- Results: discussion of the obtained results in a broader context.

8.1 Technologies for Biological Nitrogen Removal

There are several strategies for biological nitrogen removal using nitrification/denitrification: upstream, alternating, simultaneous or intermittent denitrification, SBR cycle, etc., as presented in the literature review. When planning a WWTP, it must be carefully evaluated which of these is most beneficial for nitrogen removal under different operational conditions. So far, upstream denitrification has established itself as one of the most popular technologies for municipal wastewater treatment. However, this configuration does not offer as much operational flexibility as others (e.g. cascade denitrification, intermittent denitrification, SBR, etc.). In planning new plants or retrofitting existing plants in China and other countries, it must be critically evaluated whether, given the wastewater conditions in the WWTP influent, more flexible technologies - as this study has shown (e.g. intermittent nitrification, SBR)-are more beneficial.

Several authors (e.g., (McCarty 2018), (Winkler and Straka 2019)) criticise nitrification/denitrification technologies for their high energy requirement due to intensive aeration and internal recirculation (especially in the case of upstream denitrification) and also because of possible emissions of nitrous oxide (N_2O), a gas with high global warming potential. Additionally, nitrification/denitrification requires sometimes the use of external carbon sources, which are an additional cost and environmental burden. These studies suggest as better alternatives the use of innovative processes, such as the use of Nitritation/Denitritation, Anammox, and nitritation. The application is suggested for secondary flows, but there are several current studies to apply the technology also for the main flow, especially for wastewater with an unfavourable C/N ratio.

As these technologies begin to establish in large-scale WWTP, it must be critically assessed whether they offer a better alternative to achieve the treatment targets in different countries and regions. This assessment should consider not only nitrogen removal potential and possible operational cost savings, but also start-up times and other possible operational complications. The start-up of such systems with pure cultures or highly specialised bacteria can be highly complex. As technology becomes more established, these problems will have better and faster solutions. For the time being, the development of such technologies must be closely monitored. Leaders in the field are to be found for example in the Netherlands (e.g. Paques®, etc.).

In the literature review (please refer to Chapter 2.2.1), it was noted that several countries still limit their wastewater treatment to the removal of organic matter. Retrofitting or expansion of WWTP which currently only oxidises organic matter to biological nitrogen removal is a challenge that many countries are likely to face in the future. What is clear, is that whatever alternative they choose, be it nitrification/denitrification or others, an upgrade of existing technology, including the incorporation of sensors and digital monitoring, including smart automation strategies, must come into play if water quality and respective ecosystems are to be protected. The same is true for countries or regions that already have increased their wastewater discharge standards, which poses challenges to existing WWTP.

It is also important to highlight that the simple one-on-one transfer of technologies from developed to developing countries, would not be adequate in this case, (and probably in most cases). The example WWTP showed differences in the characteristics of the influent wastewater, the discharge norms, the local policies (e.g. how the wastewater sector is financed, available budget), and cultural differences (e.g. the separate conception of wastewater and sewage sludge treatment). Moreover, China has only over a decade of experience in the wastewater sector, which difficult access to specialized local knowledge or professionals. Additionally, as discussed with the plant operator, access to personnel with experience in ICA technologies and to the technologies is not a given.

8.2 Example WWTP and Wastewater Treatment in China

According to the operational data analysis, the example WWTP shows several design problems, e.g. a too-small denitrification proportion, partially too-high sludge recirculation, potential hydraulic overload, insufficient DO online measurements, lack of online sensors and control strategies, too high and highly variable sludge age, overdosing of precipitants for phosphorus removal, etc. Moreover, the design of the sewage system may be the cause of some of the problems observed in the WWTP (i.e. low C/N ratio, increase in non-biodegradable COD in the rainy season).

These problems seem to be widespread in the wastewater industry in the country, in accordance with the information recently provided by the study of several WWTP in China. Zhang *et al.*, (Zhang et al. 2021) listed several problems such as a mismatch between the designed WWTP and the actual wastewater quality, insufficient facilities and problems in the design, low efficient facilities, insufficient equipment, etc. The relatively short experience of the country in this matter reflects in the current situation of the sector, and the sometimes, unsustainable or counterproductive coping strategies.

It results clear that the increasing environmental challenges and the accelerated development of the wastewater sector have put a lot of pressure on existing infrastructures, WWTP operators and planers. However, the increasing of norms by itself is not a sustainable measure to improve the WWTP performance and the improvement of natural water bodies.

The accelerated phase in which new norms are enforced, is an argument in favour of the use of dynamic simulation because solutions can be found (and also implemented) faster, not necessarily requiring the construction of new infrastructure. In this manner, WWTP operators can test the effectiveness of different strategies (e.g. operational, ICA), before investing in new equipment and personnel.

WWTP operators are adapting to the changing conditions and must manage with the, sometimes, deficient designs. Moreover, the fate of the sewage sludge, i.e. sewage sludge treatment and disposal, should not be disregarded or seen as a separate process, but rather be seen as an essential part of the treatment of wastewater.

On one hand, the wastewater sector in the region requires more funding for investment, operational costs and qualified personnel. On the other hand, a perspective change, including better control of the plant (e.g. measurement of the extracted sludge, control of the sludge age, incorporation of more online sensors, etc.), optimization of existing facilities (instead of building new processes downstream), the inclusion of sewage sludge management, the incorporation of energy efficiency measures, among others, is required. Additionally, more expert knowledge is required to improve the performance of WWTP sustainably.

The experience from other countries (e.g. Germany) in this matter can be very valuable. This experience is, however, not always accessible internationally, nor condensed understandably, and it requires the experience and specific knowledge to be properly interpreted and applied. Thus, this work provides a detailed and critical perspective for WWTP similar to the example WWTP, particularly in developing countries, especially China, where many of the here-tested strategies and offered perspective is not widespread. It is also an invitation to critically rethink the processes and existing biases and make data-based decisions.

8.3 Dynamic Models and Simulation

By the model calibration, which has a mostly good and sometimes medium fit for the studied parameters, the statements and the results of the tested strategies are to be understood more as a relative comparison and less as absolutely precise results.

In the case of the example WWTP, the data available was limited. There are still some questions about the operation and energy consumption of some processes e.g. automation of different processes, manual adjustments, details of the sludge line, recirculation, etc. The required process of visiting the plant and setting up a measurement campaign for calibration purposes was not possible due to the limitations imposed by the COVID-19 pandemic (the travels to China were limited in January 2020).

Simulation has been used in the past for various purposes, such as planning and also for optimisation of existing WWTPs. The latter was the main approach tested in this work, but a look at the limitations of static WWTP design was also given (see Chapter 3.4), which shows that there is potential in applying simulation for planning in this case as well.

The quality of the models and simulation results depends heavily on the quality and availability of real data. It is clear that the more information about the plant is available, the better the model fit achieved, but the experience of the modeller plays also an important role. Moreover, the detail and required model fit depend on the objectives of the work. For example, with very little data, it is possible to draw general conclusions about a WWTP performance, as there are ways to emulate the dynamic behaviour of the influent of a WWTP (e.g. Case C in SIMBA). However, if the aim is to design sophisticated control and automation systems, a very detailed description and model of the automation system are required.

The current thesis is somewhere in the middle, as lots of data for the influent and effluent of the WWTP was available, which allowed for a calibration process, but many steps in the middle (e.g. influent to the activated sludge system, sludge production, recirculation rates, etc.) are not measured. Moreover, the existing automation strategies were not accessible in detail, but only through the explanations of the plant operator.

In this case, the objective was to show how simple operational and automation strategies could be used to improve plant performance, especially from the perspective of nitrogen removal, but also considering energy requirements, total emission discharges and the amount of sewage sludge to be disposed of. The study aims to suggest plausible strategies to improve the performance of the plant, but not to predict its behaviour exactly. This would require more detailed knowledge of the various automation systems, operation (including all manual adjustments) and on-site measurements.

In this study, compliance with the standard was the primary criterion, as the main objective of a WWTP is to clean the wastewater to an acceptable quality, but different criteria, such as energy consumption/production, automation requirements, or other criteria could also be prioritised.

Phosphorous elimination was not considered in the framework of this work as it was out of its scope. The removal of anaerobic tanks as tested in some scenarios in the activated sludge (please refer to Chapters 5.1 and 6.4.2.2) will have a direct effect on the biological removal of Phosphorous. Moreover, the addition of the anaerobic digestion stage also will increase the P-backload, challenging the biological and chemical removal processes. These aspects must also be considered when deciding the best strategies to optimize the WWTP.

Dynamic simulation with SIMBA has its limitations, aspects such as sludge dewaterability, and hydraulic behaviour of the different tanks and processes. These aspects cannot be disregarded when operating a WWTP and should be evaluated with other tools.

8.4 Results

8.4.1 Biological Treatment (A2/O and SBR)

The study shows that under the studied conditions, more flexible systems deliver better results, as the influent water quality is not always favourable for upstream denitrification. However, the improvement in nitrate emissions comes with an increment in ammonia emissions. This balance must be outweighed to comply with the discharge norms. Additionally, based on the results, it results clear that other treatment strategies, besides upstream denitrification can be more adequate for the construction of future WWTP with similar conditions in China (as described in Chapter 3.1), but also worldwide.

It must be discussed, however, that according to literature, intermittent aeration is not adequate for Plug Flow Reactor (PFR)-like reactors as the tanks would show a shifting DO profile; therefore, the hydraulic behaviour of the plant must be studied before deciding in this regard. In addition, this kind of aeration is only possible with aeration elements with membrane, usually large-size plate aerators. Therefore, the investment in new aeration elements must be carefully considered, ideally when the old aeration elements are up for maintenance or replacement. The automation required to take this into account must not be disregarded either, as more sensors and control strategies are necessary. There would be also an increase in the personnel required for these purposes.

The analysis carried out in Chapter 5.4, showed that the optimization strategies have to be tailored to the discharge norm. The discharge concentrations for TN among the selected discharge norms vary between 15 and 10 mg /L, which makes a huge difference in terms of norm compliance. Meanwhile, the target discharge concentration for ammonium-nitrogen varies widely between regions and countries, between 13 and 1 mg/L. This is truly a game-changer regarding the operational requirements in a WWTP, being 1 mg/L extremely challenging to reach, as a virtual full nitrification must be reached (increasing the stakes of sufficient aeration and the capacity to flatten eventual concentration peaks).

Not only do the discharge concentrations play a role, but also the sampling strategy, whether it is a composite sample or if it is a grab sample, if a 2-h or a 24-h average is used. Here, the changes in norm compliance can be large, from virtually full norm compliance (e.g. with the EU Norm, or the Grade I-A) to 14 non-conformities per year in C3 or 46 in C5 (under the Norm from Luxembourg) (see Figure 37). This is an interesting result, considering that the same discharged wastewater quality is evaluated.

The evaluation of the discharged wastewater with the Luxembourgian norm would make most of the optimization strategies here tested (please refer to Chapter 5) insufficient, making it necessary to evaluate more complex automation strategies, the installations of a post-treatment (e.g. post-denitrification) or directly changing the type of treatment technology.

During the evaluation, carried out in Chapter 5.4, the EU norm is relatively lax, showing the least amount of norm non-compliances for the evaluated scenarios. However, the planned update to the EU Water directive (European Commission 2021) in the next years, could change this, incrementing the relevance of this work and the proposed approach and strategies.

SBR

The implementation of the SBR technology, although theoretically possible and beneficial for the overall plant performance, must be studied carefully. The use of SBR for the treatment of the wastewater present in the example WWTP is ideal from the norm compliance, air requirements and pollutants emissions perspective.

However, the automation effort must not be underestimated, even when the cycles are time-based. This can be challenging from an investment and operational costs perspective, including the personnel for programming calibrating and maintaining sensors. The coordination of several batch reactors running in parallel is far from trivial, even if it is in a time-based cycle. As the implementation of sensor-based cycles (i.e. ammonium nitrate, DO, sludge level, etc.) is the option that makes the most of the SBR technology, as it benefits from its flexibility and dynamic capacities, the automation effort is even larger.

In further studies with SBR, strategies to shorten the cycle should be tested. It would be interesting to taste an ammonium-nitrogen-based aeration strategy so that the aeration time is stopped when the ammonium is oxidized. The inclusion of a nitrate-nitrogen measurement is also key to deciding how long the denitrification cycle should be. Sensors to detect when the sludge has settled is also good practice to reduce the cycle times and avoid the release of solids to the effluent.

With the C/N ratio as the focus, this value should be measured online in the influent, to decide, for example, how many denitrification /nitrification steps are required, or if a post-denitrification step, with the dosing of raw wastewater, is required.

The use of external C-sources was not tested. This was at first a conscious decision on the philosophy of optimisation. The author considers that this should be the last resort to try to improve denitrification and not the primary improvement strategy. Moreover, the results obtained in Chapter 5, showed that this strategy is not absolutely necessary, as several tested strategies, mostly oriented to improve denitrification were very effective. More problems were observed in the removal of ammonium nitrogen (i.e. a compromise between ammonium and nitrate removal) due to the very strict discharge norms for NH_4-N). These problems can be solved with other strategies (e.g. improvements in the physical aeration system and mixing of the aeration tanks) that are out of the scope of the current study.

Similarly, the use of post-treatments, typically used in some countries such as China, was not considered, for the same reason: many improvements can be made to existing tanks before evaluating the use of external chemical agents and additional treatments. This is the strategy currently pursued by the plant operator, to install a post-denitrification filter downstream of the biological treatment.

The current study shows that this is not absolutely necessary to maintain good compliance with the standard. It should be studied whether the minimum of 5 to 6 non-compliances per year obtained with some of the operational strategies with the A2/O technology represent a fine for the plant operator, or whether they fall within the acceptable range for the monitoring perspective for plants of that size and in that region.

The parameter alkalinity in the wastewater treatment process should be monitored closely, also in the simulation. There is no measurement of the alkalinity of the wastewater in the example WWTP. This may be a key issue, as the pH measurement showed values occasionally even below 7.0, which may be an indicator of the need for lime dosing, which was not evaluated in this study. The switch from iron chloride or PFS to an aluminium-based precipitant would also be an interesting option to avoid the drop in pH.

A quick analysis of the pH and ammonium ratio showed that there is not an apparent inhibition of the biological process due to the low pH (< 7.0), but the issue should not be dismissed without measuring and following the evolution of alkalinity. It is recommended to start measuring this parameter periodically.

It may seem unfair to compare an existing system with a fictional one i.e. example WWTP A2/O vs SBR-based system. The SBR system is designed ideally, with a fairly large volume, in comparison with the example WWTP, ideally adjusted SRT, without limitations for the aeration and ideal sludge settling characteristics. However, the good results obtained with the SBR technology serve to reinforce four ideas already mentioned in previous chapters:

(1) The characterisation of the wastewater to be treated is crucial to design any type of treatment process (see Chapter 3): It was tested that the design of a WWTP depends largely on the real wastewater characteristics and the local conditions.

