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Subject/Topic:

UPGRADING A WASTE STABILIZATION POND AND ESTIMATION OF BIOGAS PRODUCTION: CASE OF UNIVERSITY OF ABOMEY-CALAVI, BENIN

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DEDICATION

I dedicate this work to:

Benin, my Country

My previous School: Ecole Superieure des Metiers des Energies Renouvelables (ESMER)

My parents: Florence I. J. Cica & Ernest YELINDO

My brothers and sister: Nedis T. S. YELINDO, Esther K. J. YELINDO, Espoir F. V. YELINDO.

DECLARATION

I hereby declare that the submitted thesis work has been completed by me and that I have not used any other than permitted reference sources or materials. All references and other sources used by me have been appropriately acknowledged in this work. I further declare that this work has not been submitted for the purpose of academic examination, either in its original or similar form, anywhere else.

Declared in Rostock, August 2023

Mahougnon E. F. YELINDO

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RESUMÉ

La croissance de la population affecte la disponibilité des terres ainsi que la production d'eaux usées. En Afrique, le traitement des eaux usées repose principalement sur le système des bassins de stabilisation des déchets. Ces bassins sont adaptés au climat chaud du continent. Cependant, avec le temps, ils doivent être rénovés parce qu'ils sont soumis à une surcharge qui entraîne la pollution des eaux souterraines et des masses d'eau. Les stations d'épuration émettent également du biogaz dans l'atmosphère. Cette étude vise à réhabiliter un bassin de stabilisation des déchets construit en 2011, sur le site de l'Université d'Abomey-Calavi, au Bénin, et à évaluer l'évolution du biogaz produit au fil des années par la station. Pour atteindre cet objectif, trois scénarios ont été élaborés : Le premier se concentre sur le redimensionnement des différents bassins de la station d'épuration avec une estimation de l'évolution du biogaz produit de 2011 à 2050. Dans le deuxième scénario, le remplacement de l'étang de stabilisation par une station d'épuration d'eaux usées pour réduire la quantité de bassin nécessaire réduisant ainsi la surface requise pour le nouveau système. L'étang de stabilisation des déchets amélioré nécessite une surface totale de 39939.55 m^2 alors que pour la même quantité d'eau à traiter, une station d'épuration des eaux usées ne nécessite qu'une surface totale de 3690,45 m^2. La station d'épuration proposée n'est pas seulement avantageuse en termes d'espace, mais aussi du point de vue de l'économie circulaire qui peut être développée pour soutenir la station. La quantité totale de méthane produite ou émise par la station d'épuration des eaux usées devrait passer de 5,7 CH_4/jour en 2011 à 144,29 kg CH_4/jour en 2050.

Mots-clés : Eaux usées ; bassin de stabilisation des déchets ; méthane, station d'épuration.

ABSTRACT

Population growth affects land area availability as well as wastewater generation. In Africa, the treatment of wastewater relies mostly on the Waste Stabilization Pond system. These ponds are adapted to the warm climate of the continent. However, with time, they need to be renovated because they are subjected to overload causing therefore groundwater and water bodies pollution. WSPs also emit Biogas into the atmosphere. This study aims to upgrade a Waste Stabilization Pond constructed in 2011, on the University of Abomey-Calavi, in Benin and to evaluate the evolution of biogas producible over the years by the plant. To achieve this goal two scenarios were developed: The first one focuses on resizing the different ponds of the WSP with an estimation of the evolution of the biogas produced from 2011 to 2050. In the second scenario, a Sewage Treatment System is used instead of a Waste Stabilization Pond to reduce the amount of pond needed reducing the area required for the new system. The upgraded Waste Stabilization Pond requires a total surface area of 39939 m^2 meanwhile for the same quantity of wastewater, a Sewage Treatment Plant requires only a total surface area of 3690.45 m^2. The Sewage Treatment Plant suggested is not only advantageous in terms of space but also in the circular economy that can be developed to sustain the plant. The total amount methane produced or emitted from the WSP is estimated to increase from 5.7 Kg CH_4/day in 2011 to 144.29 kg CH_4/ day in 2050.

Keywords: Wastewater; waste stabilization pond; methane, Sewage Treatment Plant.

ACRONYMS AND ABBREVIATIONS

BMBF	Bundesminiterium für Bildung und Forschung
WASCAL	West African Science Service Center on Climate Change and Adapted Land Use
WHO	World Health Organization
CO_2	Carbon Dioxide
Kg	Kilograms
Kg CH_4/day	Kilograms of methane per day
BOD	Biochemical Oxygen Demand
COD	Chemical Oxygen Demand
TOC	Total organic carbon
pH	Power of Hydrogen
HPO_4	Hydrogen phosphate
N	Nitrogen
P	Phosphor
K	Potassium
mg/L	Milligrams per Litters
TSS/L	Total Soluble Solid per Litters
DBP	Duckweed Based Pond
ABP	Algae Based Pond
WWT	Wastewater Treatment
WSP	Waste Stabilization Pond
FC	Fecal Coliform

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Introduction

Wastewater management is part of the indicators of the sixth sustainable development goal of the 2030 agenda which aims to afford clean water and sanitation for all. The treatment of wastewater ensures availability of water to supply the increasing demand linked to the rapid population growth. It also prevents health issues related to water quality. In Sub-Saharan, only twenty percent (20%) of the wastewater generated is safely treated. Meanwhile Africa is the second driest continent after Australia and has only nine percent (9%) of water resources to support fifteen percent (15%) of the population [1], [2]. There is therefore a need for improvement not only in the quantity of wastewater treated but also in the technologies used. The poor status of energy sectors leads to the choice of archaic technologies such as activated sludge and waste stabilization pond which are however adapted to the climate of the continent [2], [3].

In Benin, as in many other African countries, the non-treated wastewater is gradually polluting the groundwater, affecting so the quality of drinking water and causing water borne diseases as result of contamination of water bodies. To solve the issue of releasing polluting water into a water body called lake Nokoué, a wastewater treatment system is built to treat the wastewater generated from the hostel of the national campus, university of Abomey-Calavi. The system is a wastewater stabilization pond which comprises an anaerobic pond, a facultative pond, a series of two maturation ponds leading to a fishing pond before a release for irrigation[4]

Problem statement

The existing Waste Stabilization Pond was built in 2011 for about 5000 students living on the campus Hostels. Regarding the evolution of students on the campus, the actual system is no longer adapted to the number of students. The system is only limited to the hostels, which means the generated wastewater from the rest of the campus is not subjected to proper management. A rehabilitation of the WSP and its extension to the whole campus is hence needed. Additionally, the treatment of wastewater contributes 8 to 11% of global methane emission, a fact which needs to be taken into consideration in the development of water sanitation. The release of methane from wastewater treatment plant makes them less ecofriendly in contrast to their initial essence and necessitate a mitigation [5]. In the first pond of the stabilization pond, the organic matter present in the generated wastewater from the campus is digested under anaerobic conditions. This anaerobic digestion occurring in an open pond releases methane and carbon dioxide, that can be a source of energy.

Research questions

- How does the growth in population on the campus affect the size of the constructed WSP and How much methane/biogas is released over the years and what can it be used for?
- Can the use of a conventional treatment of wastewater, especially a sewage treatment system be more profitable in terms of space?

Research hypothesis

The space required for the wastewater treatment can be less by adopting a Sewage treatment instead of a WSP and the methane released is a potential biogas that can be captured and used for cooking.

Research objectives

- Proposition of a new and resized waste stabilization pond.
- Estimation of the amount of Methane released from the stabilization pond existing.
- Proposition and sizing of a sewage treatment plant for the campus.

Structure of the thesis:

The first Chapter of the thesis presents the state of knowledge about the definition and properties of wastewaters, an overview of wastewater treatment technologies in different parts of the World and an operational description of Waste Stabilization Ponds. Following this chapter, the second chapter is exposing the methodology which includes a presentation of the study area and the different scenarios developed. The third chapter presents the results and discussion.

Chapter 1: Literature review

1.1. Wastewater definition

In all aspects of life (households, in industries, restaurants, hospitals, etc.) water is being used for different purposes. After usage, the water is evacuated through pipes into nature or treated for reuse. In this line, wastewaters can be defined as any used liquids, solutions, colloids, and suspensions as well as waste solids discharged via pipelines into natural receivers (e.g., natural water reservoirs, watercourses)[6]. There are different categories of wastewater according to the source of usage and described below. Stormwaters are waters from rainfall, snowfall and hail which is usually dirtied by cleaning the road or places where they fall.

Agricultural wastewaters: agricultural wastewaters originate from farming activities such as[6]:

- Animal husbandry (this type predominantly contains animal excreta, synthetic hormones, antibiotics, and parasites).
- Plant cultivation (application of artificial fertilizers and/or various chemicals to protect plants from diseases).

Hospital wastewater: One of the most pollutant and contaminated types of wastewaters are hospital wastewater coming from laboratories, sick rooms, operation theaters, laundries, kitchens of hospitals. The concentration of microcontaminants (e.g., heavy metals, antibiotics, and painkillers) in hospital wastewaters can be about one hundred and fifty (150 times higher than that found in municipal wastewaters [7].

Domestic wastewater: domestic wastewater is water that has been used in households for personal hygiene, flush restrooms, prepare food, and retains everything that was added to the water while it was being utilized. Thus, together with the water, it is made up of human wastes (feces and urine). Domestic wastewater has around 60% organic pollutants and 40% inorganic impurities. There are main categories of domestic wastewater [6]. Gray water, which comprises domestic wastewater produced without feces contamination and with low levels of bacterial pollutants, and black water, which contains toilet outflow that is significantly infected with pathogens, can be distinguished from one another [6], [8].

Industrial wastewaters: industrial sewage is generated by commercial, industrial, or service operations and contains a variety of chemical compounds that are byproducts of technological processes utilized in industrial buildings[6].

Municipal wastewaters: municipal wastewater is wastewater generated within the limits of a city (urban and peri-urban), mainly municipal sewerage, non-sewerage wastewater from treatment systems, and combined stormwater and sewer systems. In other words, it is a combination of stormwater and domestic wastewater [9].

1.2.Wastewater properties

The characteristics of wastewater can be classified as physical, chemical, and biological.

Physical properties: The physical properties namely color, odor, temperature, or turbidity are determined using the physical methods of analysis. The color reflects how recently the wastewater was treated. A fresh sewage sample does not smell bad, but when its constituent parts begin to rot, an unpleasant smell is created. The biological activity of bacteria found in wastewater is significantly influenced by temperature, which is also known to have an impact on gas solubility and liquid viscosity. The amount of solid matter that is present in the suspended form has a major impact on the turbidity. Water clarity is decreased and takes on an opaque, foggy, or muddy look due to suspended particles and dissolved colored material. A turbidity rod or turbidimeter can be used to measure turbidity.[6]

Chemical properties: The chemical composition of wastewater helps in determining the stage of sludge decomposition, expressed as the strength or extent of degradation [10]. An important chemical parameter is dissolved oxygen, which refers to the amount of oxygen present in the dissolved state in the wastewater. Generally, wastewater does not possess dissolved oxygen, and its presence in untreated wastewater indicates that the sewage is fresh. On the other hand, its presence in treated sewage effluent indicates that considerable oxidation has been accomplished during the treatment stages. The presence of dissolved oxygen in sewage is desirable since it prevents the development of obnoxious odor. Moreover, evaluation of the amount of dissolved oxygen helps in determining the efficiency of biological treatment. Biochemical oxygen demand

(BOD) is widely used to test and control stream pollution activities. It is defined as the amount of oxygen required by the microorganisms in wastewater to carry out complete decomposition of organic matter under aerobic conditions[6] [11] .

Another test is chemical oxygen demand (COD), which is widely used to measure the amount of organic matter present in the waste. It represents the quantity of oxygen required to chemically stabilize the organic matter [12]. In contrast to BOD, COD can be used to measure biodegradable and nonbiodegradable organic matter and the test takes less time. Total organic carbon (TOC) analysis involves acidification of the sewage sample to convert inorganic carbon to carbon dioxide which is then stripped. The TOC test is accurate and correlates well with the BOD results, but it is not widely used due to the high cost of the equipment.

Determination of pH value is also an important parameter in defining chemical properties of wastewater. Fresh sewage is alkaline, and with the passage of time the pH tends to decrease due to the production of acid because of bacterial activity in anaerobic conditions or nitrification processes. With treatment of sewage, gradual degradation of the compounds occurs, and consequently the pH tends to rise. The occurrence of nitrogenous compounds in wastewater indicates the presence of organic matter. Nitrogen is an essential nutrient for microorganisms that are involved in wastewater treatment. Nitrogen compounds may occur in different forms, such as nitrites, nitrates, organic nitrogen, or free ammonia. Total Kjeldahl nitrogen is a method for the quantitative determination of any nitrogen compounds contained. It can be defined as the total concentration of organic nitrogen and ammonia [12].

