Guilherme João Mendes Gonçalves Martins

Licenciado em Ciências de Engenharia do Ambiente

Technical and economical assessment of the conversion of a conventional WWTP to reach energy neutrality and to provide water reclamation

Dissertação para obtenção do Grau de Mestre em
Engenharia do Ambiente – Perfil de Engenharia Sanitária

Orientador: Engenheiro João Luís Paciência Dinis, CTGA

Co-Orientador: Professora Doutora Leonor Miranda Monteiro do Amaral, Professora Auxiliar, FCT-UNL

Júri:

Presidente e arguente: Prof. Doutor António Pedro de Coimbra Macedo Mano
Vogal: Prof.ª Doutora Rita Maurício Rodrigues Rosa
Vogal: Prof.ª Doutora Leonor Miranda Monteiro do Amaral

FACULDADE DE CIÊNCIAS E TECNOLOGIA
UNIVERSIDADE NOVA DE LISBOA

Dezembro 2018

Technical and economical assessment of the conversion of a conventional WWTP to reach energy neutrality and to provide water reclamation

Acknowledgments

To my supervisors for their careful insight making this project possible to materialize namely,

Eng. João Dinis for always being present and giving me the necessary tools for the development of this project while guiding me from beginning to end. For the knowledge sharing and professional advice in a friendly manner.

Prof. Leonor Amaral for the kind words of motivation since the beginning of this project and for the trust and freedom that I have been given.

To the company CTGA as a whole, particularly to the management, namely Dr. Ezequiel China for letting me develop this project and also for conceding me with first-hand experience of the workplace of an engineering firm.

I’m grateful to everyone I came in contact with, everyone was very attentive and helpful, which contributed greatly to the experience and knowledge assimilation. I want to give a special mention to Eng. João Lopes da Silva, Eng. Hugo Sabino, Eng. José Paixão, Eng. Rita Gonçalves, Eng. Joana Blazer, Lab. Technician Pedro Carlos and Eng. Cristina Costa.

To my family for making this possible and for supporting me every step of the way.

To my friends that have directly or indirectly been with me throughout the elaboration of this project.

Abstract

Conventional WWTP are big energy consumers. This is an issue in terms of operation costs and a concern as global climate change constitutes a serious problem. Simultaneously, water scarcity constitutes a growing worldwide issue.

This thesis accesses the possibility of reaching energy neutrality and reducing operation costs in Espinho WWTP, by means of optimization of the treatment line, in a cost-effective manner. Moreover, the economic feasibility of providing tertiary treatment to the secondary effluent, in order to reclaim water for irrigation, is also investigated here.

It is presented an evaluation of possible processes, that could be implemented, that reduce the energy demand, such as CEPT, as well as methods for increasing the energy production in a treatment plant, like anaerobic digestion, co-digestion or the installation of photovoltaic (PV) solar panel modules. The latter is nowadays starting to be a requirement in the design/construction of new sizeable WWTP.

Adjacent to the WWTP, there is a golf course, which demands 200,000 m^3/y of water for irrigation. The water reclamation is seen as a possible to strategy to supply the needs.

The design of each treatment phase of Espinho WWTP is verified, both when operating with conventional primary treatment and chemically enhanced primary treatment (CEPT). Jar-tests with Espinho affluent wastewater were conducted, and the optimum PAX18 coagulant dosage determined was 15 mg/L, to perform CEPT.

The methods studied contribute to improving the energy efficiency of a WWTP and are presented as possible approaches to progress in the direction of energy self-sufficiency. The operation costs, as in reagents and energy, were calculated prior to the WWTP optimization and following each possible upgrade.

CEPT demonstrates to reduce the energy consumption of the aeration process by approximately 40%. On the other hand, co-digestion proves to boost the energy production in the anaerobic digestion considerably, by 84% to 154%. Additionally, PV solar panel modules have shown to supply 10% of Espinho WWTP energy demands. The implementation of the studied methods allows the WWTP to produce 68% of its total energy needs.

Keywords: Espinho WWTP; Energy neutrality; Aeration; CEPT; Co-digestion

Resumo

As ETAR convencionais apresentam-se como grandes consumidores energéticos, o que é motivo de preocupação, tendo em conta as alterações climáticas. Simultaneamente, a escassez de água constitui um problema crescente a nível global.

Esta dissertação avalia a possibilidade de se atingir a neutralidade energética e uma redução dos custos de operação da ETAR de Espinho, com a otimização da linha de tratamento de uma forma economicamente viável. Adicionalmente, é realizada uma análise económica para a hipótese de se efetuar tratamento terciário, para obtenção de água residual tratada para irrigação.

É realizada uma avaliação de possíveis processos que contribuem para a redução do consumo energético, tal como o tratamento primário quimicamente assistido (CEPT). São ainda apresentados métodos passíveis de aumentar a produção de energia numa estação de tratamento, como a digestão anaeróbia, co-digestão ou a instalação de painéis solares fotovoltaicos (PV). A instalação de painéis PV atualmente é um dos requisitos no dimensionamento e construção de novas ETAR de grande dimensão.

Nas proximidades da ETAR, há um campo de golf, que requer 200,000 m^3/ano de água para irrigação. A reutilização de água residual tratada, é uma possível estratégia para suprir estas necessidades.

É realizada uma verificação do dimensionamento de cada fase de tratamento da ETAR de Espinho, com tratamento primário convencional e com CEPT. Foram ainda efetuados ensaios de jar-test com a água residual afluente à ETAR de Espinho, tendo-se determinado uma dose ótima de 15 mg/L do coagulante PAX18 para realização de CEPT.

Os métodos estudados contribuem para uma melhor eficiência energética da ETAR e são apresentados como uma possível abordagem para alcançar a autossuficiência energética. Os custos de operação, em termos de reagentes e energia, são calculados para cada opção estudada.

Neste trabalho comprovou-se que o CEPT reduz em cerca de 40% o consumo de energia no arejamento e a co-digestão demonstrou aumentar de 84% a 154% a produção de energia no processo de digestão anaeróbia. Adicionalmente, os PV demonstraram suprir 10% das necessidades energéticas da ETAR de Espinho. A implementação dos métodos estudados permite que a ETAR produza 68% das suas necessidades energéticas totais.

Palavras-chave: ETAR de Espinho; Neutralidade energética; Arejamento; Decantação primária quimicamente assistida; Co-digestão

Contents

List of Tables

List of Figures

List of Acronyms

AcoD	Anaerobic Co-Digestion
AD	Anaerobic Digestion
AT	Aeration Tank
BNR	Biological Nutrient Removal
BOD	Biochemical Oxygen Demand
BOD_5	Five-day Biochemical Oxygen Demand
C/N ratio	Carbon to Nitrogen Ratio
CAS	Conventional Activated Sludge
CBF	Charged Bubble Flotation
CEPT	Chemically Enhanced Primary Treatment
CHP unit	Combined Heat and Power unit
COD	Chemical Oxygen Demand
d	day(s)
DO	Dissolved Oxygen
F/M	Food to Microorganisms Ratio
FOG	Fats, Oils and Grease
FW	Food Waste
h	hour(s)
HRAS	High-rate Activated Sludge
HRT	Hydraulic Retention Time
MF	Microfiltration
MLSS	Mixed Liquor Suspended Solids
NF	Nanofiltration
NOM	Natural Organic Matter
OFMW	Organic Fraction Municipal Waste
PE	Population Equivalent
PEF	Primary Effluent Filtration
PV solar energy	Photovoltaic Solar Energy
RO	Reverse Osmosis
SOTE	Standard Oxygen Transfer Efficiency

SRT	Solids Retention Time
SVI	Sludge Volume Index
TN	Total Nitrogen
TOC	Total Organic Carbon
TP	Total Phosphorus
TSS	Total Suspended Solids
UF	Ultrafiltration
VSS	Volatile Suspended Solids
WRRF	Water Resource Recovery Facility
WW	Wastewater
WWTP	Wastewater Treatment Plant
y	year(s)

1 Introduction

Domestic wastewater is a byproduct of the human activities (Mara, 2004). Wastewater is produced everyday around the world and if left untreated, it can negatively affect human health and the environment. Worldwide, the produced wastewater that is collected and receives treatment is approximately 20% (UNESCO, 2012).

The discharge of untreated wastewater leads to water pollution, mainly due to the organic matter and nutrients present. Carbon, the primary constituent of organic matter, can negatively impact the water bodies, as excessive oxidizable organic matter threatens the oxygen concentrations. Additionally, nutrient pollution due to the excess nitrogen and phosphorus is the main driver for the degradation of water quality in Europe (Lema & Suarez, 2017; Sepp et al., 2018).

Water, in quality and in quantity, is promptly declining in a global scale due to population growth, industrial and agricultural development, as well as modifications to the hydrological cycle, as a result of climate change. Water scarcity is presented as a worldwide issue and considered one of the most serious threats to society (Roccaro, 2018). Moreover, global water use over the past hundred years has increased by a factor of six, continuing to increase at a rate of 1% per year (UNESCO, 2018).

According to the International Water Management Institute a substantial amount of world's population is expected to suffer from water scarcity by 2025 (Eslamian, 2016). Currently two-thirds of world's population reside in areas that undergo events of water scarcity for at least one month a year. It should be clear that 50% of those affected are from India and China. In countries such as Somalia or Libya 80% to 90% of the population suffers from year round severe water scarcity (UNESCO, 2017).

There are three existent alternative water sources: desalination (if seawater is nearby), water importation and water reuse. The latter is often the least energy-intensive solution (Eslamian, 2016). The interest in the exploit of unconventional water resources has been growing in order to increase the drinking water supplies, as wastewater is composed of approximately 99% water and only 1% of suspended, colloidal and dissolved solids (Lema & Suarez, 2017; UNESCO, 2017). In this respect, wastewater reclamation and reuse is intended to preserve substantial volumes of fresh water by replacing fresh water utilization for non-potable uses, such as agriculture and landscape irrigation, urban cleaning, firefighting, construction, recreational activities, groundwater recharge or surface water replenishment (Meneses et al., 2010).

