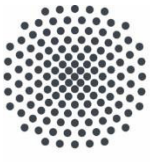

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EVALUATION OF SLOW SAND FILTRATION FOR MUNICIPAL WASTEWATER WITH A TRICKLING FILTER AS PRIMARY TREATMENT

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Scope of work

This research project aims to determinate the efficiency of slow sand filters as an additional step in municipal wastewater treatment plants with trickling filters as primary treatment. Physico-chemical and microbiological parameters are monitored in the trickling filter and slow sand filters. Different organic loads for the trickling filter allow the evaluation of different qualities of wastewater from the primary system and different hydraulic loads in the slow sand filters allowed to evaluate different operational conditions.

The results and the operational and maintenance procedures carried on during the experimental stage at the pilot and laboratory scale will be useful to determine the limits of the slow sand filters to remove key parameters in water quality with the aim of reuse, as well as to evaluate the optimal hydraulic load for the filters and the degradation procedures that occur during the initial stage until the stabilization of the systems.

The results can evidence the limitation of slow sand filtration as a final stage treatment and deepen the understanding on slow sand filters management. Further research can use this information to apply the experimental condition to a larger scale and ensure an optimal performance.

Summary / Abstract

Trickling filters and slow sand filters are low-cost treatment technology for wastewater that decreased in popularity due to the development of more complex systems that included aeration and deal better with higher organic and hydraulic loads. For small communities, the cost involved in the construction and operation of different systems, such as activated sludge technology, has affected the long-term viability of the systems or has forced the implementation of decentralized systems where the reuse of treated wastewater is not considered. In addition, slow sand filtration is a method mainly used for drinking water treatment, and its use for wastewater has been so far limited. Municipal wastewater treatment and reuse in agriculture for small communities is one of the most effective ways to fulfill treatment requirements, reduce pollution risks and reduce water demand in vulnerable areas. The present study aimed to combine two low-cost treatment alternatives, such as trickling filter and slow sand filter technology, to determine its viability for pollutant removal for water reuse. The pilot scale system included a trickling filter with a hydraulic load of 1.1 $m^3/(m^2*h)$ and three different organic loads (0.9, 0.64 and 0.67 Kg $BOD/(d*m^3)$) for three different phases, a rapid sand filter and three slow sand filters of 0.10 m diameter, 0.6 m of sand column and three different hydraulic loads of 0.15, 0.3 and 0.6 m/h.

The trickling filter reported removal rates for COD, TN, TP, *E. coli*, Total Coliforms (TC) and Enterococcus of between 80—88 %, 30—40 % and 16—32 %, 0.9—2.3 log_{10} and 1.2—2.3 log_{10}, 1.5—3.3 log_{10}, respectively. Regarding the slow sand filters, the removal rates for COD, TN, *E. coli*, Total Coliforms (TC) and Enterococcus of 18—23 %, 3—6 %, 0.8—1.35 log_{10}, 0.3—1.58 log_{10} and 0.1—1.3 log_{10}. There was not removal of TP by the slow sand filter. Although, the slow sand filter with the lowest hydraulic load (0.15 m/h) was probed to be the most effective for contaminant removal, the filter with a hydraulic load of 0.3 m/h showed similar removal values with pathogen removal values below the limits set by the World Health Organization (1000 CFU/100 mL), and less maintenance requirements as the filter with 0.6 m/h.

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List of abbreviations and symbols

BOD	Biochemical Oxygen Demand
BOD_5	5-day biochemical oxygen demand
CFU	Colony Forming Unit
COD	Chemical Oxygen Demand
d	day
EC	Electric Conductivity
m	meters
m/h	meters per hour
mg/L	miligrams per liter
MPN	Most Probable Number
Q	Inflow into the Trickling filter
Qr	Recirculated flow
SSFs	Slow Sand Filters
TC	Total Coliforms
THL	Total Hydraulic Load
TOL	Total Organic Load

1 Introduction

1.1 Introduction

Water stress and water scarcity, worsened by overexploitation of water sources and climate change, are global problems that require the improvement of technological options and development of new ones. In 2011, water stress affected 41 countries. Additionally, water demand will increase by 50% by 2030, with 70% of the demand being for irrigation (United Nations Development Programme 2018b).

Water is the key factor for human development as shown in the Sustainable Development Goals, where at least nine objectives relate to water in different degrees and dimensions. Additionally, the target 6.3 of Objective 6 Clean Water and Sanitation has as one of their aims to reduce by half the proportion of untreated wastewater and substantially increasing recycling and safe reuse globally (United Nations Development Programme 2018a).

As safe recycling and safe reuse are still a work in process in developing countries, complementing efforts are required to cover the demand. This effort must especially focus on small communities where highly technological options are not sustainable because of the energy cost and maintenance required.

Low-cost alternatives for municipal wastewater are trickling filters, an appropriated technology for small to medium size communities (EPA 2000) with the correct operation and maintenance. However, the treated effluent is not always below the recommended values for direct discharge into water bodies, which is why it requires further treatment to adjust to the legal requirements.

As energy is being used to achieve an adequate treated effluent, it is logical to aim to reuse treated wastewater. Vulnerable communities in countries such as Perú are affected by the retreat of glaciers, flooding and reduction of freshwater resources (United Nations 2014) with consequences in agriculture due to the reduction of water availability for irrigation. To improve water reuse, it is required water management policies, water efficient agriculture practices and water reuse.

This research aims to evaluate the efficiency of slow sand filtration as an additional step in municipal wastewater treatment facilities with a trickling filter as primary treatment.

1.2 Problem Description

Water scarcity and affordable wastewater treatment technologies in small communities are problems that can be addressed by water reuse policies. In areas where agriculture is an important economic activity, it is necessary to cover the demand, secure the economic sustainability of the treatment systems and ensure the health of the users.

Trickling filters are a very well-known and effective technology for wastewater treatment in small communities, while slow sand filtration is mainly applied to drinking water treatment (Langenbach et al. 2009). Some studies have approached slow sand filtration for wastewater treatment as a final treatment step, but with limited information regarding some physico-chemical and microbiological parameters, operational conditions and evaluation of their behavior when they are feed with treated wastewater from a trickling filter.

1.3 Objectives

The main objective of this research is the evaluation of the efficiency of slow sand filtration as an additional step to treat municipal wastewater treated with a trickling filter in order to achieve a water quality adequate for irrigation according to the recommendations set by the World Health Organization and National Peruvian Law. Additional secondary objectives are included to present a more specific view of the expected results.

Secondary Objectives:

- Evaluation of the removal of the physico-chemical parameters such as chemical oxygen demand, nitrogen compounds and phosphorus compounds in a wastewater treatment configuration with trickling filter and slow sand filtration at pilot scale at different hydraulic loads.
- Evaluation of the removal of *E. coli*, fecal coliforms and enterococcus by slow sand filtration at different hydraulic loads.
- Assessment of the viability of water reuse for agriculture according to National Peruvian Law and the World Health Organization.
- Assessment of operational conditions and maintenance for trickling filter and slow sand filter at different operational set-ups.

1.4 Motivation

Regarding small communities, the economic sustainability and maintenance of water treatment facilities is a key factor to guaranty the discharge of treated effluents, and its reuse is the reasonable next step to promote an adequate operation and maintenance, as it will also mean an additional benefit for the operator or the community involved, depending on the achieved water quality and the target objective for the reuse.

Several studies (Campos et al. 2002; Ellis 1987; Katukiza et al. 2014; Rolland et al. 2009) evaluated the performance of slow sand filtration focusing on the removal of physico-chemical parameters. Further studies have focused mainly on microbiological parameters (Bahgat et al. 1999; D'Alessio et al. 2015; Haig et al. 2014; Jenkins et al. 2011; Langenbach et al. 2009) and the latest studies have focused in micropollutants removal and as a pretreatment for ultrafiltration (Zhao et al. 2013; Zheng et al. 2010).

Ellis (1987) evaluated the treatment performance of slow sand filter treatment with different hydraulic loads and materials, using water treated by a trickling filter and monitoring parameters such as biochemical oxygen demand (BOD), nitrate and coliforms. However, the behavior of other parameters such as ammonium, nitrite and orthophosphates were not evaluated, and limited information was given regarding maintenance.

The present research focus on evaluating the removal efficiency of physico-chemical and bacteriological parameters of a trickling filter fed with municipal wastewater, and the removal efficiency of slow sand filter as a final stage of treatment, aiming to reuse the treated water in agriculture. This research evaluates the performance of the trickling filter feed in different stages by three different organic loads and evaluates the removal efficiency of three slow sand filters with three different hydraulic loads

2 Literature Review

2.1. Treatment Process of municipal wastewater

2.1.1. General overview

Municipal wastewater requires a series of processes to obtain treated effluent that is adequate to discharge into water bodies or reuse. As a first step, the preliminary treatment removes large suspended solids and inert materials. This step is a physical treatment process that allows the protection of pumps and other equipment (Riffat and Rumana 2013) and avoids the plugging of distributions systems (Grady 2011) by removing particles and debris and removes the nonbiodegradable particulate matter prior to a secondary biological treatment. As a second step, biological treatment achieves the removal of organic matter and nutrients through the production of biomass (Grady 2011). Systems that use attached growth processes, such as trickling filter or biodiscs, are recommend for its simplicity of operation, low energy requirement, low maintenance requirements and lower sludge production. However, it presents low efficiency at cold temperatures and can present problems with biofilm maintenance due to excess sloughing (Riffat and Rumana 2013). In addition, further treatment processes are required depending on the objective of the treatment or law requirements.

2.1.2. Preliminary Treatment

This treatment step is the first process in wastewater treatment plants, and depending on its configuration, it can include trash racks, coarse screens and fine screens. In plants with trickling filters for biological treatment, fine screens act as primary treatment (Riffat and Rumana 2013). For all these systems, a waste recovery system needs to be applied to guarantee the constant entry of water, which can either be manual or mechanical.

2.1.3. Trickling Filter

a. General overview

A trickling filter is an attached growth wastewater treatment system that consists of a bed of inert material or medium to which the microorganisms are attached. Trickling filters generally include a secondary settling tank or clarifier (Drinan and Whiting 2001) to separate produced biomass (Grady 2011) . In this case, the medium has high porosity and surface area (e.g. rock, gravel, plastic material) (Riffat and Rumana 2013). Usually the media consist of substance capable of withstanding weather conditions for many years (Drinan and Whiting 2001). The microorganisms form a biofilm and treat wastewater as it flows over the filter, through degradation of the organic matter as the biofilm grows and metabolizes. The treatment process produces sludge that is later separated to a primary clarifier. Additionally, the recirculation of effluent to the biological reactor can range from 100 to 300 % (Riffat and Rumana 2013).

This system is appropriate for small- to medium sized communities (population between 501—10000 (EPA 2008)) and it is comprised of: containment structure, wastewater application system, underdrain system and ventilation system (Grady 2011). Some of the characteristics of the systems are: the containment structure consists of a shallow tank filled with the inert medium. Trickling filters with plastic or synthetic media, also known as biotowers, present depths up to 12 m due to the lightweight media and porosities of about 94 % (Riffat and Rumana 2013). The distribution of the wastewater is done by a rotary distributor arm at the top of the tank (Riffat and Rumana 2013). Additionally a ventilation system to ensure the passage of air and keep an aerobic environment (Gray 2005). The flow of wastewater and air enters into contact with the microorganisms in an alternate way (Drinan and Whiting 2001). Also an underdrain system transports the treated wastewater and sloughed biofilm to the secondary clarifier (Riffat and Rumana 2013).

Depending on the treatment objectives, trickling filters can have different sizes, configurations and operational settings. Table 2.1 shows the benefits from the different types of processes that can be prioritized with a trickling filter. The roughing filters have as their treatment objective, the partial removal of organic matter. In the case of carbon oxidation filters, they are required to a relatively complete removal of organic matter. All these treatment objectives are defined by the total organic loading applied to the trickling filters (Grady 2011).

Table 2.1 Trickling Filter Process Comparison (Grady 2011)

Treatment Objectives	Benefits	Drawbacks
Roughing	Economical, particularly for high strength wastewaters Simple to design and operate Process and facility design well known	Further treatment typically required prior to discharge Generally, requires secondary clarification
Carbon Oxidation	Economical Simple to design and operate Process and facility design well known	Performance is consistent, but may not reliably meet stringent performance standards Generally, requires secondary clarification Limited operational flexibility
Combined carbon oxidation and nitrification	Simple to design and operate	Performance is consistent, but may not reliably meet stringent performance standards Generally, requires secondary clarification Limited operational flexibility
Separate stage nitrification	Simple to design and operate	Performance is consistent, but may not reliably meet stringent performance standards Limited operational flexibility

Among the general advantages of the trickling filter system, it can be mentioned simplicity of operation, low energy requirement, low maintenance, ability to handle shock loads, lower sludge production. Moreover, trickling filters are popular for carbon oxidation and nitrification (Grady 2011) due to the lack of problems of sludge bulking in secondary clarifiers (Riffat and Rumana 2013). The overall performance of a trickling filter depends on hydraulic and organic loading, temperature, and recirculation (Drinan and Whiting 2001).

On the other hand, the disadvantages include: low efficiency at cold temperatures, diffusion limitations, problems with biofilm maintenance due to excess of sloughing, higher biochemical oxygen demand (BOD) and solids concentration in the effluent and the possibility of odor problems (Riffat and Rumana 2013).

In addition to municipal wastewater, roughing filters can also be used to pretreat industrial wastewater with high concentration of readily biodegradable organic matter or to pretreat mixtures of municipal and industrial wastewater containing high concentrations of organic matter (Grady 2011). Trickling filters can be also be used in the case of industrial wastes including organic chemicals and plastics, and aqueous waste from synthetic fiber industries such as toluene and ethylbenzene (Drinan and Whiting 2001).

b. Media Characteristics

The selection of the media plays an important part in the designing of the system. The following properties need to be considered depending on the objectives of the treatment: chemical and biological inertness, size, unit weight, media depth, specific surface area and porosity (Grady 2011).

The filter media can be classified by their construction, such as granulated media and fixed media. While granulated media are small separate elements (such as rock, sand, plastic element types), fixed media comprises a larger construction with connected areas (Lekang and Kleppe 2000).

Each type of material has different characteristics that make it more fitting for specific circumstances. The porosity of synthetic media can rise up to 95 %, and in many cases higher porosity is desired (Riffat and Rumana 2013). In addition, light materials have a higher nitrification rate than heavy materials with the same size and degree of crushing[1] (Lekang and Kleppe 2000).

Alternatively, systems with high-rate media are known for their lower unit weight, higher specific surface area and greater void space than rock media (Grady 2011). This means that media depth has greater values (between 5—7 m), higher organic loads (up to 3.5 kg $BOD_5/(m^3*d)$) and smaller cross-sectional area. Despite the fact that, depths up to 12 m can be used (Riffat and Rumana 2013), often a depth of 6.7 m is used due to media structural considerations (Grady 2011). Inside the range of values for specific surface area, lower values (100 m^2/m^3) are recommended for wastewater in early stages of treatment (screened wastewater or primary clarifier effluent), due to its association with high biomass production rates. Also, a range of 89 to 102 m^2/m^3 are well suited for carbon oxidation and, combined carbon oxidation and nitrification (Daigger and Boltz 2011). In comparison, higher values (140 m^2/m^3) are used for secondary effluent, where the risk of plugging due to biomass production is lower. Also higher values means cheaper building costs because it occupies less space (Lekang and Kleppe 2000).

The effect of the depth is minimal when considering a fully utilize media (Grady 2011). Table 2.2 shows a comparison between rock and high-rate media.

Table 2.2 Media Type Comparison (Grady 2011)

Media Type	Benefits	Drawbacks
Rock media	Larger number of existing applications Quite effective at low to moderate organic loading rates	Relatively expensive due to structural constraints Not applicable for high loading applications Odor potential
High-rate media	Economical Applicable to a wide range of process loadings and applications Process and facility design well known	Media collapses have occurred due to improper application and/or manufacturing

Additionally, the filter medium needs to give the most homogeneous water flow (Lekang and Kleppe 2000) because complete wetting of the media ensures that all the biofilm is active (Grady 2011). The direction of the flow also affects the redistribution characteristics, when compared, a 60° cross-flow possesses good flow redistribution in comparison to a vertical media (Grady 2011). For high organic loads in roughing trickling filters, vertically oriented modular plastic media is generally accepted (Daigger and Boltz 2011).

Additionally, it allows for the avoidance of dead zones and channels that will reduce the nitrification rate (Lekang and Kleppe 2000).

[1] Wien 1995

c. Loading rates (hydraulic and organic load)

Trickling filters are defined by two types of loading rates: total hydraulic loading (THL) and total organic loading (TOL). The THL is the amount of wastewater applied to the filter surface including primary effluent and recycle flows. The TOL is calculated using only the BOD load coming with the primary effluent, but the BOD loading in the recycle flow is not included (Riffat and Rumana 2013). Table 2.3 shows different recommended TOL and THL according to different sources.

Table 2.3 Recommend total organic and hydraulic loading

Author	Type of trickling filter	Recommended value
Riffat and Rumana (2013)	High Rate	0.31—1 Kg $BOD_5/(m^3*d)$
EPA (2000)	Low-rate	<0,4 kg BOD_5/m^3
	Intermediate Rate	0.4—0.64 kg BOD_5/m^3
	High Rate	0.64—1.6 kg BOD_5/m^3
	Roughing filter	1.64—4.8 kg BOD_5/m^3
Gray (2005)	Low-rate	<3 m^3/m^3*d <0.6 kg $BOD_5/(m^3*d)$
	High-rate	>3 m^3/m^3*d >0.6 kg $BOD_5/(m^3*d)$.
Daigger and Boltz (2011)	High-rate	>1.8 m^3/m^2h
	Shallow towers using cross-flow media	0.4—1.1 m^3/m^2h

In practice, the hydraulic load typically uses ranges from 0.35 to 0.75 m/h. In the case of roughing filters, a hydraulic rate around 3 m/h without recirculation with small size and moderate depth is recommended. Due to economic reasons, most trickling filters are operated with a THL near the minimum value. It is important to mention that the theoretical performance of a trickling filter is insensitive to the hydraulic load as long as the organic loading is fixed (Grady 2011). In addition, research has shown that the performance of rock and high-rate media filters are similar for low organic loads, below 1 Kg $BOD_5/(m^3*d)$, but differs when the TOL is increased (Grady 2011).

d. Retention time

The hydraulic retention time (HRT) is the contact time of the wastewater with the media. For larger values of hydraulic loading, the greater the proportion of wastewater passing over the surface of the biofilm, resulting in a lower value for HRT and a final effluent with less quality. The HRT for high-rate systems is shorter than for low-rate filters (Gray 2005).

e. Sludge production

Sloughing depends on the hydraulic and organic load into the trickling filter. While the hydraulic load accounts for the shear velocities, the organic load controls the rate of metabolism in the biofilm layer (Riffat and Rumana 2013). As the substrate concentration at the biofilm reaches critical values, the microbial film falls (Howell and Atkinson 1976) due to the increase of the thickness of the biofilm and the shearing action of the wastewater that goes through the filter (Riffat and Rumana 2013). The sludge is then transported to the secondary clarifier or sedimentation tank for removal (Särner 1981).

For low-rate filters, the sloughed solids are generally well digested (EPA 2000) and of more stabilized nature which makes them less susceptible to denitrification (Gray 2005). Low-rate filters produce between 0.2 to 0.5 kg of dry solids per kg BOD removed, with a variation between 0.1 kg/kg and 0.5 kg/kg for summer and spring, respectively (Gray 2005).

For high-rate systems, the hydraulic retention time (HRT) is shorter, so a bigger volume of less mineralized sludge is produced. Although a seasonal variation is not differentiated, the mean production is around 0.35 kg/kg, but also depends on the characteristics of the influent wastewater (Gray 2005). The settling

characteristics of the sludge from those systems are normally poor. This is related to the low sludge age of high-rate filters. In these systems, the active organisms at the top of the filter are washed out at short time intervals due to the large sludge production and sheer stress of the water (Särner 1981). In both types of filters, the sludge production is significantly less compared to the activated sludge systems.

Trickling filters are often used to treat wastewater that produces sludge with poor settling characteristics and poor compactibility, because the sludge that sloughs off is relatively dense after passing through the system and can be removed by sedimentation (Drinan and Whiting 2001).

In the case of long sludge retention time in the settlement tank, anoxic conditions may occur, which can produce problems of denitrification and in consequence carry over sludge in the final effluent (Gray 2005).

In treatment plants, the settled sludge consisting of sloughed biofilm is usually recirculated back to the primary clarifier (Riffat and Rumana 2013).

f. Purification Process

Sylvester and Pitayagulsarn (1975) stated that trickling filter are modeled as a fixed bed reactor with the downflow of liquids. The reactions that occurred between the reactants (oxygen and organics) on the biomass surface, are first order, irreversible and isothermal (Grady 2011). Many parameters can be isolated and controlled in laboratory test. However, this is not possible when natural wastewater is used (Särner 1981). In practice, the flow through the trickling filter is intermittent and highly irregular. However, in biofilm models, it is assumed to be steady flow at a constant rate (Grady 2011).

During the purification process, the first stage is the absorption of organic nutrients onto the biofilm. Gray (2005) explained that fine particles are flocculated by extra-cellular polymers secreted by microorganism and adsorbed onto the biofilm surface. On this site, the particles and organic nutrients are broken down by extracellular enzymes from the heterotrophic bacteria and fungi present. The soluble nutrients resulting from this process are then absorbed by the biofilm and synthesized. Germain (1966) mentioned that the rate of stabilization of the organics controls adsorption capacity of the biological growth. Additionally, the BOD rate removal from domestic water exceeds the removal from dissolved material due to the ability to remove colloidal material by this biological flocculation (Germain 1966). However, Särner (1981) stated that a high load of fine suspended and colloidal particle can lower the removal rate of dissolved organics.

g. Recirculation

Recirculation is an important aspect of trickling filters (Riffat and Rumana 2013) because it allows the dilution of incoming wastewater with the returned final effluent (Gray 2005). Some advantages of recirculation are that it provides the desired wetting rate to keep the biofilm layer active, increases the dissolved oxygen of the influent and helps to dilute shock loads (Riffat and Rumana 2013). Additionally, the higher hydraulic loading has a flushing effect on the top section of the filter encouraging a thinner more active film and ensuring better utilization of the medium, it dilutes strong industrial wastewaters (Gray 2005), it allows a better flow distribution, and prevents macro fauna accumulation (Daigger and Boltz 2011).

A system with no recirculation has an applied soluble substrate concentration equal to the influent wastewater. When recirculation is used, the influent is diluted by the recirculated effluent, and the applied soluble substrate concentration is less than the concentration of influent wastewater (Grady 2011). This can allow an increment in the loading of 0.15 and 0.2 kg $BDO_5/(m^3*d)$, with little effect on the final effluent quality and a slight reduced nitrification efficiency (Gray 2005).

The recirculation ratio can be defined as the relation between the recirculated flow (Q_r) and the inflow into the trickling filter (Q). Those values can range from 0.5 to 3 (Riffat and Rumana 2013) or up to 4 (Daigger and Boltz 2011). Increasing recirculation beyond the design requirements can increase the operating costs due to the increment of the pumping requirements. Recirculation beyond this point is not generally used unless other factors require it (Grady 2011). The intent of recirculating bioreactor effluent is to decouple hydraulic and organic loading (Daigger and Boltz 2011). Also, increasing the recirculation flow beyond the amount

required to achieve the minimum THL will not improve performance (Grady 2011). In the case of domestic sewage, the values range between 1 and 2 (Gray 2005).

h. Biofilm

The biofilm is composed by microorganisms attached to the filter media, which are similar to those found in activated sludge reactor. However, trickling filters provide control of filamentous microorganisms (Grady 2011). Heterotrophic bacteria are mostly present in trickling filters, with a predominance of facultative bacteria. Grady (2011) mentioned that a diverse biota can develop, such as protozoa, worms, adult and larval filter flies (often of the genus *Psychoda*), which contribute to the stabilization of organic matter by increasing the length of the food chain. Rotifers and fungi can also be found. In case of systems exposed to sunlight, there is algae growth near the surface (Riffat and Rumana 2013).

The surface layer of the biofilm is the part where the oxidation reactions takes place. In consequence, a significant characteristic for the performance efficiency is the surface layer and not the total biomass of the biofilm in the filter (Gray 2005). It is important to consider that the active biofilm surface area decreases with increasing biofilm thickness (Daigger and Boltz 2011). A value of 0.15 mm is considered an optimum thickness for the biofilm. The critical depth is approximately 0.2 mm, where there is a predominance of bacterial biofilm. The outer 0.1 to 0.2 mm of the biofilm remains aerobic (Riffat and Rumana 2013). As the thickness increases, the inner layers of the biofilm become anaerobic, due to oxygen diffusion limitations into the inner layers. Fungal film may increase the thickness to values between 3 and 4 mm (Gray 2005). The total thickness may range from 100 µm to 10 mm (Riffat and Rumana 2013).

Biofilm can become stablished after 3–4 weeks (in summer) or up to 2 months (in winter). After this stage, the biofilm will reach its maximum purification capacity (Gray 2005). The growth pattern of biofilm can be divided into lag-phase, growth-phase and stationary phase (Honda and Matsumoto 1983). Moreover, the regrowth of biofilm occurs quickly (Riffat and Rumana 2013) and it is not constant and decreases as the biofilm weight increases (Honda and Matsumoto 1983). In the transition between lag-phase and growth-phase, the biofilm growth increases suddenly.