(2) The treatment goal of the WWTP should determine the biological treatment configuration (discussed in Chapters 6 and 7): when the norms change, the whole operation of the WWTP must be re-evaluated, because of a few tweaks here and there (such as increasing the DO set point or increase the internal recirculation in the activated sludge system) will not always be enough to achieve the treatment goals.

(3) Not every limitation must be solved by increasing the reaction volume: Although an increase in the reaction volume can be helpful to solve many challenges in nitrification and denitrification, this does not mean that every problem must be solved like this. As has been confirmed by the results from this study, there are many, less drastic, and probably less costly, strategies (e.g. incorporating online measurements, aerating a smaller volume in the existing biological treatment, reducing the HRT in primary clarifiers, etc.) that can contribute significantly to improve plant performance from multiple perspectives at the same time.

(4) The potential of simulation as a tool to optimize wastewater treatment systems is huge (see Chapters 2.5): Many ideas and approaches to optimize nitrogen removal can be found in literature, however, to understand how they interact with each other, in the complex and interdependent system as a modern WWTP, is only possible by using computer tools such as dynamic simulation. These, already-known optimization approaches chosen after a detailed plant analysis, were tested systematically and analysed from an integrated approach with the use of modelling and simulation as a tool, a better understanding of the interactions between the different, interconnected processes.

8.4.2 Anaerobic Sludge Stabilisation

China is making efforts to increase its biogas production, from diverse sources such as food waste, organic residues, etc. Sewage sludge does not seem to be considered in this plan. The use of anaerobic digestion (AD) is currently not extended in WWTP in the country and anaerobic reactors have even been removed from existing plants.

In the framework of the PIRAT-Systems project interviews with different stakeholders in the wastewater management industry, including plant operators were carried out by project partners, as detailed in (Zimmermann et al. 2022). These interviews, and also literature show that there are several arguments against AD in WWTP in China (see the introduction to Chapter 6) such as the low organic content of the sludge (and associated low biogas production), safety concerns (e.g. explosion risk), and operational problems (i.e. the reactors do not work properly). It is not clear to what extent these arguments are the opinion of a few plant operators that spread to other areas, or if these are evidence-based observations.

In praxis, an anaerobic digestion stage is far from trivial and must be carefully monitored to avoid organic overloads and inhibitions due to overfeeding or poor mass transfer. Moreover, the explosion risk is real, and all the personnel must be trained and the equipment adequate to minimize this risk.

The results from this study show that, as predicted by literature, AD can be beneficial from three important perspectives: energy production, reduction of sludge disposal costs and energy savings. In the example WWTP, ca. 37% of the energy demand can be offset with biogas production, under conservative assumptions. This means the often-named belief that there is not enough organic matter in the sewage sludge in WWTP in China to produce biogas was overturned. The adjustment of the sludge age in the biological treatment system plays a key role here, which is an aspect that seems disregarded by local plant operators.

The reduction of the amount of sludge to be disposed of is as well a national concern, as the sewage sludge amount in China has multiplied in the last decade, without a well-defined strategy to manage or finance it. However, WWTP operators do not have an incentive to reduce the amount of sludge to dispose of, as the management and funding are viewed separately, and they do not have a say in the disposal path.

Both aspects (i.e. sludge age reduction and reduction of the sludge to dispose of) contribute to lower the energy consumption for aeration and sludge thickening and dewatering post-AD.

However, the effect of the nutrient backload cannot be disregarded. The current challenges to comply with the strict discharge norms for nutrients can be a powerful argument against the implementation of AD, as there is a noticeable increase in nutrient emissions to the environment; nevertheless, this study shows that with simple optimization strategies, norm compliance can be improved.

As usual, a balance must be found between the benefits and drawbacks, but each plant operator and country must define which aspects weigh more. To decide this, several questions arise (among others):

- How clean must be the treated wastewater (once the norm is fulfilled)?
- Is fulfilling the norm enough?
- How this affects the discharge water body?
- How expensive is it to implement the changes for optimisation?
- How fast is the return on investment?
- Do I prioritize saving energy or reducing emissions even further?
- Do I have the personnel to carry out the required optimisation strategies?
- Do I have incentives to reduce the amount of sewage sludge to dispose of?

There is no single perspective under which to evaluate the performance of a technology, especially in this case, where the effects of its implementation affect the overall plant performance.

It seems, however, that other factors will be key in the development of AD in WWTP in China. Drivers such as increasing electricity prices, a decrease in own energy sources, high costs for sludge disposal, and regulations for sludge transport and disposal, could be drivers to incentivize the use of AD in WWTP, but future developments must be followed.

This work has not addressed the topic of sewage sludge dewaterability. Due to the decrease in the sludge age in the activated sludge system, a higher organic matter content in the excess sludge is expected. This can lead to problems in the dewatering processes used so far in the WWTP. However, after an anaerobic sludge treatment stage the contrary should occur, and an improvement in sludge dewatering should be expected. Still, not only is the organic content relevant for sludge dewaterability, but also the phosphate content in the sludge, which should increase after an anaerobic sludge stabilisation stage.

This poses difficulties in centrifugation, potentially increasing the amount of polymer required or worsening the solids content in the dewatered sludge. Therefore, before making drastic changes to the sludge stabilisation strategy, it must be studied how the current sludge treatment lines should be modified to cope with these changes.

8.5 General

Although the optimisation ideas presented in this work have been known for many years, are relatively simple, and many of them have been successfully applied in practice, this study shows how important a systematic analysis of existing WWTP constraints and a holistic approach to finding the gaps and overcoming them is. This means moving away from unfounded preconceptions and focusing on hard data, expert knowledge and proven optimisation strategies.

In this way, typical biases such as requiring post-treatment steps (i.e. increasing the treatment volume), expensive and/or unsustainable operational strategies (e.g. dosing of external C source without first exploring different alternatives, increasing recirculation rates and increasing operational costs, but with little effect on denitrification performance) or missing the opportunity to generate energy from biogas because of biases against anaerobic sludge digestion, can be avoided.

At the same time, as can be seen from the different scenarios, there is no perfect solution for the optimisation of a WWTP. Plant performance can be improved from different perspectives, but there is always a trade-off: the improvement in TN removal is often accompanied by a slight detriment of ammonia nitrogen removal; the process stability provided by a high sludge age will lead to a lower biogas production; the implementation of anaerobic digestion will produce energy and decrease the amount of sludge to be removed, but leads to an increase in total emissions to the environment, even when the standard is met, etc.

Therefore, it is in the hands of decision-makers and local authorities which criterion will prevail and is more relevant for a specific location, a specific WWTP, a specific water body, etc.

9 Integral approach to Improve Nitrogen Removal

This work has shown how a systematic and integral approach can help to fill the gap between the WWTP requirements, and the optimization options for nitrogen removal. This was done based on six main pillars, described in Figure 58 and detailed in the next chapters.

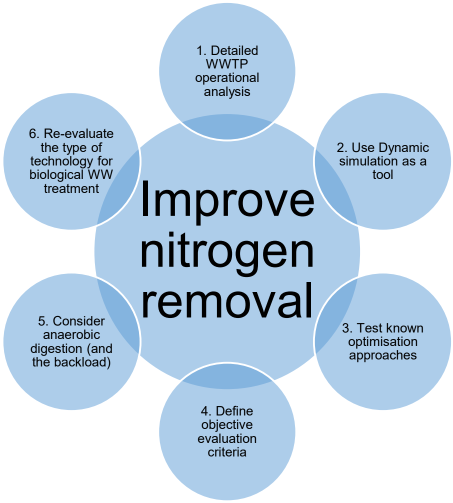


Figure 58. Pillars of the integral approach for WWTP optimisation of the dissertation

9.1 Detailed WWTP Operational Analysis

The first step to approach challenges with nitrogen removal in a WWTP is to identify the problem and challenges. This must be done systematically, with a detailed WWTP analysis, under defined criteria e.g. the guidelines provided by the ATV-DVWK 198 (2003). This guideline proposes the: evaluation of the wastewater discharge data, determination of the annual mean dry weather discharge, determination of the wastewater discharge, determination of loads and concentrations, evaluate the discharge data based on empirical values, among others.

Moreover, the graphic representation of the data in time and as cumulative frequency (as can be seen in Chapter 30 and Annex 12.3) provides a useful perspective to evaluate the data in context and allows to identify the trends in time (e.g. winter, summer, rain season, etc.).

This might seem obvious, but it is easy to lose sight of the extent of the interactions in a modern WWTP due to its complexity and it is key to understanding the problem and tackling it ahead. In this manner, the evaluation will be based on data, and not on pre-conceived ideas of how the WWTP performs.

9.2 Use Dynamic Simulation as a Tool

As shown during the development of this work, the use of dynamic simulation allows depicting realistically the variations and WWTP behaviour in time, and a broader context than static dimensioning tools. Its use is recommended in cases such as the example WWTP, which must react to the sharpening of the discharge norm and must develop sustainable operation strategies which allow complying with the normative requirements, in a reasonable time and at a reasonable cost. The expansion of a WWTP or its modernisation are examples of this approach being extremely useful.

Nitrification /denitrification-based processes are quite complex, therefore, and especially in large WWTP the use of online sensors is mandatory for a successful operation. These sensors should be integrated in a meaningful way, to contribute to the plant's performance.

For example, the use of ammonium nitrogen sensors incorporated into the aeration loop has the potential to improve both, nitrification and denitrification performances due to the efficient use of air. The use of nitrate-nitrogen measurements to control the internal recirculation is also an approach that can contribute to plant performance and energy savings in a WWTP.

These approaches, among many others, can be tested in computer modelling in a timely and cost-effective manner.

9.3 Test Known Optimisation Approaches

This work is not focused on providing or developing innovative solutions to improve nitrogen removal, but rather showing how simple (and known) strategies can be used in the context of a WWTP with upstream denitrification, challenged by normative requirements (respond to normative changes) and its design and operation.

Based on the results obtained in the different simulated scenarios, on the operational data analysis of the plant, and an extensive literature research (see Chapters 2.1, 2.2, 2.4 and 2.5), supplemented with further references (Ladiges 1994), (DWA 2006), (Maine DEP 2012), (Tchobanoglous op. 2014), this chapter provides recommendations to help improve nitrogen removal in WWTP with upstream denitrification and offers possible solutions and alternatives.

The recommendations are focused mostly on two problems: problems with ammonium removal and problems with nitrate removal.

9.3.1 Improvement of Effluent NH_4-N Values

Problems in ammonium nitrogen removal are related to inefficiencies in the nitrification step. Most problems here can be due to a poor oxygen supply or due to inhibitions (e.g. low temperature, fluctuating pH, etc.).

9.3.1.1 Operation and Maintenance of the Aeration System

The objective here is to maintain an adequate and uniform DO concentration in the nitrification basin and avoid a lack or poor distribution of DO in the aeration basin.

Adjust and check the dissolved oxygen in the aeration basin

The dissolved oxygen set point (DO_{sp}) should be between 1.5 and 3 mg/L, but ideally 2 mg/L, as no significant increases are observed at higher set points. This was observed in Chapter 4.1.2.1, in Figure 17, where the increase in DO did not contribute to improvements in nitrogen or COD removal.

DO concentrations below 0.8 mg/L, should be avoided, as they can lead to N_2O emissions (Pinnekamp et al. 2017), which is a powerful GHG. Moreover, low DO values can also contribute to problems in the settleablilty of the activated sludge. This aspect was considered in the simulation of scenarios T10 to T15 and is detailed in Chapter 5.2.1.

In general, too high DO (or too high air flows) should be avoided for several reasons. First, to minimize shear stress, which can cause floc disruption, leading to poor effluent values. Secondly, higher DO requires higher energy consumption, without leading to any benefits in the effluent values. Third, the excess of DO can be detrimental to other treatment steps (e.g. denitrification, biological P-removal).

Measure and control DO in the aeration basins consequently, carry out regular maintenance of the DO sensors

In the different simulated scenarios in SIMBA, the simulations work under the assumption that the DO sensors are measuring accurately and representatively. In reality, the WWTP operator must assure this with regular inspections, measuring in different sections of the tanks, and cleaning and calibrating the sensors.

The required number of sensors per aerated volume is dependent on the dimensions and shape of the tank, depth and type of sensor used, but they must be enough to assure a representative measurement of the conditions in the aerated basin.

The required maintenance frequency depends on the sensor as well. The typically used optical sensors for DO must be cleaned regularly (e.g. one time per week) in applications such as activated sludge, where the sensor is submerged in a medium with a large amount of solids and the optical detector is frequently blocked.

Check the aeration elements and air distribution in the aeration basin

One critical aspect is that the aeration elements and the air diffusers installed at the bottom of the tank are working properly. The WWTP operator must check if some are delivering large air bubbles or none at all, instead of the required fine bubbles stream.

The operator must look for disruptions in the air pipelines, and membranes of surfaces of the aeration elements. This can be sometimes observed on the surface of your activated sludge tank. Where a uniform pattern on the tank's surface and regular distribution of the bubbles should be observed. If this is not the case, check the aeration elements the next time the tank is emptied. Ideally, this should be scheduled for the summer months.

The emptying of activated sludge tanks for maintenance purposes was tested in 5.1.1, showing that in the case of the example WWTP, this would be possible (and highly recommended) and that it is more relevant to how the volume is distributed than the total activated sludge volume for the plant performance. This shows that under the right conditions, WWTP operators should not be afraid to empty tanks for maintenance, as in the long run, the benefits will far outweigh the temporary discomfort.

The aeration elements must be maintained regularly as well and be checked for ruptures or blockages. Proof if there is biological or chemical fouling of the membranes, the surface of the aeration elements. Sometimes a cleaning with adequate substances (e.g. weak acids) can help to recover the function of the aeration elements. Before doing this, the manufacturer's instructions must be consulted. It is also important to replace the aeration elements after the adequate lifespan. Here as well, the manufacturer's instructions are to be considered.

Moreover, it must be checked if there are visible accumulations of sand or grit at the bottom of the tank, which could be preventing an adequate performance of the aeration elements.

The mixing condition in the aeration basin can be checked by measuring the DO at different points of the aeration tank. If they differ significantly in a short period of time, then the mixing conditions must be improved, either by cleaning or replacing old aeration elements or by using a more favourable configuration of the aeration grid at the bottom of the tank.

Check the operation of the aeration system

To assure that the aeration is performing adequately, pressure loss or air leakage in the air pipelines can be checked. It must be also checked that compressors are operating correctly.

Consider an additional Redox measurement

It is recommended to include a redox potential measurement, to make sure that oxidative conditions (ORP > +100 mV) are constantly present. Redox potential sensors are cheaper and easier to maintain than DO sensors. The redox measurements cannot replace the DO measurements but are a good additional indicator of the tank conditions. It is advisable to not rely on a single measurement to control the condition of the aerated tank because sensors can show inaccurate information when the maintenance conditions are poor. As mentioned before, DO sensors are prone to fouling and the measurement will not be reliable when dirty, which could lead to excess aeration. Having a Redox measurement, in addition, allows controlling the performance of the DO sensor as well.