Wastewater treatment plants are frequently the largest individual energy consumers administered by municipalities (Gu et al., 2017a; Gu et al., 2017b). In a conventional WWTP the total operation costs relative to energy consumption, range from 25% to 40% and in some cases are as high as 65% (Gu, et al., 2017a; Guerrini, Romano, & Indipendenza, 2017). The electric energy demands represent 90% of the total energy consumption (Di Fraia et al., 2018). Nationwide it has been reported that, the WWTP represent 1% of total national electricity consumption in European countries (Di Fraia et al., 2018) and 3% in United States of America (McCarty et al., 2011; U.S.EPA, 2014).

Energy recovery, as well as water and resource recovery, in a WWTP represent the new paradigm shift. The goal of achieving energy neutrality in a WWTP is as important as water reuse (Gu et al., 2017a; Gu et al., 2017b). In this regard, WWTP are gradually becoming water resource recovery facilities – WRRF (Papa et al., 2017).

An example of this is Billund Biorefinery, an energy-sufficient treatment plant that receives both wastewater and household waste to provide treatment, while also contributing as a public energy supplier. Marselisborg WWTP is another example of a self-sufficient treatment plant from the Netherlands, that is also an energy provider (Aarhus Vand, 2018; Billund BioRefinery, 2018).

When aiming to reach energy neutrality two procedures should be considered: improving energy efficiency (with efficient blowing and mixing systems) and retrieving renewable energy from anaerobic digestion (Mattioli et al., 2017).

The energy recovered via organic matter is the type of energy most easily salvaged in a WWTP. Energy in a WWTP can be obtained via combined heat and power systems (CHP), which utilizes the biogas produced in the anaerobic digestion; biosolids incineration or pyrolysis; effluent hydropower; heat pumps; bioelectrochemical systems; and microalgae technology with the conversion of harvested microalgae to energy (Mo & Zhang, 2013).

In this context, it is here proposed an approach, applied to Espinho WWTP, to move in the direction of energy neutrality and to reduce the operation costs. An analysis to verify the viability of water reclamation is also presented.

Document Structure

This document is structured into 9 chapters.

In the first chapter it is presented an introduction of the problem studied.

In the second chapter the objectives of this work are presented.

The third chapter consists of the literature review. This section contains all the scientific publications that sustain this project.

The fourth chapter is the methodology. In this section it is exhibited all the steps and methods conducted during the elaboration of this work.

The fifth chapter comprises the results obtained throughout the development of this study.

The sixth chapter purpose is to discuss the results obtained with the results of other scientific studies and with the established objectives.

The seventh chapter consists of the conclusions and limitations of this project.

The eight chapter is the final considerations, in which it is given indications for further investigation.

The ninth chapter is the annexes, which exhibit additional information to support this work.

2 Objective

The objective of this work was to provide a methodology to improve the energy balance of a large WWTP with the purpose of coming closer to reaching energy neutrality in a cost-effective manner. This study intended to investigate the implementation of inexpensive methods to improve the energy balance and reduce the operation costs, while still maintaining adequate treatment.

Additionally, it was evaluated the implementation of a method for accomplishing water reclamation for the irrigation of a golf course, which is located near the WWTP.

3 Literature Review

3.1 Preliminary Treatment

Raw wastewater before the primary treatment requires physical and mechanical operations to remove as many elements as possible, like heavy floating objects, heavy mineral particles (sand and grit) in order not to hinder future treatment procedures. Preliminary treatment is employed to expunge or diminish the adverse effects of debris that put the functioning of downstream equipment and processes at risk (Borges et al., 2015; Degrémont, 1991; Mara, 2004).

Preliminary treatment operations include the following:

- Screening;
- Grit removal;
- Grease and scum removal.

3.1.1 Screening

Screening is normally the first unit process in a WWTP, with the intent of retaining large solids and coarse materials in the influent wastewater to the treatment plant in order to: prevent damage or clog downstream process equipment, reduce treatment process reliability and effectiveness. The types of screens used in preliminary treatment are coarse and fine screens (Demirbas et al., 2017; Tchobanoglous et al., 2014; U.S. EPA, 2000).

With the ever forward advances in technology, screens are increasingly more reliable. The interest in all types of screens has been renewed, due to the need of more compact WWTP. Screens capacities range from removing settleable solids like grit and primary sludge to refining the effluent from final clarifiers (Qasim, 1999).

Coarse screens are comprised by openings of 6 mm and above, that retain debris like rocks, branches, plastics, bottles, cans, rags. Organic matter is also removed when associated with screenings as the spacing decreases (Tchobanoglous et al., 2014).

Fine screens are comprised by openings of 0.5 mm to 6 mm. This equipment retains smaller materials including putrescible matter (such as fecal material), substantial amounts of grease and scum (Degrémont, 1991; Tchobanoglous et al., 2014). This equipment provides pre-treatment or primary treatment and in general is capable of removing 20% to 35% of BOD_5 and suspended solids (Qasim, 1999).

In terms of BOD removal, screens contribution is reduced since the solids retained are usually inorganic and would not be measured in a BOD sample even if they were organic (Alley, 2007).

A WWTP will typically remove from 4 to 90 cubic meters of screenings per 10^6 m^3 of influent wastewater (Spellman, 2010). According to Qasim (1999), a screen with a clear spacing of 25 mm produces an average amount of 20 to 36 cubic meters of screenings per million cubic meters of flow.

3.1.2 Grit and Scum Removal

In a WWT, grit removal is performed in order to remove non-digestible components from wastewater (Meroney & Sheker, 2003).

Grit consists of sand, gravel, broken glass, cinders and other materials with a settling velocity significantly greater than those of the organic material in wastewater (Davis, 2010; Tchobanoglous et al., 2014).

The process of grit removal is adequate to protect mechanical equipment from abrasion and wear; reduce the formation of heavy deposits in pipelines, aerobic tanks, aerobic digesters, conduits, and channels; and reduce the frequency of digester cleaning due to accumulated grit (Davis, 2010; Meroney & Sheker, 2003; Tchobanoglous et al., 2014).

Grit is removed by settling, more specifically discrete, or Type I, settling. This type of settling occurs when particles settle as individual entities due to low solids concentration (Qasim, 1999).

The amount of grit removed depends on: type of collection system (separate or combined); climatic conditions; soil type; condition of sewers and grades; types of industrial wastes; use of garbage grinders; and proximity to sandy bathing beaches in coastal areas (Qasim, 1999).

Grit removal is conducted in separate grit chambers like: horizontal-flow grit chambers with rectangular or square configurations; aerated grit chambers; vortex grit chambers (Tchobanoglous et al., 2014).

The removal data of grit is difficult to interpret because this material is poorly characterized and there is little information on its removal efficiencies. The available data comes from what has been collected rather than the actual grit in the influent wastewater (Tchobanoglous et al., 2014).

According to Tchobanoglous et al., (2014), horizontal-flow grit chambers with rectangular configurations can remove 100% of the particles retained in a 0.21 mm or 0.15 mm screen; horizontal-flow grit chambers with square configurations can remove approximately 95% of the 0.15 mm diameter particles at peak flow; aerated grit chambers remove close to 100% of the sedimentable grit of the influent wastewater.

An aerated grit chamber offers many advantages over the remaining systems such as: the possibility of also being used for chemical addition, mixing and flocculation before the primary treatment; grease or scum removal if a superficial skimmer is installed; reduction in odors and additional BOD_5 removal (Qasim, 1999). Vortex grit chambers provide high performance, while presenting less space requirements (Meroney & Sheker, 2003).

Grit and scum quantity varies from 5 to 200 m^3 per 10^6 m^3, average value is 30 $m^3/10^6$ m^3 (Qasim, 1999).

Grit quantities reaching the WWTP differ according to the sewage collection system implemented, as shown in the table 3.1-1.

Table 3.1-1: Quantity of grit removed from wastewater from separate and combined collection systems in aerated grit chambers (Tchobanoglous et al., 2014).

Type of collection system	**Average grit quantity (m^3/1,000 m^3)**
Separate	0.004-0.037
Combined	0.004-0.20

3.2 **Primary Treatment**

Primary treatment is generally the next step in the treatment process following the removal of coarse solids and grit. Primary treatment is materialized with the primary sedimentation of the influent wastewater, to the WWTP, with the objective of removing readily settleable solids and floating material and consequently reducing the suspended solids content (Qasim, 1999).

Primary sedimentation is characterized by flocculent, or type II, settling. This type of settling occurs in somewhat dilute suspensions in which the particles coalesce, or flocculate, increasing particle mass and consequently enhancing the settling velocity rate (Tchobanoglous et al., 2014).

3.2.1 Conventional Primary Sedimentation

Primary sedimentation takes place in a sedimentation tank or clarifier, either rectangular or circular. The sedimentation tanks if efficiently designed and operated, can remove 50 to 70% of the TSS and 25% to 40% of the BOD (Tchobanoglous et al., 2014). In figure 3.2-1 and 3.2-2, both types of clarifiers are represented.

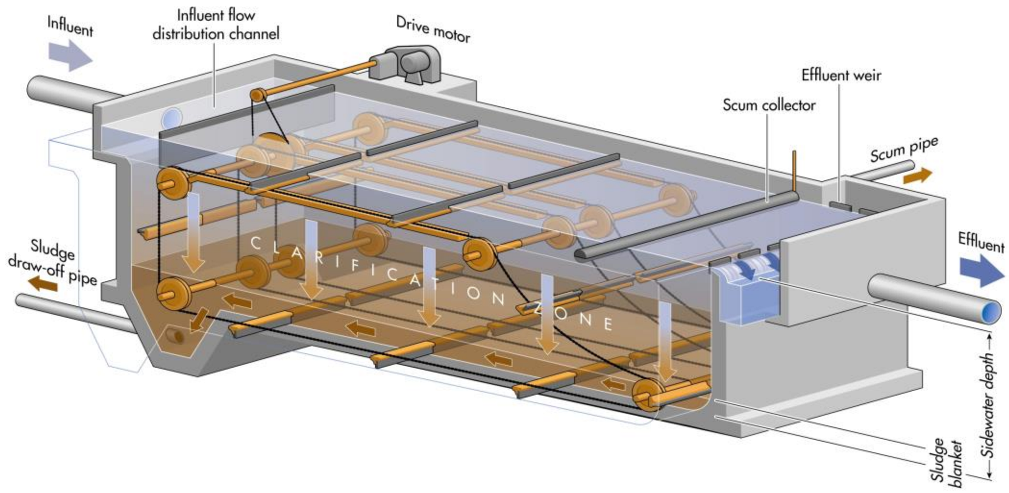


Figure 3.2-1: Conventional rectangular clarifier (Voutchkov, 2017).