The thickness of the biofilm increases until it reaches a maximum value due to the biofilm growth. With an increase of biofilm thickness, sloughing can occur. This growth is controlled primarily by the food available in the form of organic load (Germain 1966). The biofilm only develops on the surface where the supply of nutrients is constant. That is why an even distribution of wastewater affects performance (Gray 2005). The heterotrophic bacteria uses the soluble organic matter as a carbon and energy source and performs the metabolization (Grady 2011).

The increase in hydraulic loading produces a thinner, more aerobic, and more active biofilm (Grady 2011). In addition, the maximum growth also depends on the type of medium, type of organic matter and nature of the particular biological growth (Germain 1966). A proper control of the biofilm thickness through periodic flushing can improve trickling filter performance (Grady 2011). The excessive biofilm growth can reduce the volume of interstices, which can reduce ventilation (Gray 2005) and oxygen distribution.

Regarding the composition of the biofilm along the depth of the trickling filter, there are different types of bacteria which affect the performance (Grady 2011). Oxygen is required for biochemical oxidation to degrade organic compounds and to enhance nitrification of ammonium (Zhang et al. 2016). Heterotrophic bacteria, responsible for BOD removal (Daigger and Boltz 2011), grow faster than autotrophic nitrifying bacteria. That is why, in the upper level, where high concentration of organic matter and ammonia—nitrogen are found, there is a predominance of heterotrophic bacteria. Along the depth of the trickling filter, as organic matter is removed, it becomes a limiting factor for the growth of heterotrophic bacteria. In systems designed for carbon oxidation and nitrification, the upper portion has carbon oxidation reactions with heterotrophic bacteria, while in the lower portions occurs nitrification with autotrophic nitrifying biofilm. Both types are in constant competition for space (Grady 2011) and oxygen within the biofilm (Almstrand et al. 2011). Tchobanoglous et al. (2014) mentions that for BOD concentration less than 30 mg/L, nitrification initiates and

values below 15 mg/L are necessary for complete nitrification. However, Grady (2011) states that autotrophic bacteria grow when the concentration of organic matter drops below 20 mg COD/L.

At depth levels with a low organic matter concentration, the autotrophic bacteria can finally compete for space and become established in the biofilm (Grady 2011). This also means that organic carbon can indirectly influence the nitrification process (Almstrand et al. 2011). At the lower levels, there is little net growth of biofilm, due to the limiting organic load, which also translates into a predominant autotrophic thin biofilm. As a result, it is likely that dry pockets are developed in filters with high density modular plastic media (Daigger and Boltz 2011).

In systems designed only for carbon oxidation, the presence of autotrophic nitrifying bacteria is limited due to the high organic load. Meanwhile, in systems with a separate stage nitrification the carbon content is low, so the nitrifying bacteria may be predominant.

i. Ventilation/ Oxygen

In trickling filters, the oxygen is either provided by natural draft or forced draft ventilation (Grady 2011). Forced draft ventilation requires a driving force for air, such as a ventilator (Grady 2011). By comparison, natural draft depends on temperature difference between the air inside the filter and air surrounding the trickling filter (Gray 2005) resulting in a continuous flow through the media (Grady 2011). This convective air movement allows oxygen transfer (Grady 2011) maintaining aerobic conditions (Drinan and Whiting 2001). The oxygen diffuses from the air in the interstices, first into the liquid and then into the film. However, the end products of aerobic metabolism and carbon dioxide diffuse in the other direction (Gray 2005). The solubility of oxygen in water can result in concentrations in the range of 7—12 mg/L for wastewater temperatures (Grady 2011).

Inside trickling filters, the temperature causes the air to expand when warmed or to contract when cooled, and humidity differences result in density differences (Daigger and Boltz 2011). The direction of the air flow can be upwards or downwards (Gray 2005). The airflow resulting from a natural draft will distribute itself (Daigger and Boltz 2011). In neutral density conditions, there can be an absence of air flow and development of anaerobic conditions (Grady 2011). If air inside the trickling filter is colder than the ambient air, the air will flow downward. Alternatively, if the ambient air is colder than the air inside the trickling filter, air will flow upward (Daigger and Boltz 2011). An upward flow can be considered a worst-case scenario because the oxygen demand is higher in the upper part of the filter.

The control of the ventilation system is crucial to maintain the aerobic conditions, minimize heat loss during cold weather operations and for odor control.

j. Temperature

Temperature affects different interlinked aspects of trickling filter such as biofilm, microbial metabolic rates, organic matter oxidation, nitrification, activity of grazing fauna, gas production and the direction of the air flow inside the trickling filter.

Honda and Matsumoto (1983) identified that BOD removal efficiency was at a maximum at 15 °C. Additionally, they showed that the biofilm weight in the biofilm growth phase remained virtually constant at temperatures up to 15 °C and then became lower at higher temperatures due to differences in the amount of the active biofilm. Gray (2005) explained that the reason for these behaviors lies in the high microbial activity and activity of the grazing fauna which may exceed the rate of adsorption, causing a reduction of the overall biofilm biomass. Additionally, Gray (2005) indicated that below 10 °C the rate of biofilm accumulation increases rapidly, due to a decrease of the oxidation rate, reduction of the activity of the grazing fauna and an unaltered rate of adsorption, which can result in the filters becoming clogged.

It is important to point out that during spring, there is an apparent sloughing of the biofilm due to the increase in temperatures and the biofilm accumulations during months (Gray 2005). Although grazers maintain minimum biofilm accumulation for a long period after sloughing, temperature is the primary factor that controls biofilm accumulation (Gray 2005).

During summer, gas production is present, and it causes a fraction of fine solids not to settle into the secondary settling tank and to be carried to the final effluent, requiring additional treatment steps such as sand filtration (Gray 2005).

Air temperature also controls the amount of dissolved oxygen in water (Tchobanoglous et al. 2014).

For Grady (2011), limiting temperature changes can avoid the retarding of biological reactions during temperature drops and performance problems.

k. BOD removal and its relationship with nitrification

Trickling filters are designed to remove 5-day biochemical oxygen demand (BOD_5) and suspended solids (Drinan and Whiting 2001). Table 2.4 shows the removal rates for BOD_5 and total suspended solids (TSS). The removal rate of organic matter in trickling filters is affected by several factors such as: wastewater inflow rate and organic load rate, adsorption activity of particles, temperature and diffusion of oxygen and BOD into the biofilm, oxygen diffusion being a limiting factor (Riffat and Rumana 2013; Särner 1981).

Table 2.4 Removal rates for trickling filters (Modified from Drinan and Whiting (2001))

Type of filter	Hydraulic load (m^3/m^3*d)	Sloughing frequency	Recirculation	BOD_5 removal rate (%)	TSS removal rate (%)
Standard rate	3.34—12	Seasonal	No	80 to 85	80 to 85
High rate	30.75—120	Continuous	Yes	65 to 80	*65* to 80
Roughing filter	>120	Continuous	Not normally included	40 to *65*	40 to *65*

As the organic contaminants come into contact with the microorganism in the biofilm attached to the filter media, the biofilm adsorbs organics and stabilizes the previously adsorbed organics (Germain 1966). Thanks to the spaces between the media, the oxygen present allows the microorganisms to aerobically decompose the solids (Drinan and Whiting 2001). This reactions produce stable waste, in which the rate of stabilization of the organics controls the adsorption capacity of the biological growth (Germain 1966). The stable waste can become part of the biofilm or return to the wastewater (Drinan and Whiting 2001). The biofilm is sloughed off once it reaches critical values.

The reaction rate decreases when the BOD removal reaches values of approximately 90 %. Beyond this point, the filter becomes a sludge digester and not a water treatment unit. This is because the reaction nears completion when the by-products of auto-oxidation processes in the biological growth make up for a large proportion of effluent BOD (Germain 1966).

In the case of domestic wastewater, there is an apparent ability of the filter to remove colloidal material as well. Germain (1966) explained that the reason is the biological flocculation and not oxidation reactions. Although, Särner (1981) states that the removal efficiency of suspended and colloidal particles is not clear, a high load of fine suspended and colloidal particles lowers the removal rate of dissolved organics. According to Eding et al. (2006), particulate matter may also negatively affect nitrification due to clogging and occupation of surface area because of bacteria biomass and organics.

For industrial wastewater, the adsorption capacity of biological growth can reach a maximum value. In addition, based on experience in wastewater treatment plants, beyond this maximum value, the amount of BOD removed will be constant (Germain 1966).

Morover, trickling filters are a popular option for carbon oxidation and nitrification. A process with nitrification will require more oxygen than without nitrification (1.59 against 1.42 kg O_2/kg organic matter) and it will lower the pH value in the wastewater and the biofilm if there is not enough alkalinity in the system. The design of trickling filter for carbon oxidation and nitrification is generally empirical due to the complex kinetics and lack of fundamental research supporting this type of trickling filter (Daigger and Boltz 2011).

Table 2.5 shows some results obtained and recommend values for this type of systems. As shown, organic loads between 0.2 and 0.4 kg $BOD_5/(m^3*d)$ can be recommended when a high percentage of nitrification wants to be achieved.

Table 2.5 Recommended values for carbon oxidation and nitrification filters

Author	Recommended load	Obtained result
Grady (2011)	>2.5 $m^3/(m^3*d)$	Nitrification is reduced
Tchobanoglous et al. (2014)	0.08 — 0.4 kg $BOD_5/(m^3*d)$	90 % nitrification in plastic media
	0.22 kg $BOD_5/(m^3*d)$	50 % nitrification efficiency
	0.24 kg $BOD_5/(m^2*d)$	90 % ammonia-removal
Stenquist et al. (1974)	0.356 kg $BOD_5/(m^3*d)$	89 % ammonia removal (effluent of 2 mg/L) in plastic media
Daigger and Boltz (2011)	<0.2 kg $BOD_5/(m^3*d)$	90 % nitrification efficiency

l. Nitrogen removal and nitrification process

Among the different compounds present in domestic wastewater, nitrogen oxidized forms, such as nitrite and nitrate are found in very low concentrations. Nitrogen bound in organic matter (proteins and amines) is also normally found and assumed to be available for bacteria synthesis through conversion to ammonia due to hydrolysis processes (Stenquist et al. 1974).

As previously mentioned, trickling filters can be designed for combined carbon oxidation–nitrification and separate stage nitrification. It is important to note that nitrification begins when the soluble BOD concentration is less than 2 mg/L (Surampalli et al. 1995).

The processes involving the removal of nitrogen include aerobic nitrification, anoxic denitrification (Almstrand et al. 2011) and anaerobic ammonium oxidation (anammox) (Wang et al. 2015). Nitrification includes the oxidation of ammonium to nitrite by ammonia-oxidizing bacteria (AOB) and to nitrate by nitrite-oxidizing bacteria (NOB). For denitrification, the nitrate is reduced to elemental nitrogen (Almstrand et al. 2011). In the case of the ammonium oxidation processes, where N_2 is formed by oxidation of ammonium by nitrite, it is relatively uncommon (Almstrand et al. 2011).

The presence of ammonia in wastewater can present undesirable effects such as consumption of dissolved oxygen due to oxidation to nitrate, formation of chloramines when chlorine is added (reducing the effect of disinfectants) and toxicity to fish under certain conditions (Stenquist et al. 1974). The removal of ammonia is achieved due to nitrification by nitrifying bacteria, volatilization of free ammonia and metabolism into new cellular material. However, when there is low film accumulation in trickling filters and the grazing fauna is large, the concentration of ammonia is high due to the excretion products (Gray 2005). Furthermore, the ammonia loading, in addition to other operational factors determine the degree of ammonia removal (Grady 2011).

Regarding nitrification, it is an aerobic process carried out by strict aerobes autotrophic nitrifying bacteria. The presence of heterotrophic bacteria can inhibit or compete with nitrification processes. In these processes, oxygen is a limiting factor and it requires also the presence of ammonia, nutrients, low organic carbon levels (Stenquist et al. 1974) and enough alkalinity needs to be present in the wastewater (Surampalli et al. 1995).

In order to achieve carbon oxidation and nitrification in a single-stage trickling filter, in addition to the processes previously mentioned, are also necessary high residence times and consistent hydraulic, organic and ammonia–nitrogen loads (Surampalli et al. 1995).

Eding et al. (2006) explained that high organic load resulted in shorter residence time of nitrifying bacteria in the aerobic zone. Also, Zhang et al. (2015) mentioned that in conditions of variable hydraulic loads, nitrogen transformation processes often fluctuated with unsatisfactory treatment results. Almstrand et al. (2011) indicated that nitrifying bacteria may take a long time to adjust after shifts in substrate load.

The nitrification process in domestic wastewater treatment are carried out by bacteria such as *Nitrosomonas spp* and *Nitrobacter spp* (both are autotrophic), which oxidize the substrate as source of energy for growth and metabolism (Stenquist et al. 1974). The reaction is controlled by *Nitrosomon*as *spp* which are stablished before than *Nitrobacter spp* (Gray 2005). After nitrite is formed, then it is rapidly converted into nitrate, so nitrite is present in low concentrations. However, for trickling filters systems, it is reported that high concentration of nitrite is usually present. Eding et al. (2006) stated that nitrite is diffused from the biofilm layer into the recirculated wastewater when the concentration of nitrite in the wastewater was low, until an equilibrium point was met where both concentrations are equivalent. As the total ammonia—nitrogen concentration increases, the nitrite diffusion will also increase with a new point in equilibrium with a higher concentration of nitrite. This equilibrium will finally lead into an equal rate of total ammonia—nitrogen and nitrite removal.

The nitrification process can change the pH. 2 mol of H^+ are released per mole of NH_4^+ oxidized to nitrite, and the decline of pH over time can be interpreted as the time for the nitrifying community to build up (Forbis- Stokes et al. 2018). Moreover, nitrifying bacteria are sensitive to toxic compounds such as heavy metals, which can limit their growth (Gray 2005).

In the case of denitrification, there is an anoxic zone near the biomass-medium interface, which results in nitrate ions being reduced by denitrification (Gray 2005). Some denitrification can occur when the nitrate containing effluent is recirculated (Grady 2011). However, residual BOD or an electron donor is required in addition to perfect anoxic conditions and enough retention time to carry on the reaction (Zhang et al. 2015). The limiting factor is generally the electron donor. Forbis-Stokes et al. (2018) mentioned that a requirement of 4 g BOD_5/NO_3-N[2] is normally assumed for complete denitrification to nitrogen gas. Another alternative to provide the electron donor can be the particulate and slowly biodegradable COD that passes through the trickling filter or in case of denitrifying filter, a carbon-based filter medium such as wood chips.

Denitrification processes can also be present in the settlement tank due to anoxic conditions facilitated by long sludge retention time. This denitrification process can be considered a problem because it may carry over sludge in the final effluent. Sludge from high-rate systems are more susceptible to denitrification because it requires more oxygen content than sludge from low-rate systems which is more stable (Gray 2005).

m. Phosphorus

Total phosphorus (TP) removal in trickling filter takes place after building of biofilm, being the major removal mechanism of TP the microbial action (Zhao et al. 2013). Phosphorus compounds exist in three forms such as organic phosphorus, orthophosphate and polyphosphates (with detergents as the source) (Krishnaswamy et al. 2009). Polyphosphates are converted to orthophosphates with biological treatment. In plants with biological treatment, the different forms of phosphorus are removed by absorption involving microorganisms. The amount of phosphorus compounds eliminated depends on the production of biomass. The active growing microorganism extract carbon—nitrogen—phosphorus in a ratio of 50:5:1 (consider a nutritionally balance wastewater) only if the environmental conditions are favorable and the ratio of biomass to available food is appropriate. However the ratio usually is in the range of 10:5:1, which makes difficult the production of an effluent with low nitrogen and phosphorus compounds through only biological synthesis (Environmental Protection Agency 1973). A ratio BOD_5:TN:P_{Total} for aerobic wastewater treatment should be in the range 100:10:1 and 100:5:1 to fulfil the requirements of the bacteria (Winkler 2012). In case of soluble phosphate, it is considered that trickling filters do not removed it (Environmental Protection Agency 1973).

n. Common problems in trickling filters

Different problems in trickling filter can appear due to operational or environmental conditions. Problems such as flies, ponding, odor and icing can occur and must be avoided or managed.

[2] Modified from the original 4 gBOD_L/NO_3-N assuming a BOD_5 instead of BOD_L

Even though, the diverse biota that develops in the trickling filter contributes with the stabilization of the organic matter, some organisms can cause operational problems. Organisms such as flies (often of the genus *Psychoda*) can have an impact on the personnel and neighbors surrounding the treatment plant. Factors such as temperature, light intensity and wind velocity can influence the appearance of flies. From 4 °C, *Metriocnemus spp* and *Sylvicola fenestralis* may be present. From 10 °C, *Psychoda alternata* can double in number for each 1.2 °C in temperature rise and accounts for more than the 80 % of the total annual emergence of flies in trickling filters (Gray 2005).

Snails can act like grit and accumulate in posterior unit operations, reducing the effective volume of some treatment units. In addition, fly larvae, worms and snails can reduce performance in nitrifying trickling filter if they consume biomass faster than the rate than it is being produced (Grady 2011).

Ponding consists in biofilm accumulation which blocks the interstices and reduces the movement of air and the reactions that require oxygen. As the maximum availability of food occurs at the upper part of the filter, oxidations reactions and accumulation also occur there. The medium requires a compromise between specific surface area and porosity to ensure normal hydraulic conditions and oxidation reactions (Gray 2005).

Fungi can also block the passage of air flow, specially *Subbaromyces splendens* (*S. splendens*) in the medium, center or towards the base of the filter. This is caused by a reduction of the rate of biological activity and nitrification when high fluctuations of temperature in the filter occur. In areas with low temperatures, icing can also take place (EPA 2000).

Other related problems are plugging and channeling, which are more likely to occur in carbon oxidation trickling filters. This produces dry spots or areas of incomplete utilization where nuisance organisms can grow or anaerobic activity may develop (Grady 2011). Anaerobic activity can lead to disagreeable odors due to anaerobic decomposition (EPA 2000). Odors can also occur when wastewater from septic tanks is being treated in the trickling filters (Grady 2011).

o. Operation and Maintenance

The daily operation consists in maintaining pumps and equipment (Grady 2011). Additionally, the performance can be improved by proper control of the hydraulic regime that is being applied. Depending on the problems faced by the trickling filters, specific strategies can be applied to prevent, manage or control them.

According to EPA (2000), increasing the hydraulic load, to perform hydraulic flushing, can improve inadequate filter media moisture. It also minimizes the spawning area and reduces the presence of flies. Daigger and Boltz (2011) presented several cases where the increasing of hydraulic loading was effective. However, it was not the case for a pilot scale nitrifying trickling filter.

In the case of ponding, a change in operational conditions such as limiting biofilm accumulation on the surface, reducing the food-to-mass ratio, recirculation or increasing hydraulic load can be effective (Gray 2005). Odors control can be achieved by pre-aerating the wastewater, recycling plant effluent, increasing hydraulic loading to remove excess biofilm and remove debris from the filter effluent channels. Finally, icing in trickling filter can be managed by a decreased in recirculation and the use of high pressure stream to remove the ice from the orifices (EPA 2000).

2.1.4. Rapid Sand Filter

Rapid sand filtration is a type of depth filtration with a typical filtration rate between 80 to 200 L/(m^2*min). Tchobanoglous et al. (2014) describes a rapid granular medium-depth filter as a filtering medium supported on a gravel layer, which at the same time rests on a filter underdrain system. The water enters through an inlet channel and is collected in the underdrain system, which is used later to backwash the filter.

The grain size of the filtering medium is the main characteristic that affects the operation of the system. If the size is too small, the frictional resistance will be too high and much of the driving force will be used to overcome it. If the size is too large, the small particles will not be retained by the filter. During the operation,

a head loss will be observed as the material starts to accumulate in the surface of the filter, once the filter reaches a predetermined value, it must be cleaned. This process is called backwashing and it is achieved by reversing the water flow and by applying enough wash water to fluidize the medium. The material accumulated in the bed is then wash away. Pressurized air can also by injected to achieve the fluidization of the medium.

Among the different removal mechanisms, straining has been identified as the principal mechanism in the removal of suspended solids, follow by the mechanisms listed in Table 2.6. In addition, other mechanisms have a small effect and even mask by the straining action such as chemical adsorption (bonding and chemical interaction), physical adsorption (electrostatic forces, electro-kinetic forces, van der Waals force) and biological growth (Tchobanoglous et al. 2014). In the case of smaller particles, two different processes are involved: the transport of the particle near the surface and the removal of the particle by one or more removal mechanism, also known as attachment.

Table 2.6 Removal mechanism in rapid sand filters (Modified from Tchobanoglous et al. (2014))

Mechanism / phenomenon	Description
Straining	
a. Mechanical	Particles larger than the pose space of the filter medium are strained out mechanically
b. Chance contact	Particles smaller than the pore space are trapped within the filter by chance contact
Sedimentation	Particles settle on the filtering medium within the filter
Impaction	Heavy particle will not follow the flow streamlines
Interception	Many particles that move along in the streamline are removed when they come in contact with the surface of the filtering medium
Adhesion	Particles become attached to the surface of the filtering medium as they pass by. Because of the force of the flowing water, some material is sheared away before it becomes firmly attached and is pushed deeper into the filter bed. As the bed becomes clogged, the surface shear force increases to a point at which no additional material can be removed. Some material may break through the bottom of the filter, causing the sudden appearance of turbidity in the effluent.
Flocculation	Flocculation can occur within the interstices of the filter medium. The larger particles formed by the velocity gradients within the filter are then removed by one or more of the above removal mechanisms.

Rapid sand filters require smaller land surface and present lower sensitivity to temperature and water quality variations, when compared to systems such as slow sand filters (SSFs) (Haig et al. 2014).

2.1.5. Slow Sand Filter

a. Overview

Slow sand filtration (SSF) is set apart from rapid sand filtration due to low filtration rate, simpler operation, smaller sand diameter, less uniform media, the establishment of layer of material called *schmutzdecke.* In addition, it does not require coagulation pretreatment and backwash (Crittenden 2005). It is a low-tech treatment technology (Seeger et al. 2016) with less energy consumption (Verma et al. 2017). in comparison to systems such as activated sludge-intensive.

SSFs have been used predominantly for purification of potable water (Langenbach et al. 2009) because of its ability to produce consistently a high-class filtrate with low maintenance (Ellis 1987). This system integrates physical (mechanical filtration), chemical (adsorption and degradation) and biological process to remove contaminants in water (Verma et al. 2017).

Among the requirements for a proper running of the system, the inflow water requires values of turbidity less than 50 NTU, little or no colloidal clay. Furthermore, the values for the depth of the sand bed are typically between 0.9 to 1.5 m (Crittenden 2005).

The inflow water travels downward by gravity through a submerged sand bed. The maximum water level dictates the available head. Usually, the head loss happens slowly (weeks or months for drinking water treatment). When the maximum water level is reached, the filter is drained and the top 1 to 2 cm is scraped off. After maintenance procedures, the filter is placed back in service (Crittenden 2005). In addition to the sand layer, the *schmutzdecke* provides also a filtration layer (due to smaller filter media diameter) where smaller particles are retained and complex biological communities degrade organic matter (Crittenden 2005). The bacterial communities are extremely rich in taxa; not dominated by any particular phylogenetic group (Haig et al. 2014).

Rolland et al. (2009) mentioned that sand characteristics and its implementation are the most important design parameter related to treatment efficiency and sustainability. However, Katukiza et al. (2014) stated that performance is highly influenced by wastewater influent quality and hydraulic loading. Additionally, temperature, surface ripening, hydraulic retention time and surface are also important factors for designing and evaluating performance (Verma et al. 2017). Which is why, a compromise needs to be meet between sand used, loads applied, system operation to ensure performance (Rolland et al. 2009).

SSFs can also be effectively used for tertiary wastewater treatment for simultaneous removal of organics and nitrogen (Ellis 1987; Nakhla and Farooq 2003) and has shown promise for reuse of wastewater, especially in arid developing countries (Langenbach et al. 2009). It can effectively remove COD, BOD, suspended solids, turbidity, total and fecal coliforms. However, different researches implemented a variety of conditions such as different hydraulic loads, sand grain size, filter material and operational mode to determine the removal efficiency for specific types of wastewater. Ellis (1987) used grain sizes of 0.3 and 0.6 mm with significant difference between each other. In addition, for the hydraulic loads 3.5 m/d and 7 m/d, better results were found for the lower hydraulic load (3.5 m/d).

Katukiza et al. (2014) used different material for the SSF. One had only one layer of lava rock and the other was configurated with two layers, the bottom half had lava rock and upper half had silica sand. Both materials had a grain size between 1.18 and 2.56 mm. The hydraulic load was 20 and 40 cm/d with better performances for the 20 cm/d.

Langenbach et al. (2009) used hydraulic loads of 5, 10 and 20 cm/h with grain sizes between 0.25 to 0.68 mm. Nakhla and Farooq (2003) studied the impact filtration rates in the range of 0.15 to 0.38 m/h, filter depth between 0.5 to 0.15 m, grain size between 0.3 to 0.5 mm and temperatures between 10 to 39 °C. Tyagi et al. (2009) used a grain size of 0.43 mm and hydraulic loads of 0.14, 0.19 and 0.26 m/h. In general, better removal rates were found for lower hydraulic loads and smaller grain sizes.

Among the operational problems of the system, the most common is clogging, due to growth of biomass in the active layer (Zheng et al. 2010), surface development (Rolland et al. 2009), high concentrations of extracellular bacterial slimes in the sand pores, accumulation of suspended solids or precipitation and deposition of compounds such as calcium carbonate (Vidal et al. 2018).

b. Hydraulic Load

Hydraulic load can influence the performance of the SSFs (Katukiza et al. 2014) as well as the operational mode (single or intermittent feeding) (Verma et al. 2017).