9.3.1.2 Control of the sludge age (SRT)

Low concentrations or low activity of nitrifying bacteria can cause problems in nitrification. The objective is to maintain an adequate and uniform activity of the nitrifying bacteria. In general, it is recommended to implement an SRT-based control in the plant, to contribute to the operational stability and plant reliability. It is important to highlight that it is not only about the total sludge age, but more critically, assuring a sufficient aerobic sludge age.

The sludge age of the system must be adjusted according to the temperature (e.g. using Equations 5 and 7), to assure the required activity of the nitrifying bacteria. Therefore, the WWTP operator must measure the wastewater temperature (T), which is usually included in other sensors (e.g. pH, DO, ORP, etc.) and increase accordingly the SRT when the temperature drops.

Not only a too-low SRT can be problematic, but also a too-high SRT increases the air requirements unnecessarily, and in the case of anaerobic sludge stabilisation, it decreases biogas production.

As it has been observed in the operational analysis of the plant, especially during rain events, the hydraulic load of a WWTP can be challenged, affecting the sludge age as well. Therefore, the possibility of a hydraulic overload due to rain events must be predicted and prevented (e.g. increasing sludge age slightly preventively during rain events or the rainy season).

The application of an ideal sludge age (see Scenario T27 in Chapter 5.2.4) proved to be, by itself a very good strategy to improve norm compliance, in contrast to the rather erratic sludge age observed in the example WWTP.

9.3.1.3 Adequate automation and control strategies

The incorporation of ammonium-nitrate sensors into the aeration loop can improve the aeration efficiency, as air supply is targeted to the real consumption. Moreover, this can potentially improve denitrification as well, as it reduces the amount of DO being recirculated.

The controller must be adjusted to the plant requirements, setting values to comply with the required discharge values. Here, simulation can be very useful, to test which values in the controller deliver the best effluent values.

A strategy can be developed, based on multiple online measurements (DO, NH_4-N, NO_3-N) for aeration to improve the control of air supply in a Feedback-type automation e.g. Ammonia-based aeration: if NH_4-N is above the target effluent value, then DO_{sp} = 2 mg/L; if NH_4-N is below the target effluent value, then nitrification is complete, reduce air supply.

This was tested in several scenarios, in Chapters 5.2.1, 5.2.2, 5.2.3 and 5.3. The results showed that this simple approach can contribute significantly to improving norm compliance and reducing pollutant emissions. However, it was possible to see there is always a compromise between maximizing nitrate or ammonium removal.

9.3.1.4 Identification of possible inhibitions

There are several possible inhibitions affecting ammonium removal, The objective is to find causes of inhibition and reduce or avoid them. Several causes such as the ones associated with pH or temperature can be identified when carrying out the plant operational analysis described in Chapter 9.1.

Inhibition due to low alkalinity and/or low pH

Biological nitrogen removal consumes alkalinity (specifically in nitrification). Therefore, to assure a stable operation, the alkalinity must be maintained and therefore measured regularly in laboratory. If the alkalinity is too low, the targeted addition of lime is required.

Moreover, the precipitants used are to be checked. If the pH value is problematic, the use of Polyaluminium chloride (PAC) must be evaluated, as it has been shown that the pH reduction is lower than with other precipitants (Böhler and Siegrist 2008). Sodium aluminate ($NaAlO_2$), a basic flocculant can also be an alternative.

The pH value should also be measured ideally online in both influent and effluent.

Inhibition due to low temperature

As described previously in Chapter 2.1.1.1, nitrification is sensitive to low temperatures due to the decrease in nitrifying activity under low temperatures (below 8 °C), therefore the sludge age must be increased accordingly.

Other substances

If temperature and alkalinity are not the causes of inhibition, respirometric tests should be carried out in laboratory to identify if there is toxicity and other possible causes.

9.3.2 Improvement of Effluent NO_3-N values

Problems in nitrate nitrogen removal are related to inefficiencies in the denitrification step. Most problems here can be due to a lack of easily biodegradable carbon sources, a too-small denitrification volume or a lack of anoxic conditions or failures in the internal recirculation.

9.3.2.1 Influencing the Influent C/N Ratio

The objective is to increase the availability of easily biodegradable carbon sources.

Check the HRT in primary clarifiers

The first step is to check the HRT in primary clarifiers (if existent). The HRT should not be higher than 2 h, and ideally below 1 hour. This can be done permanently, taking out of operation one or more tanks (if existent), or dynamically, by measuring the influent characteristics online and switching the number of primary clarifiers in operation.

As tested in Chapter 5.1.2, the decommissioning of the PC can solve simultaneously several deficiencies in the WWTP: decrease the HRT in PC and release volume for other uses (e.g. increase anoxic volume).

Add external carbon source

If after testing other alternatives, denitrification is still not improved, and the C/N ratio is still unfavourable, the addition of external carbon sources (e.g. Sodium acetate, methanol, others) must be evaluated but always using an adequate automation strategy based on the influent measurements.

Based on the results of this study, it was observed that this strategy should not be the go-to approach when dealing with problems with denitrification. When this strategy is given last priority, it will not contribute constantly to norm compliance, but only occasionally, and more sustainable strategies can do the “heavy lifting”.

Improve conditions in the sewer

In the long term, it must be checked if it is possible to manage or improve conditions in the sewage system that reduce the C/N ratio in the influent, e.g. avoid if possible long pathways from collection to treatment point. Avoid if possible, the existence of septic tanks prior to sewage systems.

9.3.2.2 Improve the denitrification capacity

The denitrification volume proportion should be ideally, $0.2 \leq V_D/V_{AT} \leq 0.6$. If the V_D proportion is too small, the denitrification capacity of the plant is limited. To change this proportion, it can be tested if a reduction of the aerated section (reduce V_N, increasing V_D) improves the effluent values. This can increase slightly the ammonium values, but still, generate an overall reduction of the total nitrogen emissions. If this does not work, and there are unused tanks (e.g. bypassed primary clarifiers) they can be used as denitrification tanks.

If this is not suitable, alternative operation strategies such as intermittent aeration can be tested. If none of these strategies (or similar) work, an increase in the total volume should be evaluated.

All these strategies were tested successfully in Chapter 5, showing that the plant configuration must be evaluated critically and that sometimes aerating less is better for the overall plant performance.

9.3.2.3 Maintaining anoxic conditions

Anoxic conditions are required for denitrification. If an excess of DO is present or if anaerobic conditions are present (i.e. nitrate concentration is insufficient), denitrification will be impaired or not even possible.

9.3.2.3.1 Aerobic conditions

It must be checked that the denitrification tanks have anoxic conditions. The DO should be measured online and remain below 0.1 mg/L. The redox potential also can be used as a reference, and the ORP be between -50 and +50 mV, measuring the redox potential online.

If the values are above the recommended range, aeration can be reduced. Aeration should be just enough to convert all ammonia into nitrate. This can be checked with nitrate ammonium sensors and they can be incorporated into the aeration control loop. Another critical point is if DO is being transferred in the recirculation. Recirculation can be adjusted accordingly, for example with a nitrate sensor. A baffle wall can also favour O_2 degassing.

The stirring power should also be checked. An excessive stirring intensity (>> 2 W/m^3), could be causing the incorporation of air into the anoxic tank. Furthermore, it represents an unnecessary increase in energy consumption.

Reducing sludge recirculation and adjusting the stirring intensity can also decrease flocs and cell disruption due to the shear and tear of the activated sludge.

9.3.2.3.2 Anaerobic conditions

If the measured ORP is below -100 mV, there are anaerobic conditions instead of anoxic. To avoid this, it must be checked if nitrification is working properly (i.e. if there is enough nitrate present), and if the internal recirculation is enough. If not, an increase in internal recirculation rate is required to provide sufficient oxygen in the form of nitrate. This can be supported using a nitrate sensor, to adjust the recirculation rate accordingly.

Moreover, it must be checked if the anoxic tank is properly stirred to avoid septic conditions, sludge settling, etc. On time, there could be an accumulation of materials in the impeller or axis of the stirrers (e.g. hair, wet toilet paper, fabric, etc.) and this reduces the performance. A regular maintenance and cleaning routine helps to avoid this.

9.4 Define Objective Evaluation Criteria

There are no single criteria to evaluate the performance of a WWTP. Without a doubt, WWTP operators must watch closely the normative requirements and norm compliance. But nowadays that is not enough. On one hand, the increasing operational costs added to increasingly strict normative requirements for the discharge of wastewater, putting pressure on municipalities and private operators to be more efficient, considering energy demand, external chemicals use, supplies required, etc. On the other hand, the current immissions approach indicates that not only norm compliance is a factor, but also the emissions to the environment and their effect in particular water bodies must be considered.

Energy production and consumption can also be a relevant criteria to save operational costs and the application of anaerobic digestion is an alternative. This technology makes use of the energy contained in the organic matter in sewage sludge, contributing to the goal of moving towards a circular economy.

All these aspects must be weighed by decision-makers and stakeholders, and as usual, there is no one-solution-fits-all. Rightfully, different locations, countries, governments, and operators will prioritize different aspects and therefore the integral evaluation proposed in this work can contribute to easing the process. This work focused mostly on norm compliance, pollutants emissions, aeration requirements and biogas production, but other aspects could be relevant to other WWTP around the world.

9.5 Consider Anaerobic Digestion

It was discussed in detail in Chapter 2.1.3.7, anaerobic digestion of sewage sludge is not as popular in other countries (e.g. China) as it is in Germany due to several structural and cultural reasons.

However, this work showed that many of the biases and negative preconceptions against AD, particularly the ones that can be influenced by the WWTP operation, are not justified.

Although the backload coming from the sludge liquor (process water) can have a negative effect on the plant performance, most of them can be counteracted using simple operational and automation technologies, as shown in Chapter 6.

These results are an invitation to test via computer modelling, the real effect of AD in the performance of the WWTP, to look at the technology from a more objective perspective, without forgetting the advantages towards other, also relevant environmental goals (energy production, improvement of sludge disposal, etc.).

9.6 Re-evaluate the Type of Technology for Biological Wastewater Treatment

If all previously tested strategies are not delivering the expected results, and the systems do not seem to cope with the required effluent values, more flexible operational strategies must be evaluated. Two approaches were tested in the simulation:

- Make the oxygen supply more flexible → e.g. intermittent aeration.
- Make the biological treatment stage more flexible, e.g. SBR

Both strategies can serve to increase or decrease the denitrification or nitrification capacities as required, saving energy with a targeted aeration strategy, based on the effluent values.

If the nitrogen removal problem is related to biologically bound nitrogen (and therefore to suspended solids in the effluent), other measurements are required but are out of the scope of this work e.g. bulk sludge, poor settling in secondary clarifiers, foam or swimming sludge, etc.

It is important to critically evaluate past experience, and tailor the WWTP design to the influent wastewater conditions, instead of preconceived ideas on which technology will perform better.

10 Summary, Conclusions and Outlook

10.1 Summary

10.1.1 Introduction, Objectives and Methodology

This work focuses on the removal of nitrogen in domestic WWTP under unfavourable conditions such as low C/N ratio and strict discharge limits.

In recent decades, China has tightened its regulations and standards for wastewater discharge, requiring the application of the norm Grade I-A (see Table 7) and stricter norms in the Tai Hu Region. Most WWTP in the country have problems complying with the discharge standards, especially for nutrients. There are several causes for this, but the main one is the unfavourable C/N Ratio for denitrification (COD/TN < 100:10) present in the influent wastewater, caused by sewage design aspects such as unfavourable sewer conditions, upstream septic tanks and separated collection of toilet paper.

Several conditions must be fulfilled in order to carry out biological nitrogen removal such as an adequate C/N ratio, anoxic and aerobic conditions, adequate sludge age, etc., conditions that should be monitored closely. There are also different configurations possible for biological nitrogen removal (i.e., upstream, intermittent, simultaneous, or downstream denitrification) and their application is more or less favourable depending on the influent wastewater conditions, and location, among others.

Due to its unprecedented and accelerated growth over the last two decades, the wastewater sector has had to mature rapidly. In this process, technologies proven in other countries have been installed, but without much clarity or certainty about the quantity and quality of the wastewater to be treated.

This combination of factors i.e., strict discharge norms for nitrogen components and unfavourable wastewater quality is present in other regions of the world and could be increasingly present with the strengthening of wastewater discharge in many countries worldwide due to the effects of climate change and deterioration of water availability and quality. Therefore, the results of this work can be extrapolated to other regions with similar conditions (i.e., some WWTP in Germany, Switzerland, Luxemburg, Dubai, etc.), and the obtained conclusions are transferable and useful not only for the studied WWTP.

10.1.2 WWTP Description

The case WWTP is a good example of the situation described above, with a typical biological treatment step, an increase in the discharge norms and difficulties to comply with them. The plant was designed to comply with the Grade 1-A standard (GB18918-2002) effluent parameters but from 2021 the WWTP must comply with the stricter City Assessment Standard norm (CS) with discharge limits of 10 mg/L for TN and 1.5 mg/L for NH_4-N. Therefore, there is a need and opportunity for improvements in the system for nitrogen removal.

The example WWTP has a size of ca. 450,000 $PE_{COD,120}$, and it is in the Tai Hu catchment, treating mostly municipal wastewater. The WWTP consists of traditional mechanical-biological treatment with simultaneous aerobic sludge stabilisation. The plant has a mechanical pre-treatment (screens, aerated grit chamber and primary sludge settling), and a biological treatment step by activated sludge type A2/O. Before discharge, the plant carries out tertiary treatment for chemical phosphorous removal, filtration, and UV disinfection. Sewage sludge is thickened, dewatered, and then transported for disposal, by incineration or landfilling.

The plant was analysed based on information provided by the operator, observations carried out during a plant visit in 2019 and meetings with the plant operator. In the operational data analysis, it was detected that the WWTP possesses only a few online measurements and must rely heavily on manual measurements and the operators' experience. According to the literature and gathered experience in the project PIRAT-Systems, this situation is not uncommon in China.

The inflow C/N (COD/TN) ratio is variable and is commonly below the desired ratio for denitrification of 100:10. This is an indicator that the plant can profit from a more flexible operation. The hydraulic retention time (HRT) in the primary clarification stage is around 2.5 h on average, which contributes to excessive COD removal and poorer denitrification performance.

The biological tank has a volume of 96,000 m^3 (V_{AT}) and an anoxic volume of ca. 17% with respect to the total biological treatment volume (V_D/V_{AT}), which is lower than the recommended 20 to 60% (DWA 2016). The reduction of the aerated volume in favour of the anoxic volume can solve this, increasing the denitrification capacity. It must be checked, however, which is the best proportion, to avoid an impairment of the nitrification.

The dissolved oxygen set point in the aeration tanks (DO) is between 2 and 3 mg O_2/L. Literature and practical experience show that a set point of 2 mg DO/L is sufficient and that higher values do not contribute to a better oxygen transfer, but rather to a larger electricity bill.

The activated sludge stage is designed for an SRT (or sludge age) between 15 and 20 days, but an analysis of the sludge production shows that the calculated SRT is on average ca. 38 days, and it fluctuates strongly. Better control of the sludge age can contribute to better overall plant performance and less energy consumption.