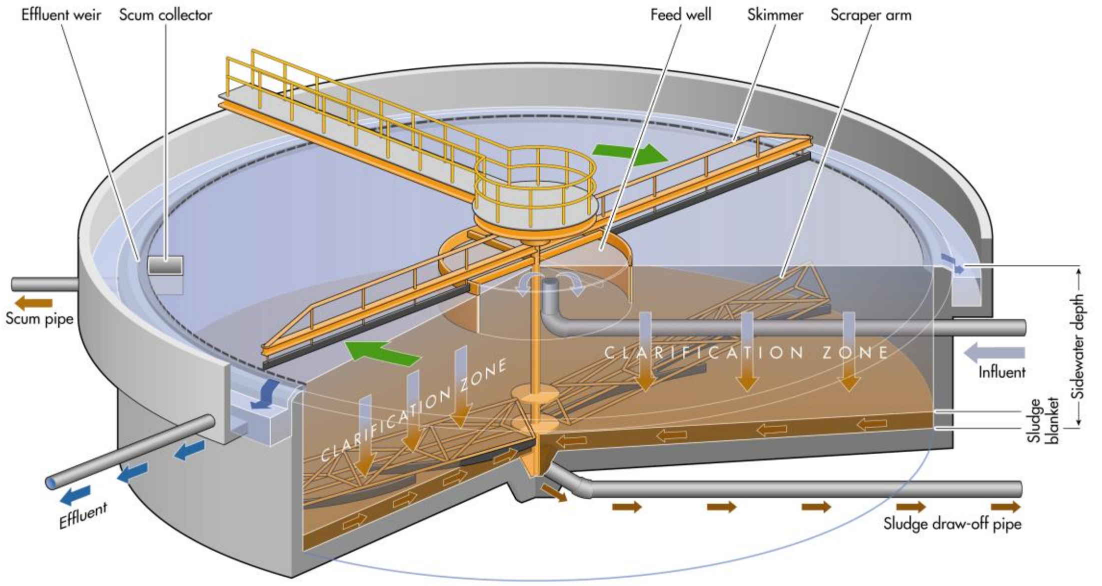


Figure 3.2-2: Conventional circular clarifier (Voutchkov, 2017).

The factors that influence primary clarifier performance are: surface overflow rate; the influent TSS concentration; the settling characteristics of the settleable solids; the nonsettleable TSS concentration; the soluble COD concentration; and the ratio of particulate COD (or BOD_5) to TSS in the primary effluent (Water Environment Federation, 2005).

Tchobanoglous et al., (2014), claims that the detention time is also a major factor for the performance of the sedimentation tank. According to Jover-Smet et al., (2017), the variable that most affects the removal of suspended solids and organic matter is the influent suspended solids load, followed by the surface overflow rate being the second most important.

The design parameters for the primary sedimentation tanks are the detention time or hydraulic retention time and the surface loading rates (or overflow rate), whose information is described in the table 3.2-1.

Table 3.2-1: Design parameters information for primary sedimentation tanks (Tchobanoglous et al., 2014).

Design parameter	Unit	Range
Detention time	h	1.5-2.5
Overflow rate	$m^3/(m^2.h)$	1.25-2.1

Primary clarifiers are occasionally designed with a shorter detention time of 0.5 to 1 h, resulting in less removal of TSS, when upstream of biological treatment processes (Qasim, 1999; Tchobanoglous et al., 2014).

Other factors that affect sedimentation tank performance are: wind induced circulation cells formed in uncovered tanks; thermal convection currents; thermal stratification in hot, arid climates; cold or warm water causing the formation of density currents. The previously mentioned factors reduce the effective volumetric capacity of the tank, due to the formation of dead spaces. (Tchobanoglous et al., 2014).

The characteristics of the sludge obtained from primary sedimentation are described in table 3.2-2.

Table 3.2-2: Primary sludge characteristics (Tchobanoglous et al., 2014)

Type of sludge	Specific gravity	Solids concentration range (%)
Primary, medium strength wastewater	1.03	4-12
Primary, from combined sewer system	1.05	4-12
Primary and waste activated sludge	1.03	2-6

3.2.2 Chemically Enhanced Primary Treatment (CEPT)

The process of chemical precipitation consists in the conversion of soluble substances to insoluble particles, that can be flocculated and separated from the liquid. The removal efficiencies are dependent on the mixing times, mixing type (either mechanical or hydraulic) and the coagulant type and dosage (Ayoub et al., 2017).

With the addition of chemicals for induced precipitation it is feasible to remove 80% to 90% of the TSS including some colloidal particles, 50% to 80% of the COD/BOD (Tchobanoglous et al., 2014), 20% of nitrogen and 95% of phosphorus (Bratby, 2006).

According to Bratby (2006), in Norway typical removal efficiencies reported with CEPT were the following: 73% of COD (370 to 99 mg/L), 81% of BOD (140 to 27 mg/L), 91% of TSS (190 to 17 mg/L), 65% of TOC (70 to 24 mg/L), 28% of TN (37 to 27 mg/L) and 94% of TP (4 to 0.25 mg/L).

According to Haydar & Aziz, (2009), CEPT with optimum doses of alum can remove almost all particulate COD and 7 to 28% of soluble COD. Nevertheless, CEPT effluent still presents high concentration of organic matter in its dissolved form.

In table 3.2-3, it is presented a comparison of the removal efficiencies between the conventional primary treatment and the chemically enhanced primary treatment (CEPT).

Table 3.2-3: Conventional primary treatment removal efficiency versus CEPT (Tchobanoglous et al., 2014)

Primary treatment processing alternatives	TSS Removal range (%)	BOD Removal range (%)
Conventional Primary Treatment	50-70	25-40
Chemically Enhanced Primary Treatment (CEPT)	80-90	50-80

The recommended surface overflow rate for CEPT ranges from 2.8 to 3.4 $m^3/(m^2.h)$, being almost twice the overflow rate of a conventional primary sedimentation process. CEPT can be designed to perform at an overflow rate of up to 4 $m^3/(m^2.h)$ without it affecting the effluent quality (Water Environment Federation, 2005).

According to Meerburg et al., (2015) CEPT proceeded by anaerobic digestion of the primary sludge, has been proposed as a candidate technology to achieve energy neutrality in wastewater treatment (Diamantis et al., 2013).

Enhanced primary treatment is essential in energy management at a WWTP, since solids removed in primary treatment, particularly the organic matter, have a high energy value prior to biological conversion to sludge, that is before the oxidation of the organics to CO_2. The energy from primary sludge can then be recovered by anaerobic digestion. Adding to this, a higher removal, in the primary treatment, of constituents that exert an oxygen demand means less aeration is required in the secondary treatment and consequently less energy expenditure. Moreover there is less excessive sludge production. (Meerburg et al., 2015; Tchobanoglous et al., 2014; Wan et al., 2016).

CEPT is however not optimized for the removal of dissolved organic matter, limiting the maximum amount that can be recovered, leaving a considerable fraction of organics to be treated in subsequent stages to meet effluent standards (Haydar & Aziz, 2009; Meerburg et al., 2015).

The sludge removed from CEPT presents a dry solids concentration that ranges from 0.5 to 3 % (Tchobanoglous et al., 2014).

The most common coagulants used for chemical precipitation are the following: (1) aluminum sulfate; (2) aluminum chloride; (3) calcium hydroxide (lime); (4) ferric chloride; (5) ferric sulfate; (6) ferrous sulfate; and (7) sodium aluminate (Tchobanoglous et al., 2014).

Polyaluminium chloride (PACl) is also presented as an inorganic polymer coagulant which presents advantages over conventional alum and ferric coagulants due to being less sensitive to temperature and pH shifts, working satisfactorily at a pH range of 5 to 8 (Gebbie, 2001). PACl contains highly positive charged polycations, which are very effective in neutralizing negative charges of colloidal particles, resulting in elevated colloidal destabilization (Ng et al., 2013). Moreover PACl is presented as a cheap coagulant alternative (De Feo et al., 2013). PAX 18 is a variant of PACl.

In figure 3.2-3, it is displayed the percentage of TP removal in line with the PACl coagulant dose added to a wastewater sample.

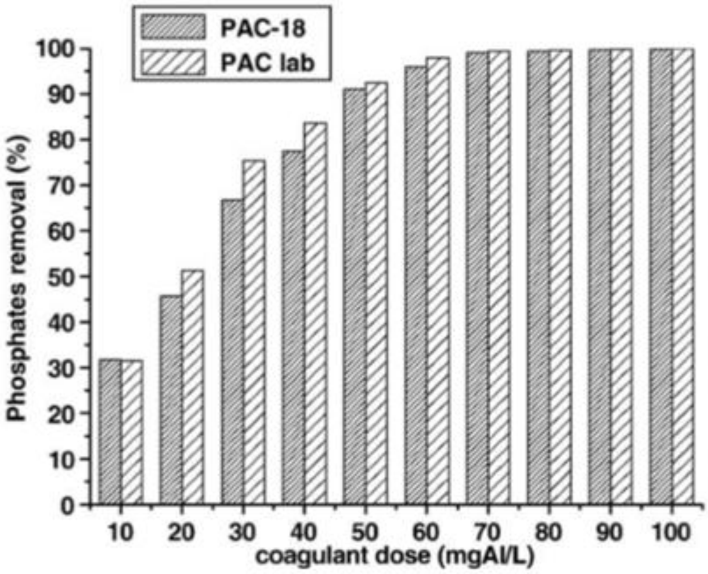


Figure 3.2-3: TP removal with PACl (PAX 18 and PAC laboratory made) (Zouboulis & Tzoupanos, 2010)

The coagulant dose required depends on the nature of the wastewater, the pH value, the phosphate level, and the point of injection (Ayoub et al., (2017). As reported by Vesilind (2003), Poon & Chu, (1999) and Tchobanoglous et al., (2014) wastewater characteristics vary and thus the selection of an appropriate coagulant and its chemical dosages should be determined from bench-scale or pilot-scale tests. The typical dosages of coagulant range from 10 to 50 mg/L. As stated by Poon & Chu, (1999) the flocculant dosage, in the form of anionic polyelectrolytes, vary from 0 to 1 mg/L (De Feo et al., 2008) to enhance the floc development.