The water to be treated percolates through the filter using the pressure of a permanent water head (Seeger et al. 2016). The hydraulic loading rate (HLR) is a measure of how much contact occurs between the

particles as water flows (Verma et al. 2017). The values range between 0.05 to 0.4 m/h (Langenbach et al. 2009). And Verma et al. (2017) mentioned that hydraulic loads as low as 0.008 m/h were implemented in the past because treatment efficiency is best achieved at lower end of the range. However, Langenbach et al. (2009) stated that filter velocities of around 0.2 m/h can be economically attractive with a slight reduction on bacteria removal when compared with lower velocities. Additionally, retained bacteria are not affected by reduced hydraulic retention times as a result of an elevated hydraulic loading rate (Langenbach et al. 2010). Seeger et al. (2016) mentioned that fine and uniform sand material with a high sand surface area achieves the best faecal indicator removal at HLRs of 0.05 and 0.10 m/h.

A higher hydraulic load increases the head-loss in the filter due to biomass accumulation or biofilm formation (Zheng et al. 2010), which can result in a reduction of the operation time (Verma et al. 2017). Tyagi et al. (2009) studied the head loss for different hydraulic loads (0.14, 0.19 and 0.26 m/h) in order to determine how it affects its operation, run before it is clogged and how the quality of the filtrate was affected. Furthermore, pilot plant studies are recommended before designing large-scale system due to the performance variation of the *schmutzdecke* (Langenbach et al. 2009).

c. Filter Media

Other key parameters for SSFs are sand grain size distribution and filter bed depth (Langenbach et al. 2009). Recommended sand grain sizes range between 0.15 mm to 0.4 mm for the effective size value (Langenbach et al. 2009). In the case of drinking water treatment a sand grain size smaller than 0.4 mm is mostly used (Zheng et al. 2010). Some models to predict total coliform removal, showed that a decrease in grain size and an increase in bed depth improved the removal of bacteria. The filter bed depth can present values higher than 0.5 m (Langenbach et al. 2009).

Different sand materials or a combination of them have been used such as coarser and quartz/silica. In most cases the materials are chosen due to its effectiveness to remove specific pollutants. For example silica sand removes more than 70 % of organic matter and around 90 % of COD (Verma et al. 2017).

The most common configuration for the different layer is small rocks at the bottom, follow by pebbles and coarse sand at the top. Also, a diffuser plate may be placed 2 cm above the standing head for homogeneous water supply and to avoid disturbance to the *schmutzdecke*. Some characteristics to take into account for the filter media are bulk density, particle density, porosity, effective diameter (de), coefficient of uniformity (UC), coefficient of curvature and specific surface area. On one hand, for larger values of de, the recovery of dissolved oxygen decreases. On the other hand, for smaller values of de, the total suspended solids removal increases due to the finer media and more frequent cleaning is required (Verma et al. 2017).

d. Schmutzdecke

Purification processes occur at or near the surface sand layer (Tyagi et al. 2009). Particles are physically strained at the surface of the filter bed (Crittenden 2005), causing also head loss (Zheng et al. 2010).

The *schmutzdecke* forms a complex biological community that degrades some organic matter (Crittenden 2005). It is composed by a mixture of humus, sand, algae, metazoan (Tyagi et al. 2009), protozoans (*Paramecium sp*.) and diatoms (Verma et al. 2017), similar to that found in an activated sludge system and trickling filters biofilm (Nakhla and Farooq 2003). It removes natural organic matter, transforms synthetic organic compounds, retains pathogens and produce microbiologically safe water (Verma et al. 2017).

Regarding the *schmutzdecke*, the ripening process is considered the start-up phase. During this process, the suspended particles are physically and/or chemically captured by the sand and the *schmutzdecke* develops on the sand surface. The effluent quality usually remains poor during this period until stable operation is reached and a high-quality effluent is achieved. The thickness of the layer can varied from 0.5 to 2.0 cm (Verma et al. 2017) up until values 2.5 cm (Zheng et al. 2010). According to Crittenden (2005) the layer starts to develop after a few days, while for Verma et al. (2017) it can take 2 weeks. After several filter runs and scrapings of the top layer, the microbial community can become established deeper, shortening the ripening period or making it non-existent (Crittenden 2005).

The mechanisms involved in pathogen retention are straining and adsorption, while for pathogen inactivation the mechanisms are abiotic and biotic (Seeger et al. 2016). Also, additional contributing factors are natural die-off (starvation), predation (by eukaryotic bacterivores and bacteria), lysis induced by bacteriophages and algal-derived reactive oxygen species (Seeger et al. 2016).

The *schmutzdecke* and 5 cm of the upper level of sand are effective in retaining fecal indicator bacteria (Langenbach et al. 2010) and viruses removal (Verma et al. 2017). The accumulated material improves straining and adsorption in the biofilm. Langenbach et al. (2010) mentions that the *E.coli* is adsorbed much better to the *schmutzdecke* when it is composed of 90 % organic material than inorganic sand grain surface. Also, increasing the hydraulic load increases the amount of particle load and the biofilm thickness.

Regarding the significance of the *schmuztdecke* in efficiency removal, it is important to mention that, Chan et al. (2018) compared new stablish and mature systems. The removal of the top layer and *schmutzdecke* on new filters impacts the quality of the effluent, while in the mature systems, the scraping does not influence the removal efficiency. This suggest that in mature systems, the bacterial communities reside in the sand bed and the *schmutzdecke*, while in new systems, the deep sand bed has a minimal impact on the bacterial community in the water. As many studies are performed in laboratory or pilot scale systems, there is emphasis on the *schmutzdecke* importance because the deep bed has not had time to develop microbial communities so its influence in treatment has been considered as minimal.

In the case of the supernatant water, it seems to protect the biofilm from shear forces caused by the inflow water (Langenbach et al. 2010). Concerning temperature effect into the biofilm, more suitable circumstances can be created for the microorganism and the removal of substances such as biopolymers (Zheng et al. 2010). Finally, Haig et al. (2014) compared the microbiological communities and water quality production in laboratory scale to the ones in full-scale systems. It was shown that it is possible to simulate both water quality production and the structure of the microbial community. Also, factors such as sand type in the filter bed, water quality, age of the filters and depth of sand samples are significant in explaining observed differences in the structure of the microbial consortia.

e. Maintenance

The presence of solids in the influent wastewater in SSFs can cause head loss in the systems and also a reduction of the operation time (Verma et al. 2017). This is also accelerated by higher hydraulic loads. Although the head loss can happen slowly for drinking water treatment (Crittenden 2005), it increases rapidly in wastewater treatment (Tyagi et al. 2009). Usual maintenance operation happen when the head loss reaches the available head in the system (Crittenden 2005) or the *schmutzdecke* has accumulated to an extent that the desired hydraulic loading rate is not achievable (Langenbach et al. 2009). In this case, a few centimeters of the top layer need to be scraped off, hydraulically cleaned and stockpiled on-site. Then the system is placed back to service and the process is repeated until the sand needs to be replenished (Crittenden 2005).

A problem related to head loss is clogging, which is caused by finer sand particles. The head loss in the system increases to the point when the filter run needs to be stopped. In this case, applying resting periods cannot restore the filter to its original condition. The solution for this problem is also the removal of the top layer through scrapping (Verma et al. 2017), indicating that clogging occurs mainly due to the fine sand at the surface layer (Langenbach et al. 2009). Also, backwashing is not recommended for these systems (Verma et al. 2017). In mature systems, the maintenance work does not affect the effluent quality as shown by Haig et al. (2014). Furthermore, a compromise must be met between the maintenance work frequency and the hydraulic loads (Langenbach et al. 2009).

f. Removal of physico-chemical parameters

Different studies have shown a variety of removal ranges for most common water contaminants due to the different operational conditions. The interaction of physical, chemical and biological processes makes it even more challenging to present consensus for removal ranges. So, removal efficiency is more focused on

individual case to case, depending on specific types of operational conditions and composition of the inflow wastewater. Tyagi et al. (2009) mentioned that the removal of physicochemical parameters is performed by straining and attachment to the filter media and previously removed particles. The removal behavior is also followed by bacterial indicators (Tyagi et al. 2009).

As seen in Table 2.7, total suspended solids (TSS) are affected by the grain size (between 0.3 and 0.6 mm). It was shown that filter depth does not significantly impact its removal efficiency (Nakhla and Farooq 2003; Tyagi et al. 2009). However, it was reported that turbidity removal is directly related to sand depth and that it is affected by the operation mode (continuous mode presents better removal) (Verma et al. 2017).

Regarding pH values, a decreased in the value of pH shows biological processes related to removal of organic matter through oxidation reactions and formation of carbon dioxide (Zheng et al. 2010).

In the case of BOD, Nakhla and Farooq (2003) showed that the removal decreases when the inflow rate is increased, larger sand size and an increased filter depth is used.

In the case of COD removal, the expected values are lower than for BOD because after the previous treatment processes, most of the COD left is non-degradable (Nakhla and Farooq 2003). Regarding sand size, higher removal values of COD were found for fine sand (33.4 to 40 %) compared to coarse sand (11.7 to 35 %) (Nakhla and Farooq 2003). Nakhla and Farooq (2003) presented the relationship between COD and TSS with a removal of 2.04 mg COD per mg TSS.

Additionally, some experiences with grey water presented by Verma et al. (2017) showed that COD removal can reach up to 90 % for grey water even when the wastewater was not readily biodegradable.

Regarding processes involving nitrogen, a relation was mentioned by Nakhla and Farooq (2003) for denitrification with a requirement on the range of 3.5 to 4.5 mg COD/mg TN.

What is more, the removal for total nitrogen (TN) was approximately 2.3 mg/L of TN in the study of Nakhla and Farooq (2003). Additional results of that study showed that at low concentrations in the inflow water (around 5 mg/L), simultaneous nitrification-denitrification processes can occur. Furthermore, there was a good correlation between nitrification, denitrification, and total nitrogen removal efficiency with filtration rates and sand size, but no impact by the bed depth in the range 0.5 to 1.5 m. Regarding residual COD, it may prevent denitrification because is not readily biodegradable.

Zheng et al. (2010) mentioned that ammonia concentration decreased sharply within the first 10 cm of the upper sand layer with the corresponding increase of nitrate content to then continue with a constant concentration. In the studies mention by Nakhla and Farooq (2003) regarding the distribution of the nitrifiers in low loaded slow sand filters, ammonia—oxidizers (*Nitrosomonas*) were more abundant and better distributed along the filter depth than the nitrite—oxidizers (*Nitrobacter*).

The removal of phosphorus in sand filters, where the objective is to remove this compound, is influenced by the hydraulic and organic load of the incoming wastewater and the physical and chemical properties of the filter material (mineralogy, grain size and pH). Among the dominant reaction in these filters, there are adsorption to iron and aluminium oxides and precipitation of phosphorus (Vidal et al. 2018). Katukiza et al. (2014) reported removal efficiencies for total phosphorus and orthophosphates with silica sand filters of 86 and 84 %, respectively. For the same conditions with activated carbon as filter media, there was a complete removal of both parameters in grey water.

Regarding other contaminants that can be remove by SSFs in different degree, Verma et al. (2017) mentioned the removal of recalcitrant xenobiotics and pharmaceuticals by sorption process, disinfection byproducts (trihalomethanes and haloacetic acids) by biodegradation process and organic micro-pollutants through its used by microorganisms for biomass built up. In addition, Zheng et al. (2010) studied the removal of biolopolymers by biodegradation process.

Table 2.7 Removal rates for different physicochemical parameters by slow sand filters

Author	Initial Values	Parameter and removal rate	Additional information
Nakhla and Farooq (2003)	10 to 12 mg TSS/L	63.9 % of TSS	Hydraulic load between 0.19 to 0.38 m/h
	8 to 22mg TSS/L	21.8 to 43.4 % of TSS	Coarse sand —0.5 mm diameter Effluent quality: 6 to 9 mg TSS/L Filter depth 1.5 m
		49.6 to 71 % of TSS	Fine sand — 0.3 mm diameter Effluent quality: 3 to 6 mg TSS/L Filter depth 1.5 m
	0.2 to 0.95 NTU	33 to 56 % of NTU	Effluent quality: 0.1 to 0.3 NTU Coarse sand —0.5 mm diameter Filter depth 1.5 m
		40 to 62 % of NTU	Fine sand — 0.3 mm diameter Filter depth 1.5 m
	2.8 to 6.1 mg BOD_5/L	58.4 to 78.5 % of BOD	Hydraulic load: 0.19 m/h Effluent quality: 1 to 2 mg BOD/L and 27 to 30 mg COD/L Filter depth 1.5 m
	32 to 58 mg COD/L	16.6 to 46.2 % of COD	
	3.6 mg TKN/L 4.6 mg TN/L	47.2 to 83.4 % nitrification 72.9 to 81.5 % of denitrification	Filter depth: 0.8 m Effluent quality:0.6 to 1.2 mg TKN/L and 1.5 to 2.75 mg TN/L Hydraulic load: 0.19 m/h
Langenbach et al. (2009)	4.4 to 14.3 mg TSS/L	70 to 84 % of TSS	Effluent from secondary clarifier on pilot scale systems Effluent quality: 1.2 to 2.3 mg/L
	-	73 to 89 % of turbidity	Effluent quality: 0.5 to 0.8 NTU
Tyagi et al. (2009)	35 to 65 NTU	91.6 % of turbidity	Sand depth of 0.54 m and grain size of 0.43 mm Hydraulic load: 0.14 m/h
	55 mg/L	78 % of BOD_5	Effluent quality: 11.5 BOD mg/L
	-	71 to 83 % of COD	Effluent quality: 27 mg/L of COD
	110 to 180 mg/L TSS	82 to 94 % of TSS	Effluent quality: 11 to 30 mg TSS/L Hydraulic load: 0.14 m/h
Katukiza et al. (2014)	1125 mg BOD_5/L	61 to 67 % of BOD_5	Higher removal values for a hydraulic load of 20 cm/d and lower for 40 cm/d Grain size: between 1.18 and 2.56 mm Material: Lava rock and silica sand
	2861 mg COD/L	69 to 70 % of BOD_5	
	996 mg TSS/L	79 to 86 % of TSS	
	24.7 mg NH_4—N/L	61 to 69 % of NH_4—N	
	3.8 mg NO_3—N/L	51 to 55 % of NO_3—N	
	2.9 mg TP/L	49 to 52 % of TP	
	2.7 mg PO_4—P/L	48 to 51 % of PO_4—P	
Ellis (1987)	24 mg TSS/L	88 % TSS	Hydraulic load: 3.5 m/d Grain size: 0.3mm Effluent from a trickling filter
	22 mg BOD_5/L	70 to 75 % BOD_5	
	106 mg COD/L	54 % COD	
	16 mg TSS/L	92% TSS	Hydraulic load: 7 m/d Grain size: 0.3mm Effluent from a trickling filter
	16 mg BOD_5/L	65 % BOD_5	
	110 mg COD/L	37 % COD	

g. Removal of bacteriological parameters

A large number of researches in slow sand filters are focused in pathogen removal as the system was primary used for drinking water purification. Jenkins et al. (2011) mentioned that the main removal mechanism for bacteria and virus in the SSF is straining, adsorption, attachment and biological activity such as predation.

For drinking and wastewater treatment, SSFs have shown 99 % of removal rate for turbidity and suspended solids (Verma et al. 2017). Also, the removal of pathogens has been well studied. While for potable water treatment, the bacterial count can be reduced by up to 99.9%, the efficiency for wastewater treatment is far less, as shown in different studies (Ellis 1987).

The pathogen removal efficiency has also been evaluated in wastewater treatment (Ellis 1987; Ellis and Aydin 1995; Jenkins et al. 2011; Lekang and Kleppe 2000), with special focus on water reuse.

As many studies have use different material, hydraulic loads and slow sand filter designs, the removal efficiencies varied through different studies. Table 2.8 shows some removal efficiencies for different bacteriological parameters.

Table 2.8 Removal rates for different microbiological parameters by slow sand filters

Author	Initial Values	Parameter and removal rate	Additional information
Ellis (1987)	1.37*10^6/100mL	91 to 96 % removal of coliform count	Effluent of TF treated by SSFs Hydraulic load: 3.5 m/d Sand size: 0.3 and 0.6 mm Occasional values over 99 %.
Langenbach et al. (2009)	10^4/100mL *E. coli*	Elimination of 1.9 to 2.6 log units of E. coli	Effluent quality: 11 to 142 CFU[3]/100 mL d_{10}= 0.23 to 0.82 mm
	3.1*10^3/100mL	Elimination of 1.9 to 3.0 log units of intestinal *Enterococci*	Effluent quality: 2 to 24 CFU/100 mL d_{10}= 0.23 to 0.82 mm
Jenkins et al. (2011)	2*10^6 CFU/100mL	Elimination of 1.4 log CFU of fecal coliforms on average	Grain size: 0.14 and 0.52 mm Hydraulic heads:10, 20, 30 cm Hydraulic loads: from 0.01 m/h to 0.41 m/h In the best conditions: removal of around 1.82 log for bacteria (98.5 percent) / Intermittent SSFs
Katukiza et al. (2014)	4.2*10^6 CFU/100mL	Elimination of 1.98 to 2.52 log *E. coli*	Silica sand SSFs Grey water treatment Higher removal values for a hydraulic load of 20 cm/d and lower for 40 cm/d
	6.9*10^7 CFU/100mL	Elimination of 1.55 to 1.83 log TC	
Tyagi et al. (2009)	4.3*10^6 MPN/100mL of TC	99.95 % of Total Coliforms (TC) and Faecal Coliforms (FC)	Hydraulic load: 0.14 m/h Effluent quality: 2.1*10^3 MPN/100mL of TC 1.3*10^3 MPN/100mL of FC 3.8*10^2 MPN/100mL of FS
	2.3*10^6 MPN/100mL of FS	99.99 % of Fecal Streptococci	
D'Alessio et al. (2015)	Between 10^4 and 10^6 MPN/100mL of Total Coliforms	95% of Total Coliforms	Silica sand Grain size: 0.15 to 0.3 mm Hydraulic load: 0.05 m/h

[3] Colony Forming Units

Jenkins et al. (2011) used an intermittent SSF to evaluate and model the interaction of sand size and hydraulic head with the removal of pathogen. According to that research, the contact time appeared to be the most critical factor for bacteria removal and a long residence time operation increased the removal of bacteria by 0.29 log-units. Furthermore, when eliminating the residence time effect of the sand size change, a reduction of the sand size can increase bacteria removal by 0.16 to 0.3 log-units. However Langenbach et al. (2010) stated that in the modeling of the removal of fecal indicator bacteria, the *schmutzdecke* is the most important process and that the hydraulic rate (an increase on the hydraulic rate reduces the retention time) had no substantial impact.

Seeger et al. (2016) mentioned additional general information regarding the different ranges of pathogen removal efficiency such as 0.3 to 3.5 log-units for Total Coliforms (TC), 2 to 2.4 log-units for Faecal Coliforms (FC), 1.9 to 4.1 log-units for E.coli and 0.7 to 3.7 log-units for enterococci, without mentioning the original concentrations.

h. Related research using slow sand filters

SSFs have been used mainly to remove pathogen and particles in drinking water treatment (Langenbach et al. 2009) and most recently in pharmaceutically active compounds (D'Alessio et al. 2015). However, in recent years, many researches have shown the removal properties for domestic wastewater treatment (Bahgat et al. 1999; Ellis 1987; Jenkins et al. 2011; Tyagi et al. 2009), grey water (Katukiza et al. 2014), specific compound removal in wastewater such as phosphorus (Vidal et al. 2018), nitrification-denitrification processes (Nakhla and Farooq 2003), reuse in general (Langenbach et al. 2009, 2010) or in irrigation (Seeger et al. 2016) and as a treatment prior to ultrafiltration for organic foulant removal (Zheng et al. 2010).

Bahgat et al. (1999) studied slow sand filters as a treatment alternative for wastewater treatment in Egyptian villages, using different materials, operational modes and evaluating the nitrification activity. Ellis (1987) evaluated the viability of slow sand filters as a tertiary treatment after treatment from a activated sludge system and trickling filters using different hydraulic loads and grain size. Parameters such as coliforms, suspended solids and BOD were evaluated. Some conclusions of this study showed that the grain size was independent from the effluent quality. Also, that the removal efficiency for suspended solids and coliforms was slightly higher for effluent from a trickling filter, while for BOD_5, there were mix results when the hydraulic load was increased.

Jenkins et al. (2011) focused on bacterial, virus and turbidity removal in intermittent slow sand filters to treat domestic wastewater in developing countries, using modeling and experimental results. Some findings point to the sand size as a critical design factor on performance. In addition, the best design operation combines fine sand with long residence time operation. Tyagi et al. (2009) carried out an study to evaluated the treatment of a UASB effluent by slow sand filters. Table 2.7 and Table 2.8 show the results for that research with high removal values for turbidity, suspended solids, COD, BOD and total and fecal coliforms with the lowest hydraulic rate (0.14 m/h) from the three settings evaluated (0.14, 0.19 and 0.26 m/h). Those studies probed the efficiency of slow sand filters to treat tertiary effluents from domestic wastewater treatment systems.

Katukiza et al. (2014) used grey water pretreated with a settlement tank to evaluated the removal efficiency of water quality parameters. The results showed (see Table 2.7) that filter columns in series have a better removal efficiency with the lowest hydraulic load of 20 cm/d. Furthermore, the removal of pollutants increased with depth with the highest pollutant removal efficiency occurring in the top 15 cm layer. A more contaminant oriented removal process was studied by Vidal et al. (2018) where phosphorus was effectively removed due to alkaline phosphorus filters.

A study evaluating simultaneous nitrification and denitrification processes was done by Nakhla and Farooq (2003). This study showed that the nitrification, denitrification and total nitrogen removal efficiency were inversely proportional to the square root to the filtration rate and sand size. Also, nitrification showed

the most sensitivity to filtration rate and sand size. For example, for nitrification reactions, the flow (Q) followed the equation with a correlation coefficient (R^2) of 0.72:

$$-2.5 * Q^{0.5} - 911.6 + 1007.6 * Q^{0.02} \quad 2.1$$

Langenbach et al. (2009) focused in particle and pathogen removal with the objective of reuse. The study evaluated different hydraulic loads (0.5, 0.1 and 0.2cm/h), grand size (between 0.25 to 0.68 mm) and filter depth treating secondary clarifier effluent. The pathogen removal was achieved by all the sand filters configuration in the range of hydraulic loading from 0.05 to 0.20 m/h. Seeger et al. (2016) monitored pathogen removal in slow sand filter with different configurations such as a standard filter, recirculating filter, static cascade and rotating cascade. The rotating and static cascade showed effluents that complied with European irrigation water standards for *E. coli* and enterococci. The cascade system is also easier to implement and maintain.

A different approach was presented by Zheng et al. (2010). The research focused on slow sand filter removing biopolymers to improve posterior ultrafiltration. As biopolymers were verified to be biodegradable, most of the removal took place in the upper layer.

2.1.6. Microbiological parameters in wastewater treatment

The removal of different pathogen in wastewater treatment facilities requires indicators. Coliforms are considered more of an indicator for wastewater treatment efficiency than for faecal pollution (Institute for Environment and Sustainability 2003). *E. Coli* reflects the behavior of pathogenic enteric bacteria and it is a suitable indicator for faecal contamination, although thermotolerant and total coliforms can also be used in the case of irrigation for agriculture (Joint Research Centre 2017).

In the case of protozoa, *Clostridium perfringens* spores can indicate the effectiveness of filtration processes and are recommended as indicators for treatment control in case of disinfection-resistant pathogens (Joint Research Centre 2017). *E. coli*, enterococci, and *Clostridium perfringens* are more associated with the faecal waste of warm-blooded animals (Cabelli and Miescier 1982).

Regarding viruses, as they are more resistant to environmental conditions and treatment technologies (filtration and disinfection) than bacteria, total coliphages, F-specific coliphages and somatic coliphages are usually used as indicators due to their share properties with human viruses (Joint Research Centre 2017).

2.2. Guidelines for water reuse

As many studies have focus on the benefits of further wastewater treatment and consequently reuse, the main target for reuse has been irrigation. Pathogen concentrations are the limiting factor due to its potential contamination of crops, and risks to human health (from workers to consumers). Parameters such as COD and BOD are required in wastewater treatment facilities. However, pathogen removal is also considered if reuse is one of the objectives. Different guidelines and regulation set coliforms as indicators of pathogen removal, such as total, faecal and thermotolerant. The World Health Organization sets the concentrations for those parameters to below 1000 CFU/100 mL for unrestricted irrigation of human food crops for raw consumption (Blumenthal et al. 2000). In the case of European guidelines, the values range between 200 to 100 CFU/100 mL depending on the country for *E. coli* (Thermotolerant). Additionally, the conversion process of nutrients that allows them to be available for the crops is an additional benefit (Seeger et al. 2016).

In the case of Peruvian law, ranges for irrigation are less restrictive as European guidelines, the limits for thermotolerant coliforms and E. coli are 1000 MPN/100 mL for a non-restrictive use and 2000 MPN/100 mL for thermotolerant coliforms for restrictive use (Ministerio del Ambiente 2017). In addition, values for nitrite and nitrate together is 100 mg/L for irrigation and animal consumption, and a maximum of 10 mg/L for only nitrites.

As the WHO reports its limits in CFU and different countries report in MPN (Most Probable Number), Cho et al. (2010) evaluated the difference between these two method. The results showed that for *E. coli* the concentrations in MPN are larger in one order of magnitude than with the CFU, except for measurements in winter. In the case of Enterococci bacteria, the values in MPN are lower than those in CFU.