The determination of the prevailing form of nitrogen compound in the sewage can provide information about the stage of decomposition. The free ammonia indicates the very first stage of decomposition of organic matter, nitrites indicate the intermediate stage in the oxidation of ammonia, and nitrates represent the final product in the oxidation of ammonia. Nitrates and nitrites are practically absent in raw sewage. Total phosphorus in sewage is present in the form of phosphates. In domestic sewage, the prevailing form is HPO4 22, which is responsible for pH changes. Phosphorus compounds act as nutrients for the growth of microorganisms and algae, eventually leading, under specific conditions, to the eutrophication of lakes or reservoirs. Sulfates and sulfides are formed during the decomposition of various sulfur containing compounds in wastewater. This decomposition leads to the evolution of hydrogen sulfide gas, causing corrosion of concrete sewer pipes and releasing very obnoxious odors [10], [13]. Heavy metals, due to their nonbiodegradable nature, are one of the most persistent pollutants in wastewater. The most common toxic heavy metals in wastewater are lead, mercury, arsenic, cadmium, chromium, nickel, copper, and zinc. The release of high amounts of heavy metals into water bodies causes serious environmental and health problems and may lead to an upsurge in wastewater treatment cost. Heavy metals have a detrimental effect on the growth of aquatic organisms and can cause severe upsets in biological wastewater treatment plants [14].

Biological properties: The biological characteristics of wastewater are mainly governed by the microorganisms present, which predominantly include fungi, bacteria, archaea, algae, protozoa, viruses, etc. However, bacteria are found to be the most common contaminants [15][11]. They play an important role in the oxidation and decomposition of sewage. Most of the microorganisms found in wastewater are harmless and nonpathogenic; nevertheless, some of them are disease-producing pathogens that can negatively affect public health. The detection of pathogenic organisms in a sample of wastewater is difficult, as they exist in low concentration consequence, large volumes of samples must be tested. In this sense, the final concentration of pathogenic organisms per unit volume in a water body may be considerably low, making detection through laboratory examination difficult. However, this obstacle is overcome through the search for indicator organisms of contamination. These organisms are predominantly non-pathogenic but give a satisfactory indication of whether the wastewater is contaminated by human or animal feces, and thus its potential to transmit dangerous diseases. The organisms most used as indicators are bacteria of the coliform group, which are present in large quantities in human feces. Moreover, coliforms present slightly higher resistance in water in comparison to enteric pathogenic bacteria[11].

1.3. Wastewater treatment

According to Cooper (2001), modern awareness of the necessity for sanitation and treatment of polluted waters first emerged from the widely cited example of John Snow in 1855. He established that a cholera outbreak in London was brought on by sewage-contaminated water drawn from the Thames River [16]. Numerous physical (suspended and organic materials), chemical (nutrients, salts, heavy metals, and organic compounds, including persistent and emerging ones), and biological (pathogens of varied sizes and pathogenicity) components are found in wastewaters, and need to be removed before discharging the wastewaters in water bodies[9].

Treatment and discharge procedures in developed countries can vary significantly between regions, between rural and urban users, and between urban high- and urban low-income consumers [17]. Centralized aerobic wastewater treatment facilities and lagoons for both domestic and industrial wastewater are the most utilized wastewater treatment techniques in developed nations. In most developing nations, the levels of wastewater treatment fluctuate. Domestic wastewater can be processed in closed or open sewers, pit latrines, centralized facilities, septic systems, unmanaged lagoons, or waterways [18]. While major industrial facilities may have thorough implant treatment, industrial wastewater is occasionally discharged directly into water bodies [17].

The majority of domestic and industrial wastewater is released untreated or merely after initial treatment in many developing nations. About fifteen percent (15%) of the wastewater that is collected in Latin America is treated at various levels at treatment facilities. Ninety seven percent (97%) of Venezuela's sewage is released untreated into the environment, according to a technical report by the Caribbean Environment Program. Even in a highly industrialized nation like China, sewage is discharged without treatment in roughly fifty percent (55%) of cases[16]. Sewage generated by Tehran's population is completely untreated and poured into the city's groundwater, which is unusual for a Middle Eastern nation of its level of development[19][16]. Most of the municipalities' sewage treatment works in South Africa, when some amount of wastewater treatment is observed, are in a poor operational state and are not properly maintained, which contributes to the polluting of groundwater[20]. Consequently, the dependents of various water bodies face very substantial health and socioeconomic risks. Wastewater treatment is lacking in the majority of sub-Saharan Africa [1].

According to a review on wastewater treatment practices in Africa, activated sludge and stabilization ponds are the most used technologies on the continent with a representation up to 68-100% [21] Situation which justifies the technology used on the university of Abomey-Calavi. In developing countries nations and regions with warm climates, stabilization ponds are generally advised because of the following reasons: a favorable climate (high temperature and sunlight), easy operation, and little to no equipment are all requirements. There is also enough land available in a variety of locales [21]

1.4.Stabilization ponds process treatment of wastewater

Systems for waste stabilization ponds are constructed to achieve different forms of treatment in up to three steps in succession, depending on the organic strength of the input waste and the desired effluent quality. For flexibility and ease of maintenance, any design must have at least two trains of ponds functioning simultaneously. Strong wastewaters with BOD_5 contents above 300 mg/l are frequently treated in first-stage anaerobic ponds because of their high volumetric removal rate. When anaerobic ponds provide an unacceptable risk to the ecosystem, weaker or even stronger wastes (up to 1000 mg/l BOD_5) may be dumped straight into main facultative ponds. The overflow from first-stage anaerobic ponds is sent to secondary facultative ponds, the second stage of biological treatment [22].

Anaerobic pond

Deep treatment ponds known as anaerobic ponds are devoid of oxygen and promote the development of bacteria that decompose wastewater. The sewage starts anaerobically decomposing in the anaerobic pond since there is no oxygen present. Anaerobic pond are usually used for wastewaters with high BOD such as domestic wastewaters and waters from slaughterhouses, piggery wastes, dairies, and beverage industries. Like an open septic tank, the anaerobic pond functions. The anaerobic conversion occurs in two steps, to put it simply: Acidification and methane generation are both caused by acid-forming bacteria, also known as acidogenic bacteria, and methane-producing archaea, respectively. During the first phase, the organic matter is converted to simpler molecules and acids. The following stage is characterized by BOD removal by converting the organic matter mainly into methane and carbon oxide [23].

The load of a considerable concentration of BOD per unit of volume of the pond creates a condition in which the oxygen consumption rate is several times greater than the oxygen produced[21]. Hence in the oxygen balance, the production by photosynthesis and atmospheric reaction are negligible. Methane and carbon dioxide are released as a result of anaerobic bacteria breaking down the organic stuff in the wastewater. They have a relatively short retention time (for BOD of up to 300 mg/l, one day is sufficient at temperature > 20ºC) and function incredibly well in warm climates (may obtain 60-85% BOD elimination). By sludge formation and ammonia emission into the atmosphere, anaerobic ponds diminish nitrogen N and harmful bacteria.

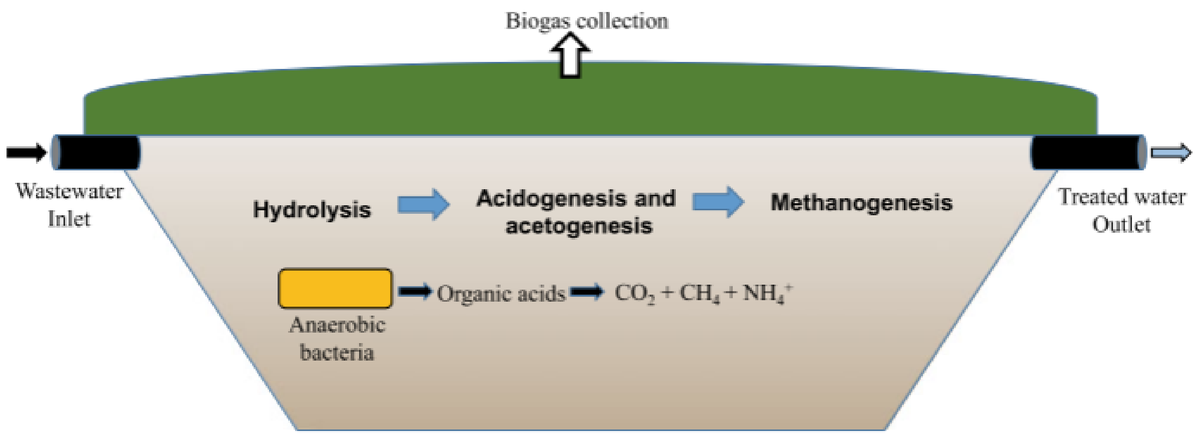

Figure 1:anaerobic pond schematic[7]

The anaerobic pond functions as a whole to:

- Distinguish between solid and dissolved material, as materials settle as bottom sludge.
- Allow dissolution of more organic matter present in the wastewater.
- Degrade biodegradable organic substance.
- Keep solids that are not biodegradable and undigested in bottom sludge.
- Permit the partially treated effluent to exit.

Characteristics: the sytem was designed for a population around 5000 people. The volume of the anaerobic pond calculated was 23,10 m^3.

Facultative pond

There are two different types of facultative ponds (1-2 m deep): primary facultative ponds that receive raw wastewater and secondary facultative ponds that receive settled wastewater (often the effluent from anaerobic ponds). In this case, the facultative pond is a secondary one. Since the majority of the oxygen needed by the bacteria to remove BOD is produced by algal photosynthesis, these systems are built to remove BOD at relatively modest surface loadings (100–400 kg BOD/ha/d at temperatures between 20°C and 25°C). This allows for the growth of a robust algal population.

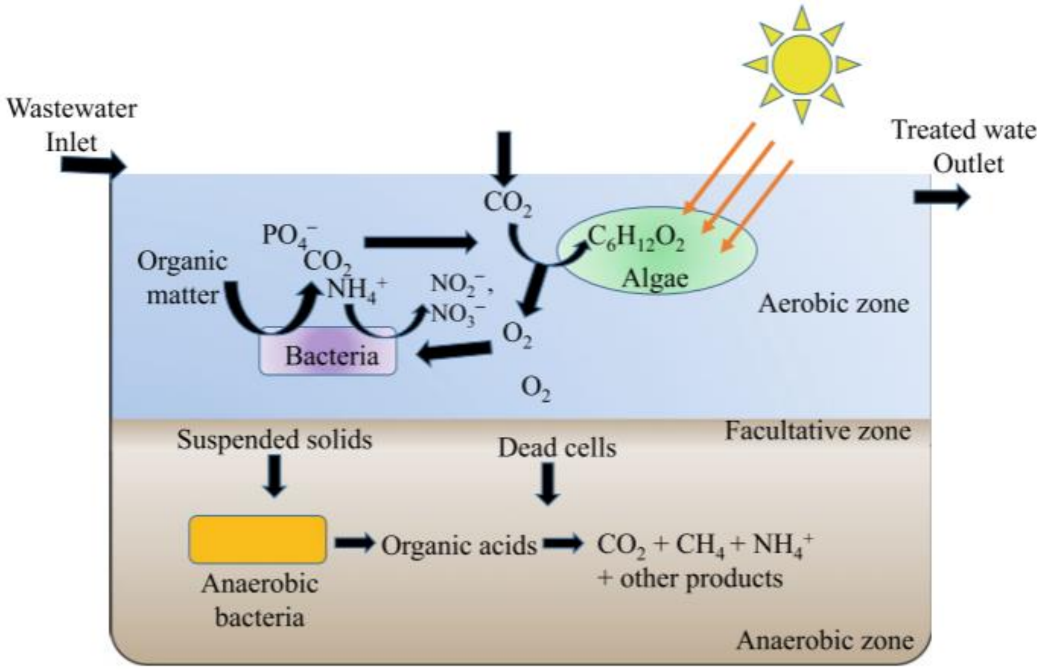

Figure 2:Operation of a facultative pond [7]

In the presence of oxygen, the effluent from the anaerobic pond (secondary facultative pond) is transformed into carbon dioxide, water, and new bacterial and algal cells.

The facultative pond will remove the odor and eliminate the majority of pathogenic bacteria. The facultative pond performs the following functions as a whole:

- Anaerobic separation, dissolution, and digesting of organic material.
- Breaking down of most residual organic materials by air near the pond surface.
- Reduction of the number of pathogenic bacteria.
- Enable the air to absorb 20% to 30% of the ammonia that is present in the effluent.
- Store non-degradable materials and digestion byproducts in bottom sludge.
- Permit the discharge of treated effluent into a river or another treatment system (such as another pond, a wetland system, or for land application).

Maturation Ponds

Maturation ponds are typically made for tertiary treatment, which involves the elimination of pathogens, nutrients, and perhaps algae. In order to allow light to reach the bottom and maintain aerobic conditions throughout the entire depth, they are often quite shallow (normally around 1 m depth, though Mara (1997) argues that at this decreased depth emergent plant growth and mosquito breeding problems can develop). The ponds, also known as facultative ponds, come after a subsequent treatment. The required retention period to obtain a specific effluent pathogen concentration determines the size and quantity of maturation ponds needed in series. Maturation ponds serve as a safety net for facultative pond failure in the absence of pathogen effluent limitations and are helpful for [22].