The size and dimensions of the WWTP are checked, based on two international approaches: DWA-A 131 (Germany) and Metcalf & Eddy (USA). Both approaches show very different treatment volumes, as the German approach puts more emphasis on having enough biomass for nitrification and enough volume for denitrification. However, both approaches indicate that the biological treatment volume is too small to comply with the norm CS.

10.1.3 Model

The WWTP was modelled in SIMBA, based on the ASM3 (Henze 2000) with modifications and parameters following the recommendations by the HSG research group (*asm3h*). To carry out the modelling, the guidelines provided by the HSG group (Langergraber et al. 2004) were followed.

The model fit was carried out for one year and evaluated according to different statistical measurements. The evaluation indicated a very good model fit for the parameter MLSS, a good fit for the effluent COD, TN, and NO_3-N, and a medium fit for the effluent NH_4-N. This model was used as the base scenario ("Base") for comparison with several scenarios, showing the effect of the incorporation of optimisation strategies in the overall plant performance.

10.1.4 Strategies Tested

The tested strategies can be grouped into two main categories: Operational strategies and Automation and control strategies. In the operational strategies, the By-pass (partial or total) of the primary clarifiers was tested, as well as the increase of denitrification volume. This increase was done in three different ways: (1) Decrease of the aerated volume in favour of the anoxic volume; (2) Use of by-passed primary clarifiers ad denitrification volume; (3) Conversion of the anaerobic tank in an anoxic tank (changing the internal recirculation point).

The automation and control strategies that were tested are Decrease of the DO_{sp}, incorporation of ammonium and/or nitrate measurements in the automation loop for aeration, adjustment of the sludge age according to temperature and Intermittent aeration (time-based, ammonium-based).

The different strategies were tested separately, and the number of norm non-compliances in a year where compared. Based on these results, the best strategies were selected and then tested in different combinations. The combinations were compared based on three parameters: (1) Number of norm non-compliances in a year; (2) Air requirements; (3) Emissions load.

The strategies that showed the best reduction in norm non-compliances are the ones that tend to improve the denitrification capacity, either by increasing the denitrification or by aerating intermittently. Moreover, they contribute to saving energy because the aerated volume is reduced.

The results show that the poor tank configuration i.e., too low denitrification volume, can be counteracted by reducing the aerated volume because the plant has an activated sludge volume that is large enough. This consistently proves to be the most effective – and probably the easier to apply– strategy.

The automation of the aeration loop based on the effluent values for ammonium will also serve to improve the overall plant performance, especially if used in combination with an increased V_D/V_{AT}.

The intermittent aeration also serves to improve the norm compliance, not only because it tends to increase the denitrification capacity, but also because it provides more operational flexibility, especially with the incorporation of ammonium sensors to the aeration loop, because the plant can react dynamically to changes in the influent quality (i.e. C/N ratio). Moreover, as it is only aerated when required, the air requirements are much lower.

It is easy to see from the strategies studied that usually either denitrification or nitrification is significantly improved. Significantly better denitrification is, therefore, at the expense of slightly worse nitrification.

The results obtained in the best combination scenarios were evaluated additionally for different discharge norms: Chinese norm Grade I-A, EU Water directive, German norm

AbvW, and the Luxembourgian discharge norm for the WWTP Beggen. These norms, some laxer and some stricter than the CS norm, give a broader perspective to the WWTP performance and tested optimization strategies. The evaluation shows that even under extremely strict norms (e.g. NH4-N < 1 mg/L, 2-h composite sample) the best combination strategies have the potential to significantly improve norm compliance. This is an indicator that the results obtained in the example WWTP are transferable outside China.

10.1.5 Incorporation of AD

Despite China having problems with its sludge stabilisation and disposal, many large WWTP in China do not make use of the inherent energy contained in the sewage sludge to produce biogas and the reduction of the sewage sludge volume. In Germany, it is recommended to use anaerobic sludge stabilisation from a plant size of 30.000 PE or even lower, depending on the conditions, and most WWTP in China are well above this plant size.

The example WWTP, with 450,000 PE could profit from this process, but the backload generated by the mixed sludge liquor after the anaerobic digestion (AD) process must be taken into account.

To incorporate an AD stage in the example WWTP, the base model was modified with an anaerobic reactor and the sludge age was adjusted according to the wastewater temperature. The AD reactor uses the IWA ADM and Siegrist model approach (Siegrist et al. 2002b), which is typical for this kind of application.

Without any countermeasure, the incorporation of the AD stage could increase the number of norm non-compliances from 31 to 77 in total in a year. Therefore, the incorporation of similar combinations of strategies as the ones tested for the plant without AD was tested in this new scenario. The dosing of centrate was incorporated and an alternative mixed liquor treatment type Anammox was tested.

Here, besides evaluating the already named (1) number of norm non-compliances in a year, (2) Aeration requirements and (4) pollutants emissions, (4) the biogas production per year is incorporated.

In all tested scenarios at least 10% fewer air requirements are observed due to the reduction of the SRT. There is less biomass in the system and therefore air requirements are lower to maintain the desired DO set point.

With the incorporation of AD, the increase in the V_D/V_{AT} proportion and maintaining the total activated sludge volume (V_{AT}) is less effective. However, the ammonium-based aeration control shows, that it is a powerful control strategy to reduce nitrogen emissions and save aerations costs.

The biogas production fluctuates between 14.7 and 16.1 L/(PE_{BOD}·d), and it is in the lower range for biogas production, but it can cover ca. 38% of the required energy in the WWTP.

Tests on the model including AD show that the introduction of an anaerobic sludge treatment step can contribute not only to energy savings through biogas production, but also to savings in aeration and sludge disposal costs. Furthermore, with appropriate nitrogen removal strategies, it is possible to almost completely counteract the negative effects of the backload generated by anaerobic sludge treatment, considering regulatory compliance and pollutant emissions.

10.1.6 New Biological Treatment Stage (SBR)

Due to its characteristics, flexibility, and small footprint, the SBR technology is evaluated as suitable for the treatment of wastewater under the conditions of the example WWTP. The control can be time-based sequential control (TSC) or real-time control (RTC).

The design of the new SBR treatment stage is carried out according to the DWA-M 210 (2009), following the TSC approach. There, a system with eight reactors for a cycle time of 8 hours was designed. The total volume of the SBR system (220,000 m^3) is comparable with the current volume of the activated sludge stage plus secondary clarifiers. The system requires an upstream and a downstream equalisation tank, of 20,000 m^3 each.

The SBR-base scenario showed a norm compliance of 100%, showing a better performance than all the tested scenarios with the A2/O technology. The air requirements are almost half that in the Base scenario due to the more suitable conditions for denitrification.

The SBR scenario was challenged, shortening the cycles and adding an anaerobic sludge stabilisation stage, involving the reduction of the total reaction volume. Both scenarios showed again complete norm compliance, but some differences in the pollutant emissions.

The use of the SBR technology is successful because it was designed explicitly for the quality of the treated wastewater, and the denitrification capacity is increased.

10.2 Conclusions and Outlook

Wastewater treatment in China has evolved very rapidly in the last decades and it faces significant challenges regarding the norm compliance perspective and also in sludge disposal. Moreover, the example of the studied WWTP can serve other countries with similar conditions, as the low C/N ratio is not a problem found exclusively in China, but also in Europe and other regions as well.

Additionally, it is expected that in the future, more and more natural water sources are threatened by the discharge of nutrients. Climate change will continue to pose a challenge to water availability, increasing the pressure on WWTP to deliver treated wastewater with higher quality worldwide. Therefore, the improvement in automation and operational strategies in WWTP will be necessary steps to comply with the increasingly strict requirements.

The operational decisions required to improve the overall plant performance require a deep knowledge of the wastewater treatment process and also to question the already established technologies (e.g. upstream denitrification) as the most viable solutions. A study of the wastewater to be treated, both in quantity and quality is essential to design plants that fulfil the purpose and the required norms, and simulation studies can help to decide the best technologies to do it.

The construction of new WWTPs must not only take into account past experiences, but also future demands (e.g. changing population, changing water demand quality or allocation, or as discussed in this work, the sharpening of norms for wastewater discharge or sludge treatment), and it must be assessed whether the technologies traditionally used are the most efficient to cope with future challenges. Likewise, the tendency to simply replicate technologies that have been used so far must be questioned, as this approach does not always take into account the specific conditions of the location where they are planned. In this study, it was tested for example, that SBR is a technology that provides more operational flexibility, allowing for better control of the treated wastewater.

This work shows that a throughout systematic analysis and study of the wastewater to be treated, and an operational data analysis can indicate very clearly where are the deficiencies in a WWTP. Combining this knowledge with dynamic WWTP modelling allows different solution strategies to be planned, tested, and objectively evaluated.

However, for the analysis, an integral approach must be carried out, as a single indicator or single optimization approach will not suffice to improve the plant performance from different perspectives (e.g. norm compliance, pollutants emissions, energy requirements, etc.).

While modelling is a powerful tool for testing optimisation strategies at low cost, it is not a trivial task and can be time-consuming. Furthermore, it requires expert knowledge of the water treatment process and the plant to be modelled.

However, in the long term, from the perspective of WWTP planners and operators, dynamic modelling and planning is worthwhile, as it can objectively indicate which changes or investments are most appropriate, and also allows prioritisation of measures and actions. All this is under the requirement of having enough and good quality data for modelling.

According to the simulations carried out in this work, simple automation and operational strategies can serve to reduce energy consumption and simultaneously improve discharge values and norm compliance. In the pre-treatment and biological treatment stages, sometimes small changes in operating and automation strategies can contribute to large savings in energy and resource consumption, for example, reduction of the DO set point, bypass of primary clarifiers, reduction of the aerated volume, temperature-based control of the sludge age, and incorporation of ammonium and nitrate sensors to the aeration control loop, among others.

As expected, the studied scenarios with a combination of strategies show better results in all analysed categories: norm compliances, emissions, and energy consumption. In general, the strategies that allow for system flexibility, i.e., intermittent aeration and SBR are the most successful.

Following the same approach, the incorporation of anaerobic digestion for sewage sludge stabilisation is not only possible but also beneficial from the energy and sludge disposal perspective. But to make it possible, not only the initial investment is key, but also several operational conditions must be closely monitored e.g., sludge age, load to the digesters, etc.

With these results, it can be argued that the incorporation of additional treatment steps can be avoided, and the focus must be on the optimisation of the existing plant rather than on additional processes. This work shows how much it can be done to improve the plant performance without expanding the existing infrastructure, but rather by investing in automation. This is a relevant finding for WWTP not only in China.

It results clear that the stricter the discharge norms, the more complex the required automation system. Simple feedback DO set point-based aeration regulation is not enough, and a single DO measurement in an aeration basin will not cut it either.

This fundamental change in the way to approach wastewater treatment will require more qualified personnel; sensors and automation systems require programming, maintenance, calibration and continuous improvement.

Moreover, there is always a trade-off when optimizing a WWTP, as no perfect solution exists, and the decision makers must choose which criteria are pursued with the optimization (e.g. decrease pollutants emissions, decrease energy consumption, etc.). These objectives are not necessarily conflicting, but any optimisation measure will have both positive and negative effects that need to be considered.

The future will surely bring new challenges in the wastewater field. As climate change modifies water allocation worldwide, there is a trend toward water scarcity and desertification in many regions around the world. This forces stakeholders and decision-makers to provide integral solutions for water management, closing the water cycle. Wastewater treatment, directly at the end of the cycle, is one of the key steps to achieve this.

As better laboratory measurement methods have developed, micropollutants of different kinds, pharmaceuticals, microplastics, PFAS, etc. have been found in treated wastewater, showing negative effects on the environment and also limiting the possibilities for reuse. This makes this topic extremely relevant nowadays, and we will see developments and more large-scale applications in the years to come, as treated wastewater will have to be much cleaner.

Just as important as nutrient removal from wastewater is becoming the recovery of such nutrients for example, as fertilizers. The circular economy approaches can now profit from technological advances and the current research (as is the case of the project PIRAT-Systems), and the emphasis that different countries have given to increase resource self-sufficiency, and (more) resource independence.

The same occurs with energy consumption, production, and efficiency in WWTP. The recent (and not so recent) energy crises, as well as climate change and the (necessary) trend to use fewer resources in general, highlight once again the importance of looking at all processes in detail and “making” more wastewater treatment, with fewer resources.

The studied WWTP is a good example both for the operation and upgrading of existing WWTP, as well as for the planning and design of new WWTP. From the example WWTP, several recommendations and lessons can be learned, to be taken into account in locations with similar challenges. Lessons such as the improvement of the automation systems, or even the way the plant is operated, but mostly a change of perspective based on the analysis of measured and collected data can be beneficial.

There are no shortcuts to reaching the required quality for future challenges in wastewater treatment. Luckily, we already have the technology and the knowledge to make it possible.

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12 Annexes

12.1 HSG Guidelines

The HSG-Sim, a group of academics from German-speaking countries in Europe, developed a guideline to carry out the modelling and simulation of WWTP. The process is divided into seven phases and the flowchart is shown in Figure 59.

As in other calibration protocols, the first step is the definition of the objectives of the study and its boundaries, followed by the collection of information on plant layout, operation and performance. With this data, a preliminary model for the WWTP under study is carried out considering sub-models for hydraulics, settler, controllers and biological compartments. After that, the quality of the plant data is verified using mass balances, e.g. with Phosphorous as a parameter (Sin et al. 2005).

Previous to dynamic calibration of the model, the hydraulic sub-model must be calibrated, then a pre-simulation using a steady state model is performed and the results are compared with average plant data. In addition, a sensitivity analysis is performed to determine the parameters with the most influence on the simulation results (Langergraber et al. 2004).

The fifth phase, data collection for the simulation study, aims to close the data gaps found in previous stages. A measurement campaign is set up and performed to collect data about the plant dynamics for use in the dynamic calibration of the full-scale model. The measurement characteristics (frequency, location and type) are determined based on the evaluation of the model obtained in the preceding steps. A 10 day long campaign is advised to include the plant performance of at least one weekend. At this step, also a data quality and consistency check are applied (Sin et al. 2005).

During the sixth phase, the dynamic calibration of the model is performed. This starts with the assessment of the initial conditions, by simulating several weeks depending on the SRT of the plant. The calibration process follows, adjusting parameters based on the results of the sensitivity analysis. An iterative procedure is used and the success of the model calibration is judged visually, considering the peak and median values of the simulation results (Sin et al. 2005).

The HSG guidelines also advise performing model validation. The calibrated model is verified with an independent set of data, from a different monitoring campaign e.g. with plant data obtained under conditions different than those of the calibration period (i.e. different temperatures, sludge ages, etc.) (Langergraber et al. 2004). In the final step, the calibrated and validated model can be used to simulate different scenarios, according to the objectives of the study. This can be done for example by using a performance index. The protocol suggest also to document thoroughly all details and steps followed until the study goal was reached (Sin et al. 2005).