2.3. Key points

Trickling filters:

- Trickling filters are an attached growth wastewater treatment process appropriate to small- to medium sized communities (population between 501—10000 (EPA 2008)) due to its simplicity of operation, low energy requirement, low maintenance, ability to handle shock loads and lower sludge production.
- Depending on the treatment objectives, trickling filters have different operational conditions: roughing, carbon oxidation, combined carbon oxidation and nitrification, separate stage nitrification.
- The hydraulic loads recommended for high-rate media is approximately 1.8 m/h, although lower values have been reported. Total organic loads ranges from 0.3 to above 1.0 Kg BOD/(m^3*d).
- Sludge production depends on biofilm growth. When the biofilm concentration reaches critical values, the biofilm falls (sloughing) and the sludge is collected in a sedimentation tank for further treatment. High-rate systems can produce 0.35 kg/kg of sludge which is lower than for activated sludge system. In addition, they have shorter hydraulic retention time and the sludge is usually less mineralized as low-rate system.
- Recirculation allows the dilution of the incoming wastewater, increases the dissolved oxygen at the upper parts, helps dilute shock loads, controls the biofilm thickness and decouples the hydraulic and organic loading. Increasing the recirculation beyond the required amount to ensured complete wetting, usually does not improve performance.
- The biofilm is composed by microorganism such as heterotrophic and autotrophic nitrifying bacteria, protozoa, fungi, worms, larval filter flies. An optimum thickness for the biofilm is 0.15 mm. For biofilm thickness larger than 0.2 mm, only the 0.2 mm of the outer layer remains anaerobic. An increase in hydraulic loading can produce a more active, thinner and aerobic biofilm, where the biofilm excess is slough down the system.
- In carbon oxidation and combined carbon oxidation and nitrification systems, the upper part of the trickling filter is usually composed by heterotrophic bacteria due to the rapid growth and the abundance of organic matter. Only when low concentration of organic matter is reached, the autotrophic bacteria starts to develop in the lower part of the filter and nitrification occurs. Both types of bacteria are constantly competing for space.
- A limiting factor for the removal of organic matter is the oxygen diffusion. When the BOD removal reaches values of approximately 90 %, the reaction rates decrease, and the filter turns into a sludge digester unit. Low organic load can improve the nitrification rate.
- The removal of total phosphorus in trickling filter is mainly due to microbial action, while soluble phosphate is not removed.
- Some problems usually present in trickling filters are flies, ponding, odor and icing.
- Hydraulic flushing of TF allows to reduce the thickness of the biofilm, reduce the presence of flies and ensure the passage of adequate ventilation.

Rapid sand filters

- The most important removal mechanisms in rapid sand filters is straining. This system allows the removal of suspended solids, being the grain size its main characteristic for operation. Backwashing is required to clean the system and prevent clogging.

Slow sand filters

- Slow sand filters have been used mainly for drinking water treatment. However, more studies highlight its use for wastewater treatment, especially tertiary treatment.
- Slow sand filters have lower hydraulic loads than the rapid sand filter and backwashing is not usually performed.
- The performance of slow sand filter is influenced by wastewater quality, hydraulic loading, temperature, surface ripening, hydraulic retention time and surface.
- Slow sand filters are susceptible to clogging due to the biomass growth on the upper layer known as *schmutzdecke*.
- Grain size is a key parameter for slow sand filters. The grain size values can range between 0.15 to 0.4 mm. The type of material and size can remove specific pollutants.
- The *schmutzdecke* is a complex mixture of humus, sand, algae, metazoan, protozoans and diatoms, and similar as the mixture found in activated sludge systems. The thickness of the layer can vary between 0.5 to 2.5 cm and it can take two weeks to develop.
- The most common process in slow sand filters involves pathogen retention with straining and adsorption. Additionally, other factor such as natural die-off, predation and lysis can contributed. In mature systems, the removal of the *schmutzdecke* has a minimal impact in efficiency.
- Head loss in slow sand filters due to the *schmutzdecke* requires maintenance to ensure the passage of water. When the head loss reaches the available head, the upper layer is scrapped off.
- Slow sand filters have proven to be effective in removing water quality parameters such as BOD (58—78 %), COD (37—83 %), ammonium (61—69 %), nitrate (51—55 %), total nitrogen (73—82 %), total phosphorus (49—52 %) and microbiological parameters such as total and fecal coliforms (0.3—3.5 log units), Enterococci (0.7—3.7 log units) and E. coli (0.7—4.1 log units) for different operational conditions and filter materials.
- New studies have evaluated the removal efficiency of different compounds such as recalcitrant xenobiotic, pharmaceuticals, organic micro-pollutants, biopolymers with positive results.

3 Experimental setup and conditions

3.1. Experimental Setup

The experimental set-up and laboratory analysis of physico-chemical parameters were done in the Institut für Siedlungswasserbau, Wassergüte- und Abfallwirtschaft (ISWA) at University of Stuttgart. The microbiological parameters were analyzed by the TZW DVGW-Technologiezentrum Wasser, Karlsruhe.

The units were composed sequentially by a 4.5 m^3 storage tank with municipal wastewater pretreated with sieves of 1 mm of opening, a trickling filter (TF) with an inflow of 76.5 l/h (1.84 m^3/d) and a sedimentation tank under it. A downflow rapid sand filter was included with an inflow of 40 l/h (0.96 m^3/d) of the treated effluent from the TF and 3 downflow slow sand filters (SSFs) in parallel with sand of 60 cm of height and different hydraulic loads. The water was transported through the system by pumps to the downflow rapid sand filter. The last stage composed by the downflow SSFs was fed with peristaltic pumps. The dimensions for each treatment unit are detailed in Appendix 1, while Figure 3.1 shows the complete treatment process.

3.2. Operational Conditions

The TF was fed with municipal wastewater with three different conditions for each stage of the experiment, as detailed in Table 3.1.: high organic load, medium organic load and low organic load. For each stage, three SSFs were fed with the treated water with different hydraulic loads: 0.15, 0.3 and 0.6 m/h.

Each organic load into the TF was obtained through dilution, with the values detailed in Table 3.1. The organic load was monitored through Chemical Oxygen Demand (COD) determination, so the relationship between Biochemical Oxygen Demand (BOD) and COD was considered 50 %. The values for the total organic load were calculated based on the procedure explained in the Technical guideline ATV-DVWK-A281. The hydraulic load for the TF was kept constant during the duration of the experiment with a value of 26.5 $m^3/(m^2*d)$ (1.1 $m^3/(m^2*h)$). The value is higher than the 0.8 $m^3/(m^2*h)$ recommended for high-rate systems with plastic media by ATV-DVWK-A281, but inside the recommended ranges of 0.4—1.1 $m^3/(m^2*h)$ (Daigger and Boltz 2011).

Table 3.1 Organic loads fed into the trickling filter and hydraulic loads for the slow sand filter

Organic load for the trickling filter	COD load	Average COD value for the inflow wastewater	Average BOD value for the inflow wastewater	Hydraulic Loads into the slow sand filters
High load	0.41 kg COD/m^3	412 mg/L	206 mg/L	0.15, 0.3 and 0.6 m/h
Medium Load	0.28 kg COD/m^3	284 mg/L	142 mg/L	0.15, 0.3 and 0.6 m/h
Low Load	0.27 kg COD/m^3	267 mg/L	134 mg/L	0.15, 0.3 and 0.6 m/h

The experiment allowed a total of 9 different set-ups to be analyzed, as results of the factors: organic load into the TF and hydraulic load into the SSFs.

Each of the treatment units were operated according to the flows detailed in Table 3.2.

Table 3.2 Flows for each treatment unit

Treatment Unit	Unit	Value
Storage tank	m^3	4.5
Tricking filter	m^3/d	1.84
Rapid sand filter	l/h	40
Slow sand filter- Filter 1	m/h	0.15
Slow sand filter- Filter 2	m/h	0.3
Slow sand filter- Filter 3	m/h	0.6

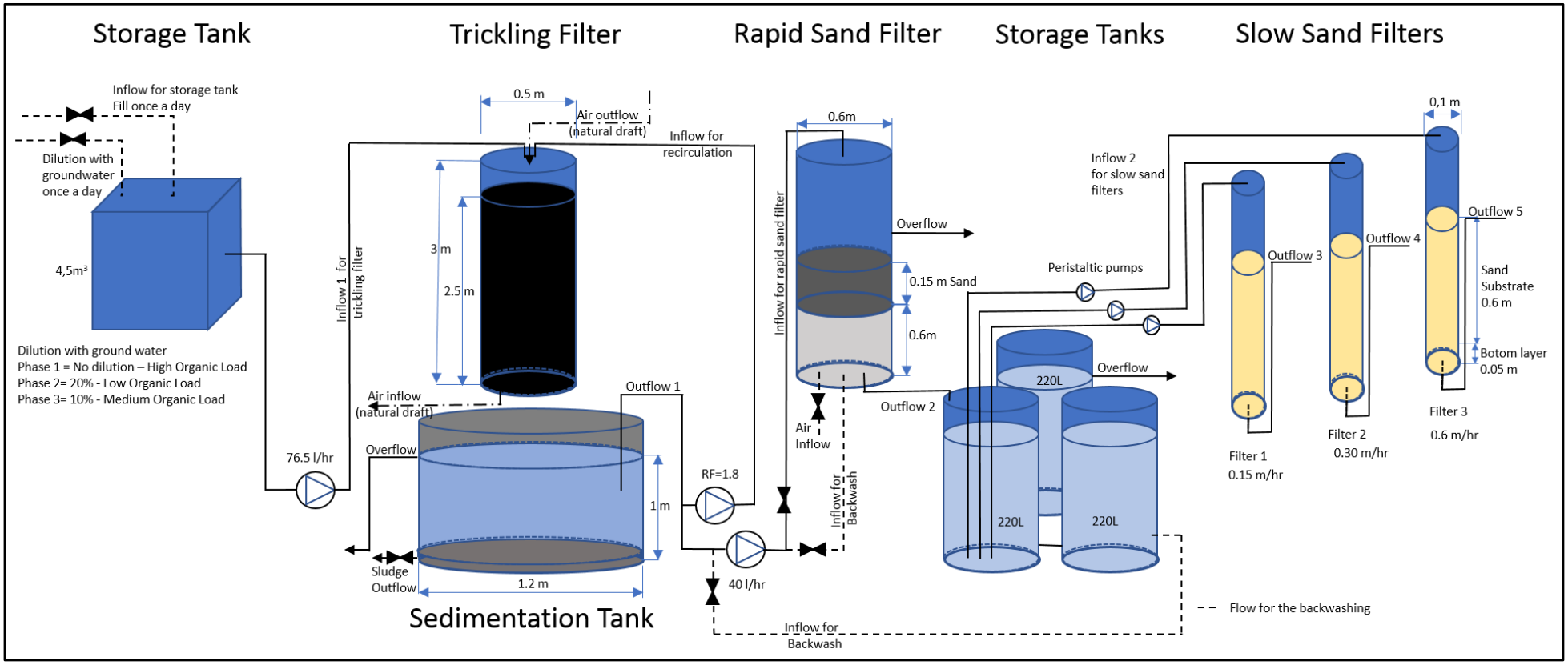


Figure 3.1 Schematic of the treatment process

The storage tank was filled daily with municipal wastewater pretreated by sieves (size 1 mm). The dilution was applied to this tank when required according to Table 3.1. The wastewater inside the tank was continuously recirculated to avoid sedimentation and to allow a uniform mixing when diluted. The TF was fed by a pump running continuously during the whole experiment. The recirculation factor to ensure the complete wetting of the TF was set to 1.8 and the hydraulic load was 1.1 $m^3/(m^2*h)$. The rapid sand filter was fed continuously by a pump. The same pump was also used for the backwashing of the filter when required. The slow sand filter 1, 2 and 3 were fed with peristaltic pumps on a continuous regime and empty bed contact time (EBCT) of 106, 53 and 27 minutes, respectively.

Regarding the operation and maintenance, the TF was flushed once a week or when sloughing of particles was observed. The flushing was performed with the same pumps used for recirculation. In the case of the rapid sand filter, the backwashing with treated effluent and compressed was performed when the filter was clogged. The frequency depended on the concentration of suspended solids of the trickling filter effluent. In the high organic load stage, the backwashing was performed every day, while for the low organic load, the frequency was two times a week.

3.3. Experiment implementation

The different stages of the process were summarized in the Table 3.3.

Table 3.3 Different stages during the experiment

Stage	Duration in days	Description
Start-up	1-68 [68]	The trickling was monitored to determine the adaptation to the organic load.
Installation of slow sand filters and beginning of their operation	69-104 [35]	Implementation of different treatments before slow sand filter to avoid clogging in the system. Installation of a rapid sand filter on day 78. The stage concluded with the change of the monitoring points to evaluate more accurate results.
First experimental stage—Phase 1	104-134 [31]	High organic load into the trickling filter. Operation of the three slow sand filters
Second experimental stage—Phase 2	134-180 [47]	Low organic load into the trickling filter. Operation of the three slow sand filters
Third experimental stage—Phase 3	180-207 [28]	Medium organic load into the trickling filter. Operation of the three slow sand filters

During the start-up phase, the TF was monitored. As the inflow wastewater had a high organic load (COD_{Total} between 400 and 600 mg/L), dilution was performed with groundwater. The TF was set initially with a recirculation factor of 1 until day 39, so the trickling filter could adapt, and the biofilm could develop. Since day 39, the recirculation was changed to a factor 1.8, reducing the periodical dilution with groundwater. On day 69, the slow sand filters started working. Until day 78, different options where implemented to allow a continuous operation of the SSF. As the trickling filter was fed with high organic load and the suspended solids concentrations were too high to feed the SSFs directly with the effluent from the trickling filter, the system was configured to run in the following way: trickling filter, rapid sand filter, storage tanks and SSFs.

In the case of all the SSFs, the removal of the *schmutzdecke* was done when the water column was close to a value of 30 cm to avoided overflowing of the filters.

On day 104, the monitoring points were changed to reduce the effect of the storage tanks performing biological treatment and Phase 1 started. Additionally, new monitoring point was implemented just before the wastewater inflow to the SSFs and the hoses that fed the storage tanks were modified to reduce the residence time of the wastewater after the rapid sand filter and before the SSFs.

On day 111, there was a momentary change in the conditions of the container that fed wastewater into the trickling filter, which caused that sludge from the tank to be dragged into the trickling filter and the container tank under the trickling filter. The high content of sludge caused an increase in the suspended solids. Through flushing of the trickling filter and the container tank, the excess sludge was eliminated.

On day 134, before changing from phase 1 to 2, 40 cm of sand from the SSFs was removed and cleaned with water in all the filters to remove the *schmutzdecke* and the bacteria present before the start of the next experiment. On day 137, the organic load on the trickling filter was changed by a constant dilution of groundwater of 0.9 m^3 in the 4.5 m^3 tank that feed the trickling filter and fed low organic loads into the trickling filter. The recirculation factor was kept unchanged at 1.8.

On day 179, the filters were cleaned to start with phase 3. On day 180, the organic load on the trickling filter was changed to a dilution of 0.45m^3 in a 4.5m^3 tank and continued in the same set-up until the end of the experiment. On day 201 and 202, a lower wastewater inflow was detected into the storage tank. This caused a higher dilution with groundwater inside of the storage tank, which influence the inflow concentration into the trickling filter. On day 203, the dilution was once again working to the setting conditions. The monitoring continued until day 207 when the last monitoring was performed including the bacteriological parameters.

3.4. Analyses performed

3.4.1. Monitoring points

The removal efficiency of the trickling filter was monitored from the beginning of the experiment on day 1 with two monitoring points and the monitoring of the rapid sand filter began on day 78. In case of the SSFs, the monitoring began with the change of monitoring points on day 104 with a total of seven monitoring points as observed in Figure 3.2 and detailed in Table 3.4.

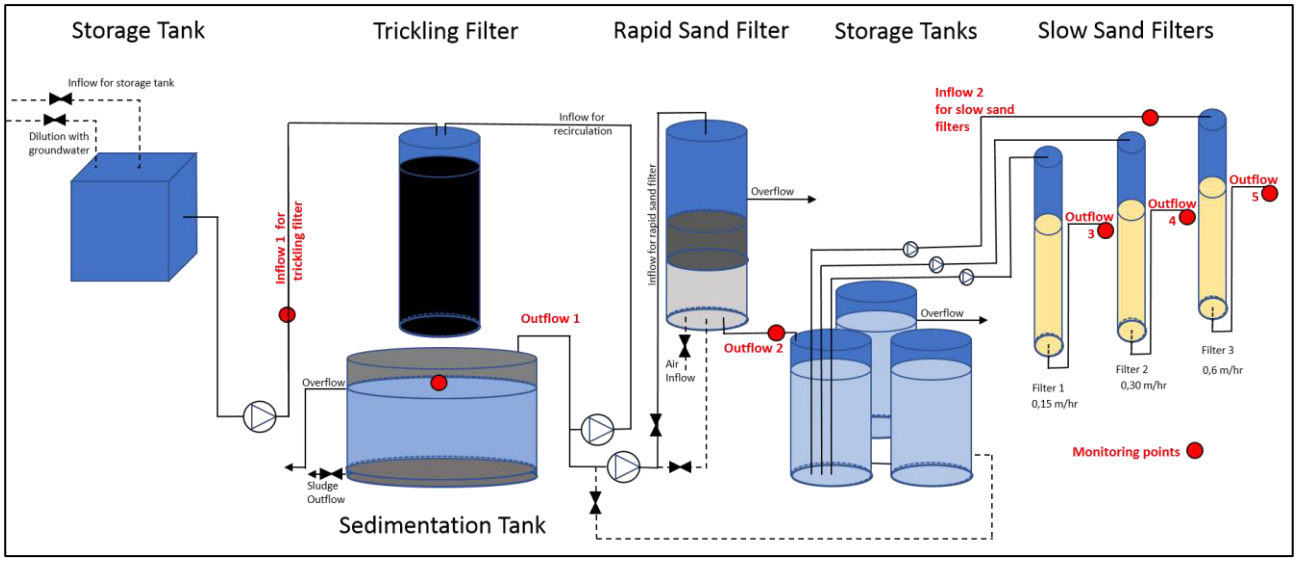


Figure 3.2 Monitoring points

The sampling was done between 9 and 10 am on the sampling day with a punctual sample and the analysis was performed immediately after.

Table 3.4 Description of monitoring points

Monitoring point	Description
Inflow 1 (in TK)	Wastewater from the storage tank before entering the trickling filter. Depending on the phase, the wastewater was diluted

Outflow 1 (out TF)	Treated wastewater from the trickling filter. From day 1 to 125, the sample was composed by the water falling directly from the trickling filter. From day 126 until the end of the experiment the sample was taken from the surface of the tank directly under the trickling filter. This was done to avoid the effect of the sludge in the uniformed sample.
Outflow 2 (out RSF)	Treated wastewater after the rapid sand filter and before the storage tanks.
Inflow 2	Treated wastewater before the slow sand filters. This point was added to obtain more accurate results of the treatment effect of the slow sand filters and avoid the effect of the residence time of the storage tanks.
Outflow 3 (SSF1)	Treated wastewater after the slow sand filter 1
Outflow 4 (SSF2)	Treated wastewater after the slow sand filter 2
Outflow 5 (SSF3)	Treated wastewater after the slow sand filter 3

3.4.2. Monitoring parameters

The parameters analyze for all the monitoring points are summarized in Table 3.5 and Table 3.6. In the case of COD, the analysis was performed for un-filtrated samples (COD_{Total}) for all monitoring points. In addition, COD was also analyzed for the monitoring points Inflow 1 (in TK) and Outflow 1 (out TF) after filtering with a membrane of 0.45 µm of pore size ($COD_{Dissolved}$).

Table 3.5 Physicochemical parameters for each monitoring point

Parameter	Technique	Determination Method
	In situ	
Temperature (T)	Electrometric	Temperature Sensor NTC 30kΩ at 25 °C
Electrical conductivity (EC)	Electrometric	Four electrodes measurement
pH	Electro-chemical	Reference electrolyte solution KCl 3 mol/L, Ag^+ free
Dissolved oxygen (OD)	Optical oxygen sensor	Optical measurement based on photoluminescence
Turbidity	DIN ISO 27027	Nephelometric
	Laboratory	
Biochemical Oxygen Demand (BOD)	DIN EN 1899-1:1998	Dilution and seeding method with allylthiourea addition
Chemical Oxygen Demand (COD_{Total} and $COD_{Dissolved}$)	ISO 15705:2002 EPA 410.4	Silver-catalyzed oxidation with potassium dichromate / sulfuric acid at 148 °C during a two-hour period. Machery-Nagel Rapid Test
	Standard Methods 5220 D	Reactor Digestion Method. Hach Rapid Test
Suspended Solids (SS)	DIN 38409 Teil 2	Determination of non-filterable substances
Total Phosphorus (TP)	DIN EN ISO 6878-D11	Ammonium molybdate spectrometric method
	Standard Method 4500-P E	Ascorbic Acid Method
Orthophosphates (PO_4-P)	DIN EN ISO 6878:2004	Ammonium molybdate spectrometric method

Ammonium (NH_4-N)	DIN 38406-5	Determination of ammonia-nitrogen through sodium dichloroisocyanurate and sodium salicylate
Nitrite (NO_2-N)	DIN EN 26777:1993	Molecular absorption spectrometric method
Nitrate (NO_3 -N)	DIN 38405-9:2011-09	Spectrometric determination of nitrate
Total nitrogen (TN)	DIN EN ISO 11905-1	Photometric determination with 2,6-dimethylphenol in sulphuric acid / phosphoric acid mixture. Machery-Nagel Rapid Test

Table 3.6 Bacteriological parameters for each monitoring point

Parameter	Technique	Determination Method
E. Coli / Bacterial Coliforms	DIN EN ISO 9308-2	Detection of Most Probable Number (MPN) on the Colilert®-18/ Quanti-Tray® system
Enterococcus	DIN EN ISO 7899-2	Membrane filtration, culture on S+B-Agar and verification on GÄA-Agar

Additionally, the Total Kjeldahl Nitrogen[4] (TKN) was calculated based on the values of TN, NO_2—N and NO_3—N.

3.4.3. Monitoring frequency

The monitoring frequency can be divided into three parts. The first part was during the start-up of the system, from day 1 to day 26. From day 27 to day 103, the system was being adjusted so the monitoring was not performed in all seven monitoring points, but the frequency was of 3 times a week. The last part was from day 104 to day 207 (end of the experiment) with a monitoring frequency of 2 times a week. Table 3.7 shows the frequency for the physico-chemical parameters. In the case of BOD, the monitoring was performed four times between the days 78 and 103. The suspended solids were only monitored for the points Outflow 1 (out TF) and Outflow 2 (out RSF). The biological parameters were monitored one time at the end of each experimental stage as shown in Table 3.3.

Table 3.7 Monitoring frequency of the physico-chemical parameters during the experiment

Phase	Days	Description	Parameters	Frequency
Start-up phase	1–26 [26]	Start-up period of the trickling filter	NH_4—N, NO_2—N, NO_3—N	Once a week
			COD	Once a week
Adjustment phase	27–103 [77]	The rapid and slow sand filters were implemented, and the monitoring points were defined	T, EC, pH, OD, turbidity, COD, PO_4—P, NH_4—N, NO_2—N, NO_3—N	Three times a week
			TN	Two times a week
			TP	One time a week
Experimental phase	104–207 [104]	Operation of all the treatment units and all the monitoring points were already defined	T, EC, pH, OD, turbidity, COD, SS, PO_4—P, NH_4—N, NO_2—N, NO_3—N	Two times a week
			TN, TP	One time a week

Regarding the laboratory equipment, the list of equipment used for the evaluation of in-situ parameters and laboratory analysis is detailed in the Appendix 2.

[4] $TKN = TN - (NO_2 - N) - (NO_3 - N)$

4 Results and Discussion

4.1. Start- up phase - trickling filter and rapid sand filter implementation

4.1.1. Organic loading and Biochemical Oxygen Demand into the trickling filter

The total organic load (TOL) and the average BOD_5 were 0.72 Kg BOD/m^3 and 151 mg/L, respectively. Appendix 3 shows more detailed values used for the calculations for the TOL. A BOD_5 removal efficiency of 85 % was set for the trickling filter due to the experimental results detailed in Appendix 4. The TOL corresponds to the recommended values for high-rate systems between 0.31—1.00 Kg BOD/(m^3*d) (Riffat and Rumana 2013) or 0.64—1.60 kg BOD_5/m^3 (EPA 2000)

4.1.2. Monitored values during the start-up phase for trickling filter and rapid sand filter

The TF performed a good oxygenation of the treated water with values of dissolved oxygen of around 7.34 mg/L. The pH was predominantly neutral with a range between 7.5—7.7 for the TF, with lower values in the outflow wastewater. The temperature also was between 19—20 °C for the TF. The in-situ parameters were stable during this phase, as seen in the Appendix 5.

Regarding the Chemical Oxygen Demand (COD), high variability was detected for the inflow and outflow of the trickling filter for the COD_{Total} and $COD_{Dissolved}$, as shown in Table 4.1. This is due to the dilution performed to adapt the filter to domestic wastewater conditions. The removal efficiency of COD_{Total} for the TF and RSF was 73 and 69%, respectively with final COD_{Total} concentrations of 107 and 19 mg/L, respectively. More stable COD_{Total} values were reported for the outflow of the RSF, as seen in Table 4.1.

In the case of phosphorus species, after the treatment with the TF, orthophosphates (PO_4—P) values increased on average by 14 % (+0.7 mg/L). This can be attributed to hydrolyzation of total phosphorus (TP) by the development of biofilm in the TF. The TP concentration before the TF was 7.4 mg/L, with a removal of 10 % (1.3 mg P/L) by the TF. In addition, the removal of TP rate due to the RSF was 23 % (1.24 mg P/L), mainly due to the filtering of particles.