According to Water Environment Federation (2005), the use of iron salts can decrease the efficiency of downstream disinfection with UV light. Adding to this metal coagulants may generate downstream pH inhibition problems in subsequent biological processes (biological treatment or sludge digestion), since each mg/L of alum potentially decreases the alkalinity by 0.5 mg/L as $CaCO_3$ (Bratby, 2006).

Subsequently an investigation was conducted by a team of Canadian researchers with the use of high polymer dosages (<8 mg/L) and the results with polymer-only coagulation (direct flocculation) were of increased removal of suspended solids, at higher overflow rates, than coagulation with ferric chloride and polymer (Water Environment Federation, 2005). Studies have shown that direct flocculation of organic-based industrial wastewater (e.g. food, paper and pulp, textile effluents) can achieve a removal efficiency of 90 % of COD and TSS (Lee et al., 2014).

3.2.3 High-Rate Clarification

High-rate clarification consists of physical or/and chemical treatment with special flocculation and sedimentation systems to achieve rapid settling. This treatment process can be conducted via ballasted flocculation or lamella plate clarification (Tchobanoglous et al., 2014).

High-rate clarification advantages are the following: compact units and thus reduced space requirements; start up times are rapid and peak efficiency can be achieved within 30 minutes; the effluent produced is highly clarified; high overflow rate is attained (Tchobanoglous et al., 2014).

Lamella plate clarification consists of a sedimentation process that occurs in a sedimentation basin with lamella plates installed to enhance the settling characteristics by increasing the settling area. Prior to this process there is the addition of chemicals such as coagulants and polymer followed by a three-stage flocculation, via three separate zones with continuously decreasing mixing energy gradient (Tchobanoglous et al., 2014).

The removal efficiencies (BOD_5 and TSS) of lamella plate clarification with no prior stage of coagulation/flocculation are similar to those obtained via conventional primary clarifiers, when operating at the same overflow rate based on projected area. The same can be said of lamella plate clarification with previous coagulation/flocculation stage and CEPT (Water Environment Federation, 2005).

Ballasted flocculation consists of a process of flocculation with added coagulant, polymer and a ballasting agent (generally silica microsand) followed by an operation of clarification with either lamella plate settling or conventional gravity clarification. The microsand serves as the nucleus for the attachment of the destabilized solids so the floc particles develop and grow. The microsand applied for wastewater treatment generally ranges from 100 to 150 mm and features a specific gravity greater than 2.6 to enhance settling (Tchobanoglous et al., 2014).

In table 3.2-4, it is presented the efficiency of removal and the overflow rate of the high-rate clarification processes mentioned above.

Table 3.2-4: Parameter efficiencies for high-rate clarification processes (Tchobanoglous et al., 2014)

Process/Parameter	**Overflow Rate ($m^3/(m^2.d)$)**	**BOD Removals (%)**	**TSS Removals (%)**
Lamella Plate Clarification	1200	35-40	65-75
Ballasted Flocculation	1,800-3,500	40-60	40-80

Lamella plate clarification without prior coagulation/flocculation presents an overflow rate of 10 to 15 m/h at peak flow (Water Environment Federation, 2005).

3.2.4 Mechanical Technologies

Given the new paradigm shift of WWTP, in which energy and resource recovery is considered critical, some technologies surface for various primary treatment applications, such as: microscreening of raw wastewater, charged bubble flotation and primary effluent filtration (Tchobanoglous et al., 2014).

Microscreening provides filtration of raw wastewater downstream of coarse solids removal. This equipment can achieve a removal of BOD and TSS ranging from 25% to 35% and 60% to 70%, respectively, being similar or slightly better than conventional primary sedimentation while possessing a significantly smaller footprint (Tchobanoglous et al., 2014). Some studies demonstrate a TSS removal efficiency higher than 90 % with the addition of chemicals (Ljunggren, 2006).

Multiple publications report the major drawback of microscreening being the clogging of the filter. This problem can be solved with an attentive supervision and correct cleaning procedures (Ljunggren, 2006).

Charged bubble flotation (CBF), used for the treatment of screened raw WW, can replace three unit processes such as: grit removal (except the largest/densest particles), primary clarification and primary scum handling. The CBF process can also be employed as an alternative to CEPT or primary effluent filtration. The CBF is characterized by: a footprint as small as a fifth of the size of a conventional primary clarifier; high solids separation efficiency, being able to handle high concentrations of suspended solids (up to 15,000 mg/L); and low power requirements. This process can achieve a removal of BOD and TSS ranging from 50% to 70% and 70% to 99%, respectively (Tchobanoglous et al., 2014).

Primary effluent filtration (PEF) comprises an effective process of filtration of the primary clarification effluent. This technology can achieve a removal of BOD and TSS ranging from 25% to 35% and 45% to 70%, respectively (Tchobanoglous et al., 2014).

A current promising primary treatment technology, which is already implemented in full-scale WWTP in Norway, is fine mesh sieves.

Paulsrud et al. (2014) conducted a study to compare fine mesh sieves with conventional primary clarifiers. Fine mesh sieve sludges were retrieved from 19 WTTPs in Western and Northern Norway and primary clarifier sludges were acquired from 9 Southern Norway WWTP. In figure 3.2-4, it is presented the fine mesh sieves sludge solids concentration, which can be compared with primary clarifier sludge solids concentration demonstrated in figure 3.2-5.

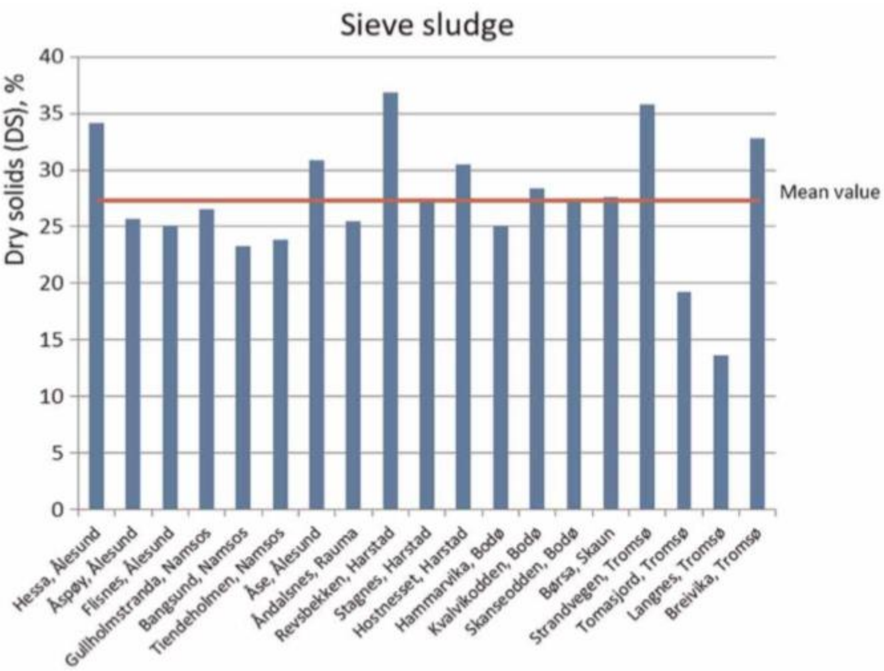


Figure 3.2-4: Fine mesh sieve sludge concentration from various full-scale WWTP (Paulsrud et al., 2014).

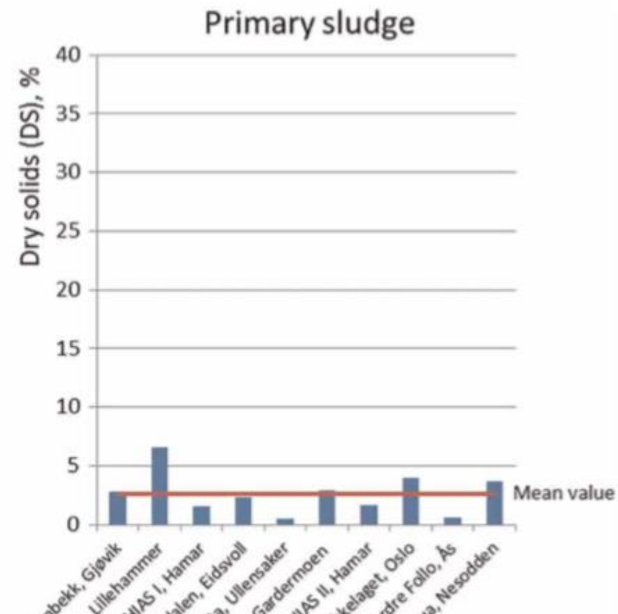


Figure 3.2-5: Primary clarifier sludge from several full-scale WTTPs (Paulsrud et al., 2014).

Fine mesh sieves revealed a mean sludge solids concentration of 27.3 %, while primary sludge from conventional primary clarifier exhibited a mean value of 2.7 %. Additionally sieve sludge presented higher methane potential, due to higher volatile solids content, via biomethane potential tests (Paulsrud et al., 2014).

3.3 Biological Treatment

The biological treatment is commonly the phase downstream of the primary treatment. This phase overall objectives are as follows: oxidize the dissolved and particulate biodegradable constituents into acceptable end products; capture and incorporate the suspended and nonsettleable colloidal solids into biofilm or biological floc; transform or remove nutrients, such as nitrogen and phosphorus, or even specific trace organic constituents and compounds (Tchobanoglous et al., 2014).