Algae Based Pond: the potential of algae-based waste stabilization pond systems to effectively remove pathogens and organic pollutants is well established, and there is a persistent interest in these systems on a global scale. The effluent, however, may occasionally contain high algal concentrations of up to 100 mg TSS/L [24], which can seriously clog up sophisticated irrigation systems. These kinds of environmentally friendly wastewater treatment methods, which are within developing nations' financial and technological reach, require further development.

Duckweed Based Pond : to boost nutrient recovery in a so-called duckweed-based pond, an aquatic plant (duckweed) could be introduced to algae-based waste stabilization ponds. Duckweed-based pond (DBP) systems are inexpensive and do not require specialized machinery, a lot of electricity, or skilled labor. In contrast to algae-based ponds (ABP), DBP systems have the potential to produce biomass, which has been shown to be a superior source of feed for producing fish or poultry and produces high-quality effluent for irrigation. The application of stabilization pond systems will be enhanced by better effluent quality and nutrient recovery, which may provide significant economic benefits for many developing nations. Diverse studies have demonstrated the efficacy of duckweed systems in treating wastewater [22].

Fishing Pond

Fish are raised in ponds that can be distinct from or a part of maturation ponds. On occasion, fish are also raised in the primary oxidation pond's end compartment. In the presence of sunlight, the phytoplankton grows quickly on the surface of the pond while absorbing the oxidation byproducts of the bacterium and algae symbiosis. The phytoplankton biomass produced in the aerobic ponds can be consumed directly by fish. In the fishpond that receives wastewater, several fish species, such as *Cyprinus carpio*, *Catla catlax*, *Cirrhina mrigala*, *Labeo rohita*, *Hypophthalmichthys molitrix*, and *Aristichthys nobilis*, including the *genus Tilapia*, are cultured [4], [7].

1.5. Conventional Wastewater treatment process and technologies

1.5.1. Preliminary Treatment:

Wastewater screening is a crucial part of the primary water treatment. It is used to clean wastewater from larger solids like rags, sticks, plastic, etc., before subjecting it to other treatment processes. The purpose of screening in wastewater treatment is to:

- protect the equipment downstream from clogging and damage.
- remove large debris that can adversely affect the efficiency of treatment processes.
- shield final discharge waterway from contamination.
- Grit is also removed.

Large object screening

Wastewater screening is a crucial part of the primary water treatment. It is used to clean wastewater from larger solids like rags, sticks, plastic, etc., [25] before subjecting it to other treatment processes. The purpose of screening in wastewater treatment is to:

- protect the equipment downstream from clogging and damage.
- remove large debris that can adversely affect the efficiency of treatment processes.
- shield final discharge waterway from contamination.

According to the size of the apertures in the screening element and the removal method, screens are often divided into three categories.

- Coarse screens
- Fine screens
- Microscreens

Coarse screens: as shown in Figure 3, clear openings on coarse screens range from 6 to 150 mm (0.25 to 6 inch). Parallel bars, rods, or wires, wire mesh, or perforated plates with apertures that are often round or rectangular make up coarse screens. As a result, it is also known as "bar rack" and is used to remove coarse solids like rags and other objects that could clog or harm other accessories.

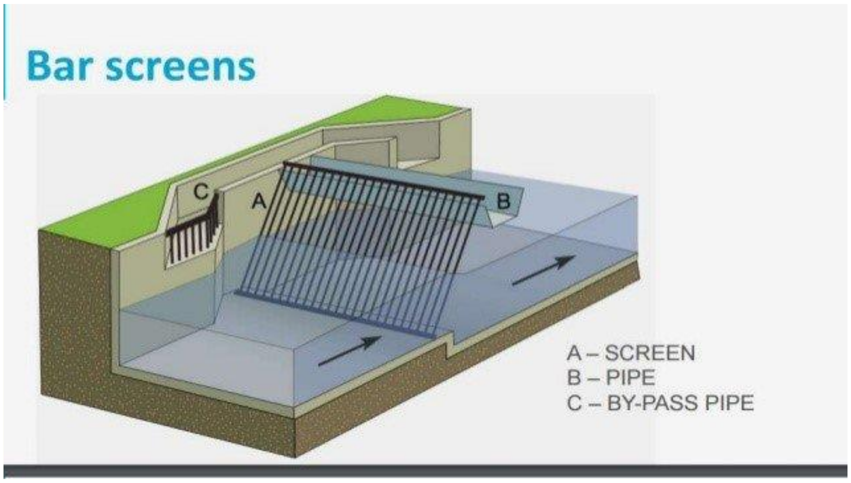

Figure 3:Bar screens (Slideshare)[25].

Mechanically cleaned screens.

The two categories of coarse screens are based on the wastewater screening process that was used to clean them:

- Hand cleaned screens
- Mechanically cleaned screens (Figure 4).

Figure 4: Mechanically cleaned screens (Mojan engineering)[25].

Its main goals are to lessen operational issues, improve screening effectiveness, and lower maintenance requirements. machine-cleaned bar. There are four main categories for wastewater screens.

Chain-driven displays: A chain was automatically employed to wipe the screen in these types of screens. According to whether the rakes return to the bottom of the bar screen from the front or back and how the screen is raked from upstream or downstream, it is divided into front and back chain driven screens:

The reciprocating rake (also known as a climber screen) goes to the base of the screen, engages the bars, and pulls the screens up to the top of the screen where they are removed. As opposed to other types of screens, which employ many rakes, this screen just uses one. They are therefore only partially capable of managing huge screening loads.

Catenary screen: They possess a rake that is secured to a rack by the weight of a chain. They have a front return chain-driven screen that is front cleaned. The rakes won't further jam in heavy things that have already become wedged in between the bars; instead, they will move over them.

Continuous belt screen: This self-cleaning screen can remove both coarse and fine particles. The driving chains have a lot of rakes attached to them. The depth of the screen channel often affects how many screening components are needed.

Fine screens: Fine screens for wastewater screening have clear apertures that are less than 6 mm wide. They were made from wedge-shaped wire components with smaller apertures, wire cloth, and perforated plates as presented in Figure 5.

Figure 5: Perforated fine screen (water online)[25]

They are also employed to eliminate the primary effluent's fine particles. Fine screens can be divided into:

- Fixed or static wedge wire screen
- rotating-drum screen
- Screen with steps

Static wedges write screens: They are made for a flow rate of 400 to 1200 L/m^2 min of screen area and have a clear opening of 0.2 to 1.2 mm. These screens should be installed on a large floor space, and they should be cleaned once or twice a day [25].

Drum screens: Figure 6 shows an example of drum screen. This type uses a cylinder that rotates in the flow channel to hold the screening or straining medium. The wastewater enters the drum from either end and exits through the screen outlet, with the solids being collected inside the drum or on top of the unit [25].

Figure 6: Rotating drum fine screens (Allegri Ecologia)[25].

Step screens: It is made up of two step-shaped sets of thin, vertical plates, one of which is fixed and the other mobile. As they alternate over the channel's width, the fixed and moving step plates combine to produce a single screen face. The adjustable plate rotates vertically. With the help of this, the solids that have accumulated on the screen face are raised up to the next predetermined step landing and moved to the top of the screen where they are discharged into an outlet [25].

Micro screens: They are gravity-flowing rotating drum screens that operate at a low speed that is continuously backwashed (up to 4 r/min). The filtering materials should be put around the outside of the drum and have apertures that range from 10 to 35 mm. The influent enters through a drum with a fabric covering. Through backwashing, the solids retained are collected and transported for disposal [25].

Grit removal

Sand, gravel, cinder, and other heavy particles that have a greater specific gravity than the wastewater's organic biodegradable solids are referred to as grit. Grit also comprises large organic particles like food waste as well as eggshells, bone chips, seeds, and coffee grounds. Grit accumulation in anaerobic digesters and aeration basins, grit deposition in pipelines and channels, and unnecessary abrasion and wear of mechanical equipment are all prevented by the removal of grit. Grit removal facilities normally come after screening and comminution and before primary clarifying. This keeps big solids from getting in the way of grit handling machinery. Grit removal should come first in secondary treatment facilities lacking initial clarity [26]. Aerated grit chambers, vortex-type grit removal systems, short-term sedimentation basins, horizontal flow grit chambers, and hydro cyclones are just a few of the various types of grit removal systems that exist [25]. When choosing a grit removal method, several aspects must be considered, including the amount and nature of the grit, potential negative impacts on downstream operations, head loss requirements, space requirements, removal efficiency, organic content, and cost. The kind of grit removal system used for a particular facility should be the one that strikes the optimal balance between these various factors [25].

Aerated Grit Chamber: In aerated grit chambers presented on Figure 7, the removal of grit is accomplished by spiraling the wastewater flow. By adding air to the grit chamber along one side, a perpendicular spiral velocity pattern is created, which causes the tank to flow. While lighter organic particles are suspended and eventually carried out of the tank, heavier particles are propelled and diverge from the streamlines, falling to the tank's bottom.

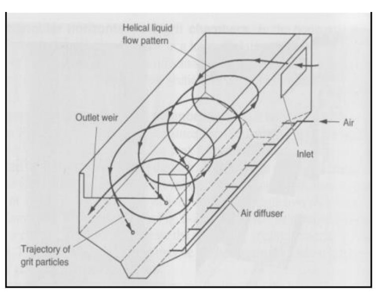

Figure 7: Aerated grit chamber [25], [27]

- Aerated grit chambers benefits are presented below [25]:

• Consistent removal efficiency over a broad flow range.

• A well-controlled rate of aeration can remove a relatively low putrescible organic content.

• Pre-aeration can be used to improve the performance of downstream units by lowering septic conditions in entering wastewater.
• Chemical addition, mixing, pre aeration, and flocculation may all be done in aerated grit chambers, making them flexible.

- Disadvantage

The aerated grit chamber may emit volatile organic compounds and scents that could be dangerous. In addition, aerated grit chambers use more energy than other grit removal techniques, and the aeration system needs to be maintained and controlled.

Vortex-Type Grit Chamber: In the cylindrical tank which sets up the vortex-type grit chamber, the flow enters tangentially to produce a vortex flow pattern. While effluent exits the tank at the top, grit settles by gravity into the bottom of the tank (in a grit hopper). A grit pump or an air lift pump can be used to remove the grit that collects in the grit hopper.

- Advantages

• Up to 73 percent of grit with a diameter of 140-mesh (0.11 mm/0.004 in) is removed by these systems.
There are no submerged bearings or parts that need to be maintained, and vortex grit removal systems provide consistent removal efficiency throughout a wide flow range.

- Disadvantages

• A vortex grit removal system has a smaller "footprint" (horizontal dimension) than other grit removal systems, which is useful when there is a shortage of available space.
• The average head loss through a vortex system is only 6 mm (0.25 in). Additionally, these systems use less energy.
• The unique nature of most vortex grit removal devices adjusts challenging.
• Rags frequently end up in paddles.
• Due to their depth, vortex units typically require extensive excavation, which raises the expense of construction, particularly if there is unrapable rock nearby.
• The grit sump frequently clogs, necessitating high-pressure agitation with air or water to break up the compacted grit.
Detritus Tank: A detritus tank, also known as a square tank degritter, is a short detention settling

tank with a constant level. To remove organic material from these tanks, a grit-washing process is necessary. Grit is removed and sorted from the grit sump using a rake and auger in one design option.

- Advantage

Since all bearings and mechanically moving components are above the water line, detritus tanks don't need flow control. This kind of unit experiences very little head loss.

- Disadvantages

• Because the entrance baffles cannot be modified, detritus tanks have a difficult time maintaining equal flow distribution over a wide range of flows.

• Because this kind of removal system removes a lot of organic material, especially at low flows, grit washing and classifying are necessary.

• The rake arm attached to this system's shallow installations (less than 0.9 m [3 ft]) may lose grit due to the agitation it causes.

Horizontal Flow Grit Chamber: The grit removal mechanism with horizontal flow is the most traditional. By keeping the upstream velocity constant at 0.3 m/s (1 ft/s), grit is eliminated. By using rectangular control sections or proportional weirs such as Parshall flumes, velocity can be controlled. In this arrangement, the lighter organic particles are carried out of the channel while the heavier grit particles settle to the bottom of the channel. A conveyor that has scrapers, buckets, or plows removes the grit. The grit is elevated for washing or disposal using screw conveyors or bucket elevators. Grit chambers are frequently cleaned manually in smaller facilities.

- Advantage

Horizontal flow grit chambers are adaptable because the output flow control mechanism can be adjusted to change performance. The process of construction is simple. Grit that doesn't need to be further classified, can be eliminated with efficient flow control.