In regards of the nitrogen species, ammonia-nitrogen (NH_4-N) represented 86 % (57 mg/L) of the Total Nitrogen (TN) (66.7 mg/L) fed into the TF. After a removal of 27 % of TN (18 mg/L), the effluent was composed of 42 % of NH_4-N (27.9 mg/L) and 21 % nitrate (NO_3—N) (14.2 mg/L). Nitrite (NO_2—N) was under 1 mg/L, so no nitrification inhibition process was observed. Although, the organic load applied was higher than the recommended range for BOD removal and nitrification (0.2—0.4 kg BOD_5/(m^3*d), nitrification reactions were observed. Conditions such as a high content of dissolved oxygen in the treated water (seen in the in-situ parameters) and BOD_5 concentrations of the effluent lower than 30 mg/L were favorable for the reaction. Regarding the Total Kjeldahl Nitrogen (TKN), the BOD_5/TKN relation for domestic wastewater was expected to be 5.5 (using the loads per capita in ATV-DVWK-A 281). However, the calculated value of 2.3 was evidence of higher TKN and TN concentrantions than expected.

4.1.3. Summary of the start-up phase

During the start-up phase, the trickling filter adapted to domestic wastewater treatment in a high-rate load with COD_{Total} and $COD_{Filtrated}$ removal rates in the range of 52—89 % and 50—80 %, respectively. The high variability on the removal rates is due to the dilution applied to avoid an overload of organic matter. The BOD_5 results also showed removal rates of 85 %, from a concentration of 150 mg/L before the TF to 23 mg/L after the TF on average. The removal rates fell into the ranges reported by Tchobanoglous et al. (2014) with a BOD_5 removal between 70—90 % and final BOD_5 concentration value below 30 mg/L. Regarding the TP and PO_4—P, the TF removed 10 % of TP (1.34 mg P/L) due to biofilm growth and a slightly increase in PO_4—P (+0.7 mg/L) was an indication of anaerobic zones inside the biofilm in the filter media. Regarding nitrogen compounds, the TN removal rate was around 27 % (18 mg/L), NH_4-N was removed in 29.5 mg/L while the NO_2—N was under 1 mg/L. NO_3—N increased on average 14 mg/L. This showed nitrification and denitrification processes inside the TF, even though the organic load was higher than recommended for this reaction to take place (between 0.2 and 0.4 kg BOD_5/(m^3*d)). The evaluation of the reactions in the TF during

this phase, gave an idea of the predominant processes in the following phases. The BOD_5/TN/TP ratio recommended for needs of bacteria in an aerobic treatment such as TF is recommended to varied between 100:10:1 and 100:5:1. However this phase presented values of 27:9:1. This shows a deficit on BOD_5 to improve further degradation processes and biofilm development and explains the high nitrification rates for the TF.

Table 4.1 Physicochemical results monitored during the start-up phase

Sampling points	**Units**	**COD_{Total}**	**$COD_{Dissolved}$**	**TP**	**PO_4—P**
Inflow 1 (in TK)	mg/L ±	417.5 ± 171.0 (28)	129.0 ± 51.4 (27)	7.4 ± 2.6 (8)	5.2 ± 1.61(22)
Outflow 1 (out TF)	Std. dev	106.5 ± 52.1 (28)	39.4 ± 13.1 (27)	6.0 ± 0.9 (8)	5.9 ± 1.8 (22)
Outflow 2	(n values)	18.98 ± 3.9 (7)		4.1 ± 0.6 (3)	3.9 ± 0.8 (7)
Sampling points	**Units**	**TN**	**NH_4—N**	**NO_2—N**	**NO_3—N**
Inflow 1 (in TK)	mg/L ±	66.6 ± 15.4 (23)	57.3 ± 15.3 (31)	0.01 ± 0.01 (27)	0.5 ± 0.2 (30)
Outflow 1 (out TF)	Std. dev	48.6 ± 11.4 (23)	27.9 ± 15.7 (31)	0.9 ± 0.4 (27)	14.2 ± 7.9 (30)
Outflow 2	(n values)	30.0 ± 1.98 (4)		0.9 ± 0.5 (7)	21.3 ± 2.8 (7)

4.2. First phase

4.2.1. Organic loading into the trickling filter

The TOL and the average BOD_5 were 0.90 Kg BOD/m^3 and 198 mg/L, respectively (see Appendix 3). A BOD_5/COD relation of 0.5 was assumed (Appendix 4). This phase was considered as the one with the highest organic load into the TF.

4.2.2. In situ parameters

Regarding the dissolved oxygen in each monitoring point, the TF and RSF kept the values between 6.1 and 6.5 mg O_2/L (see Appendix 5). After the treatment by SSFs, the dissolved oxygen values were indirectly proportional to the total hydraulic loads (THL). For the lowest hydraulic load (0.15 m/h), the highest oxygen concentration was measured (6.9 mg O_2/L) and for the highest hydraulic load (0.6 m/h), the average oxygen concentration was 6.4 mg O_2/L. In addition, the pH values for all the monitoring points were between 7.4 to 7.7. After the TF, the pH and conductivity presented lower values (7.5 of pH and 1160 uS/cm) than the inflow (7.6 of pH and 1324 uS/cm). In the case of the slow sand filters (SSFs), the pH values were higher after the treatment (from 7.4 to between 7.5 and 7.7). The turbidity removal rates by the TF and RSF were 79 and 82 % (initial values of 133 and 28 NTU), respectively. In the case of all the SSF, the turbidity values at the end of the treatment were below 1.7 NTU with values for the hydraulic loads of 0.15, 0.3 and 0.6 m/h of 1.7, 1.4 and 1.3 NTU, respectively.

4.2.3. Chemical Oxygen Demand removal

The TF removed 80 and 61 % of COD_{Total} and $COD_{Dissolved}$, respectively (see Appendix 7) (initial values of 412 and 114 mg/L). Furthermore, the RSF removed 44 % of COD_{Total} (initial value of 80 mg/L). However, the removal rates of the COD_{Total} for the TF and RSF are inversely proportional. As the TF removal efficiency increased from 77 to 89 %, the removal efficiency of the RSF decreased from 37 to 12%. This can be explained by the fact that an increase in the removal rate of the COD_{Total} of the TF meant a reduction of the suspended solids and sludge in the treated wastewater. A sequentially reduction in efficiency of the RSF was mainly due to the reduction of particles to be removed.

On average, the $COD_{Particulate}$ and $COD_{Dissolved}$ in the outflow of the TF were approaching similar values (38 and 42 mg COD/L, respectively), as seen in Figure 4.1. In addition, when the removal efficiency of COD_{Total} improved in the TF, the predominant fraction of COD was dissolved and inorganic. As seen in Appendix 4, the BOD_5/COD for the effluent was on average 0.3. This mean that the RSF was able to only remove the non-dissolved fraction present in the suspended solids.

Regarding the SSF removal efficiency in relation with the Inflow 2 (before the SSFs), SSF1 had a higher removal efficiency of COD_{Total} (21% with a final concentration of 31 mg/L), follow by SSF3 (19% with a final concentration of 33 mg/L), and SSF 2 (19 % with a final concentration of 33 mg/L). However, the differences between each other are non-significant (around 2.3 mg COD/L). As the degradation processes for the SSF are predominantly biological, the COD_{total} removal efficiency depended directly from the organic fraction available to be degraded present at this point of the treatment. However, the removal values of COD_{total} increased up to 34 to 36% during lower efficiency values for the trickling filter (77 %) or were as low as 3 to 12 % with higher TF efficiency (90 %). This shows that the efficiency depended mainly on the organic fraction available as the predominant fraction was inorganic. Appendix 4 shows that the BOD_5 values for the slow sand filter were in all cases lower than 3 mg/L.

4.2.4. Suspended Solids in the RSF

The suspended solids removal by the RSF was shown to be very consistent during this phase with an average removal of 89 % and final concentration of 8.2 mg SS/L (see Appendix 8).

Regarding the influence of the suspended solids in the SSF, as detailed in Appendix 9, the removal frequency of the *schmutzdecke* in the SSFs increased with the hydraulic load applied. It went from 2 times for SSF1 with the THL of 0.15 m/h to 3 times with the SSF3 with the THL of 0.6 m/h. The suspended solids that were not removed by the system were expected to be part of the development of the *schmutzdecke* in the SSFs. The reduction of suspended solids from 63.3 to 8.2 mg/L was necessary to avoid the rapid clogging of the SSFs. During the start-up phase, rapid clogging was observed due to the underperformance of the sedimentation tank. In consequence, the installation of an additional non-biological treatment step (in this case the RSF) was required.

4.2.5. Phosphorus compounds

During this phase, the total phosphorus (TP) concentration was consistently removed by the trickling filter. The removal rate of TP was on average 16 % (1.3 mg/L). This is mainly due to absorption of microorganisms and biofilm growth in the TF. As the biofilm sloughs, the phosphorus compounds are removed from the system through the settled sludge. In addition, the inflow is composed on average by 62 % of orthophosphates (PO_4—P) (4.9 mg/L). However, after the trickling filter, PO_4—P represented 86 % (5.7 mg/L) of the TP content (6.7 mg/L). The degradation process of polyphosphates by microorganisms in the trickling filter increased the concentration of PO_4—P by 20 % (+0.8 mg/L), explaining their increased in ratio from 0.62 to 0.86 of PO_4—P/TP. As one of the mechanisms for conversion from polyphosphates to orthophosphates requires an anaerobic environment, it can be assumed that there was a presence of anaerobic zones in the biofilm which benefited the release of orthophosphate.

The RSF retained phosphorus compounds present in the suspended solids removing only 9.0 % of TP (0.6 mg/L), leaving an PO_4—P content of 93 % (5.6 mg/L) in relation to TP (6.0 mg/L). As the storage tanks were installed to secure the continuity of the system, the concentration of phosphorus is the result of the mixing of different treated wastewater during the day. This explained the slight increase in concentration from Outflow 2 to Inflow 2 (right before being fed into the SSF) (+0.3 mg/L) (see Appendix 10).

Regarding the SSFs, a slight removal rate of TP was observed for all the filters with percentages of 1.3, 1.1 and 0.6 %, for SSF1 (0.09 mg/L), SSF2 (0.07 mg/L) and SSF3 (0.04 mg/L), respectively. This removal efficiency was inversely proportional to the hydraulic load, which means that to a lower hydraulic load, the higher the removal percentage. Regarding the content of orthophosphates, the difference between the inflow and outflow cannot be considered significant for any of the SSFs. In addition, the ratio PO_4—P/TP remained the same as the inflow of the SSFs (0.9). It indicates that with evidence of TP removal, a small fraction of PO_4—P was also removed to keep the ratio. This can be attributed to biofilm development in the *schmutzdecke*.

4.2.6. Nitrogen compounds

In the inflow of the TF, NH_4—N represented 75 % (50.7 mg/L) of TN (67.6 mg/L) (see Appendix 11). The TN values were above the expected for municipal wastewater, which was also observed in the start-up phase.

The BOD_5/TKN was 2.93, far from the expected ratio of 5.5. Furthermore, the TF removed 30 % of TN (21 mg/L), leaving around 46.6 mg/L of TN. The different values for each sampling during this phase are detailed in Appendix 11.

The remaining nitrogen compounds were mainly composed by NH_4—N and NO_3—N with 42 % (19.5mg/L) and 49 % (22.7 mg/L) respectively with a conversion rate of NH_4—N into NO_3—N of 63 % (22 mg/L were converted from NH_4—N to NO_3—N).

The removal of TN and conversion of NH_4—N can be attributed to biofilm growth and reactions in the anoxic zones in the biomass-media interface. This facilitates denitrification reactions, which also explained the increase of orthophosphates. Although, the organic load was higher than recommended for combined carbon oxidation-nitrification (<0.4 kg $BOD_5/(m^3*d)$), the nitrification rate was higher than expected in the Technical guideline ATV-DVWK-A281. The limited number of nitrifying bacteria and residual organic matter in the system can explain the remaining NH_4—N (19.5 mg/L) after the TF. As the values of nitrite (NO_2—N) were below 0.9 mg/L, there was no sign of problems regarding incomplete nitrification processes. The TKN removal rate was 65%, which accounts for the removal of TN and the conversion of NH_4—N, considered as nitrification efficiency by the equations presented by Nakhla and Farooq (2003).

Although the RSF was only installed as a physical treatment to reduce suspended solids, it also presented a removal rate of 2 % of TN (1.2 mg/L) which was mainly present in the SS. In the case of NH_4-N, the removal concentration was of 0.9 mg/L. This can be explained by the presence of nitrifying bacteria in the water with high dissolved oxygen concentration, which continued to carry on nitrification reactions between the treatment units. The storage tanks accounted for a reduction of 3.3 mg/L of TN, a conversion of 0.9 mg/L of NH_4—N and a reduction of 0.7 mg/L of NO_3—N. This indicates also the presence of denitrification reactions. Although an anoxic environment is required, the reduction of dissolved oxygen in the point Inflow 2 (- 0.6 mg O_2/L) can evidence the use of oxygen for the oxidation of NH_4—N.

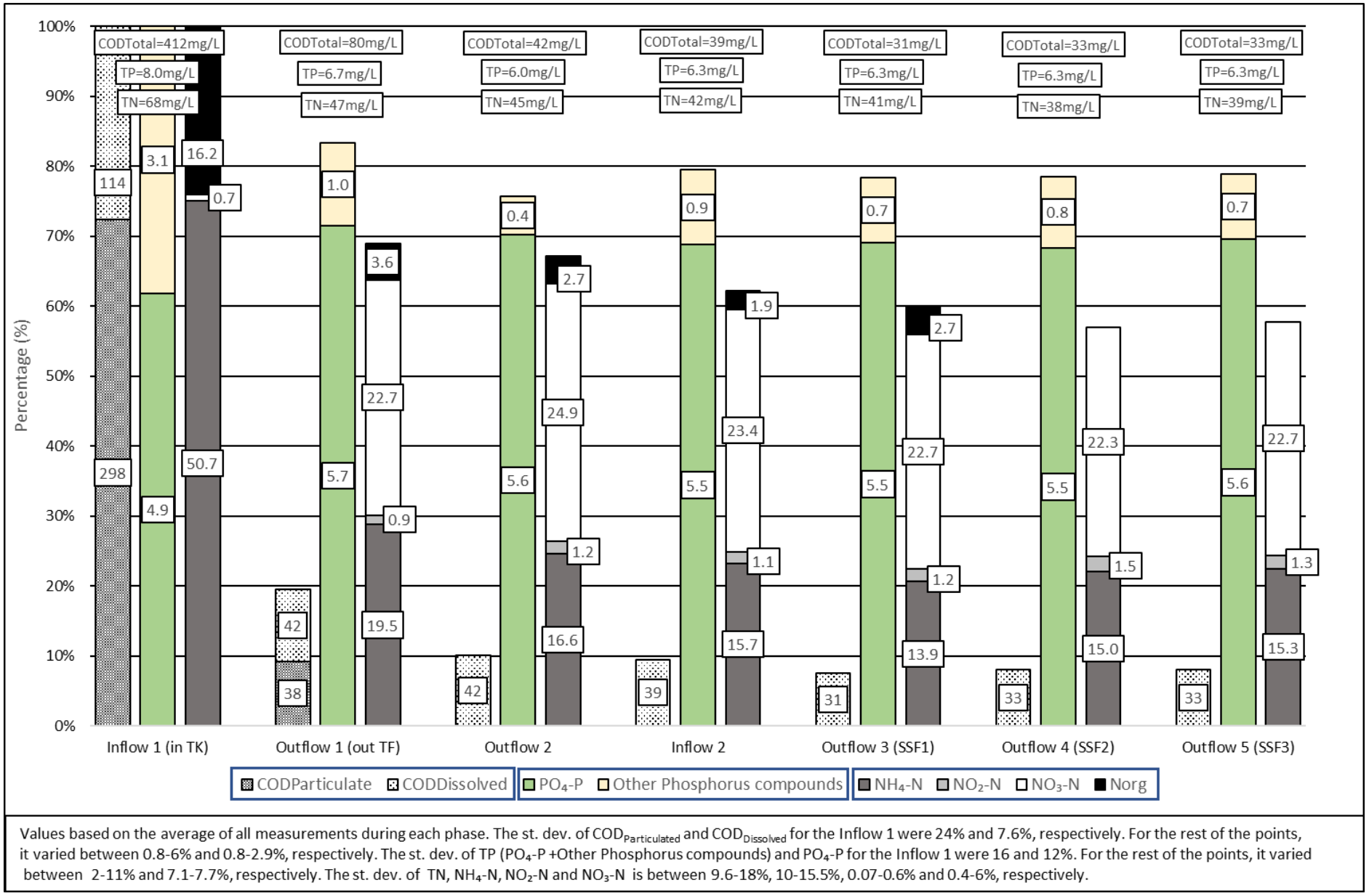
Regarding the SSF, the removal rate of TN was 7 % (2.9 mg/L), 6 % (2.3 mg/L) and 5 % (1.9 mg/L) for SSF1, SSF2 and SFF3, respectively. This shows an increase in removal of TN with a lower hydraulic load. The same behavior was also observed for the NH_4—N conversion, but with concentrations of converted NH_4—N of 1.8, 0.7 and 0.4 mg/L on average. Regarding NO_2—N, the concentration increased in relation with the concentration in Inflow 2 (between 0.1 and 0.4 mg/L), but it was below 1.6 mg/L and below the values of converted NH_4-N (between 1.7 and 0.4 mg/L), which is evidence of complete nitrification reactions.

In relation with NO_3-N results, the concentration is related to the conversion of NH_4—N, NO_2—N and denitrification processes (evident by the TN removal). Although the final average values of NO_3—N, showed a lower NO_3—N concentration for the SSF2 (22.3 mg/L), follow by SSF1 (22.6 mg/L) and SSF3 (22.6 mg/L). Considering the amount of NH_4—N converted, the increased of NO_2—N and the NO_3—N already present in the inflow, it can be calculated that higher concentrations of NO_3—N are removed in the SSF1 (THL of 1.5 m/h). These values are also inversely proportional to the hydraulic loads with average removal concentration of 2.4, 1.5 and 0.8 mg/L for SS1, SSF2 and SSF3, respectively.

4.2.7. Summary of the phase 1

The treatment with TF and SSF has shown to raised up the dissolved oxygen content to above 6.4 mg/L and a reduction of turbidity from 133 NTU in the inflow to values below 1.7 NTU for all SSFs in the final effluent. The BOD_5/TN/TP ratio for the inflow wastewater in this phase was 25:8.5:1. The TF removed 80 and 61 % of COD_{Total} and $COD_{Dissolved}$, with an effluent being predominant dissolved and inorganic with a BOD_5/COD of 0.33. This shows a deficit of organic matter to carry on biofilm development in an aerobic treatment (recommended values varied between 100:10:1 and 100:5:1). Considering the removal of 1.3 mg/L of TP, it would correspond a use of 13 mg/L of TN for biofilm development, leaving 8 mg/L of TN for denitrification processes. Regarding the BOD_5 content, it would correspond a use of 130 mg/L also for biofilm development. A part of the remaining was used for denitrification process detected on the biofilm (4 g BOD_5/NO_3—N). This balance left a deficit of organic carbon for further biofilm growth and denitrification reactions after the TF

with a ratio BOD_5/TN/TP of 6:7:1. The RSF removed the residual $COD_{Particulated}$ in the suspended solids and reduced the SS in 89 %. The RSF reduced the SS concentration from 63.3 to 8.2 mg/L, allowing a higher time interval for the removal of the *schmutzdecke* in the SSFs. As the inflow into the SSFs was predominantly inorganic, the removal rate of COD_{total} was on the range of 19 and 21 % with no significant difference among the different hydraulic loads in the SSFs. In all cases the BOD_5 had values lower than 3 mg/L. Further nitrification and denitrification reactions in the SSFS were limited. A removal of COD was observed with values similar for all three SSFs. In addition, the concentration values for TP and PO_4—P were not modified by the SSFs, mainly due to inertness of the sand for adsorption reactions with phosphorus compounds. This phase showed that organic content limited the expected reactions in the TF and the SSF, with similar removal values for all the SSFs. In the case that the system was to be applied into a larger scale, the hydraulic load of 0.3m/h would be recommended. The reasons are a smaller plant size in comparison to the lowest hydraulic load (0.15m/h) with similar removal values and less requirement for the *schmutzdecke* removal in comparison with the highest hydraulic load.



Values based on the average of all measurements during each phase. The st. dev. of $COD_{Particulated}$ and $COD_{Dissolved}$ for the Inflow 1 were 24% and 7.6%, respectively. For the rest of the points, it varied between 0.8-6% and 0.8-2.9%, respectively. The st. dev. of TP (PO_4-P +Other Phosphorus compounds) and PO_4-P for the Inflow 1 were 16 and 12%. For the rest of the points, it varied between 2-11% and 7.1-7.7%, respectively. The st. dev. of TN, NH_4-N, NO_2-N and NO_3-N is between 9.6-18%, 10-15.5%, 0.07-0.6% and 0.4-6%, respectively.

Figure 4.1 Removal rates for COD, phosphorus and nitrogen compounds during Phase 1 [104-134] for all the monitoring points

4.3. Second phase

4.3.1. Organic loading

On average, the TOL and BOD_5 were 0.66 Kg BOD/m^3 and 134 mg/L, respectively. This TOL was the lowest load from the three phases. The THL was kept with the same values as the previous phase (1.1 $m^3/(m^2*h)$).

4.3.2. In situ parameters

The TF increased the value of dissolved oxygen up to 7.3 mg O_2/L, while a slightly decreased was observed for the RSF and just before the inflow into the RSF (6.9 mg O_2/L). The final values of dissolved oxygen were the highest for the lowest hydraulic load (0.15 m/h) with 7.5 mg O_2/L. In addition, the dissolved oxygen decreased with higher hydraulic loads (7.3 mg O_2/L for SSF3 with THL of 0.6 m/h). Regarding pH, the values were stable and range between 7.6 and 7.8 for all the monitoring points. In regards of EC, the highest values were registered for the inflow. The treatment by the TF reduced the values from 1248 to 1055 uS/cm. After that, the values of EC were stable for the rest of the process for all monitoring point with a range between 1048 and 1058 uS/cm. The temperature values were stable between 22 and 23 °C. In the case of turbidity, the average into the trickling filter was 100 NTU, with a removal rate of 97 % and an outflow of 2.5 NTU. The values of turbidity in the effluent of the SSFs were between 0.9 and 1 NTU, with no significant difference between the different hydraulic loads and the removal rate (range between 29 and 36 % of turbidity).

4.3.3. Chemical Oxygen Demand removal

The average COD_{Total} and $COD_{Dissolved}$ inflow into the TF was 267 mg COD/L and 92 mg COD/L, respectively. The removal efficiencies of COD_{Total} and $COD_{Dissolved}$ were 89 % and 76 %, respectively. As the ratio $COD_{Dissolved}/COD_{Total}$ before and after the TF were 0.35 and 0.85, this showed that the particulate fraction contributed the most with the COD before the treatment with the TF. However, after the treatment with the TF, most of the $COD_{Particulate}$ was removed (98 %), leaving a small fraction in the suspended solids. As shown in Figure 4.2, after the RSF, the COD_{Total} was mostly $COD_{Dissolved}$ with a concentration of 22 mg/L.

As the efficiency removal of the TF increased, the efficiency of the RSF decreased, the same behavior was observed in the previous phase. The TF presented removal rates for COD_{Total} as high as 94 % or low as 85% with removal rates for the RSF of 15 and 29 %, respectively. This is explained by the limited capacity of the RSF to removed suspended particles smaller than the grain size and the reduction of particles to be removed due to the TF.

The SSF removed a small portion of the COD mainly constituted of dissolved components. For the hydraulic loads 0.15 (SSF1), 0.3 (SSF2) and 0.6 m/h (SSF3), the removal rate for the SSFs in reference at the Inflow 2 were 23, 22 and 20 %, respectively. As the treated wastewater into the SSF was composed mainly by dissolved and inorganic components, with low values of organic matter, further degradation was limited. Although, the higher removal value for COD_{Total} was for the SSF1 (23 %), the difference in concentration for the final effluent in the different hydraulic loads was less than 1 mg COD_{Total}/L (the values ranged between 16.0 and 16.8 mg/L). This shows a limited COD_{Total} removal, which is consistent with the low BOD_5 values set for this phase, showing a limitation for further reducing COD values.

4.3.4. Suspended Solids in the RSF

The suspended solids (SS) removal by the RSF was shown to be very consistent with an average of 41 % during this phase. The Outflow 1 from the TF had lower concentrations of suspended solids (4 mg/L) due to the change of the organic load and high performance of TF shown in the COD_{Total} removal values (90 %). The Outflow 2 had values of 2.9 mg/L of SS (see Appendix 8).

Furthermore, the low concentrations of SS also reduced the frequency of the *schmutzdecke* removal. For the SSF1 (0.15m/h) and SSF2 (0.30m/h), during the whole duration of this phase (47 days), there was no *schmutzdecke* removal. However, for the SSF3, the removal of the *schmutzdecke* was required four times, as seen in Appendix 9. Although, there was a direct relationship between the concentration of SS, pressure loss and *schmutzdecke* removal: to higher concentration of SS, higher pressure loss and more frequent

schmutzdecke removal. The relationship was variable, not proportional and did not have any discernable patron to be predictable. This can be attribute to the variable SS concentrations going into the SSFs daily.