The biological treatment via conventional activated sludge reactor, has a low removal efficiency of emerging contaminants such as stimulants (caffeine, nicotine), analgesics (like ibuprofen), pesticides, beta blockers and surfactants (Ahmed et al., 2017).

The removal of dissolved and particulate carbonaceous BOD by oxidation and the stabilization of the organic matter is materialized biologically with microorganisms (bacteria) (Sperling, 2007; Tchobanoglous et al., 2014). Microorganisms provide the biological treatment by feeding off the nutrients present in wastewater, such as carbon, nitrogen and phosphorus. In aerobic treatment, the optimum BOD_5:N:P ratio is 100:5:1. Aerobic microorganisms require oxygen (for respiration) to develop their normal functions (Davies, 2005; Forster, 2003).

This process of organic matter removal, relies on microbial metabolic activity and on organic matter adsorption to the surface of microorganisms (Zhang et al., 2014). During this process, microorganisms produce additional biomass which is removed in the downstream process of secondary settling, since biomass specific gravity is slightly greater than that of water (Sperling, 2007; Tchobanoglous et al., 2014). According to Henze et al. (1997) in sludge originated from domestic wastewater, there is: 80 to 120 g of TN per kg of VSS; 10 to 25 kg of TP per kg of VSS (Henze et al., 1997).

3.3.1 Conventional Activated Sludge

Conventional activated sludge (CAS) is the most common suspended growth biological process for municipal wastewater treatment.

The design and operating parameters for the biological treatment system are the F/M ratio, the organic volumetric loading rate and the SRT or sludge age.

There are several methods for the design of the biological reactor and its diverse configurations. The German standard ATV-DVWK is one of the methods used, characterized by being a conservative approach (ATV-DVWK Specialist Committees, 2000).

In the table 3.3-1, it is presented the design criteria of complete-mix activated sludge reactor.

Table 3.3-1: Complete mix activated sludge (CMAS) design criteria (Tchobanoglous et al., 2014).

SRT or sludge age **d**	**F/M** **kg BOD/(kg MLVSS.d)**	**Volumetric loading** **kg BOD/(m^3.d)**	**MLSS** **mg/L**	**HRT** **h**
3-15	0.2-0.6	0.3-1.6	1,500-4,000	3-6

In a WWTP with biological treatment without nitrification, sludge age should not surpass 5 days for an affluent BOD_5 load of up to 1,200 kg/d or 4 days for an affluent BOD_5 load greater than 6,000 kg/d at 12 °C (ATV-DVWK Specialist Committees, 2000).

The dissolved oxygen (DO) and the oxygen uptake rate in the aeration tank along with the sludge volume index (SVI) and the sludge blanket level in the second clarifier are important operating parameters (ATV-DVWK Specialist Committees, 2000; Tchobanoglous et al., 2014). The SVI and the mixed liquor suspended solids concentration (MLSS) are determinant parameters for the sizing of the biological reactor and secondary settling tanks (ATV-DVWK Specialist Committees, 2000).

Biological reactor or aeration tank (AT) requires aeration to maintain optimal DO concentrations for the development and growth of microorganisms. Do concentration should not be the limiting factor in the biological treatment, therefore DO concentration in the biological reactor should never be inferior to 0.5 mg/L (U.S. EPA & American Society of Civil Engineers (ASCE), 1983). Aeration can be achieved via diffused air aeration or mechanical aeration. For surface slow speed mechanical aerators the oxygen transfer capability ranges from 1.5 to 2.1 kg O_2/kWh (Tchobanoglous et al., 2014).

The biological reactor is followed by a secondary clarifier, which is designed to satisfy the parameters shown in table 3.3-2.

Table 3.3-2: Design parameters information for secondary sedimentation tanks (Tchobanoglous et al., 2014).

Design parameter	Unit	Range
Detention time	h	1.5-2.0
Overflow rate	$m^3/(m^2.h)$	0.8-1.2

3.3.2 Membrane Bioreactor

Membrane Bioreactors (MBR) constitute an alternative to CAS system, that does not require a secondary sedimentation basin since secondary clarification occurs via membrane separation. MBR presents different configurations in terms of materials and pore sizes. The standard configurations are with ultrafiltration hollow-fiber and microfiltration flat plate. Other membrane configurations are nanofiltration (NF) or reverse osmosis (RO) (Arévalo et al., 2012; Judd, 2010).

MBRs do not portray only advantages. This technology is less sustainable than CAS, presenting higher energy consumption as well as elevated operation costs. WWTP in the Netherlands have been switching their MBR installations into CAS due to its costs and energy consumption: 0.26 €/m^3 (58% higher than that of single CAS) and 0.77 kWh/m^3 (114% higher). This technology, however, can be advantageous considering stringent discharge permit limits or space restrictions (Hao et al., 2018; Judd, 2010).

3.4 Sludge Thickening

Thickening is a process conducted to increase the solids content of the sludge by eliminating part of its liquid fraction. It is characterized by being a physical procedure and occurs by co-settling, settling, flotation, centrifugation and drainage (by a gravity belt or a rotary drum screen thickener) (Degrémont, 1991; Tchobanoglous et al., 2014).

Gravity thickeners are designed for:

1. Overflow rate of: 15.5 to 31 $m^3/(m^2.d)$ for primary sludge; 4 to 8 $m^3/(m^2.d)$ for waste activated sludge; and 6 to 12 $m^3/(m^2.d)$ for combined sludge (Qasim, 1999).
2. Solids capture of: 85 to 98 % for primary sludge; 60 to 85 % for waste activated sludge; and 85 to 92 % for combined sludge. (Qasim, 1999)
3. Solids Loading (table 3.4-1).

In the next table it is presented the sludge solids concentration that results from a gravity thickener, as well as the solids loading for design purposes.

Table 3.4-1: Typical solids concentration of sludges and solids loading for gravity thickeners (Water Environment Federation, 2011).

Type of sludge	Solids Concentration, %		Solids Loading, kg/(m^2.d)
	Unthickened	Thickened	
Combined primary and waste activated sludge	0.5-1.5	2-6	25-70
	2.5-4	4-7	40-80
Primary sludge	1-6	3-10	100-150
Chemical sludge with alum	0.5-1.5	2-4	10-50
Chemical sludge with iron	0.5-1.5	3-4	10-50

As shown in table 3.4-1, waste activated sludge, as well as, chemical sludge do not thicken with ease by gravity. For these types of sludge, thickening via dissolved air flotation (DAF) or mechanical thickening are considered more appropriate (Water Environment Federation, 2005).

Mechanical thickeners, in comparison with gravity thickeners, require higher energy costs and maintenance, but allow for a superior thickening of the sludge. Mechanical equipment is eligible when space is a limitation (Qasim, 1999).

Thickening reduces sludge volume which consequently lessens further sludge processing costs. This ensues a smaller digestion volume required as well as diminished heating necessities and less volume of sludge for disposal (Puchajda & Oleszkiewicz, 2008).

3.5 Anaerobic Digestion

Anaerobic digestion main objective is to provide sludge stabilization and volatile solid destruction to reduce pathogens, eliminate and inhibit offensive odors and reduce or eliminate the potential for putrefaction (Cao & Pawłowski, 2012).

In domestic wastewater between 60 to 80 % of the total suspended solids (TSS) are volatile suspended solids (VSS) (Henze et al., 1997). A conventional anaerobic digester can achieve a reduction in total volatile solids in the range of 50% to 60% (Qasim, 1999). VSS destruction is critical for the reduction of sludge volume, which consequently decreases the cost of disposal (Arnaiz et al., 2006).

The Water Environment Federation suggests a formula to calculate the maximum VS destruction percentage value, depending on the hydraulic retention time (Water Environment Federation et al., 2012). The WEF formula is the following:

$$\text{VS destruction } (\%) = 13.7 \times \ln(\text{HRT}) + 18.9$$

During anaerobic digestion, four chemical and biochemical reactions occur: hydrolysis, acidogenesis, acetogenesis and methanogenesis. This process is developed in the absence of oxygen resulting in the decomposition of organic matter and in the reduction of inorganic matter with the end products being stabilized sludge and the production of methane gas and carbon dioxide (Tchobanoglous et al., 2014). Hydrolysis is usually the rate limiting phase (Appels et al., 2008).

In the figure 3.5-1, the anaerobic digestion reactions are represented by sequential order.

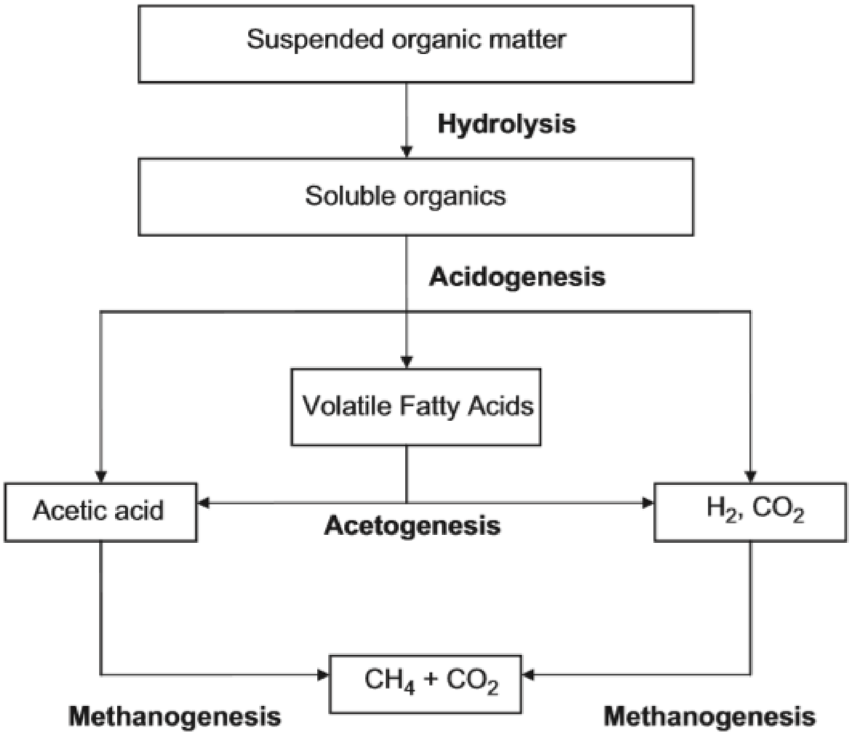


Figure 3.5-1: Anaerobic digestion reactions (Appels et al., 2008).