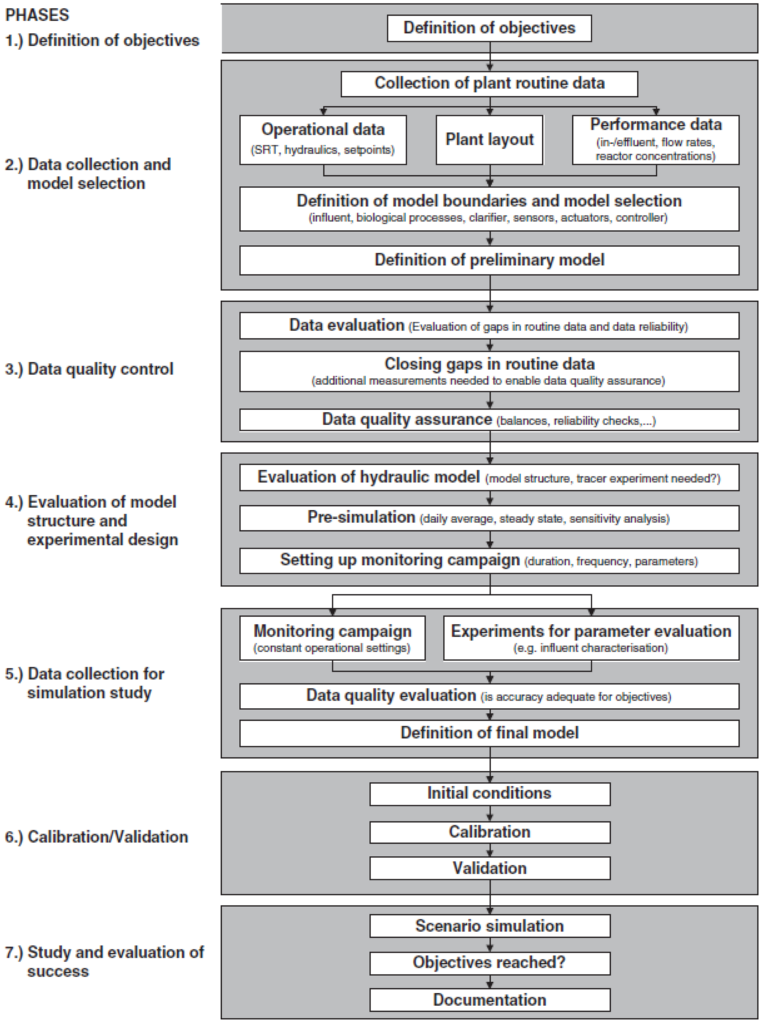

Figure 59. Flowchart of a simulation study according to the HSG guideline (Langergraber et al. 2004)

12.2 SIMBA

Not only ASM is relevant for the modelling of wastewater treatment plants. To model the complete process of a wastewater treatment plant and sewerage systems, several models and approaches are used for different processes, described in Table 47.

Table 47. Commonly used models and approaches in SIMBA for simulation of WWTP

Process	Model /Approach	Description
Influent	Fractionation of components in the influent	COD and nitrogen compounds are fractionated. The fractionation can be changed to calibrate the model and it is based on the fractions defined in the DWA-A131 (DWA 2016)
Primary settler	Dynamic model for clarifier according to (Otterpohl and Freund 1992)	It is a model with a fully mixed reactor and a consigned elimination behaviour. The resulting primary sludge can be calculated by means of a mass balance (Alex et al. 2015)
Activated sludge	ASM, IWA (Henze 2000)	As described in 2.3.1
Biological Phosphorous elimination	EAWAG Bio-P module according to (Siegrist et al. 2002a)	The module uses modified processes from ASM2d but neglects the fermentation of readily degradable substrate. Biomass decay is modelled in the form of endogenous respiration as in ASM3. The glycogen pool and biologically induced P-precipitation are not taken into account (Siegrist et al. 2002a)
Secondary clarifier	3 layers model	The secondary clarifier model uses 3 layers. • Variable volume top layer: to model the clear water zone during the sedimentation phases, • Variable volume middle layer: to model the thickening and storage of the sludge. • Fixed volume bottom layer: to model the sludge concentration at the bottom. The simulated mixture of wastewater and activated sludge flows into the second layer. The outflow of clean treated water through the upper layer is simulated as an overflow, while the return sludge from the lower layer is simulated by a given flow rate (ifak 2019).
Anaerobic digestion	Siegrist (Siegrist et al. 2002b)	The Siegrist approach is similar to the ADM1 model described in (Batstone et al. 2002), but it considers an additional mineral fraction XMI, and an additional fraction XD (for the nitrogen balance) and it is possible to adjust it with ASM3 regarding nitrogen and solids content of each fraction (ifak 2018).
Sludge thickening	Module for mechanical sludge thickening (Band filter)	A fraction (0-1) is given for the effectivity of TS retention, which determines the TSS in the filtrate
Sludge dewatering	Modules for sludge dewatering (Centrifuge)	A fraction (0-1) is given for the effectivity of TS retention, which determines the TSS in the filtrate

12.3 Graphs of the Process in Time

12.3.1 Influent Flow

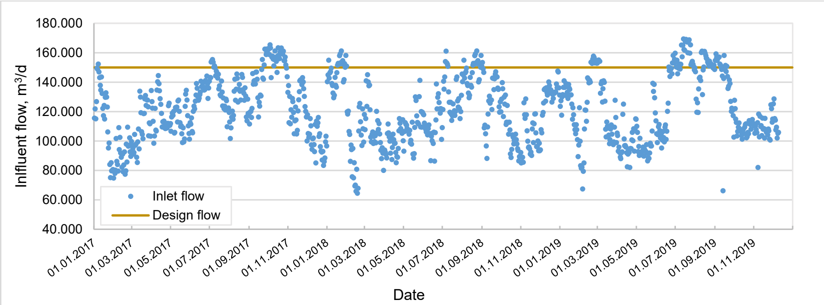

Figure 60. Infuent flow example WWTP between 2017 and 2019

12.3.2 COD and BOD

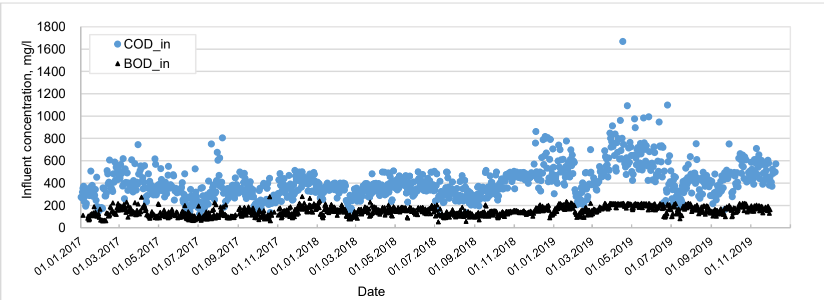

Figure 61. Influent COD and BOD concentration example WWTP between 2017 and 2019

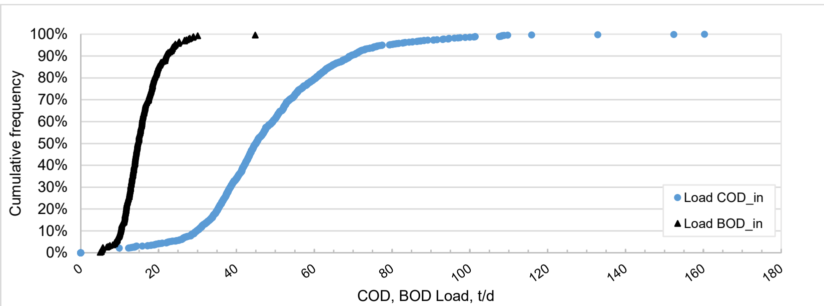

Figure 62. Cumulative frequency for COD and BOD loads in the influent of the example WWTP between 2017 and 2019

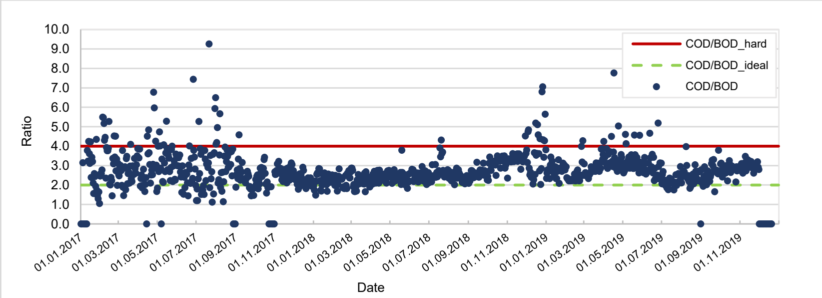

Figure 63. Influent COD/BOD ratio in the example WWTP between 2017 and 2019

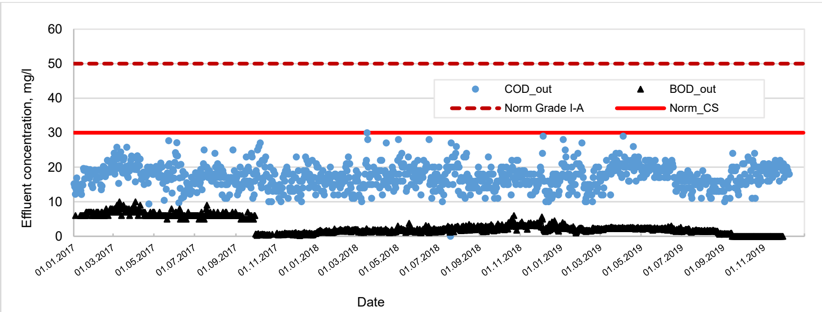

Figure 64. Effluent COD and BOD concentration example WWTP between 2017 and 2019

12.3.3 TN and NH_4-N

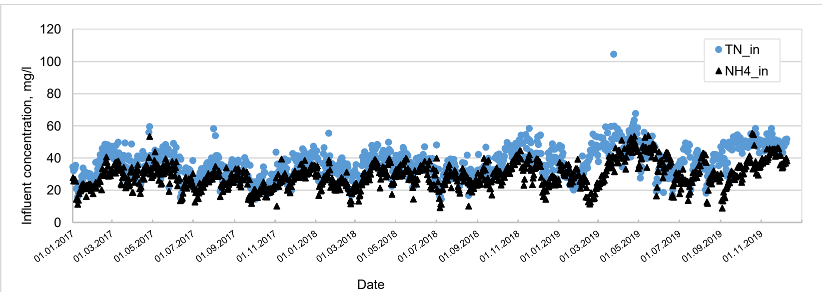

Figure 65. Influent Total Nitrogen (TN) and ammonium nitrogen (NH_4-N) concentration example WWTP between 2017 and 2019

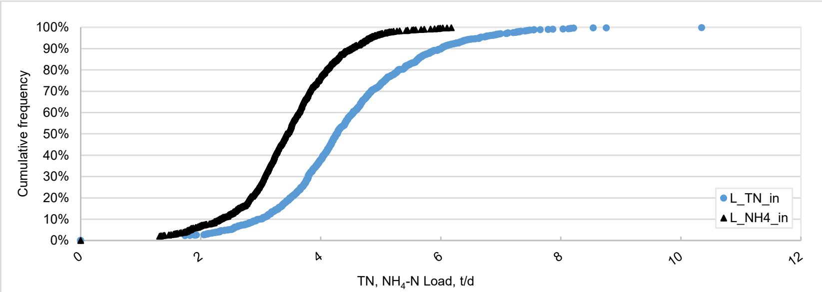

Figure 66. Influent Total Nitrogen (TN) and ammonium nitrogen (NH_4-N) concentration example WWTP between 2017 and 2019

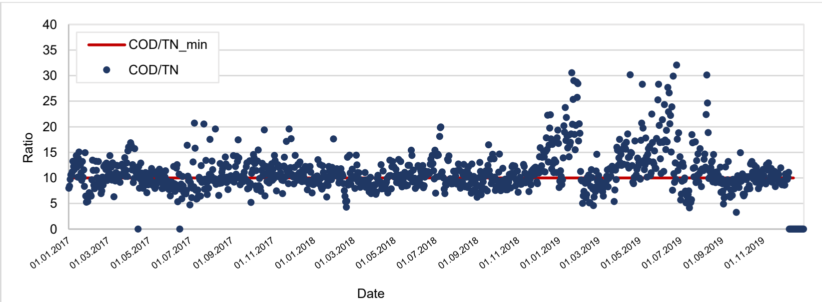

Figure 67. Influent C/N ratio example WWTP between 2017 and 2019

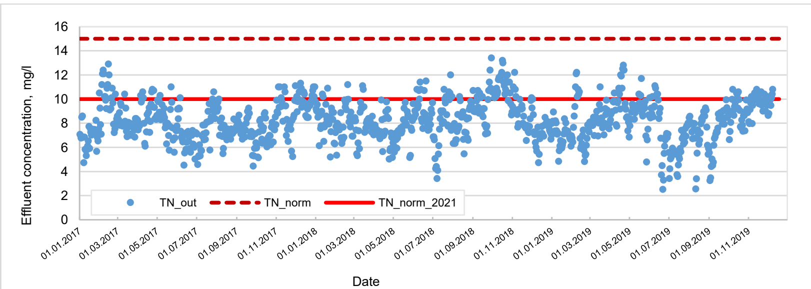

Figure 68. Effluent Total Nitrogen (TN) concentration example WWTP between 2017 and 2019

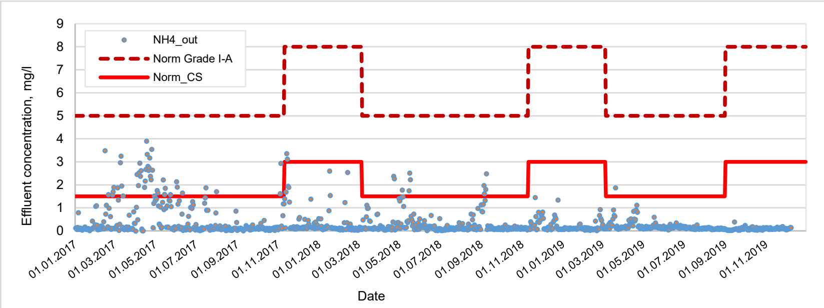

Figure 69. Effluent ammonium nitrogen (NH_4-N) concentration example WWTP between 2017 and 2019