4.3.5. Phosphorus compounds

The value for TP into the TF was 6.4 mg/L. The removal of TP was on average 22% due to the TF (1.6 mg/L). This also can be attributed to the absorption of microorganism, biofilm growth and adsorption due to the filter media. The removal out of the system is done due to the slough of the biofilm into the settlement tank. The ratio between PO_4—P/TP before the trickling filter was 0.7. After the TF the ratio was also 0.7. This can be attributed to the degradation of polyphosphates into PO_4—P and the use of phosphorus compounds to biofilm growth. However, the increment of PO_4—P was around 0.1 mg/L, which cannot be considered substantial. The values for TP and PO_4—P for the rest of the treatment units were stable (around 4.8 mg/L for TP and between 4.2 and 4.3 mg/L for PO_4—P) with no removal or transformation reactions.

4.3.6. Nitrogen compounds

The TN into the TF reported average values of 48.2 mg/L, with an 81 % being composed by NH_4—N (39 mg/L). The treatment with the TF removed TN in 39 % (19 mg/L). Regarding the load of nitrogen in the system, the ratio BOD_5/TKN was 2.7, which also presented high values of nitrogen compounds from the expected ratio of 5.5 for municipal wastewater. The removal of TN can be attributed to biofilm growth and reactions in the anoxic zones in the biomass-media interface, which facilitate denitrification reactions. The removal of TKN by the TF was around 88 % (42.1 mg/L), counting for the TN removal and transformation of NH_4—N.

Regarding the behavior of NH_4—N, after the TF, only 8% of the remaining TN was NH_4—N with a concentration of 2.3 mg/L. The concentration of NO_2—N in all the monitoring points was below 0.3 mg/L during the whole phase, so no indication of incomplete nitrification reactions was observed. The concentration of NO_3—N was 22.6 mg/L, representing 78 % of the remaining TN after the TF. The organic load on this phase facilitated nitrification reactions in the TF that were higher than expected in accordance with the Technical guideline ATV-DVWK-A281.

The concentration of TN after the RSF was kept around 28.7 mg/L with a difference less than 0.3 mg/L between the effluent of the TF and the effluent for the RSF. After the RSF, the value of NH_4—N decreased to 0.8 mg NH_4—N/L on average for the whole duration of the phase 2 (47 days). This can be attributed to further nitrification processes inside sedimentation tank, because the RSF was only a physical treatment to reduce suspended solids. After the RSF, the concentration of NO_3—N increased in 0.8 mg/L so the inflow into the SSFs was around 1.5 mg/L of NO_3—N.

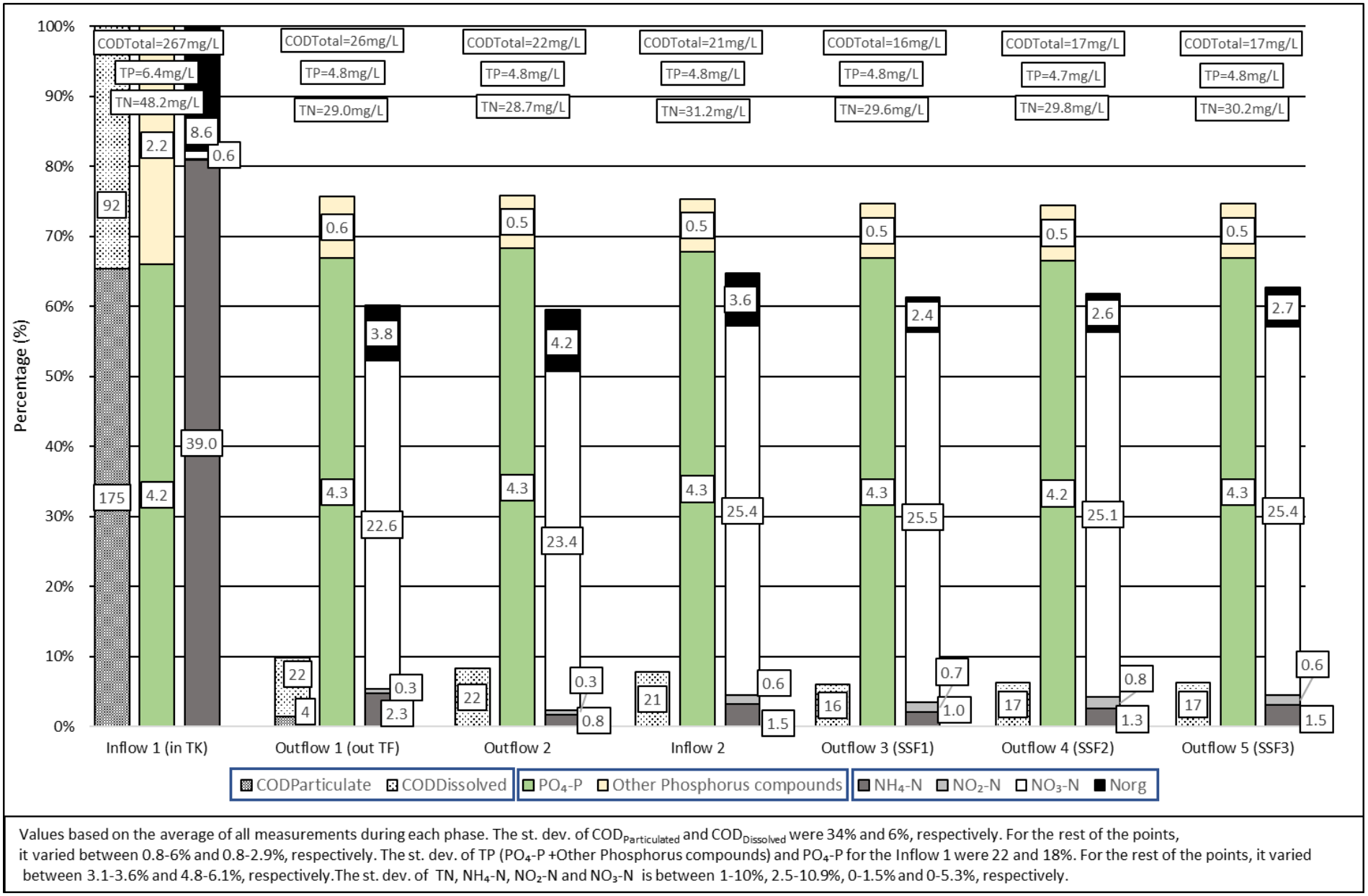
For the inflow into the SSFs, the TN concentration was 31.2 mg/L, which was slightly higher (+2.5 mg/L) than the previous treatment unit due to the storage tank effect to buffer treated wastewater with different concentrations during the day.

The values of NH_4—N before the SSF were higher than after the RSF due to the storage tanks (same as with the TN behavior) (+0.7mg/L). In regards of the NH_4—N removal by the SSFs, after day 168 (35 days after the beginning of phase 2) constant removal values were detected for all the SSFs.

For hydraulic loads of 0.15, 0.3 and 0.6 m/h, the concentration removed for NH_4—N were 0.5, 0.3 and 0.2 mg/L, respectively. As mention before, the NO_2—N values were below 0.3 mg/L, so nitrification processes were also observed in the SSF. The final values of NO_3—N were higher for SS1 (25.5 m/L), followed by SS3 (25.4 mg/L) and SS2 (25.1 mg/L). However, a balance of nitrogen species into each SSF showed that the conversion from NH_4—into NO_3—N was higher for lower hydraulic loads with 0.4, 0.3 and 0.3 mg/L for SS1, SSF2 and SSF3 respectively. Denitrification processes were detected in the SSFs, the highest value was for the lowest hydraulic load (SSF1) with 1.6 mg/L. The TN removed for SSF2 and SSF3 was 1.4 and 1.0 mg/L, respectively (see Appendix 12).

4.3.7. Summary of the phase 2

The treatment with TF and SSF has shown to increase the dissolved oxygen content to around 7.2 mg/L and a reduction of turbidity from 100 NTU in the inflow to values below 0.9 NTU for all SSFs in the final effluent. The BOD_5/TN/TP ratio for the inflow wastewater in this phase was 20:8:1. The TF removed 89 and 76 % of COD_{Total} and $COD_{Dissolved}$. The effluent from the TF was predominant dissolved and inorganic with a $COD_{Dissolved}/COD_{Total}$ of 0.85 and a COD_{Total} of 26 mg/L. A TP concentration removal of 1.6 mg/L would have required a TN concentration removal of 16 mg/L and 160 mg BOD_5/L, which is more than the average BOD_5 available of 128 mg/L. In consequence, it is likely that part of the TP and TN removal was also related to adsorption into the biofilm and filter media and denitrification processes. This can also explain the low concentration of suspended solids after the TF with an average value of 4 mg/L. In addition, the BOD_5/TN/TP in the effluent was 2.6:6:1, leaving a deficit of organic carbon for further biofilm growth and denitrification. The RSF removed the residual $COD_{Particulated}$. Also, the SS were reduced in 41 % with final values of 2.9 mg/L, this reduced the frequency of *schmutzdecke* removal for all the SSFs, which could be translated into less maintenance requirements. As the inflow into the SSFs was predominantly inorganic, the removal rate of COD_{total} was on the range of 20 and 23 % with no significant difference among the different hydraulic loads in the SSFs. In addition, the concentration values for TP (4.8 mg/L) and PO4—P (between 4.2 and 4.3 mg/L) were not modified by the SSFs. Regarding the nitrogen compounds, the NH_4—N and NO_3—N showed slightly nitrification reactions and denitrification, respectively. However, it was not significant among the different hydraulic loads (0.15, 0.3 and 0.6m/h). It is important to mention that, the removal of NH_4—N only was detected for all the SSFs after day 35 since the beginning of phase 2. This shows that in case of lower concentration of pollutants, the system requires more time for the *schmutzdecke* development and to perform degradation reactions. For this phase, as the limiting factor was the organic carbon available for further reactions, the hydraulic load of 0.3 m/h showed to be the most efficient considering a potential plant size and maintenance related to *schmutzdecke* removal.



Values based on the average of all measurements during each phase. The st. dev. of $COD_{Particulated}$ and $COD_{Dissolved}$ were 34% and 6%, respectively. For the rest of the points, it varied between 0.8-6% and 0.8-2.9%, respectively. The st. dev. of TP (PO_4-P +Other Phosphorus compounds) and PO_4-P for the Inflow 1 were 22 and 18%. For the rest of the points, it varied between 3.1-3.6% and 4.8-6.1%, respectively.The st. dev. of TN, NH_4-N, NO_2-N and NO_3-N is between 1-10%, 2.5-10.9%, 0-1.5% and 0-5.3%, respectively.

Figure 4.2 Removal rates for COD, phosphorus and nitrogen compounds during Phase 2 [134-180] for all the monitoring points

4.4. Third phase

4.4.1. Organic loading

On average, the TOL and BOD_5 were 0.69 Kg BOD/m^3 and 142 mg/L, respectively. This TOL was the intermediate load from the three phases. The THL was kept with the same values as the previous phase.

4.4.2. In situ parameters

The treatment with the TF increased the values of dissolved oxygen up to 6.7 mg O_2/L. The RSF also increased the dissolved oxygen up to 6.9 mg O_2/L. Inflow 2 had the lowest value with 5.9 mg O_2/L, likely due to nitrification processes inside the storage tanks. The SSFs registered higher values up to 7.8 mg O_2/L for the lowest hydraulic load (0.15 m/h) and 6.9 mg O_2/L for the highest hydraulic load (0.6 m/hr).

Regarding the pH, the values range between 7.5 to 7.7, with a slightly decreased after the treatment by the TF. This can indicate nitrification processes being carried put in the TF. In regards of the EC, the highest values were registered for the Inflow 1 with an average of 1317 uS/cm with the highest removal being attribute to the TF. For the rest of the monitoring points, the values range between 1105 to 1128 uS/cm. The temperature in the treatment process ranged between 20 and 21 °C.

In the case of turbidity, the Inflow 1 registered values of 107 NTU with a removal of 97 % by the TF (see Appendix 6). After the TF and RSF, the values of turbidity were on average 3.3 and 1.8 NTU, respectively. The value of turbidity in Inflow 2 (before the SSFs) was around 1 NTU. The final values of turbidity for the SSF1, SSF2 and SSF3 were 0.6, 0.5 and 0.6 NTU, respectively. This showed no significant influence for different hydraulic loads in the turbidity for these conditions.

4.4.3. Chemical Oxygen Demand

The average values for COD_{Total} and $COD_{Dissolved}$ into the TF were 284 and 122 mg COD/L with removal efficiencies of 88 and 75 %, respectively. The ratio $COD_{Dissolved}/COD_{Total}$ before and after the TF was 0.4 and 0.9. This indicates that a predominant fraction before the TF is composed by $COD_{Particulate}$. After the treatment with the TF, most of the $COD_{Particulate}$ was removed leaving a small fraction to be removed by the RSF.

As shown in Figure 4.2, after the RSF, the COD_{Total} was mostly $COD_{Dissolved}$ with a concentration of 26 mg/L.

The SSFs removed a small portion of COD_{Total} left in the wastewater. The highest removal values were for the lowest hydraulic rate. The final COD values for SSF1 (0.15 m/h), SSF2 (0.3 m/h) and SSF3 (0.6 m/h) were 21.4, 22 and 22.8 mg/L, respectively. The low removal rates (between 6 and 13 %) showed that as the COD was mainly composed of dissolved and inorganic components, further degradation of organic compounds was limited.

Although, the highest removal value for COD was for the SSF1, the difference in concentration for the final effluent in the different hydraulic loads was less than 1.3 mg COD_{Total}/L. This shows a limited COD_{Total} removal, which is consistent with the low BOD_5 values left at this point in the treatment in this phase.

4.4.4. Suspended Solids in the RSF

The suspended solids (SS) concentration into the RSF was around 5.3 mg/L with an average removal rate of 39 %. The effluent from the RSF (Outflow 2) had concentration of SS of around 3.6 mg/L (see Appendix 8).

Regarding the frequency of removal of the *schmutzdecke*, for the SSF1, it was not necessary the removal of *schmutzdecke* during the duration of this phase (28 days). The SSF2 required the scraping of the *schmutzdecke* after 21 days. For the SSF3 (higher hydraulic load) the removal was necessary after 15 days and it was kept until the end of phase 3, as seen in Appendix 9. A higher frequency of *schmutzdecke* removal and higher-pressure loss in the SSFs correlates to higher SS concentrations in the inflow of the SSFs. However, this correlation is variable and not proportional in time.

4.4.5. Phosphorus compounds

The inflow concentration of TP in the TF was on average 7.5 mg/L. After the treatment with the TF, the value of TP was reduced up to 5.2 mg/L with a 32 % removal (2.3 mg/L). The ratio PO_4—P/TP before the TF was around 0.7 with a PO_4—P concentration of 4.9 mg/L. After the treatment with TF, the PO_4—P concentration was on average 5 mg/L with a PO_4—P/TP ratio of 0.97. This indicates that the main TP removal was due to

biofilm growth and adsorption into the filter media. There was no significant increase in the PO_4—P concentration to indicate degradation reaction of polyphosphates into PO_4—P (+0.1 mg/L). The removal of TP and increase of PO_4—P due to the RSF was also no significant (-0.02 mg TP/L and +0.1 mg PO_4—P/L, respectively). This is also consistent with the predominant presence of PO_4—P in the wastewater. Regarding the TP and PO_4—P in the SSFs, the difference between Inflow 2 and the Outflow for each SSF, was less than 0.1 mg/L and 0.2 mg/L, respectively. So, no significant removal was observed for those two parameters.

The final effluent for TP and PO_4—P after the SSF was around 5.3 and 5.2 mg/L, respectively. The slightly higher values for PO_4—P in relation to TP in the SSF2 can be explained by the margin of error in the measurement methodology.

4.4.6. Nitrogen compounds

The TN into the TF was on average 64 mg/L with an 73% being composed by NH_4—N. The treatment of the TF reduced the TN concentration in 26 mg/L (40 %), as seen in Appendix 13. The ratio BOD_5/TKN in the inflow of the TF was 2.2, representing higher values of nitrogen compounds from the expected ratio for municipal wastewater. The removal of TN can be attributed to biofilm growth and reactions in the anoxic zones in the biomass-media interface, which facilitate denitrification reactions. Regarding the TKN removal rate, the TF removed around 74 % which included the removal of TN and the NH_4—N transformation.

In the case of the NH_4—N, the TF removed 73 % of the NH_4—N (40 mg/L), reducing the concentration from 55 to 15 mg/L. The concentration of NO_2—N between the Inflow 1 and the Outflow 2 was below 0.7 mg/L, which indicates complete nitrification reactions. The NO_3—N concentration after the TF was on average 21 mg/L.

The RSF reduced the TN concentration from 38 to 37 mg/L (3 %). In the case of NH_4—N, the concentration was reduced from 14.9 to 10.5 mg/L. The reduction can be attributed to continuous nitrification reactions. The concentration of NO_2—N was stable in the RSF with a value of 0.7 mg/L. In the case of NO_3—N, an increment of 2.1 mg/L can be also attributed to continuous nitrification reactions.

In regards of the SSF, the inflow into the SSFs registered on average 37.6 mg/L of TN. The TN concentration is higher than after the RSF by 0.7 mg/L, this can be attributed to the mixing inside the storage tanks from the continuous loading during the day. Inside the storage tanks, nitrification reactions can also be observed by the slight reduction of NH_4—N (-0.1 mg/L) and the increased of NO_2—N (+0.1 mg/L) and NO_3—N (+0.3 mg/L). Regarding the removal of TN inside the SSFs, the lowest hydraulic load (0.15 m/h) reported the highest TN concentration removal with a value of 2 mg/L for the SSF1 (5 %). SSF2 and SSF3 had removal concentration values of TN of 1.65 (4 %) and 1.58 mg/L (4 %), respectively. In regards of the NH_4—N, a reduction in NH_4—N was observed, with the highest values for the lowest hydraulic load. The values for NH_4—N removal concentration were 1.5, 1.3 and 1.1 mg/L for the SSF1, SSF2 and SSF3, respectively. The concentration of NO_2—N in the SSFs increased from 0.8 mg/L to values between 1.6 and 1.7 mg/L. As these concentrations accounted for most of the NH_4—N conversion, some incomplete nitrification can be observed. Regarding the NO_3—N concentration, lower values of NO_3—N were observed for the lowest hydraulic load (-0.7 mg/L for SS1). Also, after a balance of nitrogen compounds, the higher conversion rate from NH_4—N to NO_3—N was for SSF1 with 1.3 mg/L, followed by SSF2 and SSF3 with 0.9 and 0.5 mg/L.

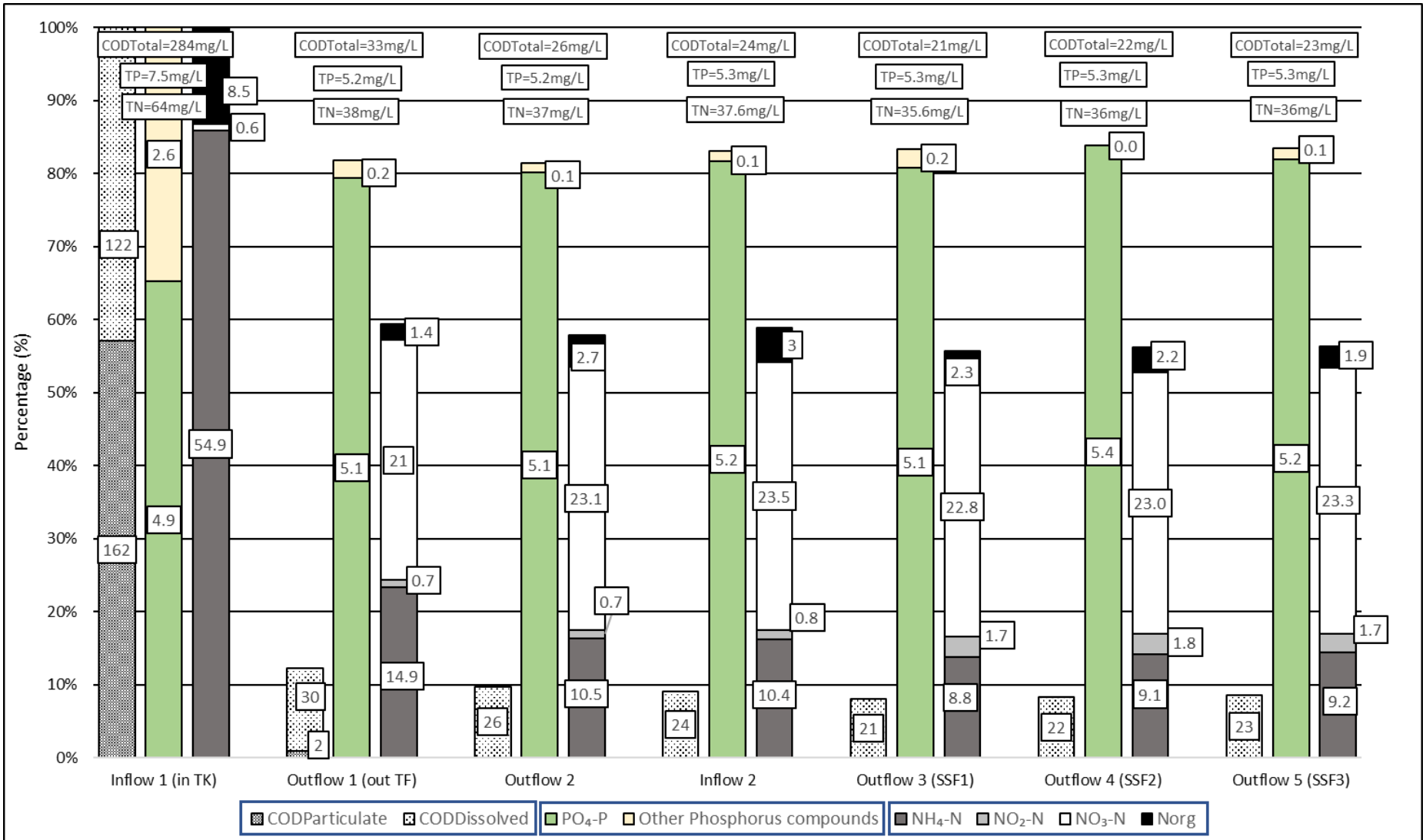
4.4.7. Summary of the phase 3

The treatment with TF and SSF has shown to raised up the dissolved oxygen content to around 6.8 mg/L and a reduction of turbidity from 107 NTU in the inflow to values below 0.6 NTU for all effluents of the SSFs.

The BOD_5/TN/TP ratio for the inflow wastewater in this phase was 18:8.5:1. The TF removed 88 and 75 % of COD_{Total} and $COD_{Dissolved}$, respectively. The effluent from the TF was predominantly dissolved and inorganic with a $COD_{Dissolved}/COD_{Total}$ of 0.93 and a COD_{Total} of 32 mg/L. A TP concentration removal of 2.3 mg/L would have required a TN removal of 23 mg/L and 230 mg BOD_5/L, which is more than the average BOD_5 available of 136 mg/L. In consequence, it is likely that part of the TP and TN removal was also related to adsorption into the biofilm and filter media and denitrification processes additional to biofilm growth, which can also

explain the low concentration of suspended solids after the TF with an average value of 5.3 mg/L. In addition, the BOD_5/TN/TP in the effluent was 3:7:1, leaving a deficit of organic carbon for further biofilm growth and denitrification.

The RSF removed the residual $COD_{Particulated}$. Also, the SS were reduced in 39 % with final values of 3.6 mg/L, this reduced the frequency of *schmutzdecke* removal for all the SSFs. However, the SSF2 required maintenance after 21 days, while the SSF3 required it after 15 days. As the inflow into the SSFs was predominantly inorganic, the removal rate of COD_{total} was on the range of 6 and 12 % with no significant difference among the different hydraulic loads in the SSFs, as the removal concentrations ranged from 2 to 3 mg COD/L. In addition, the concentration values for TP and PO_4—P were not modified by the SSFs. Regarding the nitrogen compounds, NH_4—N and NO_3—N showed slightly nitrification reactions (concentration removal between 1.1 to 1.5 mg/L) and denitrification (0.5 to 1.3 mg/L), respectively. However, it was not significant among the different hydraulic loads. For this phase, as the limiting factor was the organic carbon available for further reactions, the hydraulic load of 0.3 m/h showed to be the most efficient considering a potential plant size and maintenance related to *schmutzdecke* removal.



Values based on the average of all measurements during each phase. The st. dev. of $COD_{Particulated}$ and $COD_{Dissolved}$ were 28.5% and 11%, respectively. For the rest of the points, it varied between 2-3.4% and 2.2-3%, respectively. The st. dev. of TP (PO_4-P +Other Phosphorus compounds) and PO_4-P between 13.5-20% and 12.3-17.4%, respectively. The st. dev. of TN and NH_4-N in Inflow 1 were 8.5 and 6%, respectively. For the rest of the point, it varied between 2-4% and 2.6-4.6%, respectively. For NO_2-N and NO_3-N, the st. dev. were between 0.02-1.4% and 0.1-3.9%, respectively.

Figure 4.3 Removal rates for COD, phosphorus and nitrogen compounds during Phase 3 [180-207] for all the monitoring points

4.5. Bacteriological parameters

During the three phases, bacteriological parameters were measured: E. coli, Total Coliforms (TC) and Enterococci. TFs are expected to remove between 0 to 2 log_{10} units of bacteria. Figure 4.4 shows the results for the monitoring during the three phases. It can be observed that the removal of *E. coli* for the TF varied between 0.9 to 2.4 log_{10}. with initial values between 1.1*10^5 and 5.8*10^5 for the different phases (see Appendix 14) which is inside the expected removal range for this type of systems. The removal mechanism that can explained the reduction of *E. coli* is adsorption by the biofilm and sedimentation from the sloughed biofilm into the settlement tank. The RSF only removed 0.2 log_{10}.