The typical biogas production in anaerobic digesters ranges from 0.8 to 1.1 m^3/kg volatile solids destroyed (Water Environment Federation. et al., 2012).

The biogas produced can be utilized to produce thermal and electrical energy in a cogeneration unit, as its composition in methane ranges from 60 to 65% (Tchobanoglous et al., 2014).

Pre-treatment can be ensued in order to make organic matter more responsive to utilization by acidogens and methanogens (Li & Yu, 2016). Biological pre-treatment is conducted by enhancing the hydrolysis process in an additional stage before the main digestion. A thermophilic pre-treatment (2 days of HRT) in comparison with a mesophilic pre-treatment was studied prior to mesophilic anaerobic digestion revealing an increase on methane production and solids destruction of 25 % (Carrère et al., 2010; Kalogo & Monteith, 2013).

Thermal hydrolysis (temperatures over 100 ºC) has proved to be successful as pre-treatment techniques. According to Pilli et al., (2015), via lab scale studies, the optimum pre-treatment conditions for enhanced biogas production are 160 to 180 ºC for 30 to 60 minutes. Full-scale studies demonstrated that high temperature pre-treatment reduce sludge volume and increase biogas production and sludge dewaterability. At temperatures over 190 ºC, biogas production decreases due to the formation of toxic refractory compounds, that decrease sludge biodegradability (Pilli et al., 2015).

Ozonation, sonication and mechanical shear are also presented as pre-treatment techniques that cause sludge disintegration resulting in an increase of the bioavailability of the sludge being digested (Carrère et al., 2010; Kalogo & Monteith, 2013; Water Environment Federation. et al., 2012).

According to Tchobanoglous et al. (2014), Lettinga (1995) and Water Environment Federation et al. (2012), the factors that determine optimal conditions of the process are the following:

1. SRT and HRT, being directly proportional to the extent of each reaction. Sufficient residence time allows the bacteria to grow enabling the process of digestion and the destruction of VSS.

   In table 3.5-1, it is presented the design SRT values, which are the same as HRT for complete-mix digesters (Tchobanoglous et al., 2014), and the operating temperature of the reactor.

Table 3.5-1: suggested SRT for mesophilic complete-mix anaerobic digesters (Tchobanoglous et al., 2014)

Operating Temperature (ºC)	**Minimum SRT (d)**	**Design SRT (d)**
18	11	28
24	8	20
30	6	14
35	4	10
40	4	10

In practice for a complete-mix mesophilic digester SRT values generally range from 15 to 20 days, providing adequate solids stabilization (Qasim, 1999). Although, SRT values of 10 days are sufficient to ensure the methanogenic population doesn't suffer from washout, SRT values above 15 days show relatively small incremental changes in volatile solids destruction.

1. Temperature, responsible for determining the rate of digestion, especially the rate of hydrolysis and methanogenesis. Mesophilic digesters operate between 30 to 35ºC, while thermophilic digesters operate in the range of 50 to 57ºC. Thermophilic digestion is characterized by being advantageous, having an increased efficiency and improved dewatering (Qasim, 1999).

   Maintaining stable temperatures is crucial for the bacteria, particularly the bacteria responsible for the production of methane. Temperature variation greater than 1ºC/d in sludge temperature affects performance.

2. Volatile solids (VS) loading for sustained conditions should be in the range of: 3.2 to 6.4 kg VS/(m^3.d) for thermophilic anaerobic digestion; 1.6 to 2.4 kg VS/(m^3.d) for mesophilic anaerobic digestion.
3. Alkalinity, used to monitor the health of the digestion process via the ratio of volatile acids to alkalinity, which should be in between 0.05 to 0.25. Volatile acids and carbon dioxide consume alkalinity, a well-established digester has a total alkalinity of 2000 to 5000 mg/L.

4. pH, should be maintained at 6.8 to 7.8 for the occurrence of stable methanogenic activity. If the pH drops below 6 the methane formation ceases (Qasim, 1999).
5. Presence of inhibitory substances, which disrupt the process of digestion. These substances are certain heavy metals, high nitrogen concentration from ammonia, chlorinated organic compounds, amino acids and industrial chemical products.
6. Bioavailability of nutrients and trace metals, which enhance biological growth and consequently improve the process of digestion.

Another criterion for adequate operation of the AD process is mixing. This aspect is critical to provide even distribution of microorganisms, organic matter, inoculation of fresh feed, temperature and the homogenization of the sludge inside the reactor (Lindmark et al., 2014).

Mixing can be achieved by mechanical mixing (via propellers and agitators), hydraulic mixing (by recirculation of AD sludge) or pneumatic mixing (by pumping biogas to the bottom of the reactor, promoting the mixing of its contents as the bubbles ascend to the surface). Mechanical mixing is usually the most power efficient (Lindmark et al., 2014).

3.6 Sludge Dewatering

Dewatering consists of a physical unit operation that separates solid matter of sludge or biosolids from water. This process produces a high solids content stream called “cake” and a liquid stream designated as centrate (Tchobanoglous et al., 2014). Dewatering achieves a superior volume reduction than that attained with thickening, resulting in posterior reduced costs of handling and of management of the sludge. Dewatering processes lead to sludge solids concentration from 4 to 20 % (Turovskiy & Mathai, 2006).

Municipal WWTP sludge generally presents a negative value of zeta potential, meaning sludge particles are negatively charged resulting in its electrostatic repulsion and inhibiting particle aggregation. The addition of correct dosages of cationic polymers or inorganic flocculants (at a suitable pH) promote charge neutralization, eliminating the electrostatic repulsion and encouraging particle aggregation (Tuan et al., 2012). Conditioning of the sludge is most commonly materialized with coagulation of colloids (Novak, 2006).

Sludge chemical conditioning is usually via polyelectrolytes, Fe, Al or lime to promote floc development and consequent improved dewaterability (Chen et al., 2006). Typical chemical conditioning doses of ferric chloride and lime for anaerobically digested sludge range from 30 to 50 kg/1,000 kg and 100 to 130 kg/1,000 kg of dry solids respectively. These inorganic chemicals increase sludge mass by 15 to 30 %, increasing disposal costs and the sludge presents less fuel value for incineration. Conversely polymers do not increase sludge mass nor reduce sludge energy potential for incineration and can be dosed in much lower quantities (Krishnamurthy & Viraraghavan, 2005; Novak, 2006; Sharma & Sanghi, 2013). Polymer doses are based on the centrate clarity obtained, being dependent on sludge type, polymer type and equipment type. In-situ empirical tests are critical for the determination of the optimum dose (Murthy et al., 2004).

The use of inorganic polymers in the dewatering process results in sludge with 2 to 5 % higher solids content in comparison with organic polymers (Andreoli et al., 2007).

Thermal conditioning is another possible method (Chen et al., 2006). Moreover, acoustic conditioning with ultrasounds via ultrasonic vibrations and magnetic conditioning are reported as promising technologies (Sharma & Sanghi, 2013).

Sea water intrusion can result in lower dewaterability of the sludge. Additionally sludge storage, or elevated residence time, in the anaerobic digester lowers posterior dewaterability as flocs disintegrate and conductivity increases (Christensen et al., 2015; Tuan et al., 2012).

The criteria for the selection of the dewatering equipment is (Water Environment Federation, 2008):

- Type and quality of the sludge (concentration of the feed sludge);
- Mode of operation (continuous, discontinuous and capable working hours per day);
- Polymer cost (dose required and unit price);
- Cost of sludge disposal or downstream processing;
- Cost of recycle (relative to the capture of solids performance of the equipment);
- Feed rate, as in solids loading mass of dry solids per hour and hydraulic loading to take profit of the maximum throughput of the equipment installed.

In the table 3.6-1, it is represented the performance of most common dewatering equipment.

Table 3.6-1: Performance data of dewatering technology for anaerobically digested sludge (Primary + WAS) (Turovskiy & Mathai, 2006).

Equipment type	**Feed solids concentration (%)**	**Conditioning Chemicals**			**Cake solids concentration (%)**	**Solids capture (%)**
		Polymer dosage (g/kg dry solids)	FeCl3 (%)	Lime (%)		
Solid Bowl Centrifuges	2 - 6	3 - 10	-	-	15 - 27	85 - 98
Belt Filter Press	3 - 6	3 - 8	-	-	20 - 25	90 - 98
Filter Press	6 - 8	-	5	10	20 - 45	90 - 98

The mechanical dewatering equipment that produces a higher solids content cake is the filter press, followed by centrifuges and belt presses. The filter press operates in a discontinuous manner (working in cycles of 3 to 6 h) and provides a cake 6 to 10 % dryer (Andreoli et al., 2007).

The most common equipment are centrifuges and belt filter presses. Centrifuges in comparison with belt filter presses present a simple, confined compact process with a more efficient odor control, less frequent cleaning requirements and lower water consumption (Cheremisinoff, 2002). Belt filter presses portray a lower capital cost, lower power consumption, are quieter and aren't deemed of expert maintenance. However, centrifuges often achieve a higher sludge cake solids concentration at a lower polymer consumption rate (Mamais et al., 2009).

In the table 3.6-2 and 3.6-3, it is represented the total annual costs of the dewatering process via belt filter press and centrifuge in the WWTP of Volos and Lavrio, Greece. The WWTP of Volos is a medium to large size plant with 130,000 PE (population equivalent), while the WWTP of Lavrio is a small to medium facility with 10,000 PE.

Table 3.6-2: Annual sludge dewatering total costs of a medium – large sized WWTP (capital, operation and maintenance) of Volos WWTP in Greece (Mamais et al., 2009).