- Disadvantages

• In a variety of flows, it is challenging to sustain a 0.3 m/s (1 ft/s) velocity.

• Excessive wear is experienced by the submerged chain, the flight apparatus, and the bearings.

• Excessive volumes of organic material will be removed from channels without efficient flow control, necessitating grit cleaning and sorting.

• Excessive head loss occurs (usually 30 to 40% of flow depth).

• When proportionate weirs are used, high velocities may be produced at the channel bottom, causing bottom scour.

Hydrocyclone: To separate grit from organics in grit slurries or to remove grit from primary sludge, hydrocyclone devices are frequently used. Sometimes, grit and suspended sediments are removed from wastewater flow directly using hydrocyclones by pumping at a head ranging from 3.7 to 9 m (12 to 30 feet). Due to the centrifugal forces created, heavier grit and suspended materials collect on the sides and bottom of the cyclone, while scum and lighter solids are evacuated from the center through the cyclone's top.

- Advantage

Grit and suspended solids from wastewater can both be removed using hydrocyclones. Potentially, a primary clarifier and a hydrocyclone can both remove the same number of solids.

- Disadvantage

Hydrocyclones need energy because a pump is used to remove grit and suspended materials. To get rid of sticks, rags, and plastics, coarse screening is necessary before these units.

1.5.2. Primary treatment or Sludge removal

After grit removal, the sewage is directed to a first clarifier. The main goal of primary treatment is to remove inorganic and organic solids. These solids (which are not as heavy as sand or grit but heavy enough to sediment) are removed by sedimentation. Sludge is collected from the bottom of the tank and after subjecting to appropriate treatment can be used as a fertilizer. In this step, along with the simultaneous removal of suspended solids, impurities floating on the surface are also removed effectively by skimming [6].

Sedimentation occurs when there is a density difference between the dispersed phase particles and the dissipative phase particles. Thus, heterogeneous mixtures (suspensions) with a density higher than the density of liquids can be subjected to a sedimentation process. The sedimentation process depends on the concentration, size, shape, and weight of the sediment solid particles and the viscosity of the liquid. In a typical activated sludge plant, sedimentation is used in three of the treatment steps: in grit chambers (discrete settling), in which inorganic matter is removed; in the primary clarifier (Flocculent settling) and in the second clarifier (zone settling). Flocculent settling is defined as an agglomeration of the settling particles accompanied by changes in density and settling velocity [6], [27].

The effectiveness of the primary clarification is a matter of appropriate water flow. If the water flow is too fast, the solids don't have time to sink to the bottom resulting in negative impact on water quality downstream. If the water flow is too slow, it impacts the process up stream [6].

1.5.3. Biological process

Biological deterioration characterizes this phase. The primary objective of this stage is to hasten the decomposition of organic matter. Air pumps or forceful mixing are used to generate this acceleration. In these circumstances, ammonia first becomes nitrates, and then bacteria extract oxygen from nitrates to make nitrogen. Both anaerobic and aerobic conditions can be used for biological treatment [28]. For highly contaminated sewage, anaerobic conditions are necessary. Sewage treatment facilities employ aerobic wastewater treatment more frequently. According to and Zia et al. (2019), it involves aerobic microorganisms (protozoa and aerobic bacteria) decomposing organic materials (fat, proteins, and carbohydrates) [29]. These microorganisms oxidize the organic material in wastewater to produce water and carbon dioxide as a source of energy and nutrition for their survival [30].

The reactors in biological tanks can be stationary or mobile. According to Wiesmann et al. (2006) and Lewandowski and Boltz (2011) [29], [31]. The trickling filter (Lewandowski and Boltz, 2011; Sabba et al., 2018), which sprinkling the filter with previously mechanically cleaned wastewater, is the oldest technique that replicates the natural process of self-purification of water happening in nature [31], [32]. Whether the bed is constructed of rocks, slag, coke, or plastic materials, a biofilm—a thin, gelatinous layer made of bacteria, fungi, and even insect larvae—begins to form on the surface of the bed conditions. These treatment systems, which are typically built in sealed cylindrical tanks, remain one of the most often used treatment methods. Water continuously runs through the bed in trickling filters, yet the bed does not sag, allowing for continuous air exchange. Using a spraying mechanism, the entire surface of the bed is equally coated with sewage, and treated wastewater is collected beneath the bed. Wastewater is continuously in touch with ambient air supplied to the treatment plant, which aerates the wastewater.

The treated wastewater is gathered beneath the bed, and the spraying mechanism equally distributes sewage over the entire surface. By continuously encountering the ambient air supplied to the treatment plant, wastewater is continuously aerated. The SFBB reactor (Lewandowski and Boltz, 2011; Sabba et al., 2018; Schlegel and Koeser, 2007) is another fixed-bed type bioreactor. In SFBB, the reactors are most frequently made of plastic and the beds are permanently attached beneath the sewage's surface [31], [32] [33]. Under-the-bed diffusers deliver compressed oxygen, which is needed to boost the effectiveness of biological processes. A rotating biological contactor (RBC), a new sort of bioreactor unit, is distinguished by a completely different design.

Finally, wastewater treatment processes in MBBRs are carried out in a manner that is very similar to activated sludge technology, but they have some significant advantages, such as smaller volume, higher stability of removal process, easier steering and control, and easier settling and maintenance of bacterial population [6].

1.5.4. Secondary treatment

After aeration, wastewater flows to circular tanks called secondary clarifiers, where very small solids (frequently called activated sludge) will sink to the bottom. As activated sludge mainly consists of bacteria, it is often returned to step 4 to increase the bacterial concentration in the aeration tanks [6].

1.5.5. Sludge use

The production of enormous amounts of sewage sludge, which in addition to having a high organic content, may also contain harmful materials like heavy metals and persistent micropollutants, represents one of the biggest issues in wastewater management. Depending on the wastewater treatment method used, the weight of particles in sewage sludge ranges from 0.25 to 12% [34].

Since sludge burning is linked to high energy demands and prices and landfill disposal is subject to numerous legal restrictions, traditional handling methods for sludge, such as agricultural usage, may be risky.

Thermochemical treatment of sludge in the absence of oxygen or in oxygen-starved conditions is one alternative management approach that can be used to stop combustion. Sludge may undergo chemical processes that produce fuels that can be used to produce heat and/or energy while also removing organic load under strictly controlled circumstances and high temperatures (350–1000°C) [6]. Gasification, which yields syngas, and pyrolysis, which yields bio-oil, are two

examples of processes. These are viable substitutes for sludge incineration, however they both still have substantial operational expenses, especially when employing high temperatures.
Additionally, extra attention should be paid to monitoring operating conditions to prevent the creation of any hazardous byproducts, such hydrogen cyanide.

Chapter 2: Materials and methods

2.1. Data collection

The data used for this study was mainly collected through literature review, previous works focused on the wastewater treatment in West Africa considering the University of Abomey-Calavi in Benin.

2.2. Study area

Figure 8: Entrance of the Campus

The University is in Abomey-Calavi, a city in the south – Benin. Benin is a tropical country located in the West between 6°30 and 12°30 latitude and 1° and 3°40 longitude. The average annual temperature is 26.5 °C with an average annual sunshine of 2862 hours, an average height annual rainfall of 1300mm and an average evaporation of 7200 mm/day.[4] Majority of the wastewater produced in discharged in the lake Nokoué shown in Figure 9.

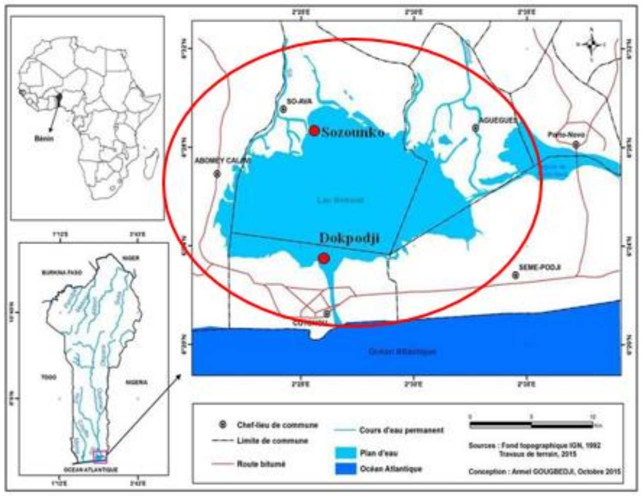

Figure 9: View of the closeness of City of Abomey to Nokoué lake

The following Figure 10 shows the Waste Pond Stabilization built in 2011 for the treatment of wastewater from the students' hostels. These hostels, are located 460 m from the station, is collected by a mini-sewer system gravity to the treatment system. The quality required for the effluent was fixed considering its reuse in fish farming and agriculture. The standard for the reuse of treated wastewater effluents in agriculture used the WHO guidance on wastewater reuse in agriculture as reference.

The proposed system includes an anaerobic pool, an optional pool and two series of maturation tanks, one algae-based treatment, the other macrophytes based treatment. The macrophytes used are lenses of water collected in the lowlands of Cotonou. The effluents of the series are converged on two (02) separate fishponds. The effluent of the basins at Fish is used for irrigation of vegetable crops [4].

Figure 10: View of the waste pond stabilization on campus

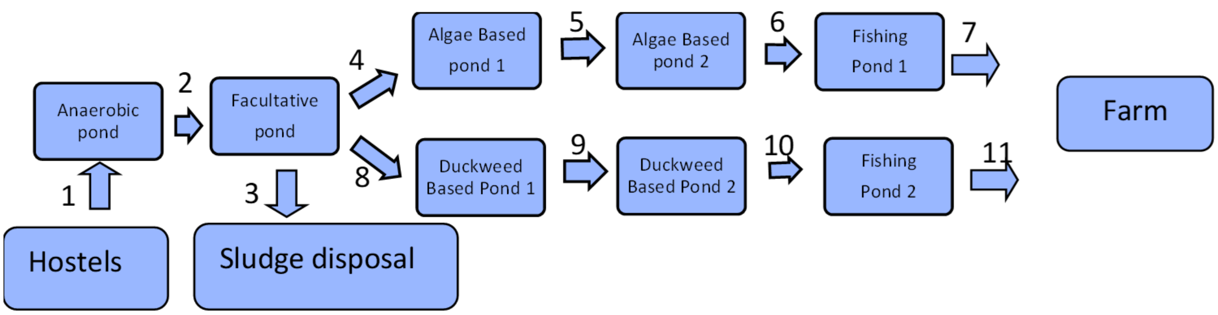

Figure 11: Flow diagram of the actual wastewater treatment system on the campus

1: Flow of wastewater from the campus hostel to the treatment plant

2: flow of wastewater from anaerobic pond to facultative pond

3: Sludge removal

4: Flow of wastewater from facultative pond to the first Algae based pond

5: Flow of wastewater from the first Algae based pond to the second one

6: Flow of the wastewater from the second Algae based pond to a fishing pond

7: Flow of the wastewater from a fishing pond to the farm

8: Flow of the wastewater from the facultative pond to first Duckweed based Pond

9: Flow of the wastewater from the first Duckweed based Pond to the second one

10: Flow of the wastewater from the second Duckweed based pond to the second fishing pond

11: Flow of the wastewater from the second fishing pond to the farm

The performance of the plant was evaluated through the monitoring of physico-chemical parameters such as pH, turbidity, total suspended solids (TSS), chemical oxygen demand (COD), biochemical oxygen demand at 20°C after fife (5) days incubation (BOD_5), nitrate, nitrite, and phosphate. From the laboratory analysis, BOD_5 the removal efficiencies observed are: 93% and 90% for COD, 97% and 93% respectively for the duckweed-based system and algae-based system; the percentage of removal of suspended solids is approximately 98% for the two systems.

The characterization of the effluent has shown a considerable rate of biodegradability (COD/BOD5 = 2.34) and therefore well suited to the fishing application proposed. The study of system performance showed that the Lens system is more efficient in the removal of materials organic and nutrients than the algae system; The system is more effective in eliminating pathogenic germs. The quality of effluents obtained at the outlet of both systems meets the WHO recommendation for reuse in agriculture [4].

2.3. Data processing and analysis

The different calculations and equations solving have been done using excel and the processing is structured into three scenarios.

2.3.1. Scenario 1 : Proposition of a resized Waste Stabilization Pond and Estimation of methane emissions.

In this scenario, the sizes of the different ponds are revised and adapted to the whole campus. To the existing plant is added a composting platform used mix the sludge with organic solid waste for making compost and supply the farm. To the sizing is added an estimation of the methane emitted from both the existing and the old pond.