12.4 Design of an Activated Sludge Stage According to DWA-A 131

12.4.1 COD Fractionation and Sludge Production

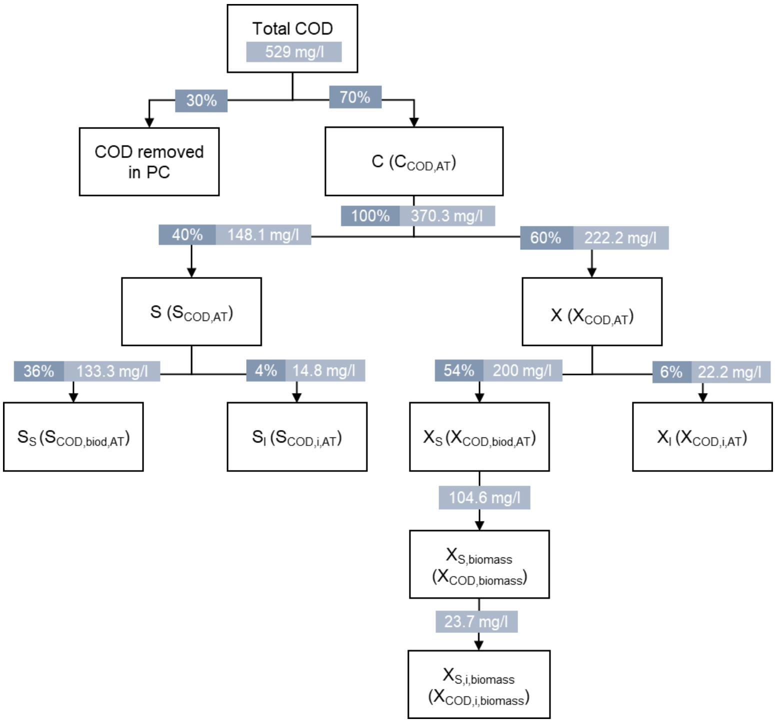

Figure 70. Estimated fractionation of the inlet COD at the example WWTP

12.4.2 Calculation of the Sludge Mass and A2/O volume (DWA-A 131)

Table 48. Calculation of the sludge mass and required volume of the A2/O stage, according to the DWA-A 131

Parameter		Value	Unit
Wastewater temperature	T	12	°C
Process factor	PF	1.5	
Sludge age	SBR	25.0	d
Sludge production factor	F_T	0.8	
COD in the inlet of biological step	$C_{CSB,abb,ZB}$	333.3	mg/L
Yield coefficient	Y	0.67	g VSS /g COD_{rem}
Decay coefficient	b	0.17	1/d
Formed biomass (aerobic sludge stabilization)	$X_{CSB,BM}$	50.2	mg/L
Proportion of inorganic substances in the filterable substances	f_b	0.2	-
Daily wastewater inlet flow	$Q_{d,konz}$	149,588	m^3/d
inert particulate inlet COD	$X_{CSB,inert,ZB}$	66.7	mg/L
fraction of inert COD from particulate COD	f_A	0.3	-
Inlet COD concentration to biological treatment	$C_{CSB,ZB}$	370.3	mg/L
Inlet particulate COD concentration to biological treatment	$X_{CSB,ZB}$	222.2	mg/L
Inlet soluble COD concentration to biological treatment	$S_{CSB,ZB}$	148.1	mg/L
formed biomass	$X_{CSB,BM}$	50.2	mg/L
endogenous decay of the biomass remaining inert solids	$X_{CSB,inert,BM}$	34.6	mg/L
Share of readily degradable COD in degradable COD	f_{CSB}	0.15	-
filterable substances of the inlet	$X_{TS,ZB}$	50.0	mg/L
daily sludge production from the carbon elimination	$ÜS_{d,C}$	18,702	kg/d
Biological phosphorous elimination	$X_{P,BioP}$	2.22	mg/L
Inlet TP concentration	$C_{P,ZB}$	5.77	mg/L
Outlet concentration	$C_{P,AN}$	0.18	mg/L
Phosphorus required for the cell structure of the heterotrophic biomass	$X_{P,BM}$	1.85	mg/L
Precipitated phosphate	$X_{P,Fäll,Fe}$	1.5	mg/L
Sludge from P-elimination	$ÜS_{d,P}$	2.542	kg/d
Total Sludge production	$ÜS_d$	21,245	kg/d
Sludge mass in the reactor	$M_{TS,AT}$	531,119	kg
Total TN inlet concentration	$C_{N,ZB}$	48.0	mg/L
MLSS	MLSS	3.5	g/L
Activated sludge volume	V_{AT}	151,748	m^3

12.4.3 Calculation of the Sludge Mass and A2/O volume (Metcalf & Eddy)

Table 49. Calculation of the sludge mass and required volume of the A2/O stage, according to Metcalf & Eddy

Parameter Norm compliance	Symbol	Value Grade I-A	 CS	Unit	Comment
Temperature	T	12	12	°C	Assumption
total COD in the influent to the biological treatment	tCOD	370.3	370.3	mg/L	
total BOD in the influent to the biological treatment	tBOD	136.5	136.5	mg/L	
Soluble BOD	sBOD	68.25	68.25		50% tBOD
biodegradable COD	bCOD	218.4	218.4	mg/L	bCOD = 1.6 · BOD
soluble COD	sCOD	148.12	148.12	mg/L	40% tCOD
Non-biodegradable COD	nbCOD	151.9	151.9	mg/L	tCOD-bCOD
readily biodegradable COD	rbCOD	333.27	333.27	mg/L	36% + 54% tCOD
slowly biodegradable COD	sbCOD	22.218	22.218	mg/L	6% tCOD
non-biodegradable soluble COD	$nbsCOD_e$	79.87	79.87	mg/L	
Non-biodegradable particulate COD	nbpCOD	72.03	72.03	mg/L	
Suspended solids in the influent of the biological treatment stage	SS	28.22	28.22	mg/L	85% removal in PC, HRT = 2 h
Volatile suspended solids in the influent of the biological treatment stage	VSS	22.58	22.58	mg/L	80% VSS/TSS
COD related Volatile suspended solids	VSS_{COD}	9.84	9.84	mg/L	
non biodegradable volatile suspended solids	nbVSS	7.3195	7.3195	mg/L	
Total Kehldhal nitrogen influent	TKN	51.1	51.1	mg/L	
Ammonium nitrogen in the influent	NH_4-N	37.7	37.7	mg/L	
Organic nitrogen	OrgN	13.4	13.4	mg/L	
Half velocity constant for ammonium nitrogen	K_{NH4}	0.5	0.5	mg/L	
Target effluent concentration for ammonium nitrogen	$NH_4\text{-}N_{,out}$	**1.5**	**0.5**	mg/L	Assumption
maximum growth rate autotrophic bacteria	$\mu_{max,\ AOB}$	0.520	0.520	g/g*d	
specific endogenous decay coefficient autotrophic bacteria	b_{AOB}	0.135	0.135	g VSS / gVSS ·d	
Dissolved oxygen conc. in the aeration tanks	DO	2	2	mg/L	

Table 49. (continued) Calculation of the sludge mass and required volume of the A2/O stage, according to Metcalf & Eddy

Parameter	Symbol	Value		Unit	Comment
Norm compliance		Grade I-A	CS		
Dissolved oxygen half velocity constant	$K_{o,AOB}$	0.5	0.5	mg/L	
Growth rate autotrophic bacteria	μ_{AOB}	0.177	0.073	g/g·d	
Theoretical SRT	SRT_{theo}	5.7	13.8	d	
Peak to average TKN Load	SF	2.80	2.80	d	
Corrected SRT	SRT	15.8	38.5	d	
Influent flowrate to the WWTP	Q	149,588	149,588	m^3/d	
Heterotrophic bacteria synthesis yield coefficient	Y_H	0.45	0.45	g VSS / gVSS ·d	
Substrate (biodegradable BOD)	S_0	218.4	218.4	mg/L	
specific endogenous decay coefficient heterotrophic bacteria, 20°C	$b_{H,20}$	0.12	0.12	1/d	
specific endogenous decay coefficient heterotrophic bacteria	b_H	0.088	0.088	g/g·d	
maximum growth rate COD oxidation, temperature corrected	$\mu_{m,T}$	3.492	3.492	1/d	
Half velocity constant for COD oxidation	$K_{s,COD}$	8.0	8.0	mg/L	
fraction of biomass that remains as cell debris	f_d	0.15	0.15	g VSS/ g biomass VSS	
Substrate	S	0.361	0.269	mg/L	
Nitrification yield coefficient	Y_n	0.150	0.150	g VSS / gVSS ·d	
Sludge production	$P_{x,bio}$	7,717	5,204	kg VSS/d	
Sludge production	$P_{X,VSS}$	8,812	6,299	kg VSS/d	
Sludge production	$P_{X,TSS}$	11,014	7,873	kg TSS/d	80% VSS/TSS
MLSS	X_{TSS}	3,500	3,500	mg/L	assumption
Nitrification volume	V_N	49,835	86,562	m^3	
Hydraulic retention time nitrification tanks	HRT_{nitri}	8.0	13.9	h	
Oxidized NO_x (produced in nitrification)	$NO_{3,nitri}$	40.9	40.9	mg/L	80% of influent TKN
Target NO3-N effluent	$NO_{3,out}$	10	7	mg/L	Assumption
Return sludge recirculation rate	R	0.75	0.75	-	Assumption

Table 49. (continued) Calculation of the sludge mass and required volume of the A2/O stage, according to Metcalf & Eddy

Parameter Norm compliance	Symbol	Value		Unit	Comment
		Grade I-A	CS		
internal recycle ratio	IR	2.34	4.09	-	
F/M Nitrification	F/M	0.32	0.18	g COD /(g MLSS·d)	
Denitrification HRT as percentage of nitrification	% HRT_{deni}	11.5%	31.0%	%	
Denitrification HRT	HRT_{deni}	0.92	4.31	h	
Total activated sludge volume	V_{AT}	55,566	113,396	m^3	
Denitrification proportion	V_D/V_{AT}	0.10	0.24	-	From WWTP design
Denitrification volume	V_D	5,731	26,834	m^3	
Food to microorganisms ratio, BOD based	F/M_b	1.82	0.51	g BOD /(g MLSS·d)	
Specific denitrification rate	$SDNR_b$	0.37	0.16	g NO_3-N/(g MLVSS ·d)	
Mixed liquor volatile suspended solids	MLVSS	2,800	2,800	mg/L	80% VSS/TSS
Mixed liquor volatile suspended solids denitrification	$MLVSS_b$	1,956	1,494	mg/L	
Specific denitrification rate	SDNR	0.256	0.086		
Influent flow to anoxic tank	$Q_{in, anox}$	461,943	724,028	m^3/d	
Nitrate nitrogen contained in recirculation sludge	NO_3-N_{RAS}	8	8	mg/L	Assumption
Nitrate to denitrify	NO_X-N_{feed}	3,695,546	5,792,223	g/d	
Denitrification capacity	NO_r	4,115,472	6,442,562	g/d	

12.5 Model Fit Calculation

Table 50. Comparison of the statistical values for the selected evaluation parameters

Parameter	COD, mg/L		TN, mg/L		NO_3-N, mg/L		NH_4-N, mg/L		MLSS, mg/L	
	O	M	O	M	O	M	O	M	O	M
Mean	16.96	17.15	8.11	6.86	7.01	7.00	0.303	0.164	5.748	5.891
Median	17.00	17.74	8.11	6.77	7.09	6.92	0.132	0.069	5.893	5.922
85%-Quantile	20.00	20.33	9.78	9.17	7.84	8.88	0.445	0.211	7.204	6.447
SD	3.55	3.11	1.51	2.15	1.32	2.16	0.505	0.465	1.295	534

12.6 Sensitivity Analysis

Table 51. Sensitivity analysis in the example WWTP pre-calibration model

Para-meter	Base value	Value + 10%	NO_3-N	S_{NO3}	NH_4-N	S_{NH4}	COD	S_{COD}	Sludge prod.*	S_{sludge}
			mg/L	-	mg/L	-	mg/L	-	Mg/d	
		Base	6.2755	-	0.2536	-	17.356	-	19.97	-
TSS/ COD	0.475	0.5225	6.2997	0.51	0.2566	0.06	17.354	-0.04	20.97	20.91
f_B	0.3	0.33	6.2605	-0.50	0.2537	0.00	17.351	-0.17	20.49	17.10
f_S	0.05	0.055	6.3041	5.72	0.2528	-0.17	19.055	339.80	19.92	-11.34
f_A	0.3	0.33	6.3039	0.95	0.2547	0.04	17.358	0.07	20.31	11.27
RS	120,000	132,000	6.1641	0.00	0.2531	0.00	17.36	0.00	20.29	0.00
RZ	240,000	264,000	6.1337	0.00	0.2533	0.00	17.357	0.00	19.97	0.00
Q_{air}	$9.04 \cdot 10^5$	$1.04 \cdot 10^6$	6.2755	0.00	0.2526	0.00	17.356	0.00	19.97	0.00
DO_{sp}	3	3.3	6.3534	0.26	0.2491	-0.02	17.355	0.00	19.97	-0.02
SRT	27.75	30.525	6.3004	0.01	0.2439	0.00	17.369	0.00	19.45	-0.19

*Sludge production = Excess sludge + primary sludge

12.7 Estimation of the Excess Sludge Production

The excess sludge production as VSS and TSS (see Figure 71) was estimated according to Metcalf & Eddy (Tchobanoglous op. 2014):

$$P_X = Y \cdot Q \cdot (S_0 - S) \qquad \text{Equation 8}$$

Where:

S = BOD

Y = 0.5 kg VSS /kg BOD removed

Q = inlet flow activated sludge = $Q_{in} + Q_{RS}$

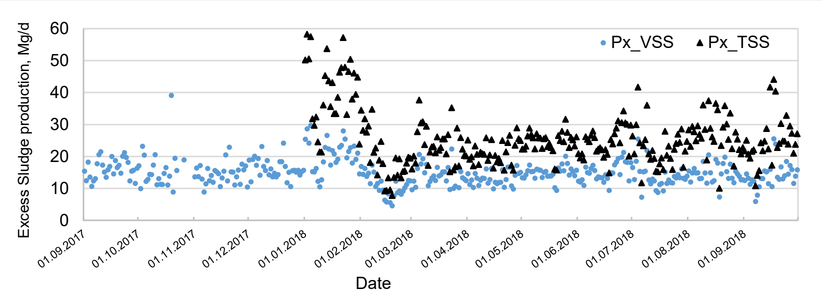

Figure 71. Estimation of the excess sludge production according to M&E

12.8 COD in the Simulated Scenarios in Chapter 5

The COD concentration in the effluent in selected scenarios is presented in Figure 72. The remaining scenarios, not shown here, show identical trends.