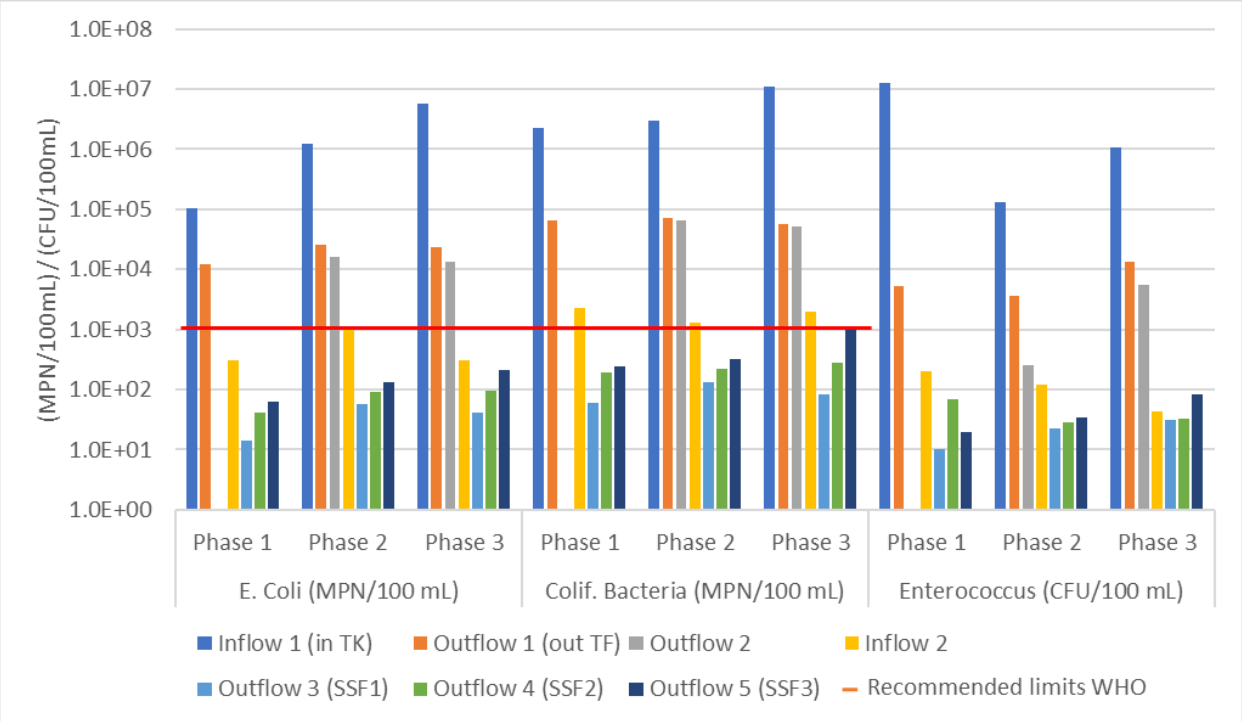


Figure 4.4 Values of pathogens E. coli, Colif. Bacteria and Enterococcus in the wastewater treatment system for all the phases.

Regarding the SSFs, the lowest hydraulic load (0.15 m/h) registered the highest removal rate of *E. coli* with values between 1.35, 1.30 and 0.87 log_{10}. The removal rate of *E. coli* was reduced with higher hydraulic loads, the lowest removal rates were for SSF3 with values between 0.16 and 0.9 log_{10}, while the SSF2 reported values between 0.4 and 1 log_{10}. The removal mechanisms for the SSFs are mainly straining, adsorption, attachment and biological activity such as predation. In addition, the values for all the SSFs were below the recommended limits for treated wastewater reuse according to the WHO (1000 CFU/100mL) and National Peruvian law (1000 MPN/100mL), except SSF3 during phase 3. Although, the measured values used the Most-Probable Method, which usually reports higher values than Coliform-forming Units method for E.coli (Cho et al. 2010), which is why, it is expected that the values were even lower. Regarding the values for Total Coliforms (TC) (Coliform Bacteria), similar behavior was observed for the removal values. The TF removed between 1.5 and 2.3 log_{10} due to the same process as for the *E. coli*. In case of the SSFs, SSF1 removed between 1 to 1.58 log_{10}, with the highest removal rates compared with the SSF2 and SSF3. SSF2 and SSF3 had removal rates between 0.77 and 1 log_{10} and 0.3 and 0.98 log_{10}, respectively. Although each phase had different dilution, the content of pathogens was variable. Regarding the content of TC, all the SSFs reported values below the recommended for reuse in agriculture. The same relation regarding MPN-method and CFU-method can be applied for TC. In regards of Enterococcus, the removal rates for the TF were around 1.5 to 3.3 log_{10}, with initial values between 1.3*10^5 and 1.3*10^7. The RSF reported variable efficiencies as low as 0.4 log_{10} or high as 1.4 log_{10}. The SSF1 removed the highest values of Enterococcus with a removal rate of 0.15, 0.7 and 1.3 log_{10} for phase 3, phase 2 and phase 1, respectively. In the case of SSF2 and SSF3, the

removal varied between 0.1 to 0.4 log_{10}. and 0.5 to 1 log_{10}, respectively. Although for phase 3, SSF3 reported the highest value for the outflow of the SSF.

The removal rates for *E. coli* of all the SSFs were lower than removal rates reported by Langenbach et al. (2009) (1.9 to 2.6 log units) and Katukiza et al. (2014) (1.98 to 2.52 log units). In the case of TC, SSF1 reported values similar to Ellis (1987) and Katukiza et al. (2014), while the rest of SSFs reported lower values. Additionally, all the SSFs reported lower values than the ones mention by Tyagi et al. (2009).

The removal of pathogens *E. coli* and TC was around the recommended limits for reuse in agriculture. While SSF1 can achieved higher removal values, all the systems, with exception of SSF3 removed pathogen concentrations below the recommended values of the WHO. In this case, a higher hydraulic load such as SSF2 can perform the necessary removal to fall inside the limits values without falling into higher construction costs with a smaller plant size. The hydraulic load for SSF3 can be recommended in a system configuration only if an in-series is considered, because it will allow a further treatment without risk of high pathogen content for water reuse in agriculture.

Regarding the Enterococcus, the WHO has not limit values for this parameter for water reuse, as it more related to recreational uses of water.

4.6. Performance comparison between phase 1, 2 and 3

During phase 1, the treatment with the TF reduced the turbidity to values below 28 NTU with a removal rate of 79 %, while during phase 2 and 3, the values were reduced to below 3.3 NTU with removal rates of 97% for both phases. In all cases, the inflow values of turbidity into the SSFs were lower than the recommended value of 50 NTU (Crittenden 2005). The SSFs presented removal rates of turbidity between 31 and 49 % for all the phases. Although the turbidity removal rates are lower than the ones reported by Langenbach et al. (2009) (73—89 %) and Tyagi et al. (2009) (91.6 %), the final values for phase 2 and 3 ranged between the final values reported by Langenbach et al. (2009). Tyagi et al. (2009) used smaller sand diameter, which can explain the lower turbidity removal values.

Regarding the SS, during phase 1, the RSF was needed to lower the concentration of suspended solids to values around 8 mg/L to ensure a continuous run of the filter. For phase 2 and 3, the concentration of suspended solids after the TF was lower than 6 mg/L. Which is why, the RSF can be treated as a failsafe mechanism more than a requirement in the treatment sequence.

Regarding the BOD_5, the TF had a removal rate of 85 % on average which is a slight higher removal rate for high-rate filters as reported by Drinan and Whiting (2001) (80—85 %).For the SSFs, Nakhla and Farooq (2003) reported removal rates between 58 to 79 % of BOD with a final effluent of 1 to 2 mg/L of BOD. Although the BOD_5 was not constantly monitored, the measurements done (see Appendix 4) showed most times values below 3 mg/L or removals higher than 50 %.

Regarding the COD, all phases showed high removal rates in the TF for COD_{Total} with values around 80 % for the highest load conditions and above 88 % in low and medium load (phase 2 and 3). The concentrations of $COD_{Filtrated}$ in all the phases were closer to the COD_{Total} after the TF. The capacity of the TF to reduce the COD_{Total} concentration determined the ranges of COD in the final effluent, due to the limited removal rates of the SSF for all the phases. The SSF removed between 18 to 23 % of COD_{Total} for phase 1 and 2 with similar mg COD/L removed for SSF2 (5.6 mg COD/L) and SSF3 (3.9 mg COD/L) for both cases. Phase 3 reported lower values with removal efficiencies between 6 to 12 % for COD_{Total}. This removal percentage are lower than the ones reported by Tyagi et al. (2009) (71—83 %) although the final effluent (27 mg COD/L) quality was similar with 31—34, 16—17 and 21—23 mg COD/L for the final effluent for phase 1, 2 and 3, respectively.

Regarding the phosphorus compounds, the TF showed a consistent concentration removal of TP of 1.3, 1.6 and 2.3 mg/L for phase 1, 2 and 3, respectively. The concentration of TP removed was similar for the high organic load and low organic case. This is likely to be related to a stable growth of biofilm after a maximum development of biofilm was achieved during the first phase, which then was follow by a low organic load

phase. The higher removal values of phase 3 (medium load) can be evidence of an increment of biofilm growth that follows an increment in the organic load. The SSFs showed no considerable removal of TP in any case, showing that the removal of suspended solids by the RSF, also removed almost all the TP that was present in the suspended solids, leaving dissolved compounds that did not interact with the sand in the filter media. In regards of the PO_4—P, only for phase 1, higher values of PO_4—P were detected for the outflow of the TF (+0.8 mg/L). This can be explained by anaerobic zones into the trickling filter during high load of organic matter and biofilm growth. In case of the other two phases, the difference between the measured values in inflow and outflow of the TF were around 0.02 and 0.05 mg/L of TP, which fall into the error ranges of the determination method.

Regarding the nitrogen compounds, there was a clear difference in inflow concentrations of the TN for each phase. The TF removed concentrations of 21, 19 and 26 mg/L of TN for phase 1, 2 and 3, respectively. The higher removed concentration in phase 3 can be explained also due to a higher biofilm growth, consistent with the higher TP consumption explained previously. On average values, the TN removal of the TF during the 3 phases was around 30 and 40 %. In regards of the SSF, the removal concentrations were higher for the phase with higher organic load (phase 1), with removal between 1.8 and 2.8 mg/L of TN. In the case of the phase 2 and phase 3, the removal concentrations of TN were between 0.9—1.6 and 1.5—2 mg/L. The TN removal for phase 1 for the SSFs was similar to the values reported by Nakhla and Farooq (2003) (2.3 mg/L). The values for each filter showed in all the phases, higher removal for the lowest hydraulic load. However, it is important to notice that the final effluent reported by Nakhla and Farooq (2003) was lower (1.5 to 2.7mg/L of TN) than the effluent values in this research. This shows that the removal of compounds such as TN, depend partially to the initial concentration into the SSF and a maximum amount of reactions that can take place inside the SSF system. In addition, the removal rate was no proportional, this means that decreasing by half the hydraulic load, did not removed twice as much TN, instead it only increased the removal concentration of TN on average between 0.1 to 0.5 mg/L.

Regarding the concentration of NH_4—N, during all the phases, the TF transformed 31, 36 and 40 mg/L of NH_4—N for phase 1, 2 and 3, respectively, with removals of 63, 94 and 73%. The higher removal rates are attribute to smaller initial concentrations, instead of higher removal concentrations. The removal percentage of NH_4—N during phase 2 was higher than the reported values by Tchobanoglous et al. (2014) (90 % ammonia removal) and Stenquist et al. (1974) (89 % ammonia removal) for even lower organic loads. The TF was the main treatment unit that determined the concentration of NH_4—N in the rest of treatment system. During phase 1, 2 and 3, the final concentrations of NH_4—N in the SSF were between 13.9—15.3, 0.9—1.5 and 8.8—9.2 mg/L, respectively. This shows that nitrification reactions in all SSF are limited. In additions, the amount of NH_4—N transformed into the SSFs depended on the concentration fed to the SSF. As seen in the highest load (phase 1), the concentration of NH_4—N removed was between 0.4 and 1.8 mg/L. For the median load (phase 3) the removed concentrations of NH_4—N were between 1.1 and 1.5 mg/L. For the lowest load the removal rate of NH_4—N was between 0.21 and 0.53 mg/L.

The concentration of NO_2—N was observed to be below 1.5 mg/L for phase 1 and 2 for all treatment units. This showed that complete nitrification reactions were performed during the treatment for these phases. In the case of phase 3, an increment from 0.8 to 1.7 mg NO_2—N/L due to the SSFs showed some incomplete nitrification reactions.

The concentration of NO_3—N in the TF increased between 20 and 22 mg/L for all the phases. The RSF increased the aeration in the treated wastewater and the storage tanks allowed more nitrification reactions. In the SSFs, the amount of NH_4—N converted into NO_3—N range between 0.8—2.4 mg/L, 0.3—0.4 mg/L and 0.5—1.3 mg/L for phase 1, 2 and 3, respectively. Higher conversions from NH_4—N to NO_3—N were observed for the lowest hydraulic loads in all phases (2.4, 0.4 and 1.3 mg/L for phase 1, 2 and 3, respectively).

The TF performed most of the nitrification reactions. The removal rates of TKN due to the TF were 65 % (43.2 mg/L), 88 % (42.1 mg/L) and 77 % (44.2 mg/L) for phase 1, 2 and 3, respectively. This showed that nitrification reactions in the TF were almost stable during the whole experiment, if the initial TKN concentration was higher than the removal concentrations (between 42.1 and 44.2 mg/L).

The results for the different phases show a better performance for lower hydraulic rates. In addition, reducing by half the hydraulic load does not result in doubling the removal or conversion rates. Comparing the different hydraulic load in each phase, it can be mention that different conversion reactions and removals are very similar between each hydraulic load. Langenbach et al. (2009) proposed filter velocities of around 0.2 m/h as economically attractive. However, as observed from the results, it is also possible to implement hydraulic load as 0.3 m/h. This value ensures a relative treatment regarding physico-chemical parameters and secures the removal of pathogen such as *E. coli* and TC below the recommended values to reuse in agriculture.

4.7. *Schmutzdecke* behavior

The increase of the pressure loss depended on the suspended solids fed into the system. As seen in Appendix 9, phase 1 had the highest concentration of suspended solids, which was translated in a frequent removal of the *schmutzdecke* before it reached the limit value of 0.3 m. On average, during phase 1, the removal was performed every 9, 7 and 5 days for SSF1, SSF2 and SSF3, respectively. However, during the last 2 weeks of the experiment, a better performance of the TF reduced the SS into the filter, increasing the duration to more than 10 days for all the filters. During phase 2 (lowest organic load into the TF), SSF1 and SSF2 did not required the removal of the *schmutzdecke* during the whole duration of the phase (47 days). For SSF3, the average duration was above 9 days. In case of phase 3, SSF1 did not require *schmutzdecke* removal. SSF2 and SSF3 required the removal after 21 and 15 days, respectively. It was observed that there is not a constant pressure loss for any phase or filter. In addition, SSF3 in all phases required the most maintenance. Considering performance and maintenance, SSF2 with a hydraulic load of 0.3 m/h showed to be the most optimal hydraulic load for all the phases. This values is slightly higher of the 0.2 m/h recommended by Langenbach et al. (2009).

5 Conclusions

- The trickling filter removed between 79 to 97 % turbidity of the inflow wastewater. The slow sand filter removed between 31 and 49 % of turbidity for all the experimental phases with a final turbidity values for all filters in all the phases of 1.7 NTU.
- The rapid sand filter was installed as a failsafe mechanism during the phase with higher organic load into the trickling filter to reduce the suspended solids concentration from 63.3 to 8.2 mg/L. The suspended solids after the treatment with the trickling filter in phase 2 and 3 had concentrations below 6 mg/L.
- The trickling filter removed between 79 to 89 % of the COD_{Total} (final concentration of around 80, 26 and 33 mg/L for phase 1, 2 and 3, respectively) while the slow sand filter removed between 18 to 19 % (final effluent around 33 mg/L), 20 to 23 % (final effluent around 16 mg/L) and 6 to 13 % (final effluent around 22 mg/L) for phase 1, 2 and 3, respectively.
- The BOD_5 removal in the treatment system was mainly performed by the trickling filter (values around 85 %). The SSF reported values of BOD_5 removal of around 50 % (final effluent values below 3 mg BOD_5/L).
- The inflow wastewater used in this research presented a higher nutrient content (nitrogen compounds) than the expected for domestic wastewater effluents, with a BOD_5/TN/TP ratio of 25:8.5:1, 20:8:1, 18:8.5:1 for phase 1, 2 and 3, respectively.
- The trickling filter removed between 1.3, 1.6 and 2.3 mg/L of total phosphorus for phase 1, 2 and 3, respectively. An increased of orthophosphates was detected during phase 1 with the higher organic load. The slow sand filter did not remove total phosphorus or orthophosphates.
- The trickling filter removed concentrations of 21, 19 and 26 mg/L of TN for phase 1, 2 and 3, respectively, with a removal percentage between 30 to 40 %. The slow sand filter removed concentrations of total nitrogen between 1.8 and 2.8 mg/L, 0.9 to 1.6 mg/L and 1.5 to 2 mg/L for phase 1, 2 and 3, respectively. The reduction by half of the hydraulic load only increased the removal concentration of TN on average between 0.1 to 0.5 mg/L.
- The trickling filter transformed 31, 36 and 40 mg/L of ammonia—nitrogen for phase 1, 2 and 3, respectively, with removals of 63, 94 and 73%, respectively. For the slow sand filter, the concentrations of removed ammonia—nitrogen were between 0.4 to1.8 mg/L, 0.2 to 0.5 mg/L and 1.1 to 1.5 mg/L for phase 1, 2 and 3, respectively, showing limited nitrification reactions.
- The nitrite—nitrogen concentrations for the treatment units and all cases were below 1.7 mg/L.
- The increment of nitrate—nitrogen in the outflow of the trickling filter varied between 20 to 22 mg/L for all phases.
- The removal rates of TKN due to the TF were 65 % (43.2 mg/L), 88 % (42.1 mg/L) and 77 % (44.2 mg/L) for phase 1, 2 and 3, respectively.
- In the slow sand filters, the amount of ammonia—nitrogen converted into nitrate-nitrogen range between 0.8 to 2.4 mg/L, 0.3 to 0.4 mg/L and 0.5 to 1.3 mg/L for phase 1, 2 and 3, respectively.
- The removal rates of TKN due to the TF were 65 % (43.2 mg/L), 88 % (42.1 mg/L) and 77 % (44.2 mg/L) for phase 1, 2 and 3, respectively.
- The results for the removal of physico-chemical parameters still do not show consensus on the removal rates for slow sand filters, which still manages to require case-to-case experiments.
- A slow sand filter with a hydraulic load of 0.3 m/h can achieved similar removal rates for contaminants such as COD and nitrogen compounds than hydraulic loads of 0.15 and 0.6 m/h, and pathogen removal within the limits set by the WHO and Peruvian Law for water reuse in agriculture.

- A grain size between 0.4 to 0.8 mm was proven to be effective for the removal of bacteriological parameters such as *E. coli*, total coliforms and Enterococcus.
- The trickling filter removed E. coli, Total Coliforms and Enterococcus in the range of 0.9 to 2.3 $\log_{10}$, 1.5 and 2.3 $\log_{10}$ and 1.5 to 3.3 $\log_{10}$, respectively.
- The E. coli removal for the slow sand filter with hydraulic load of 0.3 m/h was 0.5, 0.8 and 1 $\log_{10}$. For the hydraulic load of 0.15 and 0.6 m/h, the removal rates were between 0.9 to 1.4 and 0.2 to 0.9 $\log_{10}$, respectively.
- The removal of Total Coliforms for the slow sand filter with hydraulic load of 0.3 m/h was between 0.7 and 1 $\log_{10}$. For the filter with hydraulic load of 0.15 and 0.6 m/h, the removal rates were between 1 to 1.6 $\log_{10}$ and 0.3 and 1.0 $\log_{10}$, respectively.
- The removal of Enterococcus for the filter with hydraulic loads of 0.15, 0.3 and 0.6 m/h were between 0.15 to 1.3 $\log_{10}$, 0.1 to 0.4 $\log_{10}$ and 0.5 to 1 $\log_{10}$, respectively.
- After the scraping of the schmutzdecke, there was not a detection of a worsening of the effluent quality.
- The sand material used in this research for the slow sand filters did not interact with the remaining orthophosphates in the treated wastewater.
- For low inflow concentrations in the slow sand filter of ammonia—nitrogen, there is an adequation time to be considered before detecting removal of ammonia—nitrogen. For higher inflow concentrations, the removal is observed shortly after.
- The pressure loss in the slow sand filter showed that a hydraulic load of 0.3 m/h requires less maintenance than a load of 0.6 m/h with similar removal values of a load of 0.15 m/h, which in all the phases presented the higher removal values.
- An additional treatment unit after the trickling filter to improve nitrification reactions would also avoid the necessity of a system such as the rapid sand filter.
- An additional trickling filter for nitrification-denitrification reactions can improve the treated water quality in case it is required to be discharged in a water body.

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Appendix

Appendix 1 Dimensions of the treatment units

Parameter	Unit	Value
Storage tank		
Volume	m^3	4.5
Tricking filter		
Height of Cylinder	m	3
Height of substrate	m	2.5
Radius	m	0.25
Superficial area (As)	m^2	0.196
Volume with substrate material (V)	m^3	0.491
Porosity	%	0.90
Volume of water in the filter	m^3	0.445
Specific surface area of material	m^2/m^3	150
Rapid sand filter		
Height of Cylinder	m	0.6
Height of substrate	m	0.15
Sand diameter	mm	0.4 — 0.8
Radius	m	0.3
Superficial area (As)	m^2	0.28
Slow sand filter		
Height of filter	m	1
Height of sand layer	m	0.6
Height of bottom layer	m	0.05
Volume of filter with substrate	Liters	5.11
Sand diameter	mm	0.4 — 0.8
Bottom layer diameter	mm	1.6 — 2
Sand density	kg/m^3	1470
Real sand density	kg/m^3	2685
Porosity	%	44.1
Sand used in each filter	kg	6.84
Bottom layer density	kg/m^3	1417
Bottom layer used in each filter	kg	0.567
Volume of water in each filter	Liters	2.25

Appendix 2 Laboratory equipment

Equipment for In-situ parameters

Equipment	Parameter
Turb 430 IR	Turbidity
WTW Multi 3430/ WTW pH Electro	pH
WTW Multi 3430/WTW FDO® 925 IDS	Dissolved Oxygen
WTW Multi 3430/WTW TetraCon® 925 IDS	Temperature/Conductivity

Laboratory equipment for wastewater analysis

Equipment	Parameter
Digestor Nanocolor Vario 4 Machery-Nagel	Ammonium, nitrite, orthophosphate, COD, total nitrogen
Spectrophotometer Nanocolor VIS-II, FP-200 Macherey-Nagel	Ammonium, nitrite, orthophosphate, COD, total nitrogen
Digestor LT 200 Hach Lange	COD, Total Phosphorus
DR 2800 Hach Lange	COD, Total Phosphorus
UV/VIS Spectrophotometer V-550	Ammonium, nitrate
Scale Precisa 321 Lx 220ASCS	General use
Mixer Heidolph MR Hei-Mix L	General use
Drying Oven Memmert	General use

Appendix 3 Organic and Hydraulic loads for each phase of the study

Parameter	Unit	Start-up	Phase 1	Phase 2	Phase 3
BOD concentration (Si)	Kg BOD/m^3	0.151	0.206	0.134	0.142
Flow rate (Qi)	m^3/d	1.84	1.84	1.84	1.84
Recirculation factor	R	1.83	1.83	1.83	1.83
Recirculation flow (Qr)	m^3/d	3.36	3.36	3.36	3.36
Removal efficiency*	%	0.85	0.85	0.85	0.85
BOD conc. of recirculation flow (Sr)	Kg BOD/m^3	0.023	0.023	0.023	0.023
BOD conc. of inf. Fed to the filter (So)	Kg BOD/m^3	0.07	0.09	0.06	0.07
Organic Load	Kg BOD/(d*m^3)	0.72	0.93	0.66	0.69
Hydraulic Load	m^3/(m^2*d)	26.46	26.46	26.46	26.46
	m^3/(m^2*h)	1.10	1.10	1.10	1.10

(*) Based on the average values of BOD_5 for the trickling filter during the whole experiment

Appendix 4 BOD_5 values and BOD_5/COD relation

Parameters	Sampling points	Day of measurement					Average
		27	**92**	**99**	**106**	**118**	
BOD_5 (mg/L)	Inflow 1	201.12	125	102	213	112	150.6
	Outflow 1	14.88	20	26	28	27	23.18
	Outflow 2		<3	<3	5	9	
	Inflow 2					6	
	Outflow 3		<3	<3		3	
	Outflow 4		<3	<3		<3	
	Outflow 5		<3	<3		<3	
COD total (mg/L)	Inflow 1	402	194.5	194	486.5	366	328.6
	Outflow 1	51.6	41.5	82.5	103.5	86.5	73.12
BOD_5/COD	Inflow 1	0.50	0.64	0.53	0.44	0.31	0.48*
	Outflow 1	0.29	0.48	0.32	0.27	0.31	0.33