2.3.1.1. *Sizing of the new Waste Stabilization Pond*

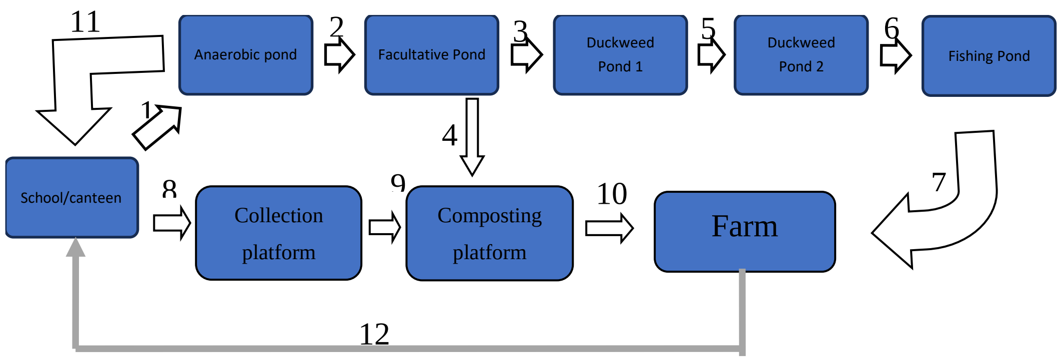

Figure 12: Proposed wastewater system treatment

1: wastewater flow from the campus hostels and restaurants to the anaerobic pond

2: wastewater flow and digestate from the anaerobic pond to the facultative pond

3: wastewater flow from the facultative pond to the first maturation pond (Duckweed Pond 1)

4: Sludge removal from the facultative pond and disposal in a composting plant

5: wastewater flow from the first duckweed pond to the second one

6: wastewater flow from the second duckweed pond to the fishing pond

7: wastewater flow from the fishing pond to the farm

8: organic solid wastes supply from the restaurants and canteen to a collection platform

9: organic solid wastes supply from the collection plant to the composting plant for mix with the sludge

10: compost supply to the farm

11: Biogas supply to the restaurants and canteen of the campus.

12: Provision of vegetables from the farm to restaurants and canteens

The calculations of the new sizes of each block are done considering a lifetime of 30 years using the campus population in 2020. The water consumption is assumed to be the same as the quantity consumed in Niger and Burkina Faso available in the literature and the generated wastewater is assumed to be 85% of the water consumed [25].

Important data

- Population

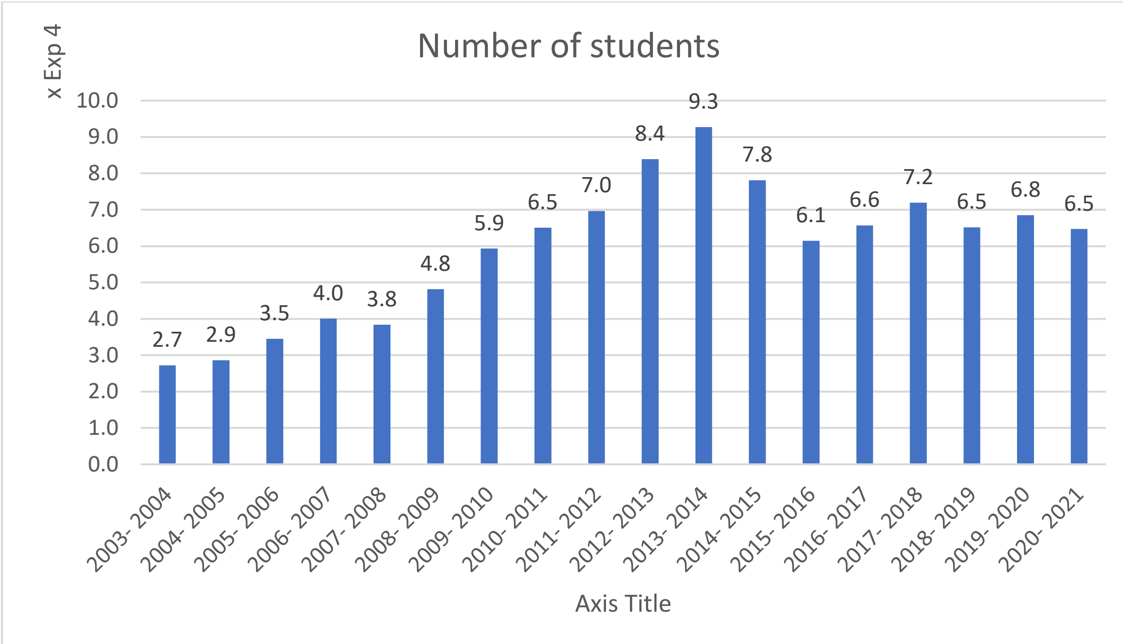

Figure 13:Evolution of the student's population on the campus over 18 academic years

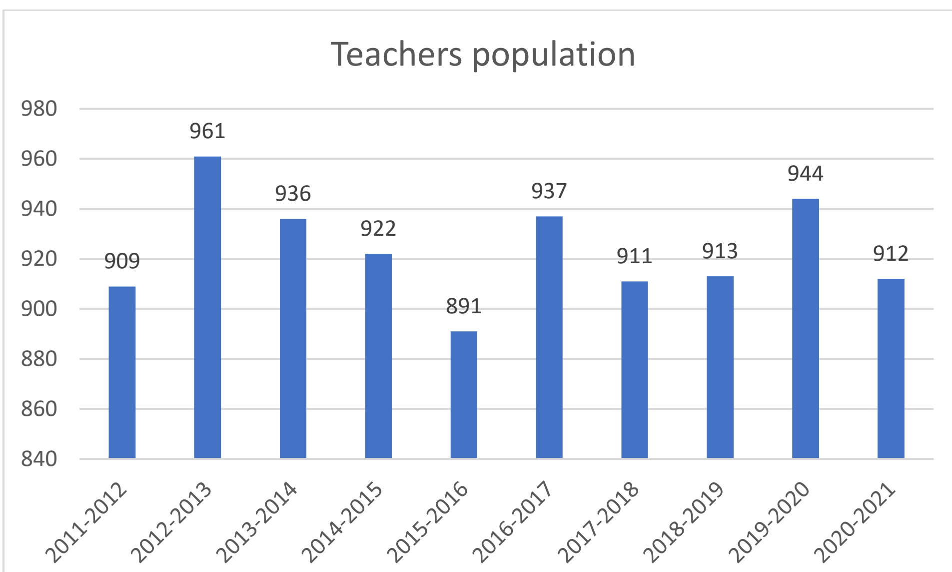

Figure 14:Teachers population over 10 Academic years

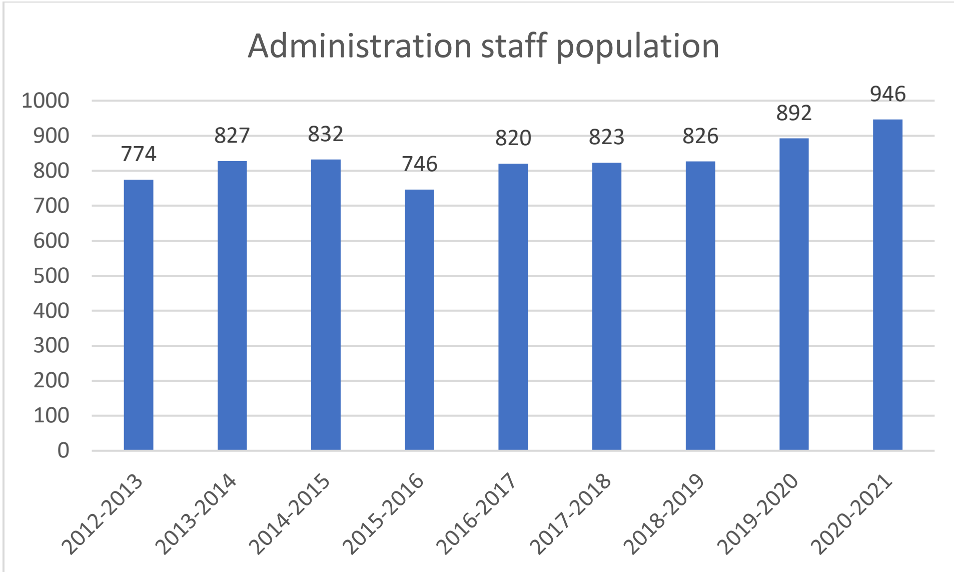

Figure 15: Population of the administration staff of the campus

From Figures 13,14 and 15, the maximum capacity of the campus can be estimated using the pic of each histogram presented. The number of students has reached a maximum over eighteen (18) years during the academic year 2013-2014 with a value of 92,703 students. The maximum of lecturer's population on the campus was reached over ten (10) years during the same academic year with a value of 961 Lecturers. The maximum of administration staff is 946 reached during 2020-2021. The estimation of the campus population in 2050 is based on the maximum capacity.

Population in 2050= 92,703 + 961+946 = 96,610 inhabitants

- Influent flow

Water consumption:

The water consumption is estimated to be 2.7 m^3/y / capita in Niger and Burkina Faso, with an extension to West Africa including Benin [26].

So, Water consumption per day = $(96{,}610\ x\ 2.7)/365$

= 714.65 m^3/d

Influent flow = 85% x Water consumption per day

= $0.85x\ 714.65$

Influent flow, Q = 607.45 m^3/d

Influent flow = 607451.92 L/d

- Influent BOD, S_o = 485 mg/L

2.3.1.1.1. Anaerobic pond sizing

a) Required volume.

The volume required is given by:

$$V = {}^{Lo}/_{Lv} \qquad (1)$$

Where:

V= volume required for the fond (m^3)

Lo= total (soluble + particulate) influent BOD_5 load ($kgBOD_5/d$)

Lv= volumetric loading rate ($kgBOD_5/m^3.d$)

- Volumetric organic loading rate: this parameter is a function of the temperature.

Table 1:Permissible volumetric loading rates for the design of anaerobic pond as a function of temperature [23]

Mean air temperature in the coldest month- T(°C)	Permissible volumetric loading rate Lv ($KgBOD/m^3.d$)
10 to 20	0.02T-0.10
20 to 25	0.01T+0.10
>25	0.35

The coldest month is Benin is July with a temperature of 29°C[27]. So, the permissible loading volumetric loading rate for the anaerobic pond is $Lv = 0.35\ \mathrm{KgBOD/m3.d}$.

Determination of the volume:

Influent BOD load Lo [kg BOD /day] = Influent BOD So [mg/L] x influent flow [L/day] x 10^{-6}

= 485 mg/L x 607451.92 L/d x 10^{-6}

Lo = 294.61 $KgBOD_5/d$

- Temperature T= 29°C
- Influent BOD load $Lv = 0.35\ \mathrm{KgBOD/m3.d}$

Required volume.

$$\text{Volume} = \frac{load}{volumetric\ load}$$

$$= \frac{294.61}{0.35}$$

$$= 841.75\ \text{m}^3$$

b) Detention time

The retention time is preferrable to be between 3 and 6 days according to [23] and is given by the following formula:

$$t = V/Q \qquad (2)$$

Where:

t = detention time (d)

V= volume of the pond (m^3)

Q= average influent flow (m^3/d) or influent flow (m^3/d)

- Verification of detention time:

$t = \frac{V}{Q} = \frac{841.75}{607.45} = 1.39$ d ≈ 2 days

The pond has a low detention time and is equal to the formal calculations. The inlet should be at the bottom in contact with the settled sludge.

c) Depth

Anaerobic ponds are deep to ensure the preponderance of anaerobic conditions and prevent the pond from functioning as a facultative pond. In fact, the better, the deeper the pond. Deep excavations, however, are typically more expensive.

H = 3.5 to 5.0 meters is the range of values that are typically used [23].

The anaerobic pond may have an additional depth of at least 0.5 m at the inlet and covering at least 25% of the pond's surface if there hasn't been any prior grit removal. Grit chamber units, however, are seen to be advantageous because they minimize issues with grit accumulation near to the entrance pipe and because of their simplicity.

e) Geometry (length/ breadth ratio)

The typical length/breadth (L/B) ratios of anaerobic ponds, which are square or slightly rectangular, are:

L/B = 1 to 3 is the length to breadth ratio[23].

- Required area dimensions.

Let's take the formal depth of the anaerobic pond, H= 2.80 m

$$Area = \frac{volume}{depth} \quad (3)$$

$$= \frac{841.75}{2.80}$$

Area = 300.63 m^2

f) Concentration of effluent

Estimation of effluent BOD removal BOD concentration from the anaerobic pond

As shown in the following table, the BOD removal efficiency is a function of temperature:

Table 2: BOD removal efficiency as a function of temperature[21]

Mean air temperature of the coldest month T (°C)	BOD removal efficiency E (%)
10 to 25	2T+20
>25	70

The corresponding temperature in this case being greater than 25°C, the BOD demand is estimated at 70%.