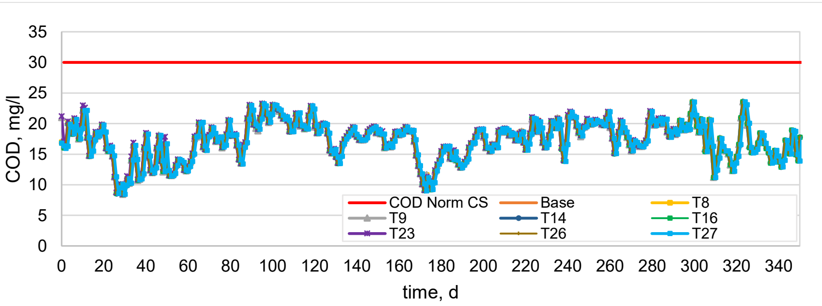

Figure 72. COD in the effluent in selected scenarios from T8 to T27

12.9 Water Recirculation in Combination Scenarios

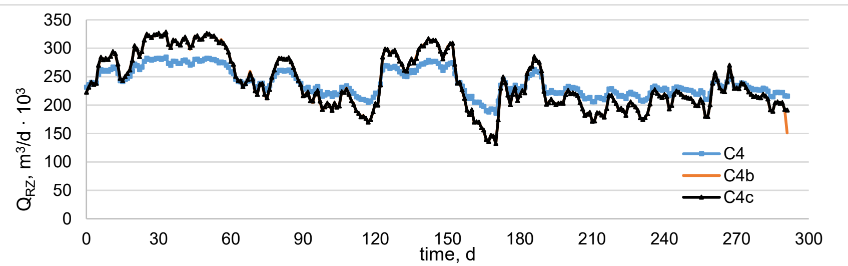

Figure 73. Comparison of the water recirculation (Q_{RZ}) in scenarios C4, C4b and C4c

12.10 Design of an Anaerobic Sludge Stabilisation Stage

Table 52. Calculation of the sludge production for the example WWTP with anaerobic sludge stabilization

	Parameter	Symbol	Values	Unit	Comment
Primary sludge	Primary sludge flowrate	Q_{PS}	539	m³/d	From plant data
	Primary sludge concentration	TS_{PS}	28.3	g/l	From plant data
	Daily primary sludge production	$F_{PS,d}$	15,254	kg/d	Calculated
Sludge production from carbon elimination (Activated sludge)	Temperature	T	12	°C	Assumption
	Sludge age (SRT)	SRT	8.22	d	According to (DWA-A 131)
	Sludge production factor	F_T	0.8	-	$FT = 1{,}072^{(T-15)}$
	Proportion of inorganic substances in the filterable substances	f_b	0.2	-	for pre-treated wastewater
	Daily wastewater inlet flow	$Q_{in,d}$	149,588	m³/d	85% percentile
	Inert particulate inlet COD	$X_{COD,\, i,AT}$	66.7	mg/l	
	Fraction of inert COD from particulate COD	f_A	0.3	-	Between 0.2 – 0.35. Recommended: 0.3 (DWA-A 131)
	Inlet COD concentration to biological treatment	$C_{COD,AT}$	370.3	mg/l	
	Inlet particulate COD conc. to biological treatment	$X_{COD,AT}$	222.2	mg/l	60% of the inlet COD to biological treatment
	Inlet soluble COD conc. to biological treatment	$S_{COD,AT}$	148.1	mg/l	40% of the inlet COD to biological treatment
	Fraction of SS from biodegradable COD	f_{COD}	0.15	-	0.15 – 0.25 (DWA-A 131)
	Filterable substances of the inlet	$X_{TS,AT}$	50.0	mg/l	
	Formed biomass	$X_{COD,biomass}$	104.6	mg/l	
	Endogenous decay of the biomass remaining inert solids	$X_{COD,I,\, biomass}$	23.7	mg/l	
	Daily excess sludge production from carbon elimination	$\mathbf{F_{ESd,C}}$	23,688	kg/d	According to Equation 3
Sludge production from biological P-elimination	Biological phosphorous elimination	$X_{P,BioP}$	2.22	mg/l	$X_{P,BioP} = 0.006 \cdot C_{COD,AT}$
	Inlet TP concentration	$C_{P,AT}$	5.77	mg/l	
	Outlet concentration	$C_{P,AN}$	0.18	mg/l	60% of the maximum discharge value
	Phosphorus required for the cell structure of heterotrophic biomass	$X_{P,biomass}$	1.85	mg/l	$X_{P,biomass} = 0.005\ C_{COD,AT}$
	Backload P	$C_{P,AT,BL}$	0.29	mg/l	5% of the inlet TP (Fimml 2010)
	Inlet TP concentration	$C_{P,AT + BL}$	6.06	mg/l	
	Precipitated phosphate	$X_{P,ppt,Fe}$	1.8	mg/l	
	Sludge from biological P-elimination	$\mathbf{F_{ESd,P}}$	2,836	kg/d	According to Equation 3
Tertiary sludge	Tertiary sludge flowrate	Q_{TerS}	300	m³/d	From plant data
	Tertiary sludge concentration	TS_{TerS}	5.6	g/l	From plant data
	Daily tertiary sludge production	$\mathbf{TerS_d}$	1,670	kg/d	Calculated

$$F_{ES,d,C} = Q_{in,d} * \left(\frac{X_{COD,i,AT}}{1,33} + \frac{X_{CSB,BM} + X_{COD,i,biomass}}{0.92 * 1.42} + X_{i,TS,AT}\right)/1000$$ Equation 9

$$F_{ES,d,P} = Q_{in,d} * \left(3 * X_{P,BioP} + 6.8 * X_{P,ppt,Fe}\right)/1000$$ Equation 10

The estimated excess sludge production is 26,524 kg/d ($F_{ES,d,C}$ +$F_{ES,d,P}$) and the total sludge to thickening ($F_{PS,d}$ + $F_{ES,d}$ + $F_{TerS,d}$) is 41,778 kg/d.

12.10.1 Digester and Peripheral Equipment Dimensioning

Egg-shape digester

An Egg volume and area are calculated according to Equations 11 and 12 and Figure 74.

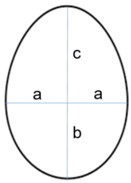

Figure 74. Egg shape dimensions

$$A = 2 * \pi * a^2 + \pi * a\left(\frac{b^2}{\sqrt{b^2 - a^2}} * \cos^{-1}\left(\frac{a}{b}\right) + \frac{c^2}{\sqrt{c^2 - a^2}} * \cos^{-1}\left(\frac{a}{c}\right)\right)$$ Equation 11

$$V = \frac{2 * \pi}{3} * a^2 * (b + c)$$ Equation 12

Table 53. Dimensions of anaerobic digesters

Parameter	Symbol	Value	Unit
Total reactors volume	V	22,000	m^3
Number of reactors	n	2	-
Individual digester volume (goal)	$V_{i,goal}$	11,000	m^3
Equatorial radius	a	13.0	m
Short polar radius	b	15.1	m
Long polar radius	c	16.0	m
Total height	H	31.1	m
Individual digester volume (obtained)	V_i	11,008	m^3
Footprint area of each reactor	S_i	531	m^2
Egg area (egg surface)	Ai	2,406	m^2

Centrate and sludge tank

Table 54. Dimensions of the sludge and centrate tanks

Parameter		Symbol	Value	Unit
Sludge tank	Volume	V	2,000	m^3
	Number of tanks	n	1	
	Radius	R	10	m
	Height	H	6.4	m
	Footprint area of each tank	S_i	314.2	m^2
Centrate Tank	Volume	V	3,800	m^3
	Number of tanks	n	1	
	Radius	R	10	m
	Height	H	12.1	m
	Footprint area of each tank	S_i	314.2	m^2

12.10.2 Thermal Balance

The heat losses are calculated in Equation 13.

$$H_L = U \cdot A \cdot \Delta T \qquad \text{Equation 13}$$

Where:

H_L = heat loss, W

U = overall coefficient of heat transfer, W / m^2 ·°C

A = area through which heat loss is occurring, m^2

ΔT = temperature drop across the surface in question, °C or K

The overall coefficient of heat transfer depends on the construction material of the digester, as described in Table 55. It was tested that, even in a scenario with high heat losses (concrete with insulation and concrete floor in contact with moist earth) the heat produced is enough to heat the digester in the winter months, with a temperature of 4°C (see Table 56).

Table 55. Coefficient for heat transfer according to materials (Tchobanoglous op. 2014)

Coefficient for heat transfer (U)	Units	Value
Concrete walls, 300 mm thick, insulated (above ground)	W / m^2 ·°C	0.6 – 0.8
Concrete walls, 300 mm thick, not insulated (above ground)	W / m^2 ·°C	4.7 – 5.1
Concrete floor 300 mm thick (in contact with dry earth)	W / m^2 ·°C	1.7
Concrete floor 300 mm thick (in contact with moist earth)	W / m^2 ·°C	2.85
Fixed concrete cover with 25 mm insulation	W / m^2 ·°C	0.9 – 1.0

Table 56. Thermal energy balance in the worst-case scenario

Parameter		Worst case scenario	Units
Air temperature (minimum)	T_{air}	4	°C
Overall coefficient of heat transfer, above ground	U_{above}	0.8	W/(m² ·C°)
Overall coefficient of heat transfer, under ground	U_{below}	2.85	W/(m² ·C°)
Individual reactor surface	A_i	2,406	m²
Total surface	A	4,813	m²
Total surface above ground	A_{above}	3,850	m²
Total surface below ground	A_{below}	963	m²
Temperature of the reactor	$T_{reactor}$	37	°C
Temperature difference between air and reactor	ΔT	33	°C
Heat loss	H_L	192.2	kW
Thermal energy required	$E_{th,required}$	4,612	kWh
Thermal energy produced	$E_{th,produced}$	14,671	kWh
Percentage of the thermal energy consumed	%	31.4%	-

12.11 Design of an SBR

12.11.1 Determination of Design Parameters and Effluent Quality Requirements

This process was done in Section 3.4, based on data between 2017 and 2019, according to worksheet ATV DVWK-A 198 (ATV-DVWK 2003). The biological stage was designed to comply with the norm City Assessment Standard (CS).

12.11.2 Determination and Calculation of the Process Parameters

This calculation is done according to the worksheet DWA-A 131 (DWA 2016) in the following steps:

Determination of the volume fraction for denitrification in an iterative process: In an iterative process, the chosen V_D/V_{AT} ideal for the treatment of the example wastewater with aerobic sludge stabilization is 0.44. When considering anaerobic sludge stabilization, this value increases slightly to 0.46.

Calculation or selection of the sludge age SRT, according to the intended treatment target according to worksheet DWA-A 131: The sludge age for aerobic sludge stabilization and denitrification, is described in Section 3.4. For the selected design temperature of 12 °C, it is 25 days. For anaerobic sludge stabilization, the sludge age is calculated as described in 6.1, and it is 12.6 days.

Determination of sludge production: The wastewater fractionation is calculated based on the 85%-percentile of the inlet COD concentration between the years 2017 and 2019, as detailed in Annex 12.4.1. The design was carried out for an activated sludge plant with upstream denitrification. For the P-elimination, a biological P-removal and chemical Phosphorous precipitation with a dosing of Ferric salts were considered.

Sludge production with anaerobic sludge stabilisation stage

Due to the size plant, an anaerobic sludge stabilization is recommended, as described in previous chapters. To estimate the backload from anaerobic digestion, 1.5 g N/(PE·d) (Fimml 2010) was applied. For a plant size of 345,000 $PE_{BOD,60}$, approx. 4.9 mg N/L in the return sludge liquor are estimated. The sludge production and nitrogen balance calculation are detailed in Table 57.

Table 57. Sludge production, Nitrogen balance and determination of the denitrification capacity under different operational conditions with aerobic and anaerobic sludge stabilization

	Parameter	Symbol	Values			Unit	Comment
	Scenario		**SBR-0**	**SBR-1**	**SBR-2 AD**		
	Cycle duration	t_{cycle}	**8**	**6**	**8**	h	
General parameters	**Sludge stabilisation**		**Aerobic**	**Aerobic**	**Anaerobic**	-	
	Denitrification volume proportion	V_D/V_{AT}	0.44	0.44	0.45	-	X=1.00 (denitrification requirements)
	Temperature	T	12	12	12	°C	Assumption
	Sludge age (Sludge retention time)	SRT	25.0	25.0	12.4	d	According to DWA-A 131
Sludge production from carbon elimination	Proportion of inorganic substances in the filterable substances	f_b	0.2	0.2	0.2	-	for pre-treated wastewater
	Fraction of inert COD from particulate COD	f_A	0.3	0.3	0.3	-	Recommended: 0.3 (DWA-A 131)
	Daily wastewater influent flow	$Q_{in,d}$	149,588	149,588	149,588	m^3/d	85% percentile
	Inert particulate influent COD	$X_{COD,inert,AT}$	66.7	66.7	66.7	mg/l	
	Inlet COD concentration to biological treatment	$C_{COD,AT}$	370.3	370.3	370.3	mg/l	
	Inlet particulate COD conc. to biological treatment	$X_{COD,AT}$	222.2	222.2	222.2	mg/l	60% of the inlet COD to bio. treatment
	Inlet soluble COD concentration to biological treatment	S_{COD_AT}	148.1	148.1	148.1	mg/l	40% of the inlet COD to bio. treatment
	Fraction of SS from biodegradable COD	f_{COD}	0.15	0.15	0.15	-	0,.15 – 0.25 (DWA-A 131)
	Filterable substances of the inlet	$X_{TSS,AT}$	50.0	50.0	50.0	mg/l	
	Formed biomass	$X_{COD,biomass}$	50.2	50.2	85.2	mg/l	
	Endogenous decay of the biomass remaining inert solids	$X_{COD,i,biomass}$	34.6	34.6	27.6	mg/l	
	Daily sludge production from carbon elimination	$F_{ES,d,C}$	18,702	18,702	21,913	kg/d	

Table 57 (continued). Sludge production, Nitrogen balance and determination of the denitrification capacity under different operational conditions with aerobic and anaerobic sludge stabilization

	Parameter	Symbol	Values			Unit	Comment
	Scenario		**SBR-0**	**SBR-1**	**SBR-2 AD**		
	Cycle duration	t_{cycle}	**8**	**6**	**8**	h	
General para	**Sludge stabilisation**		**Aerobic**	**Aerobic**	**Anaerobic**	-	
	Denitrification volume proportion	V_D/V_{AT}	0.44	0.44	0.45	-	X=1.00 (denitrification requirements)
Sludge production from phosphorous elimination	Biological phosphorous elimination	$X_{P,BioP}$	2.22	2.22	2.22	mg/l	0.006 $C_{COD,AT}$, with BioP (0.005 - 0.007)
	Influent TP concentration	$C_{P,AT}$	5.77	5.77	5.77	mg/l	Influent + backload for anaerobic sludge stabilisation
	Influent TP concentration + backload P	$C_{P,AT,BL}$	-	-	6.06	mg/l	+ 5% of the inlet TP (Fimml 2010)
	Effluent P concentration	$C_{P,AN}$	0.18	0.18	0.18	mg/l	60% of the maximum discharge value CS norm
	P required for the cell structure of heterotrophic biomass	$X_{P,biomass}$	1.85	1.85	1.85	mg/l	0.005 $C_{COD,AT}$
	Precipitated phosphate	$X_{P,Prep,Fe}$	1.5	1.5	1.8	mg/l	
	Sludge from P-elimination	$F_{ES,d,P}$	2,542	2,542	2,836	kg/d	
Nitrogen balance	TN inlet concentration	$C_{N,AT}$	48.0	48.0	48.0	mg/l	85%-percentile
	Backload TN	$C_{N,AT+BL}$	-	-	51.5	mg/l	1.5 g TN/ (PE·d)
	NH_4-N influent concentration	$S_{NH4,N}$	2	2	2	mg/l	85%- percentile
	Effluent concentration of organic nitrogen	$S_{orgN,AN}$	37.7	37.7	37.7	mg/l	DWA-A 131
	Effluent concentration of NH_4-N	$S_{NH4,AN}$	0	0	0	mg/l	Assumption / design
	Concentration of inorganic nitrogen	$S_{inorgN,norm}$	7.5	7.5	7.5	mg/l	Maximum according to CS Norm · 75%
	Effluent concentration of NO_3-N	$S_{NO3,AN}$	5.25	5.25	5.25	mg/l	0.8 to 0.6 $S_{inorgN,norm}$
	N_{org} incorporated into the biomass	$X_{orgN,biomass}$	3.51	3.51	5.97	mg/l	0.07 COD (DWA-A 131); 0.04-0.05 BOD (DWA-M 210)
	N_{org} bound to inert particulate matter	$X_{orgN,inert}$	3.04	3.04	2.83	mg/l	0.03 ·($X_{COD,inert,biomass}$ + $X_{COD,inert,AT}$) (DWA 131)
	NO_3-N concentration to denitrificate	$S_{NO3,D}$	34.2	34.2	35.4	mg/l	$C_{N,AT}$ - ($S_{orgN,AN}$ + $S_{NH4,AN}$ + $S_{NO3,AN}$ + $X_{orgN,biomass}$ + $X_{orgN,inert}$) (DWA-A 131)
	Sludge mass in the reactor	$\mathbf{M_{TS,AT}}$	531,119	531,119	418,776	kg	

12.11.3 Definition of Process Design, Cycle Strategy and Design Parameters

Upstream equalisation tank

The process will have an equalization tank upstream and downstream of the biological treatment step. The equalization tanks will serve to equalize 4 hours at the average flowrate, i.e. 20,000 m^3.