*It was considered for the calculations a theorical value of 0.5

Appendix 5 In situ parameters measured for each phase during the experiment

Phases	Sampling points	Dissolved Oxygen	pH	Electric Conductivity	Temperature	Turbidity
		mg/L ± Std. dev (n values)	Value ± Std. dev (n values)	uS/cm ± Std. dev (n values)	°C ± Std. dev (n values)	NTU ± Std. dev (n values)
Start-up Phase [1-104]	Inflow 1	-	7.7 ± 0.13 (27)	1285 ± 217 (27)	19.64 ± 1.26 (27)	94.3 ± 20.6 (3)
	Outflow 1	7.34 ± 0.46 (26)	7.5 ± 0.17 (27)	1117 ± 222 (27)	19.65 ± 1.32 (27)	19.2 ± 11.7 (3)
Phase 1 [104-134]	Inflow 1 (in TK)	-	7.6 ± 0.11 (8)	1324 ± 202 (8)	23.16 ± 1.48 (8)	132.9 ± 36.7 (7)
	Outflow 1 (out TF)	6.52 ± 0.46 (8)	7.5 ± 0.12 (8)	1160 ± 94 (8)	23.09 ± 1.41 (8)	28.3 ± 16.6 (8)
	Outflow 2	6.12±0.9(8)	7.4 ± 0.23 (8)	1142 ± 80 (8)	23.04 ± 1.39 (8)	4.3 ± 3.5 (8)
	Inflow 2	5.52 ± 0.78 (8)	7.4 ± 0.09 (8)	1133 ± 85 (8)	22.95 ± 1.34 (8)	2.9 ± 2.6 (8)
	Outflow 3 (SSF1)	6.94±0.33(8)	7.7 ± 0.1 (8)	1111 ± 98 (8)	22.85 ± 1.33 (8)	1.7 ± 0.8 (8)
	Outflow 4 (SSF2)	6.89 ± 0.28 (8)	7.6 ± 0.11 (8)	1124 ± 97 (8)	22.78 ± 1.45 (8)	1.4 ± 0.7 (8)
	Outflow 5 (SSF3)	6.41±0.42(8)	7.5 ± 0.12 (8)	1125 ± 91 (8)	22.98 ± 1.37 (8)	1.3 ± 0.5 (8)
Phase 2 [134-180]	Inflow 1 (in TK)	-	7.6 ± 0.14 (13)	1248 ± 57 (13)	22.67 ± 1.38 (13)	99.9 ± 29.1 (13)
	Outflow 1 (out TF)	7.25±0.49(13)	7.6 ± 0.11 (13)	1055 ± 82 (13)	22.75 ± 1.27 (13)	2.5 ± 1.2 (13)
	Outflow 2	7.2 ± 0.49 (13)	7.6 ± 0.11 (13)	1048 ± 88 (13)	22.85 ± 1.27 (13)	1.7 ± 1.4 (13)
	Inflow 2	6.96±0.89(13)	7.6 ± 0.15 (13)	1047 ± 81 (13)	22.75 ± 1.39 (13)	1.3 ± 0.6 (13)
	Outflow 3 (SSF1)	7.53 ± 0.37 (12)	7.8 ± 0.1 (12)	1052 ± 66 (12)	22.58 ± 1.22 (12)	0.9 ± 0.5 (12)
	Outflow 4 (SSF2)	7.43±0.34(12)	7.8 ± 0.08 (12)	1058 ± 66 (12)	22.54 ± 1.18 (12)	0.9 ± 0.6 (12)
	Outflow 5 (SSF3)	7.26 ± 0.49 (12)	7.7 ± 0.16 (12)	1056 ± 64 (12)	22.68 ± 1.26 (12)	1 ± 0.5 (12)
Phase 3 [180-207]	Inflow 1 (in TK)	-	7.6 ± 0.12 (8)	1317 ± 118 (8)	20.35 ± 1.36 (8)	106.7 ± 44 (8)
	Outflow 1 (out TF)	6.69 ± 0.61 (8)	7.5 ± 0.36 (8)	1128 ± 79 (8)	20.18 ± 1.4 (8)	3.3 ± 1.2 (8)
	Outflow 2	6.94±0.82(8)	7.5 ± 0.23 (8)	1112 ± 79 (8)	20.11 ± 1.44 (8)	1.8 ± 1.5 (8)
	Inflow 2	5.88 ± 0.67 (8)	7.4 ± 0.17 (8)	1119 ± 68 (8)	20.09 ± 1.42 (8)	1 ± 0.3 (8)
	Outflow 3 (SSF1)	7.82±0.41(8)	7.7 ± 0.15 (8)	1105 ± 65 (8)	20.1 ± 1.55 (8)	0.6 ± 0.2 (8)
	Outflow 4 (SSF2)	7.21 ± 0.35 (8)	7.6± 0.14 (8)	1108 ± 65 (8)	20.08 ± 1.5 (8)	0.5 ± 0.1 (8)
	Outflow 5 (SSF3)	6.88±0.57(8)	7.5 ± 0.15 (8)	1119 ± 71 (8)	20.3 ± 1.5 (8)	0.6 ± 0.2 (8)

Appendix 6 Removal of turbidity in each phase

	Turbidity removal (%) ± Std. dev (n values)		
Sampling points	**Phase 1 [104-134]**	**Phase 2 [134-180]**	**Phase 3 [180-207]**
Trickling filter	78.5 ± 14.7 (8)	97.4 ± 1.4 (13)	96.8 ± 1.2 (8)
Rapid sand filter	81.5 ± 10.3 (8)	42.5 ± 18 (12)	46.1 ± 32.1 (8)
Slow sand filter 1	36.8 ± 20.9 (7)	31.9 ± 21.9 (11)	42.4 ± 19.5 (8)
Slow sand filter 2	36.3 ± 26.2 (8)	35.5 ± 24.4 (11)	44.6 ± 14.6 (8)
Slow sand filter 3	49.6 ± 24.2 (7)	29.9 ± 20.2 (9)	40.4 ± 15.2 (8)

Appendix 7 Values of COD_{Total} and $COD_{Dissolved}$ and removal rate in each phase

Sampling points	Phase 1 [104-134]		Phase 2 [134-180]		Phase 3 [180-207]	
	COD_{Total}	**$COD_{Dissolved}$**	**COD_{Total}**	**$COD_{Dissolved}$**	**COD_{Total}**	**$COD_{Dissolved}$**
	Measured values (mg/L) ± Std. dev (n values)					
Inflow 1 (in TK)	412.38 ± 98.67 (8)	114 ± 31.38 (8)	267.1 ± 89.83 (24)	92.38 ± 15 (13)	283.73 ± 80.83 (9)	121.88 ± 31.17 (8)
Outflow 1 (out TF)	80.19 ± 24.15 (8)	42.2 ± 4.11 (8)	26.15 ± 9.42 (24)	22.27 ± 3.06 (13)	32.64 ± 9.77 (9)	30.38 ± 7.44 (8)
Outflow 2	41.53 ± 12.01 (5)		21.7 ± 6.88 (12)		26 ± 8.57 (9)	
Inflow 2	39 ± 10.86 (5)		20.67 ± 3.91 (13)		24.22 ± 6.02 (9)	
Outflow 3 (SSF1)	31.08 ± 5.69 (4)		16.04 ± 2.49 (12)		21.44 ± 6.33 (9)	
Outflow 4 (SSF2)	33.37 ± 3.27 (4)		16.79 ± 2.33 (12)		22 ± 6.28 (9)	
Outflow 5 (SSF3)	33.37 ± 4.09 (4)		16.82 ± 2.16 (12)		22.78 ± 6.4 (9)	
Treatment Unit	**Efficiency removal (%) ± Std. dev (n values)**					
Trickling filter	79.45 ± 8.41 (8)	61.03 ± 9.14 (8)	89.44 ± 4.48 (24)	75.6 ± 3.3 (13)	88.16 ± 3.76 (9)	74.87 ± 3.28 (8)
Rapid sand filter	44.16 ± 20.28 (3)		22.32 ± 17.84 (10)		20.31 ± 13.89 (9)	
Slow sand filter 1	20.59 ± 9.52 (3)		23.22 ± 13.61 (11)		12.62 ± 9.58 (9)	
Slow sand filter 2	18.93 ± 16.31 (3)		21.79 ± 13.31 (10)		9.53 ± 10.46 (9)	
Slow sand filter 3	19.3 ± 14.31 (3)		20.41 ± 10.03 (11)		6.7 ± 8.2 (9)	

Appendix 8 Values of suspended solids and removal rate in each phase

Sampling points	Phase 1 [104-134]	Phase 2 [134-180]	Phase 3 [180-207] (*)
	Measured values (mg/L) ± Std. dev (n values)		
Outflow 1 (out TF)	63.3±59.6(7)	4.1±2.4(12)	5.3±2.0(6)
Outflow 2	8.2±7.2 (7)	2.9±2.6(12)	3.6±1.6(6) (**)
Treatment Unit	**Efficiency removal (%) ± Std. dev (n values)**		
Rapid sand filter	88.5±7.2(6)	40.5±52.1(12)	39.1±28.4(5)

(*) Between the days 201 and 204, unusual low inflow values were detected, which were not considered for removal average

(**) On day 207, the RSF was clogged due the concentration of suspended solids in the system. Backwashing was performed before taking the sample so the removal rate for this day was not considered in the average values.

Appendix 9 Frequency of *schmutzdecke* removal in the slow sand filter in each phase

Filter	Phase 1 [104-134]	Phase 2 [134-180]	Phase 3 [180-207]
Filter 1 (0.15m/h)	7	47(*)	28(*)
	12	-	-
	10(*)	-	-
Filter 2 (0.3m/h)	7	47(*)	21
	7	-	7(*)
	16(*)	-	-
Filter 3 (0.6m/h)	7	8	15
	5	5	13(*)
	3	11	-
	15(*)	13	-
	-	4(*)	-

(*) Values measure at the end of each phase when removal of *schmutzdecke* was not yet required

Appendix 10 Values of Total Phosphorus and Orthophosphates for each phase

Sampling points	Phase 1 [104-134]		Phase 2 [134-180]		Phase 3 [180-207]	
	Total Phosphorus (TP)	**Orthophosphates (PO4-P)**	**Total Phosphorus (TP)**	**Orthophosphates (PO4-P)**	**Total Phosphorus (TP)**	**Orthophosphates (PO4-P)**
	Measured values (mg/L) ± Std. dev (n values)					
Inflow 1 (in TK)	7.98 ± 1.29 (6)	4.93 ± 0.94 (8)	6.36 ± 1.39 (7)	4.2 ± 1.15 (13)	7.53 ± 1.5 (6)	4.91 ± 1.12 (8)
Outflow 1 (out TF)	6.65 ± 0.9 (6)	5.7 ± 0.57 (8)	4.81 ± 0.23 (7)	4.25 ± 0.39 (13)	5.2 ± 1.42 (6)	5.05 ± 1.31 (8)
Outflow 2	6.04 ± 0.72 (6)	5.6 ± 0.58 (8)	4.82 ± 0.2 (7)	4.34 ± 0.34 (12)	5.18 ± 1.36 (6)	5.1 ± 1.26 (8)
Inflow 2	6.34 ± 0.26 (4) (*)	5.49 ± 0.56 (8)	4.79 ± 0.21 (7)	4.31 ± 0.35 (13)	5.28 ± 1.03 (6)	5.19 ± 1.02 (8)
Outflow 3 (SSF1)	6.25 ± 0.23 (2) (*)	5.51 ± 0.57 (8)	4.75 ± 0.23 (7)	4.25 ± 0.31 (12)	5.3 ± 1.02 (6)	5.14 ± 0.96 (8)
Outflow 4 (SSF2)	6.265 ± 0.19 (2)	5.45 ± 0.57 (7)	4.73 ± 0.23 (7)	4.23 ± 0.34 (12)	5.32 ± 1.04 (6)	5.37 ± 1.25 (8)
Outflow 5 (SSF3)	6.295 ± 0.16 (2)	5.55 ± 0.61 (8)	4.75 ± 0.23 (7)	4.25 ± 0.35 (12)	5.31 ± 1 (6)	5.21 ± 0.93 (8)
Treatment Unit	**Efficiency removal (%) ± Std. dev (n values)**					
Trickling filter	16.1 ± 7.85 (6)	-19.68 ± 29.42 (8)	22.07 ± 13.23 (7)	-7.18 ± 23.63 (12)	31.94 ± 11.03 (6)	-1.79 ± 9.08 (8)
Rapid sand filter	8.97 ± 6.15 (6)	1.74 ± 3.86 (8)	-0.28 ± 1.65 (7)	-0.57 ± 2.1 (12)	-0.26 ± 3.06 (6)	-1.64 ± 3.08 (8)
Slow sand filter 1	1.34 ± 1.78 (2) (*)	-0.24 ± 1.66 (8)	0.85 ± 1.58 (7)	0.25 ± 1.44 (12)	-0.49 ± 1.9 (6)	0.58 ± 3.37 (8)
Slow sand filter 2	1.09 ± 1.07 (2)	0.06 ± 1.32 (7)	1.09 ± 0.88 (7)	0.83 ± 2.94 (12)	-0.82 ± 1.5 (6)	-3.64 ± 11.22 (8)
Slow sand filter 3	0.6 ± 1.54 (2)	-1.07 ± 1.53 (8)	0.67 ± 1.17 (7)	0.52 ± 2.24 (12)	-0.71 ± 1.34 (6)	-1.07 ± 4.83 (8)

(*) Only the measurements for the last 2 sampling campaigns were considered

Appendix 11 Values of Nitrogen species and removal rates for Phase 1

Sampling points	TN	NH_4-N	NO_2-N	NO_3 -N
		Measured values (mg/L) ± Std. dev (n values)		
Inflow 1 (in TK)	67.59 ± 11.88 (7)	50.68 ± 10.47 (8)	0.02 ± 0.05 (8)	0.66 ± 0.32 (8)
Outflow 1 (out TF)	46.59 ± 8.3 (7)	19.47 ± 8.18 (8)	0.89 ± 0.15 (8)	22.68 ± 3.98 (8)
Outflow 2	45.39 ± 6.5 (7)	16.61 ± 7.26 (8)	1.24 ± 0.13 (8)	24.89 ± 2.11 (8)
Inflow 2	42.06 ± 6.67 (5)	15.69 ± 6.98 (8)	1.14 ± 0.38 (8)	23.38 ± 1.61 (8)
Outflow 3 (SSF1)	40.5 ± 6.36 (4)	13.94 ± 7.54 (8)	1.24 ± 0.34 (8)	22.66 ± 2.02 (8)
Outflow 4 (SSF2)	38.25 ± 8.41 (2)	14.97 ± 7.5 (8)	1.51 ± 0.37 (8)	22.28 ± 1.92 (8)
Outflow 5 (SSF3)	38.7 ± 8.91 (2)	15.33 ± 7.35 (8)	1.33 ± 0.25 (8)	22.74 ± 1.74 (8)
Treatment Unit		**Efficiency removal (%) ± Std. dev (n values)**		
Trickling filter	30.09 ± 11.57 (7)	62.65 ± 11.78 (8)	-	-
Rapid sand filter	1.59 ± 11.86 (7)	14.45 ± 12.51 (8)	-	-
Slow sand filter 1	6.54 ± 2.36 (4)	15.08 ± 17.79 (8)	-	-
Slow sand filter 2	5.83 ± 1.54 (2)	6.79 ± 16.12 (8)	-	-
Slow sand filter 3	4.83 ± 2.55 (2)	3.72 ± 8.27 (8)	-	-
Treatment Unit		**Efficiency removal (mg/L)(*) ± Std. dev (n values)**		
Removal-Trickling Filter	21 ± 9.63 (7)	31.21 ± 6.48 (8)	0.87 ± 0.18 (8)	22.02 ± 3.94 (8)
Removal-Rapid sand filter	1.21 ± 4.61 (7)	0.92 ± 2.76 (8)	0.35 ± 0.12 (8)	2.21 ± 2.5 (8)
Removal-Slow sand filter 1	2.85 ± 1.28 (4)	1.75 ± 1.68 (8)	0.11 ± 0.54 (8)	-0.72 ± 1.87 (8)
Removal-Slow sand filter 2	2.3 ± 0.14 (2)	0.72 ± 1.67 (8)	0.37 ± 0.4 (8)	-1.09 ± 1.67 (8)
Removal-Slow sand filter 3	1.85 ± 0.64 (2)	0.36 ± 0.83 (8)	0.19 ± 0.35 (8)	-0.63 ± 1.47 (8)

(*) Calculated by extracting the value from the inflow and outflow of the system evaluated

Appendix 12 Values of Nitrogen species and removal rates for Phase 2

Sampling points	TN	NH_4-N	NO_2-N	NO_3 -N
		Measured values (mg/L) ± Std. dev (n values)		
Inflow 1 (in TK)	48.2 ± 4.71 (7)	39.02 ± 5.23 (25)	0.03 ± 0.05 (13)	0.56 ± 0.13 (13)
Outflow 1 (out TF)	28.96 ± 1.68 (7)	2.27 ± 2.04 (25)	0.31 ± 0.16 (13)	22.6 ± 2.17 (13)
Outflow 2	28.68 ± 2.96 (6)	0.82 ± 1.66 (20)	0.28 ± 0.32 (13)	23.35 ± 2.55 (13)
Inflow 2(*)	31.17 ± 1.24 (3)	1.51 ± 1.78 (7)	0.62 ± 0.48 (4)	25.43 ± 2.09 (4)
Outflow 3 (SSF1) (*)	29.55 ± 0.61 (3)	0.98 ± 1.23 (7)	0.67 ± 0.7 (4)	25.5 ± 2.02 (4)
Outflow 4 (SSF2) (*)	29.77 ± 1.4 (3)	1.25 ± 1.53 (7)	0.75 ± 0.65 (4)	25.14 ± 2.42 (4)
Outflow 5 (SSF3) (*)	30.21 ± 1.31 (3)	1.5 ± 1.61 (6)	0.62 ± 0.51 (4)	25.38 ± 2.19 (4)
Treatment Unit		**Efficiency removal (%) ± Std. dev (n values)**		
Trickling filter	39.27 ± 8.5 (7)	94.29 ± 4.99 (25)	-	-
Rapid sand filter	0.77 ± 7.38 (6)	80.31 ± 27.82 (20)	-	-
Slow sand filter 1(*)	5.16 ± 2.16 (3)	37.92 ± 27.26 (7)	-	-
Slow sand filter 2(*)	4.51 ± 1.31 (3)	38.91 ± 25.48 (7)	-	-
Slow sand filter 3(*)	3.09 ± 0.53 (3)	34.73 ± 27.44 (6)	-	-
Treatment Unit		**Efficiency removal (mg/L) (**) ± Std. dev (n values)**		
Removal-Trickling Filter	19.24 ± 5.33 (7)	35.63 ± 8.43 (25)	0.28 ± 0.14 (13)	22.04 ± 2.19 (13)
Removal-Rapid sand filter	0.2 ± 2.1 (6)	0.23 ± 0.51 (19)	-0.02 ± 0.21 (13)	0.75 ± 0.74 (13)
Removal-Slow sand filter 1(*)	1.63 ± 0.72 (3)	0.53 ± 0.67 (7)	0.04 ± 0.26 (4)	0.07 ± 0.27 (4)
Removal-Slow sand filter 2(*)	1.4 ± 0.4 (3)	0.26 ± 0.3 (7)	0.12 ± 0.21 (4)	-0.28 ± 0.36 (4)
Removal-Slow sand filter 3(*)	0.96 ± 0.14 (3)	0.21 ± 0.33 (7)	-0.01 ± 0.05 (4)	-0.04 ± 0.28 (4)

(*) Only the results after day 168 (35 days after the beginning of phase 2) were considered. After that day, NH4—N was removed consistently for all the SSFs

(**) Calculated by extracting the value from the inflow and outflow of the system evaluated

Appendix 13 Values of Nitrogen species and removal rates for Phase 3

Sampling points	TN(*)	NH_4-N(*)	NO_2-N(*)	NO_3 -N(*)
	Measured values (mg/L) ± Std. dev (n values)			
Inflow 1 (in TK)	63.98 ± 5.45 (5)	54.94 ± 3.8 (8)	0.01 ± 0.01 (7)	0.58 ± 0.1 (7)
Outflow 1 (out TF)	37.99 ± 2.02 (5)	14.88 ± 2.25 (8)	0.67 ± 0.1 (7)	21.01 ± 2.33 (7)
Outflow 2	36.97 ± 2.2 (5)	10.46 ± 1.71 (7)	0.7 ± 0.28 (7)	23.1 ± 2.13 (7)
Inflow 2	37.62 ± 2.52 (5)	10.37 ± 2.68 (7)	0.8 ± 0.37 (7)	23.47 ± 1.75 (7)
Outflow 3 (SSF1)	35.62 ± 1.32 (5)	8.84 ± 2.59 (7)	1.72 ± 0.85 (7)	22.77 ± 2.3 (7)
Outflow 4 (SSF2)	35.97 ± 1.72 (5)	9.06 ± 2.48 (7)	1.75 ± 0.47 (7)	22.96 ± 2.48 (7)
Outflow 5 (SSF3)	36.04 ± 2.18 (5)	9.22 ± 2.93 (7)	1.65 ± 0.56 (7)	23.27 ± 2.22 (7)
Treatment Unit	**Efficiency removal (%) ± Std. dev (n values)**			
Trickling filter	40.27 ± 5.87 (5)	72.92 ± 3.65 (8)	-	-
Rapid sand filter	2.62 ± 4.93 (5)	30.74 ± 6.29 (7)	-	-
Slow sand filter 1	5.1 ± 4.95 (5)	15.38 ± 5.61 (7)	-	-
Slow sand filter 2	4.26 ± 3.23 (5)	13.02 ± 5.32 (7)	-	-
Slow sand filter 3	4.15 ± 2.08 (5)	12.11 ± 5.86 (7)	-	-
Treatment Unit	**Efficiency removal (mg/L) (**) ± Std. dev (n values)**			
Removal-Trickling Filter	25.99 ± 6.13 (5)	40.06 ± 3.5 (8)	0.65 ± 0.09 (7)	20.43 ± 2.3 (7)
Removal-Rapid sand filter	1.02 ± 1.8 (5)	0.09 ± 2.99 (7)	0.03 ± 0.23 (7)	2.1 ± 1.04 (7)
Removal-Slow sand filter 1	2 ± 2.1 (5)	1.52 ± 0.51 (7)	0.92 ± 0.59 (7)	-0.71 ± 0.83 (7)
Removal-Slow sand filter 2	1.65 ± 1.33 (5)	1.31 ± 0.6 (7)	0.95 ± 0.34 (7)	-0.51 ± 0.87 (7)
Removal-Slow sand filter 3	1.58 ± 0.87 (5)	1.14 ± 0.43 (7)	0.85 ± 0.31 (7)	-0.21 ± 0.71 (7)

(*) The values between the days 201 to 203 were not considered due to low loads registered in the inflow

(**) Calculated by extracting the value from the inflow and outflow of the system evaluated

Appendix 14 Values of pathogen removal for all phases

	E. Coli (MPN/100 mL)			***Colif. Bacteria (MPN/100 mL)***			***Enterococcus (CFU/100 mL)***		
Sampling points	**Phase 1**	**Phase 2**	**Phase 3**	**Phase 1**	**Phase 2**	**Phase 3**	**Phase 1**	**Phase 2**	**Phase 3**
Inflow 1 (in TK)	1.1E+05	1.2E+06	5.8E+06	2.2E+06	3.0E+06	1.1E+07	1.3E+07	1.3E+05	1.1E+06
Outflow 1 (out TF)	1.2E+04	2.6E+04	2.3E+04	6.5E+04	7.3E+04	5.7E+04	5.3E+03	3.6E+03	1.4E+04
Outflow 2	-	1.6E+04	1.3E+04	-	6.5E+04	5.2E+04	-	2.5E+02	5.5E+03
Inflow 2	3.1E+02	1.1E+03	3.0E+02	2.3E+03	1.3E+03	2.0E+03	2.0E+02	1.2E+02	4.4E+01
Outflow 3 (SSF1)	1.4E+01	5.7E+01	4.1E+01	6.1E+01	1.3E+02	8.5E+01	1.0E+01	2.3E+01	3.1E+01
Outflow 4 (SSF2)	4.1E+01	9.1E+01	9.8E+01	2.0E+02	2.3E+02	2.9E+02	7.0E+01	2.9E+01	3.3E+01
Outflow 5 (SSF3)	6.3E+01	1.3E+02	2.1E+02	2.4E+02	3.3E+02	1.0E+03	2.0E+01	3.4E+01	8.3E+01
Treatment Unit	**Efficiency removal (%)**								
Trickling filter	88.1	97.8	99.6	97.1	97.6	99.5	99.96	97.2	98.7
Rapid sand filter	97.5	38.2	98.7	96.4	10.8	96.5	96.2	93.1	59.6
Slow sand filter 1	95.5	94.9	86.5	97.4	90.0	95.8	95.0	81.1	29.5
Slow sand filter 2	86.8	91.8	67.7	91.5	82.9	85.8	65.0	76.2	25.0
Slow sand filter 3	79.7	88.0	30.7	89.6	75.2	48.1	90.0	72.1	-88.6
Treatment Unit	**Efficiency removal (log unit)**								
Trickling filter	0.93	1.67	2.40	1.54	1.61	2.29	3.38	1.56	1.90
Rapid sand filter	1.60	0.21	0.24	1.45	0.05	0.04	1.42	1.16	0.39
Slow sand filter 1	1.35	1.29	0.87	1.58	1.00	1.37	1.30	0.72	0.15
Slow sand filter 2	0.88	1.09	0.49	1.07	0.77	0.85	0.46	0.62	0.12
Slow sand filter 3	0.69	0.92	0.16	0.98	0.61	0.28	1.00	0.55	-0.28