After estimating the removal effectiveness (E), this formula can be used to determine the anaerobic pond's effluent concentration (BODeffl)[23]:

$$BODeffl = (1 - E/100).So \quad (4)$$

Where:

So is the influent total BOD concentration (mg/L);

BOD_{effl} is the effluent total BOD concentration (mg/L)

- Temperature = 29°C> 25°C so E = 70%

- S_o= 485 mg/L

$BODeffl = (1 - 70/100).485\ mg/L$

= 145.5 mg/L

g) Sludge accumulation

The rate of accumulation ranges from 0.03 to 0.10 m3/inhab.year, with the lower range being more typical in regions with mild climates. 2 to 8 cm/year is another range of accumulation rates that can be seen. These yearly increases in sludge layer thickness are consistent with accumulation rates that are less than 0.03 m3/inhab.year [23].

Any one of the following cleaning methods should be used to clean the anaerobic ponds:

• when the liquid depth is approximately 1/3 of the sludge layer.

• yearly removal of a specific volume in a predetermined month to systematically include the cleaning stage into the pond's operational strategy.

The entire sludge mass should not be removed if the removal method does not involve emptying and drying inside the pond. This would result in a complete loss of the biomass and necessitate restarting the anaerobic pond.

h) Annual accumulation

Regarding the less amount of wastewater produced, the sludge accumulation can be considered less than 0.03 m^3/inhab.year. Let's consider a value of 0.001 m^3/inhab.year

Annual accumulation = 0.001 m^3/inhab.year x 293989.84 inhab

= 293.99 m^3/year

i) Thickness of the sludge layer

$$Thickness = \frac{Annual\ accumulation}{pond\ area} \qquad (5)$$

$$= \frac{293.99/_{year} x\ 1\ year}{300.63}$$

= 0.32 m/year

= 32 cm / year

j) Time to reach 1/3 of the pond depth.

$$Time = \frac{H/3}{yearly\ yhickness} \quad (6)$$

$$= \frac{2.8m/3}{0.32m/year}$$

$= 2.90$ years

≈ 3 years

2.3.1.1.2. *Facultative pond*

The BOD concentration of the effluent is estimated to be 30% of the initial influent for the 70% removed in the Anaerobic Pond. To estimate the BOD concentration of the effluent from the facultative pond, a coefficient K is applicable depending on the previous removal of the more easily degradable organic matter in the anaerobic pond[23].

For different temperatures, the value of K can be corrected using the following equation[23]:

$$K_T = K_{20}.\theta^{(T-20)} \quad (7)$$

where: K_T = BOD removal coefficient at a temperature T (d^{-1})

K_{20} = BOD removal coefficient at a temperature of 20◦C (d^{-1})

T = liquid temperature (◦C)

θ = temperature coefficient

It should be noted that different values of θ are proposed in literature. For K = 0.35 d^{-1}, mentioned by EPA (1983), the temperature coefficient is θ= 1.085. For K = 0.30 d^{-1}, mentioned by Silva and Mara (1979), the reported value is θ= 1.05

For the most frequent case of the design according to the complete-mix model, the following range of K values may be used for design [21]:

Table 3 : Coefficient K values

Pond	K value (20ºC)
Primary ponds (receiving raw wastewater)	0.30 to 0.40 d^{-1}
Secondary ponds (receiving effluent from a previous pond or reactor)	0.25 to o.32 d^{-1}

k) Influent load

$$L = \frac{(100-E).Lo}{100} \quad (8)$$

$$= \frac{(100-70)x294.61}{100}$$

= 88.38 kg/BOD/ d

l) Adoption of the surface loading rate

Benin can be referred to as a region with warm winter and high sunshine, so according to [23], Ls is between 240 and 350 kgBOD5/ha.d. let us adopt:

Ls= 350 kgBOD5/ha.d

m) Required Area

A= $\frac{L}{Ls} = \frac{88.38}{350}$= 0.25 ha= 2525.26 m^2

n) Adoption of value of the depth

H= 2 m considering the formal system

o) Calculation of the resulting volume

V= A.H = 2525.26 m^2 x 2 =5050.53 m^3

p) Calculation of retention time

t = $\frac{5050.53}{607.45}$ = 8.31 days

q) Adoption of a value for the BOD removal coefficient

$K_T = K_{20}.\theta^{(T-20)}$, with :

T= 29^{o}C;

K_{20} = 0.30

and θ= 1.05 based on the information in section 2.2

So, $K_T = 0.30 \text{ x } 1.05^{(29-20)}$

= 0.47 /d

r) Estimation of the effluent soluble BOD

According to[23], secondary facultative pond is considered as a complete-mix model and the length/breadth ratio = 2.5. In this case,

$$S = \frac{Sof}{1+(kxt)} \qquad (9)$$

with S_{of} the influent BOD in the facultative pond; K removal coefficient and t the retention time in the facultative pond

$S_{of} = BOD_{effl}$

$$S = \frac{145.5}{1+(0.47x8.31)}$$

$= 29.88$ mg/L

$S \approx 30$ mg/L

s) Estimation of the effluent particulate BOD

Assuming an effluent SS concentration equal to 80 mg/L, and considering each 1 mgSS/L leads to a BOD_5 of around 0.35 mg/L

Particulate BOD_5 = 0.35 mgBOD_5/mg SS x 80 mg BOD_5/L

= 28 mgBOD_5/L

t) Total effluent BOD (BOD_{effl_fac})

Total effluent BOD = soluble BOD + particulate BOD

= 29.88+28

Total effluent BOD = 57.88 mg/L

u) Calculation of the Length and breadth of the Facultative Pond

A = L.B = [(L/B). B].B = [2.5.B].B = $2.5.B^2$

5050.53 m2 = $2.5.B^2$ → B = $[A/ (L/B)]^{0.5}$ = $(6200/2.5)^{0.5}$ = 79.46 m

L = (L/B) × B = 2.5.B = 2.5 × 49.80 = 31.78 m

Length L= 79.46 m
Breadth B= 31.78 m

v) Calculation of the BOD removal efficiency of the anaerobic-facultative pond system

$$E = \frac{(So-BODeff)}{So} x100 \qquad (10)$$

$$= \frac{485-57.88}{485}\ x\ 100$$

= 88.07%

w) Total net area (anaerobic + facultative pond)

Total net area = 0.030 ha + 0.25 ha

Total net area = 0.28 ha

x) Total area required for the anaerobic and facultative pond.

According to[23], the total area required is 23% to 33% greater than the one calculated above, so:

Total net area = 1.3x0.28 = 0.37 ha

y) Total detention time

t_T = 1.39 + 8.321

= 9.70 days

≈10 d

2.3.1.1.3. Maturation ponds

Design of the maturation ponds

- Population: 179.000
- Influent flow: 485 m^3/d
- Temperature: 29°C
- Fecal coliform concentration in the raw wastewater No = $15x10^7$/100mL
- Number of facultative pond: 1

Length and breadth of the Facultative Pond

- Length L= 124.5 m
- Breadth B= 49.80 m
- Depth H = 2 m
- Hydraulic detention time: t= 9.72 d

a) Coliform removal in the facultative pond

- Hydraulic regime to be adopted in the calculation: dispersed flow regime.
- Dispersion number

d= 1/(L/B) = 1/ 2.5 = 0.40 *(11)*

- Coliform removal coefficient

For dispersed flow, the value of the bacterial decay coefficient is obtained by this equation:

K_b *(dispersed flow) = 0.542 x* $H^{-1.259}$ *(12)*

$= 0.542 \times 2^{-1.259}$

$= 0.33\ d^{-1}$ (20◦ C)

Correcting K_b for 29°C:

$K_{bT} = K_{b20}.\theta^{(T-20)} = 0.33 \times 1.07^{(29-20)} = 0.61d^{-1}$

b) Effluent coliform concentration

In a context of dispersed flow and with a retention time of 8.31 days, according to [23], the effluent coliform concentration is obtained this way:

$$a = \sqrt{1 + 4K.t.d} = \sqrt{1 + 4x0.23x8.31x0.40} \quad (13)$$

$$a = 2.56$$

$$N = N_o.\frac{4ae^{1/2d}}{(1+a)^2 e^{\frac{a}{2d}} - (1-a)^2 e^{-a/2d}} \quad (14)$$

$$N = 15x10^7 \,.\frac{4x3.42e^{1/(2x0.40)}}{(1+3.42)^2.e^{3.42/(2x0.40)} - (1-3.42)^2 e^{-3.42/(2x0.40)}}$$

$N = 1.7$ x 10^7 FC/100ML

The coliform removal efficiency in the facultative pond

$$E = \frac{N_o - N}{N_o}\ x\ 100 \quad (15)$$

$$= \frac{5x10^7 - 1.7x10^7}{5x10^7} x100$$

= 88.45 %

c) Volume of ponds

There are two maturation ponds in series and the detention time of the formal system is 4 days for each pond which corresponds to a total detention time of 8 days:

V= t.Q

= 4d x 607.45 m^3/d

= 2429.81 m^3

d) Dimension of the ponds

Depth: H= 1.5 (the same as the existing one)

- Surface area of each pond:

A=V/H = 2429.81 m^3/1.5 = 1619.87 m^2

- Total surface area:

1619.87 m^2 x 2 = 3239.74 m^2

- Dimensions: adopt square ponds (L/B = 1.0)

Number of ponds: 2

Length = 40.25 m

Breadth = 40.25 m

Depth = 1.0 m

The total area require is 25% greater than the theoretical area: 1.25 x 9858.73 m^2 = 12323.41m^2

e) Coliform concentration in the final effluent

Using the same method as in [23], L/B = 1:

d = 1/(L/B) = 1/1.0 = 1.0

The value of the coliform die-off coefficient is given by:

K_b (dispersed flow) = $0.542 x H^{-1.259}$

$= 0.542 \times 1.50^{-1.259}$

$= 0.33\ d^{-1}$ (20◦ C)

For T= 29°C, the value of K_b is:

$Kb_T = K_{b20}.\theta^{(T-20)}$

$= 0.33 \times 1.07^{(29-20)}$

$= 0.60\ d^{-1}$

The effluent coliform concentration from the 1st pond in the series is:

$a = \sqrt{1 + 4K.t.d} \quad = \sqrt{1 + 4x0.60x4x1.0}$

$= 3.25$

$$N = N_o . \frac{4ae^{1/2d}}{(1+a)^2 e^{a/2d} - (1-a)^2 e^{-a/2d}}$$

$$N = 15x10^7 . \frac{4x3.25}{(1+3.5)^2 . e^{3.25/(2x1)} - (1-3.25)^2 e^{-3.25/(2x1)}}$$

$N = 3.5 \text{ x } 10^7$ FC/100 ML

2.3.1.2. *Biogas emission from the stabilization pond from the existing and proposed WSP*

The third scenario aims to identify through calculation the emission of methane by the stabilization pond built in 2011 and up to 2050 after expansion.

Parameters

The parameters needed for this estimation are the emission factor of the plant and its BOD removal capacity.

- Emission factor:

According to Gupta et al. (2012), the emission factor is the amount of GHG released from a unit of source or activity related to that emission. Additionally, certain (real) emission parameters might be considered as WWT system performance indicators. Typically, emissions factors for WWT inventory are given as kg CH4 released per kg BOD removed or per m3 of treated water. Regarding the results obtained for Mexico, the closest context to Benin is the emission factor at 30^{o}C: 0.431 kg CH4/kg BOD removed [5].

2.3.1.2.1. *Methane emission estimation from 2011.*

- Amount of BOD removed:

Population: 4558 persons

Water consumption = (4558 x 2.7)/ 365

= 33.72 m^3/d

= 1324.109 m^3/d

Influent flow = 85% x Water consumption per day

$= 0.85x\ 33.72$

$= 28.66\ m^3/d$

Influent flow = 28660 L/d

Influent BOD = 485 mg/L = 13,900 Kg BOD /day

97% of BOD is removed by the system with Duckweed ponds and 93% by the algae-based pond system:

BOD removed = 1/2(13.900 Kg BOD /day x 0.97) + 1/2(13.900 Kg BOD /day x 0.93)

BOD removed = 13.20 Kg BOD /day

Estimated Methane emission = 0.431 kg CH4/kg BOD x 13.20 Kg BOD /day

Estimated Methane emission = 5.7 kg CH4/ day

2.3.1.2.2. Methane emission estimation up to 2050.

Applying the same emission factor as in the first scenario:

- BOD removal efficiency = 88.07%
- BOD removed = 294.61 $KgBOD_5$/d *0.88
- BOD removed =334.78 $KgBOD_5$/d

Estimated Methane emission = 0.431 kg CH_4/kg BOD x 788,95 Kg BOD /day

= 144.29 kg CH_4/ day

2.3.2. Scenario 2: Proposition of a Sewage Treatment System using less space.

The present scenario is based on the use of conventional treatment process as presented in Figure 16.