Annual costs €/y	**Belt Filter Press**		**Decanter Centrifuge**	
	50 h/week	**100 h/week**	**50 h/week**	**100 h/week**
Capital cost	11,583	23,167	15,959	31,918
Chemicals/Reagents	257,242	257,242	195,149	195,149
Water consumption	22,176	22,176	53	53
Power	2,883	2,883	14,256	14,256
Labor	47,520	23,760	23,760	11,880
Maintenance	4,500	7,700	3,100	6,200
Centrate/filtrate treatment	23,319	23,319	6,088	6,088
Sludge disposal	105,600	100,800	105,600	100,800
Total Cost	**474,823**	**465,846**	**359,165**	**366,403**
Total Cost per ton of sludge	**108**	**106**	**82**	**84**

Table 3.6-3: Annual sludge dewatering total costs of a small – medium sized WWTP (capital, operation and maintenance) of Lavrio WWTP in Greece (Mamais et al., 2009).

Annual costs €/y	**Belt Filter Press**	**Decanter Centrifuge**
	30 h/week	30 h/week
Capital cost	5,457	11,120
Chemicals/Reagents	35,171	26,522
Water consumption	4,752	53
Power	675	2,851
Labor	14,256	7,128
Maintenance	1,060	2,500
Centrate/filtrate treatment	5,366	1,378
Sludge disposal	16,958	13,104
Total cost	**83,695**	**64,657**
Total cost per ton of sludge	**147**	**114**

At an economic standpoint the long-term analysis proves that centrifuges have lower overall life cycle costs, despite having an higher capital cost (Mamais et al., 2009).

3.7 Energy Recovery

Worldwide development in an economic and social level is generally followed by cumulative amounts of waste generated, meaning losses in terms of materials and energy as well as damage to the environment which impact health and quality of life. Waste management is a critical topic that requires an attentive approach. Fossil fuels are hastily diminishing, and energy consumption is increasing. Renewable energies are considered to be crucial for future development (Lema & Suarez, 2017).

Energy can be retrieved from wastewater. The energy content of wastewater is expressed as: thermal energy and chemically-bound energy of the organic matter. Chemically-bound energy portrays little losses via the sewer system, while thermal energy reuse presents elevated losses and consequently the need of its reuse as close to the source (Nowak et al., 2015).

It is estimated that municipal wastewater contains, in terms of energy, approximately 23 W/capita in organic carbon, 6 W/capita in ammonium-N and 0.8 W/capita in phosphate-P (Dai et al., 2015; Gao et al., 2014).

WWTP are big energy consumers, the average energy consumption that occurs in WWTP in Germany, the United Kingdom and the United States is 0.67, 0.64 and 0.45 kWh per cubic meter of treated wastewater. In Italy this benchmark ranges from 0.4 to 0.7 kWh/m^3 and in a sample of 177 Spanish WWTP with extended aeration the average value is 0.82 kWh/m^3 (Guerrini et al., 2017). In France CAS WWTP with a population equivalent superior than 50,000, have an energy consumption that ranges from 1.5 to 2.5 kWh/kg BOD_5 for carbon removal and 2.5 to 3.5 kWh/kg BOD_5 for nutrient removal (Lazarova et al., 2012).

Large WWTP present significant economies of scale in comparison with smaller plants. When it comes to energy usage, large WWTP are more energy efficient (Molinos-Senante et al., 2018).

Wan et al., (2016), estimates the energy consumption in conventional CAS process ranges from 0.3 to 0.6 (averaging at 0.45) kWh/m^3. As an alternative consumption in CAS process can be determined via affluent load as 3.2 kJ/g COD or 0.896 kWh/kg COD for an affluent wastewater with a concentration of roughly 500 mg/L of COD, which represents run-of-the-mill domestic wastewater. According to Guerrini et al., (2017), a WWTP with a capacity of over 100,000 PE presents an energy consumption of 0.85 kWh/kg COD, validating and supporting previous studies (Guerrini et al., 2017; Wan et al., 2016).

It is estimated that conventional activated sludge process in a WWTP requires from 0.3 to 0.65 kWh per cubic meter without nitrification. If nitrification is required, the energy needs become higher (Gikas, 2017).

Aeration in a CAS system constitutes roughly 60% of the total energy consumption of a WWTP, while the energy consumption of the sludge treatment ranges from 15% to 25% and secondary sedimentation along with the recirculation pumps consume 15% (Gu et al., 2017b; Guerrini et al., 2017; Rieger et al., 2012).

Energy expenditure varies with the WWTP configuration. In the figure 3.7-1, it is presented the typical energy consumption of a typical conventional WWTP having a population equivalent of 400,000.

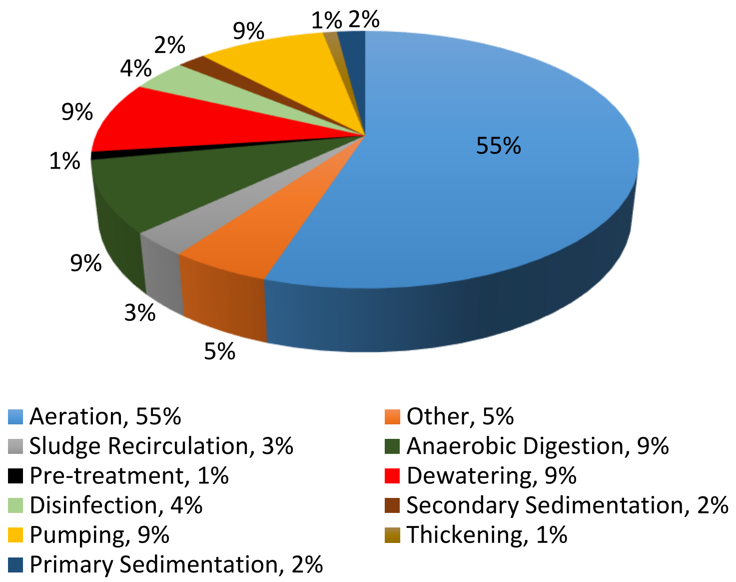


Figure 3.7-1: Average energy expenditure in a conventional wastewater treatment plant with CAS (Stamatelatou & Tsagarakis, 2015).

Reliable dissolved oxygen (DO) sensors were introduced in 1970s. This equipment improved aeration efficiency significantly. The control of DO to a set-point under performs in terms of aeration efficiency in comparison with the utilization of DO and ammonia sensors or even time based-control (Olsson et al., 2005).

3.7.1 Methods for Energy Neutrality

The current energy recovery methods that can be applied to a WWTP, are represented in figure 3.7-2.

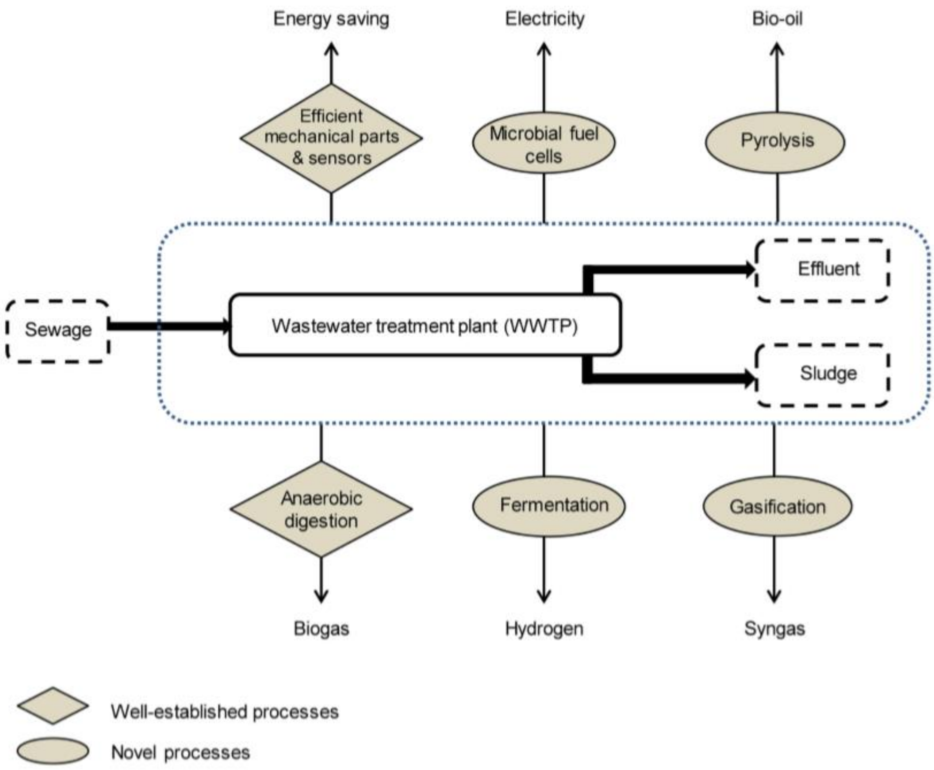


Figure 3.7-2: Processes capable of recovering energy within a WWTP, either by saving it and/or producing it (Stamatelatou & Tsagarakis, 2015).

Pyrolysis consists of a more environmentally friendly incineration of sewage sludge, which results in toxic-free byproducts due to the absence of oxygen and lower operating temperatures. Its large-scale application is limited since it requires complex and expensive equipment. Fermentation and gasification are also other technologies for energy recovery (Stamatelatou & Tsagarakis, 2015).

The focus of energy recovery from organic carbon in wastewater has been on anaerobic treatment and bioelectrochemical systems (Gao et al., 2014).

3.7.2 Anaerobic Digestion

The shift from aerobic to anaerobic treatment of municipal wastewater presents a feasible opportunity for self-sustained or even net-positive energy facilities. Anaerobic processes avoid the intensive energy expenditure of aeration and allow positive energy outputs (Li & Yu, 2016).

In anaerobic treatment, energy can be retrieved in the form of methane (CH_4), which can be further used in electricity generation. It is considered that roughly half of the biodegradable COD can be anaerobically converted to methane under optimal conditions (Nowak et al., 2015).