Determination of the reactor volume

For the number of reactors, at least two tanks are necessary, for two reasons: (1) to have enough operational safety and flexibility; (2) according to the size plant, a single reactor would be too large and mixing problems could arise. The plant is designed for an alternating, continuous feeding, with the treatment goals: carbon elimination, nitrification, denitrification, and biological phosphorus elimination. For sludge stabilization, both aerobic and anaerobic sludge stabilisation strategies are calculated.

The calculation is carried out for two conditions: the 85%-percentile conditions, and the average conditions. It is assumed that the average conditions are like “dry weather” conditions. The required volumes and further parameters are calculated based on the guideline DWA-M 210 (DWA 2009) and are detailed in Table 58. To avoid too large reactors volumes, and comply with the usually applied heights, eight reactors (n=8), were chosen.

The required biomass in the reactor is around 770 Mg for aerobic sludge stabilisation and round 440 Mg for anaerobic, and it is calculated as:

$$\mathrm{M_{TS,SBR}} = \mathrm{M_{TS,AT}} * \left(\frac{\mathrm{t_{cycle}}}{\mathrm{t_R}}\right) \qquad \text{Equation 14}$$

Where:

$M_{TS,SBR}$ = required mass in the reactor in an SBR system, kg

$M_{TS,AT}$ = required mass in the reactor in an activated sludge system, kg

t_{cycle} = total SBR cycle time, h

t_R = reaction time (nitrification and denitrification time), h

Table 58. Parameters for the determination of the reactor volume for an SBR system with aerobic and anaerobic sludge stabilisation at 12 °C, for cycles of 8 and 6 hours

	Parameter	Symbol	Values			Unit
			SBR-0	SBR-1	SBR-2 AD	
General parameters	Sludge stabilisation	-	Aerobic	Aerobic	Anaerobic	-
	Temperature	T	12		12	°C
	Cycle length	t_Z	8		8	h
	Sludge age	SRT	25.0	25.0	12.4	d
	Sludge mass in an activated sludge reactor	$M_{TS,AT}$	531.1	531.1	303.2	t
	Sludge mass in the SBR	$M_{TS,SBR}$	765.6	838.6	437.0	t
85%-Percentile conditions	Inlet flow (85%-value)	Q_{in}	6,233	6,233	6,233	m^3/h
	Sludge concentration at minimum volume (V_{min})	TS_{min}	5	5	5	g/L
	Number of reactors	n	8	8	8	-
	Volume after completion of the clear water discharge	V_{min}	19,139	20,965	10,924	m^3
	Maximum feed volume discharged per cycle	ΔV_{max}	6,233	4,675	6,233	m^3
	Volume exchange ratio	f_{AA}	0.25	0.18	0.36	-
	Total solids concentration in the reactor	TS_R	3.77	4.09	3.18	kg/m^3
Average conditions (TW) conditions	Inlet flow (average or TW)	$Q_{in,TW}$	5,060	5,060	5,060	m^3/h
	Maximum feed volume discharged per cycle	$\Delta V_{max,TW}$	5,060	3,795	5,060	m^3
	Reactor volume (TW)	$V_{R,TW}$	24,200	24,760	15,985	m^3
	Volume exchange ratio	$f_{AA,TW}$	0.21	0.15	0.32	
	Reactor volume per unit	$V_{R,unit,TW}$	3,025	3,095	1,998	m^3
	Total solids concentration in the reactor	$TS_{R,TW}$	3.95	4.23	3.42	kg/m^3
Reactor dimensions	Reactor volume	V_R	25,372	25,640	17.157	m^3
	Reactor volume per unit	$V_{R,total}$	202,978	205,119	137.257	m^3
	Reactor diameter	D	57	25	47	m
	Reactor height	H	10	6.5	10,0	m
	Reactor cross-sectional area	A	2537	491	1716	m^2

Calculation of sludge settling

Table 59 summarizes the verification of the required minimum distance between the sludge level and the lower decanter opening during the decanting phase, separately for dry and rainy weather conditions. The flocculation time is 10 min, according to the suggestion in the DWA-M 210. The minimum required clear water height is calculated as 15% of the sludge level at the beginning and end of settling.

As can be observed in Table 59, in both conditions (85%-quantile and dry weather), and for both sludge stabilisation paths, the clear water height is sufficient.

Determination of the denitrification capacity and other technical requirements (aeration and excess sludge)

The technical requirements for denitrification are verified and summarized in Table 60. This is carried out by comparing the oxygen supply and demand, considering the parameter x = 1, with:

$$x = \frac{\mathrm{OV}_{\mathrm{C,D}}}{2{,}86 * S_{NO3,D}}$$ Equation 15

Where:

$OV_{C,D}$ = Oxygen demand equivalent in denitrification (oxygen consumption of carbon elimination covered by nitrate oxygen), mg/l

$S_{NO3,D}$ = oxygen supply from denitrification, mg/l

According to the calculation, two denitrification cycles are required in both scenarios to comply with the discharge norm for nitrogen. The oxygen demand is calculated for a WWTP with upstream denitrification. The oxygen demand and excess sludge production and pump requirements are also summarized in Table 60.

Table 59. Calculation of sludge settling requirements in an SBR system with aerobic and anaerobic sludge stabilisation at 12 °C, for 8 h cycles

	Parameter	Symbol	Values			Unit
	Scenario		**SBR-0**	**SBR-1**	**SBR-2 AD**	
	Sludge stabilisation		Aerobic	Aerobic	Anaerobic	
	Cycle length	t_Z	8	6	8	h
85%-Percentile condition	Sludge level at the beginning of settling	$H_{w,0}$	9.50	6.50	9.50	m
	Sludge level at the end of settling	$H_{w,e}$	7.17	5.31	6.05	m
	Decanter capacity	Q_{Ab}	13,851	10,388	13,851	m^3/h
	Relative final sludge level height, related to $H_{W,0}$	$h_{S,e}$	0.47	0.51	0.40	-
	Sludge level at the beginning of the settling process	$v_{s,0}$	1.96	1.77	2.44	m/h
	Progress parameters of the e-function	alpha	0.39	0.56	0.46	1/h
	Begin of the decanting time	$t_{(1h)}$	1.00	1.00	1.00	h
	End of the decanting time	$t_{(2h)}$	2.00	2.00	2.00	h
	Flocculation time	t_{flock}	0.17	0.17	0.17	h
	Height of the sludge level before decanting	$H_{s(1h)}$	6.87	4.09	6.04	m
	Height of the sludge level after decanting	$H_{s(2h)}$	4.65	2.35	3.82	m
	Clear water height at decanting beginning	$H_{KW(1h)}$	2.63	2.41	2.81	m
	Clear water height at decanting end	$H_{KW(2h)}$	4.85	4.15	5.03	m
Average conditions (TW)	Inlet flow (TW)	$Q_{in,TW}$	5,060	5,060	5,060	m^3/h
	Maximum feed volume discharged per cycle (TW)	$\Delta V_{max,TW}$	5,060	3,795	5,060	m^3
	Reactor volume (TW)	$V_{R,TW}$	24,200	24,760	15,985	m^3
	Volume exchange ratio	$f_{AA,TW}$	0.21	0.15	0.32	
	Reactor volume per unit	$V_{R,unit,TW}$	3,025	3,095	1,998	m^3
	Total solids concentration in the reactor	$TS_{R,TW}$	3.95	4.23	3.42	kg/m^3
	Sludge level at the beginning of settling (TW)	$H_{w,0,TW}$	6.27	3.77	8.85	m
	Sludge level at the end of settling (TW)	$H_{w,e,TW}$	4.03	2.04	6.05	
	Sludge decanting time (TW)	$t_{Ab,TW}$	0.37	0.37	0.37	h
	Volumetric sludge index (TW)	SVI	101.86	101.86	101.86	ml/g
	Relative final sludge level height, related to $H_{W,0}$ (TW)	$h_{s,e,TW}$	0.40	0.43	0.90	-
	Sludge level at the beginning of the settling process (TW)	$v_{s,0,TW}$	2.39	2.19	2.92	m/h
	Progress parameters of the e-function	$alpha_{TW}$	0.44	0.61	3.35	1/h
	Begin of the decanting time	$t_{1h,TW}$	1.00	1.00	1.00	h
	End of the decanting time	$t_{2h,TW}$	2.00	2.00	2.00	h
	Flocculation time	$t_{flock,TW}$	0.17	0.17	0.17	h
	Height of the sludge level before decanting (TW)	$H_{s,1h,TW}$	6.27	3.77	0.54	m
	Height of the sludge level after decanting (TW)	$H_{s,2h,TW}$	4.03	2.04	0.02	m
	Clear water height at decanting beginning (TW)	$H_{KW,1h,TW}$	2.79	2.51	8.31	m
	Clear water height at decanting end (TW)	$H_{KW,2h,TW}$	5.03	4.24	8.83	m

Table 60. Determination of the denitrification capacity, and technical requirements of an SBR system with aerobic and anaerobic sludge stabilisation at 12 °C, for the 8 hours cycles

	Parameter	Symbol	Values			Unit
General	Sludge stabilisation	-	Aerobic	Aerobic	Anaerobic	
	Temperature	T	12	12	12	°C
	Cycle length	t_Z	8	6	8	-
	Denitrification proportion	V_D/V_{AT}	0.44	0.44	0.45	-
Denitrification capacity	Concentration of nitrogen to nitrification	$S_{NH4,N}$	37.7	37.7	37.7	ml/g
	Concentration of nitrogen to denitrification	$S_{NO3,D}$	34.2	34.2	35.6	ml/g
	Effluent nitrate (z =1)	$S_{NO3,AN\ (z=1)}$	9.3	6.9	12.9	mg/l
	Effluent nitrate (z =2)	$S_{NO3,AN\ (z=2)}$	4.6	3.4	6.5	ml/g
	Number of nitrification or denitrification phases during a cycle	z	2	2	2	-
Oxygen demand, Q_{in} 85%	Oxygen demand for oxidation of carbon	$OV_{d,C}$	32,241	32,255	33,304	kg O_2/d
	Oxygen demand for nitrification	$OV_{d,N}$	24,781	24,781	22,300	kg O_2/d
	Oxygen demand for denitrification	$OV_{d,D}$	14,836	14,836	15,440	kg O_2/d
	Total oxygen demand per hour	OV_h	2,922	9,574	3,082	kg O_2/h
	Impact factor for carbon respiration	f_c	1	1	1	-
	Impact factor of the nitrogen load	f_N	1.5	1.5	1.5	-
Excess sludge	Flow rate of activated sludge discharged per cycle	$Q_{ES,z}$	88.5	66.4	102.1	m^3/cycle
	Excess sludge daily flow	$Q_{ES,d}$	265.6	265.6	306.2	m^3/d
	Sludge extraction time	t_{ES}	0.30	0.41	0.44	h
	Required pump capacity, excess sludge	$Q_{Pump,ES}$	300.0	160.0	230.0	m^3/h
	Total solids excess sludge (min)	TS_{ES}	8.03	8.03	8.03	g/l
	Total daily mass of excess sludge	$F_{ES,d}$	21,245	21,245	24,496	kg/d

Definition of the SBR system cycle

The cycle parameters are summarized in Table 61. Two cycles were designed: 8 and 6 hours, for dry and rainy weather, respectively.

Table 61. Cycle times of the SBR system with aerobic and anaerobic sludge stabilisation at 12 °C, for cycles of 8 and 6 hours

Parameter	Symbol	Time, in hours			Comments
Sludge stabilisation	-	Aerobic	Aerobic	Anaerobic	
Cycle duration	t_Z	8	6	8	Assumption. Cycle duration: 6- 8 (between 4 and 12)
Duration of the sedimentation phase	t_{Sed}	1.00	1.00	1.00	Assumption.
Duration of the clear water removal phase	t_{Ab}	0.45	0.45	0.45	Usually only 15 to 60 minutes
Total duration of the filling phase	t_F	1.00	0.75	1.00	Static filling $t_F = t_{cycle}/n$, with stirring (for BioP)
Duration of the 1st filling phase	t_{F1}	0.65	0.45	0.60	
Duration of the 2nd filling phase	t_{F2}	0.35	0.30	0.40	
Duration of the idle time	t_{idle}	0.00	0.00	0.00	Assumption
Total duration of the denitrification phase	t_D	2.44	1.67	2.48	$t_R \cdot (V_D/V_{AT})$
Duration of the 1st denitrification phase	t_{D1}	1.59	1.09	1.20	
Duration of the 2nd denitrification phase	t_{D2}	0.85	0.58	1.28	
Total duration of the nitrification phase	t_N	3.11	2.12	3.07	$t_R - t_D$
Duration of the 1st nitrification phase	t_{N1}	2.02	1.38	1.50	
Duration of the 2nd nitrification phase	t_{N2}	1.09	0.74	1.57	
Duration of the reaction phase	t_R	5.55	3.80	5.55	$t_D + t_N$

Effluent equalization tank

After the SBR treatment stage, an equalization step is advisable. This can be a simple equalisation tank or even a polishing pond. According to the recommendation of the DWA- M 210, the HRT there should not exceed 2 days in dry weather conditions. Different tank sizes were tested in the simulation and a volume of 20,000 m^3 for the scenarios SBR-0 and SBR-1 and 40,000 m^3 for SBR-2 AD were chosen.

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2020 – 2022	**Lecturer**, Otto von Guericke University Magdeburg, Faculty for Process And Systems Engineering, Institute for Equipment and Environmental Technology.
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