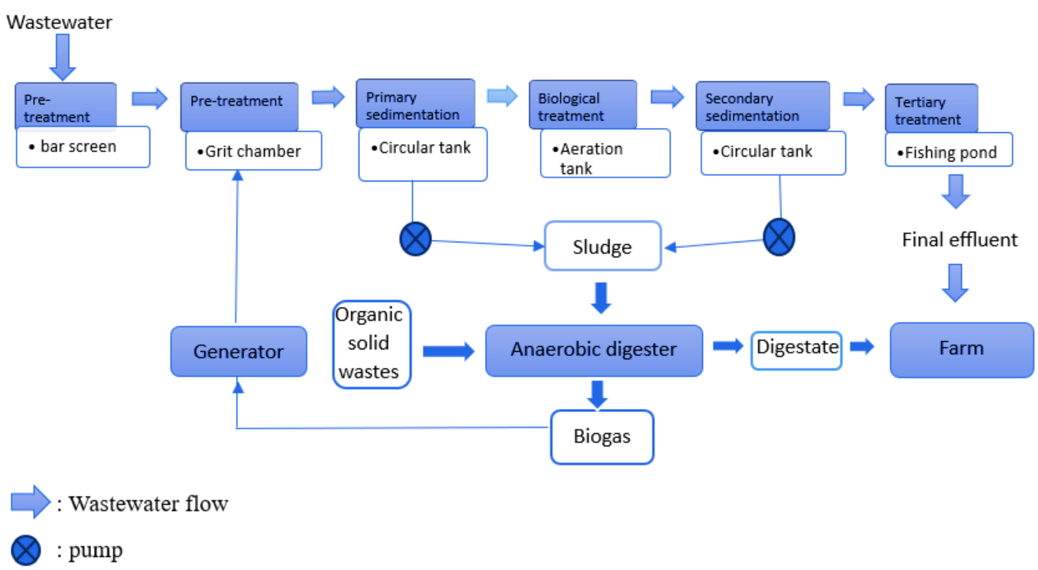

Figure 16: Proposition of a sewage treatment system for the campus

Description of the flow chat

The wastewater collected from the whole campus undergoes a series of pre-treatment passing through a bar screen (manually maintainable) followed by a grit chamber for the removal of grit. After the pretreatment a circular tank receives the wastewater for the primary sedimentation process where the settled sludge is pumped out. From this tank the water is conducted to an aeration tank for the removal of nitrogen and then into the second clarifier for the secondary sedimentation. The effluent is hence used in a fishing pond for the removal of coliform and nutrients before being used for irrigation in the farm near the Sewage treatment plant.

The sludge obtained from the primary and secondary sedimentation tank is mixed with organic solid wastes for an anaerobic digestion producing biogas and digestate. The digestate is a potential fertilizer for the farm while the biogas is used in a generator to produce electricity for the grit chamber use and the two pumps used for the sludge removal.

a. Design flow capacity

Population: 96,610 inhabitants

Influent flow, Q = 607.45 m^3/d

= 0.6 MLD

= 25.31 m^3/hr

Peak factor = 3

Design flow capacity = 25.31 m^3/hr x 3

= 75.93 m^3/hr

= 0.021 m^3/sec

b. Sizing Calculation for Collection Pit

Retention time = 4hr

Average design flow = 25.31 m^3/hr

Capacity of collection sump = 4 x 25.31

= 101.23 m^3

Assume liquid depth = 5 m

Area, required for collection pit = 101.23/5

= 20.24 m^2

The collection pit is assumed to be a circular tank so:

$20.24 = \pi r^2$

$r = \sqrt{20.24/\pi}$

= 2.53 m

c. Design of sewer chamber

Q_{max} = 0.021 m^3/sec

Assumption,

Shape of bar is a Mild Steel Flats with a size = 10 mm x 50 mm

Clear spacing between the bars = 20 mm

Inclination of bars = 80 deg

The average Velocity to Sewer = 0.8m/s

At peak flow, net inclined area required to =0.03/0.8

= 0.039 m^2

gross vertical area required = 0.039 x sin80 = 0.039 m^2

Provide submergence depth = 0.3 m

Width of channel = 0.039/0.3 = 0.13 m ≈ 0.20 m

Provide 20 bars of 10 mm x 50 mm at 20 mm clear spacing. Screen chamber will be 40 cm wide.

d. Design of Grit Chamber

The number of grit chamber is two (02) by assumption.

Design Flow = (2.5 x 0.6)/2

= 0.75 MLD

= 750 m^3/day

To account for turbulence and short circuiting, the surface loading rate is assumed to be:

Surface Loading = 500 m^3/m^2/day

Area required = 750/500

= 1.5 m^2

Provide a diameter of the chamber to be 1.31 m.

Detention time = 60 sec.

Volume = (750*60)/(24*3600)

= 0.52 m^3

Liquid depth = $\frac{Volume}{Area} = \frac{0.52}{1.5}$ = 0.35 m

Size of the grit chamber = 1.31x(0.35+0.6) with 0.6 the free board

Hence the dimensions of the grit chamber are:

Diameter = 1.31 m

Depth = 1.29 m

Area = 5.39 m^2

f. Check of horizontal velocity

Cross sectional area of grit chamber = 1.31 x 0.35

= 0.45 m^2

Velocity =750 / (1.31x0.35x24x3600)

=0.019 m/sec

= 1.9 cm/sec

The sewage flow from the Grit is assumed to be 0.05 m^3 per 1000 m^3.

Storage volume required = (607.45x8x0.05)/(24x1000)

$= 0.01\ m^3$

Grit storage area = $(\pi/4)\ x 1.31^2$

$= 1.34\ m^2$

Grit storage depth = 0.01/ 1.34

= 0.0075 m

Total liquid depth = 0.35 + 0.0075

= 0.35 m

= 0.4m

Provides grit chamber of size =1.31 x (0.4 + 0.4) = 1.31m x 0.8m

Referring in the example used as reference, the effluent from the grit chamber is carried to the aeration tank through a 600 mm wide RCC channel provided with fine bar screen (manually operated). The clear spacing between the bars shall be 10 mm.

e. Design of primary Sedimentation tank

Detention hour = 2 hr

Volume of sewage = max.Quantity of sewage/(detention time x 24)

= (607.45/2) x 24

$= 12.65\ m^3$

Provide depth = 2m

Surface area = Volume/Depth

$= 6.32\ m^2$

$(\pi/4)$ x $d^2 = 6.32\ m^2$

$d^2 = 8.05$

d=2.83 m

f. Design of Aeration tank

There will be two aeration tanks.

Average flow of each tank = 0.6 MLD /2 = 0.3 MLD = 303.72 m^3/day

Total BOD entering the Sewage Treatment Plant (STP)= 485 mg/L.

Assuming the BOD removed by the screen and grit chamber is negligible for they mainly remove inorganic solids. So the BOD of sewage coming to aeration tank = Yo =485 mg/L.

BOD left in the effluent = Y_E = 20 mg/L.

BOD removed in activated plant = 485 – 20 = 465 mg/L.

Minimum efficiency required in the activated plant = 465/485 = 0.95%

The volume of aeration tank can be designed by assuming a suitable values of cc and (F/M ratio), X_T = 3000mg/L (Between 3000 – 3500 mg/L).

F/M ratio = 0.15 (Between 0.18 – 0.1)

F/M = Q/V = Yo / X_T

V= (607.45x465)/(3000x0.15)

V = 627.69 m^3

Aeration tank dimensions.

Adopted values: Depth = 3.5 m; Width = 9 m

Length of the tank = V/ (B x D) = 627.69 / (9 x 3.5) = 19.9m

- Check for aeration period.

t = (V/Q) x 24 hr

= (627.69 x 24)/ 303.72

t = 49.6 hr

- Check for volumetric loading.

Volumetric loading = Q.$\frac{Yo}{V}$ gm of BOD_5/ m^3 volume of tank

= (303.72 x 485)/ 627.69

= 234.67 gm/m^3

- Check of return sludge ratio

$$\frac{Q_R}{Q} = \frac{X_T}{(\frac{10^6}{SVI} - X_T)}$$

Where SVI is the Sludge Volume index and SVI = 100 m/gm (between 50 – 150 m/gm)

X_T = 3000 mg/L

$$\frac{Q_R}{Q} = \frac{3000}{(\frac{10^6}{100} - 3000)}$$

$$= \frac{3000}{7000}$$

= 0.43

The value should be between 0.5 and 1. Taking SVI = 120 ml/gm

$$\frac{Q_R}{Q} = \frac{3000}{(\frac{10^6}{120} - 3000)}$$

= 0.56

➢ Check for SRT

$V.X_T = \frac{\{\propto_y.Q(Yo-Y_E).\theta_c\}}{K_E.\theta_c.t.1}$

Where:

$\propto_y = 1.0$

$K_E = 0.06\ /d$

Yo = 485 mg/L

$Y_E = 20\ mg/L$

X_T = 3000 mg/L

Q = 303.72 m^3

V = 627.69 m^3

⇨ 627.69 x 3000 = $\frac{[1x303.72(485-20)\theta_c]}{1+0.06\ \theta_c}$

⇨ $\theta_c = 66\ days$

Area of Aeration tank =313.84 m^2

g. Design of secondary clarifier

Only secondary clarifier is used.

Average flow = 607.45 m^3/day

Recirculated flow, say 50% = 303.72 m^3/day

Total inflow = 607.45 + 303.72

= 911.175 m^3/day

Provide hydraulic detention time = 2 hrs

Volume of tank = (911.175 x 2)/24

= 75.93 m^3

Assumed liquid depth = 3.5 m

Area = $\frac{75.93}{3.5}$

= 21.69 m^2

Provided Surface loading rate of average flow = 10 m^3/ m^2/day

Surface area to be provided = 607.45/10

= 60.74 m^2

Diameter = $\sqrt{60.74 \, x \, \frac{4}{\pi}}$

= 8.79

= 9 m

Actual area provided = 60.74 x 1.10

= 67 m^2

- Check for weir loading.

Average flow = 607.45 m^3/day

Weir loading = 607.45/($\pi x 10$)

= 19.33 m^3/day

Provided a peripheral loading:

Check for solids loading:

Recirculated flow = 607.45m^3/day

MLSS solids inflow = 3000 mg/L

Total solids inflow = (607.45+303.72) x 3

= 2733.51 kg/day

Solids loading = 2733.51/60.74

= 45 kg/day/m^2

Provide a clarifier with 9 m in diameter and liquid depth of 3.5 m.

h. Return Sludge Pump House

Total return flow = 303.72 m^3/day = 12.65 m^3/hr =0.21 m^3/min

Detention time = 15 min

Volume of wet well = 0.21 x 15

= 3.16 m^3

Provide depth wet well = 1m

Area wet well = 3.16 m^2

i. Design of sludge Drying beds

Sludge applied = 125 kg / day

Period of each cycle = 1.015

Specific gravity = 1.015

Solid contents = 1.5%

Volume of sludge = $\frac{125}{0.015}$ x $\frac{1}{1000x1.015}$ = 8.2 m^3/day

Considering monsoon, the total number of cycles in one year = 33

Period of each cycle = 365/33

= 11 days

Volume of sludge = 8.2 x 11 days

=90.2 m^3

Spreading a layer of 0.3m/cycle, the rea of beds required = 90.2/0.3 = 300.67

Provide 4 beds of 1.2m x 7 m, the total area needed = 33.6 m^2.

j. Filtrate Pump House and Sump

Actual BOD$_5$ 20^{o}C removed per day = 607.45 x (485 – 20)/ 1800 = 156.92 Kg

Excess water sludge, θ_c = V.$\frac{X_T}{Q_w.X_R}$

66 = $\frac{627.69 \text{ x } 3000}{Q_w.X_R}$

$Q_w.X_R$= (627.69 x 3000)/ 66

= 28531.36 g/d

$Q_w.X_R$ = 28.5 kg/d

Hence the excess sludge provided = 28.5 kg/d

Assuming the excess sludge to contain 1% solids and specific gravity = 1.015

Volume of excess sludge = $\frac{28.5}{1000\ x\ 1.015\ x0.01}$

= 2.8 m^3/d

= 0.11 m^3/hr

Taking detention time as 8 hrs

Volume of wet well = 8 x 0.11

= 0.95 m^3 for 1% concentration.

Provided liquid depth = 1m

Area required for 1% concentration of solids = 0.95 m^2

Diameter of the wet well = $\sqrt{(0.95x\,\frac{4}{\pi})}$

= 2.14 m

Chapter 3: Results and discussion

3.1. Scenario 1: Proposed wastewater treatment and estimation of methane emission

3.1.1. Description of the different blocs of the system

The proposed system is designed for 30 years and constitutes nine (09) blocks: School, anaerobic pond, facultative pond, duckweed pond 1, duckweed pond 2, fishing pond, collection platform, composting platform, and farm. The description of each block is found below.

Block School/Canteen: The school has a capacity of 96,610 people. Provided with this Parameter, the quantity of wastewater generated from the school is estimated at 607.45 m^3/day. In addition, the school also provides solid organic waste to the composting platform.

Anaerobic pond:

As shown in Table 3 the newly designed anaerobic pond requires a total area of 300.63m^2 which represents more than 21 times the one currently in use. This gap shows how important it will be to implement a new wastewater treatment system. The BOD_5 removal efficiency is 70% as well as a detention time of approximately 1.39 days. With this, its inlet should be positioned at the bottom of the pond. The sludge should be removed every 3 years corresponding to an amount of 289.83 m^3 or annually for an amount of 96.61 m^2.

Performance analysis of anaerobic and facultative ponds in two WSP systems before upgrading reveals the anaerobic ponds comply with the expected efficiencies based on the operational and climate conditions. The long-term removal efficiency of about 70% (BOD5) of APs and the mixture of the black and grey color of the sewage in them due to the generation of metal sulfides indicates their acceptable performance based on the design goals. Sedimentation followed by anaerobic digestion is the principal mechanism for the stabilization of organic carbon (as measured by BOD or COD) in anaerobic ponds and is also a major mechanism in facultative ponds (the sludge layer). Anaerobic decomposition involves a consortium of microorganisms that execute a complex process in a series of interdependent steps. In general, the anaerobic biochemical reactions involved three basic steps: (1) hydrolysis, (2) acidogenesis (fermentation)/acetogenesis, and (3) methanogenesis.

Table 4: Characteristics of the renewed anaerobic pond

Parameters	Values
Depth (m)	2.80
Influent flow (m^3/day)	607.45
Total influent BOD_5 load (Kg BOD_5/ day)	294.61
Volume required (m^3)	841.75
Detention time (Days)	1.39
Area required (m^2)	300.63
Effluent total BOD_5 concentration (mg/L)	145.50
Annual ludge accumulation/year (m^3/year)	96.61
Thickness of the sludge layer in 1 year (m/year)	0.32
BOD removal efficiency (%)	70

Facultative pond:

Table 4 shows that the total area required to build the new facultative pond is 2,525.26 m^2 and a volume of 5050.53 m^3. It will be 2 meters by maintaining the same depth as was used previously. The suspended solids concentration of the wastewater is 80mg/L with an effluent particulate of 28 mg/L and the total effluent BOD_5 given as 57.88 mg/L. The detention time considered after dividing the proposed wastewater volume by the influent flow is 8.31 days. Maintaining this facultative pond, coliform bacteria which are mostly found in ponds, rivers, streams and surface water sources are determined by three measurements with the specificity from the University of Abomey-Calavi wastewater; total coliform, fecal coliform as well as E.coli. These contaminants found in the pond are removed at a coliform removal efficiency of 88.45% as shown in Table 4. All of the characteristics in Table 4 were determined from scenario 1 as presented in the methodology for a proposed wastewater treatment.

Table 5: Characteristics of the new facultative pond

Parameters	Values
Depth (m)	2
Suspended solids concentration (mg/L)	80
Influent load L (Kg BOD/d)	88.38
Influent flow (m^3/day)	607.45
Required area (m^2)	2525.26
Volume (m^3)	5050.53
Detention time (Days)	8.31
Influent BOD_5 (mg/L)	145.5
Total effluent BOD_5 (mg/L)	57.88
Effluent soluble BOD_5(mg/L)	29.88
Effluent particulate (mg/L)	28
Breadth (m)	31.78
Length (m)	79.46
Coliform removal efficiency (%)	88.45

Maturation ponds:

As shown in Table 5, the two ponds have similar dimensions: a volume estimated at 2429.81 m^3 for each pond and a surface area of 1,619.87 m^2. With this, the aggregating surface area is 3239.74 m^2. In addition, the length of each pond is 40.25 m as well as the breadth of each pond corresponds to the same 40.25 m. The segment of the pond effluent coliform concentration form is $3.5*10^7$ FC/mL. In so doing, the coliform removal efficiency is 24% after considering the two duckweed ponds which represent the maturation ponds characteristics presented in Table 6.

Table 6: Characteristics of the maturation ponds

Parameters	Values
The volume of each pond (m^3)	2429.81
The surface area of each pond (m^2)	1619.87
Total surface area (m^2)	3239.74
Length of each pond(m)	40.25
Breadth of each pond (m)	40.25
Effluent coliform (FC/100 mL) concentration	3.5×10^7

Fishpond

The fishing pond is also considered as a maturation pond and therefore has the same size as a single maturation pond presented in Table 5. The three maturation ponds (the two duckweed-ponds and the fishing pond) receives wastewater from the facultative pond simultaneously.

Table 6 shows the wastewater characteristics received from facultative ponds in the volume of 4,859.62 m^3 with the proposed fishpond surface area of 3239.74 m^2. In accordance with sizing, it is estimated to have length (80.5 m) as well as 80.5 m in breadth. The concentration of effluent coliform is approximately $3.5*10^7$ FC/100ml.

Table 7: characteristics of the fishing pond

Parameters	Values
Volume of fishpond (m^3)	4859.62
Surface area of fishpond (m^2)	3239.74
Length (m)	80.5
Breadth (m)	80.5
Effluent coliform concentration (FC/100 ML)	3.5×10^7

Composting platform

The composting platform is reserved for the mix of sludge and organic solid waste from the different restaurants of the campus. Considering the ratio (Organic waste: Sludge) of co-composting being 2:1, the volume of this platform is equal to Annual accumulation x 3 = 96.61 m^3/year x 3 = 289.83 m^3/year. For a depth equal to 1m, the Length and Breadth have a value of 17.02m.

The surface A= V/H=289.83 m^3/1 m= 289.83 m^2

Collection platform

The collection platform is used for the storage of the organic solid waste collected from the restaurants of the campus. The collection platform has a lesser volume but can be sized at the same dimensions as the fishing pond.

Comparison of the performance of the new WSP to the existing one

Table 8:Performance and characteristics of the proposed system

Parameters	Values
BOD removal efficiency (%)	88.97
Coliform (FC/100ML)	3.5 E6
Anaerobic pond area (m^2)	300.63
Facultative pond area (m^2)	2525.26
Fishing pond area (m^2)	3239.74
collection platform area (m^2)	882.00
Composting platform (m^2)	289.83
Maturation Ponds area (m^2)	3239.74
Total area (m^2)	10477.2

Table 9: Comparison of the two systems

Parameters		Waste Stabilization Pond	
		Existing	Resized
Area (m2)	Anaerobic pond	13.75	914.82

	Facultative pond	55.65	7684.53
	Fishing pond	165.98	14788.10
	collection platform	_	882.00
	Composting platform	_	882.00
	Maturation Ponds	199.58	14788.10
	Total	434.96	39939.55

The primary design and operational parameter of FPs is surface organic loading (BOD5 surface loading rate: kg BOD5 ha−1 day−1), which relies on the concept that sufficient oxygen must be produced by phototrophs (microalgae: algae and cyanobacteria) to offset the required oxygen for waste organic oxidation by heterotrophic microorganisms.[2] The imbalance between the rates of phototrophic oxygen production and heterotrophic oxygen utilization leads to the disruption of the photo-biochemical treatment process of organic wastes. [WSP 2]

3.1.2. Biogas emission from the stabilization pond from the existing and proposed WSP

After using the emission factor from [5], the theoretical amount of methane emitted by the plant is estimated at 5.7 Kg/day in 2011 of which 144.29 kg CH4/ day is expected to be emitted by 2050 corresponding to an amount of 2080.5 kg CH4/year and 52,665.8 kg CH4/year respectively. The low amount of methane can be explained by multiple reasons. The lack of available information about the real concentration in BOD of the influent currently. The value used is the one used in the literature review, and which corresponds to a population in 2010 and has surely increased by 2021. Another explanation for this result is that the digestion of sludge produces more methane than the wastewater itself.

According to (Jonh Hobson, in his paper good practice guidance and uncertainty management in national greenhouse gas inventories, an important amount of methane is released rather from anaerobic digestion, storage and disposal of sewage sludge than an anerobic digestion of the wastewater itself. Some other considerable factor is that some of the methane generated is held in the sludge layer, transported in dissolved form into the following ponds (facultative and maturation ponds). The hypothesis made earlier about the enough amount of methane released is not verified. The calculated amount of methane is very low to be used for energy purposes.

3.2. Scenario 2: Proposition of a Sewage Treatment System using less space.

As it shows in Table 9, the Sewage Treatment System suggested requires a total surface area of 3690.25 m^2. This contributes to the reduction of required space, but it involves the consumption of electricity. To be able to cover the amount of energy to supply for the grit screen and pumps, organic solid wastes are to be added to the sludge for an anaerobic digestion. Pre-treatment area required (25.63 m^2) is essential in the treatment plant to aid in fastening the process of treatment. In other to have a suitable passing through of nitrogen for prefer treatment, the aeration tank required area (313.84 m^2) is suggested for better treatment. As observed in Table 9, proposed fishing pond (3239.79 m2) area is larger than that of aeration tank (313.84 m^2). These treatment characteristics are required for the new system which contributes to the use of lesser space and the improvement of sewage treatment as well as the usage of less electricity.

Table 10: Sewage Treatment Plant Characteristics

Process	Required area (m^2)
Pre-treatment	25.63
Primary sedimentation tank	6.32
Aeration tank	313.84
Secondary clarifier	67
Fishing pond	3239.79
Wet well for return sludge pump house	3.16
Sludge drying beds	33.6
Filtrate house and sump	0.91
Total	3690.25

Investing in waste stabilization ponds (WSPs) to treat wastewater on the campus over the past years often do not have the capital and/or inclination to replace their WSPs with more intensive treatment technologies such as activated sludge plants. Instead, as the required effluent quality gets higher, many countries look to relatively low-cost WSP upgrades to meet these requirements [28]. In this regard, the small-scale diffused aeration system investigated in the current study can be considered as one of the practical and economic options at the full scale. After upgrading of WSPs, the effluent quality of a WWTP is comparable to the other electromechanical treatment processes such as activated sludge. This system is installed on the natural treatment process with the least possible disturbance and significantly increases its efficiency.

Typically, the energy requirements for conventional activated sludge WWTPs are usually between 0.30 kWh/m3 and 0.65 kWh/m3, with the highest value being met when nitrification is applied. Due to the economy of scale, the higher the inlet flow rate, the lower the per volume unit energy requirements. Based on plant capacity, the energy consumption of the activated sludge process with 6,913 m3/day (capacity of Birjand WSPs) and 16,929 m3/day (capacity of Neyshabur WSPs) is almost 0.56 kWh/m3 and 0.45 kWh/m3 respectively according to Siatou et al. (2020). Whereas the upgraded Birjand and Neyshabur WSPs consume about 0.15 and 0.13 kWh/m3 respectively (more than 70% less energy usage). This low energy consumption, along with the minimum capital, and operation and maintenance costs for WSPs, well illustrates the economic and environmental benefits of upgrading WSPs with this aeration system.

Conclusion and perspectives

The objective of this thesis work was to upgrade the wastewater treatment system built several years ago on the campus of Abomey-Calavi and to evaluate the amount of biogas produced by the plant. The treatment used was a Waste Stabilization Pond designed only for the hostels on campus. The renovation proposed in this work has expanded its capacity to the whole campus. To achieve this goal two research questions were established:

- How does the growth in population on the campus affect the size of the constructed WSP and How much methane/biogas is released over the years and what can it be used for?
- Can the use of a conventional treatment of wastewater, especially a sewage treatment system be more profitable in terms of space?

Based on this question three scenarios were studied. In the first scenario, the WSP was resized according to the population of the whole campus and for a lifetime of 30 years the methane emission from the old and resized WSP. In the second scenario, the waste stabilization pond is replaced by a Sewage Treatment Plant of the same capacity. The sizing and calculation in the different scenarios were conducted using excel and equations from literature review.

The performance of the proposed system is close to the formal one except for the removal of coliform efficiency that has decreased. The methane produced from the wastewater treatment is too small for cooking or running a generator. How ever it is possible to use it for laboratory experiments. The upgraded Waste Stabilization Pond requires a total surface area of 39939.55 m^2 meanwhile for the same capacity a Sewage Treatment Plant requires only a total surface area of 3690.45 m^2. The Sewage Treatment Plant suggested is not only advantageous in terms of space but also in the circular economy that can be developed to sustain the plant. The total amount methane produced or emitted from the WSP is estimated to increase from 5.7 Kg/day in 2011 to 144.29 kg CH_4/ day in 2050.

The results in this work can be used to do a feasibility study of waste pond stabilization on the campus and the other public universities.

For further research and work, the following recommendation can be considered:

- This study has focused more on the technical aspect of the plant and needs to be completed with a study of the economic aspect. The use of WSP suits more the country situation in terms of energy availability. But at the same time, to face urbanization and make sure the land required for its expansion can be met, it is better to go for a sewage treatment that requires less space for the same treatment.

- Another aspect that needs to be worked on is the empirical performance of the different systems proposed and a possibility to maintain the fishpond and irrigation system of the formal plants. Because Fishing and farming can be a way of sustaining the plant economically.
- The sludge obtained for the treatment of the wastewater can be mixed with the organic solid waste generated from the campus.
- The possibility of capturing the few methane produced for lab work improving this way the building capacity of the campus.

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