The methane present in biogas ranges from 60% to 70%, the other 30% to 40% of the gas composition is CO_2 and other trace gases. Biogas can be utilized for heat, power or combined heat and power (CHP). Energy produced is utilized in the digester process for its energy demands, to maintain the elevated temperature of the reactor. The surplus energy is eligible for electricity generation, direct combustion for heating purposes, or to supply fleet vehicles by means of its compression (Ma et al., 2015).

CHP systems require a considerable volume of biogas, which limits its use to large WWTP only. According to Bastian et al. (2011), CHP systems are only cost-effective in wastewater facilities with a flow rate superior to 19,000 m^3/d. Electricity generation potential averages 350 kWh per 3,800 m^3/d of wastewater treated (Mo & Zhang, 2013). As reported by Monte (2010), the conversion of biogas to energy is only cost-effective in sizeable WWTP with a population equivalent (PE) superior to 35,000.

In the US, 94% of the operating WWTP have a flow rate inferior to 19,000 m^3/d (Mo & Zhang, 2013). In this context, currently less than 0.6 % of all operating WWTP in the US utilize biogas to produce energy (Bastian et al., 2011).

There are five types of equipment used in CHP systems, such as gas turbines, micro-turbines, steam turbines, reciprocating engines and fuel cells. The various appliances vary in terms of power and heat generation capacity (Gude, 2015)

AD processes allow the simultaneous recovery of energy, through biogas, and also the production of fertilizer with the AD digestate (Lema & Suarez, 2017).

An efficient separation of primary solids, prior to the biological treatment, before AD has shown positive results in order to improve the energy balance of the WWTP, due to more partitioning of the organic matter to the sludge phase (Carlsson et al., 2016; Li & Yu, 2016).

Advanced AD processes have proven to achieve greater energy recovery (up to 60%) with higher biogas production and its methane content as well as a more stable process. Some of the advances in AD are thermal/high pressure pre-treatment of the sludge and co-digestion of the sewage sludge with other biodegradable materials such as food wastes (FW) and fats, oils, and grease (FOGs) (Lema & Suarez, 2017).

Thermal sludge pre-treatment requires energy, but the surplus biogas production is an offset as more energy is gained and a reduced sludge volume for disposal is obtained via enhanced sludge dewaterability (Han et al., 2017).

3.7.3 Co-Digestion

Co-digestion is a process of digestion of sludge with added organic wastes, with the intent of taking full advantage of WWTP infrastructure, increasing biogas production and receiving monetary revenue from treating receiving organic wastes (Water Environment Federation. et al., 2012).

The population served by WWTP is stale or decreasing meaning some WWTP are over dimensioned. This happens in Germany, a country with 1,400 operating anaerobic digesters. Additionally, WWTP are designed to be capable of treating 20 to 30% more of expected. As a result, anaerobic digesters are low loaded and thus not very efficient. For this reason there is spare volume in the digesters to implement the anaerobic co-digestion of sludge with organic wastes to maximize energy recovery (Mattioli et al., 2017).

Co-digestion of the sewage sludge with organic-rich wastes boosts the carbon concentration and improves the carbon/nutrient (C/N) ratio, increasing biogas yield, enhancing sludge digestibility and consequently improving energy balance (Kim et al., 2017; Li & Yu, 2016).

The ideal co-substrates for co-digestion are high COD content wastes, as they present elevated organic loading rates at low volumetric loading, allowing the digester to operate only at a slightly lowered HRT/SRT, provided that COD and VS destruction does not diminish below set value (Tandukar & Pavlostathis, 2015). Anaerobic digesters perform optimally, when the added substrate material is little to no recalcitrant (Zamanzadeh et al., 2017).

The organic waste usually utilized as co-substrate is food waste (FW), from industrial processes or restaurants, FOG (fats, oils and grease) and organic fraction municipal solid waste (OFMW). FOG is one of the co-substrates which provide higher methane production, although its usage should be performed with caution as loadings greater than 30% of volatile solids may present process instability and without appropriate surface mixing, FOG will accumulate at the top of the digester (Water Environment Federation. et al., 2012). An addition of greasy sludge, from the meat industry, greater than 50% of volatile solids causes the accumulation of long chain fatty acids and potential sludge floatation resulting in the inhibition of the AD process (Budych-Gorzna et al., 2016).

The meat industry wastes consist of FOG concentrated organic wastes. FOG presents a high COD content, high biodegradability and a COD to methane conversion of 81.1% (Tandukar & Pavlostathis, 2015). The addition of 13% in total organic loading of greasy sludge, from the meat industry, as co-substrate proved to increase specific biogas production from 0.38 m^3/kg VS to 0.49 m^3/kg VS, providing almost a 30% boost in biogas production (Budych-Gorzna et al., 2016).

Utilizing FW as co-substrate in anaerobic digestion accelerates methane production rates and increases the methane yield, mainly by enhancing C/N ratio (Kim et al., 2017). Municipal wastewater biosolids present a C/N ratio of 6:1 to 9:1, while the optimal C/N ratio for AD is 15:1 to 30:1. According to the batch tests performed by Koch et al. (2016), FW from canteens present an average C/N ratio of 17.7, which was in line with other studies conducted (Kim et al., 2017; Koch et al., 2016). According to Parra-Orobio et al. (2016) co-digestion with 20% organic municipal wastes resulted in a C/N ratio in the range of 22.6 to 25.8 (Parra-Orobio et al., 2016).

According to Koch et al. (2016), the usage of FW as co-substrate is recommended, as performing co-digestion with 10% FW, could enhance energy production from 25% to 78%. Co-digestion with FW proved to cause a higher methane yield and to accelerate methane production, even with an increment of up to 35% in volatile solids loading (Koch et al., 2015; Koch et al., 2016).

Co-digestion is popularly used in digesters with unutilized volume capacity. Co-digestion was implemented in two WWTP of Austria: WWTP of Zirl demonstrated 110% energy self-sufficiency; WWTP of Strass im Zillertal, which was already energy self-sufficient, proved to be a public power supplier with 160% energy self-sufficiency (Insam & Markt, 2016).

In Rovereto, Italy, WWTP AcoD (anaerobic co-digestion) was implemented for 1 year to provide data on its effectiveness for energy recovery. Rovereto WWTP was designed for a capacity of

95,000 PE, and with the co-digestion of 10.6 ton/d of municipal organic waste, the biogas production doubled. The increase of the organic loading from 0.73 kg VS/m^3 to 1.38 kg VS/m^3 resulted in a biogas production boost from 1321 m^3/d to 2723 m^3/d, increasing the expected energy production from 4000 to 8100 kWh/d (Mattioli et al., 2017).

Although subject to digester capacity, energy recovery could be improved from 15 to 18 kWh per person per year to 30 kWh per person per year with co-digestion (Mattioli et al., 2017).

Kim et al., (2017), accredits co-digestion efficiency to the increase in biodegradability and not to the C/N ratio. According to lab-scale tests it was demonstrated that co-digestion with FW as co-substrate provided a 37% increase in degradation rate and an amplification of at least 18% in methane production rates, resulting in an enhancement of COD and VS removal from 39% to 53% in a 15 day SRT digester (Kim et al., 2017).

In a WWTP with 100,000 PE, co-digestion of primary sludge and thickened waste activated sludge with 15% FW increased volumetric COD loading by 56% and methane production by 100%, while only raising digested volatile solids by 2.9% (Kim et al., 2017).

Co-digestion is a complex process and along with the SRT and the volatile solids loading, the most important process control parameters are the specific energy loading rate (SELR) and the five-day biodegradable energy conversion (BEC_5). Both parameters require further investigation, although for SELR the optimum value seems to be 950 kJ/(kg.d) for mesophilic digesters (Water Environment Federation. et al., 2012).

The addition of organic co-substrate to the anaerobic digesters, affects the microbial population and activity and thus may explain the effectiveness of co-digestion, although investigation on this topic is required (Kim et al., 2017).

When implementing co-digestion, both the advantages and disadvantages should be clear. Some studies have reported the return of nitrogen to the water treatment phase as well as the decreased dewaterability of the digested sludge (Mattioli et al., 2017). Additionally undesirable suspended impurities like glass, metal or sediments, from the co-substrates, may cause operational failures, decrease the usable digester volume and increase the maintenance required (Aichinger et al., 2015).

3.7.4 Microbial Fuel Cells

Besides AD, bioelectrochemical systems are currently regarded as state-of-the-art technology, which present a feasible opportunity in pushing WWTP to the circular economy (Lema & Suarez, 2017).

Microbial fuel cells (MFCs) are presented as a competitive promising technology, of bioelectrochemical systems, that captures the energy potential of the dissolved organic fraction of the wastewater, converting biological energy to electricity (Ma et al., 2015). Power generation with MFCs ranges from 1 to 3600 MW/m^2, with most WWTP generating 10 to 100 MW/m^2. Excess sludge is also reduced to 20% in comparison with conventional treatment, lowering sludge disposal costs (Mo & Zhang, 2013).

In anaerobic digestion the production of electricity via biogas (methane) utilization in CHP, presents an efficiency of only 30% to 40%. It is expected that MFCs will have an higher efficiency of 50% (McCarty et al., 2011). MFCs also allows for the recovery of nutrients present in wastewater. All things considered fundamental research is still required to further advance and improve this technology (Lema & Suarez, 2017).

3.7.5 AB Process

The AB process is also an effective wastewater treatment method that can significantly improve energy recovery and allow for a net-positive energy output. It consists in the optimization of COD capture prior to biological oxidation in A-stage, reducing the energy consumption in B-stage.

The A-stage is comprised of processes that achieve a 60 % COD capture. The systems considered in A-stage are: CEPT; HRAS process; and anaerobic process. For B-stage, considering the requirement for N removal, only shortcut nitrification-denitrification and partial nitrification combined with anammox processes would meet the criteria due to decreased affluent COD to the B-stage (Wan et al., 2016).

When CEPT functions as A-stage, the soluble COD affluent to the B-stage will be high enough resulting in the growth inhibition of anammox bacteria versus heterotrophic denitrifying bacteria. Thus shortcut nitrification-denitrification should be considered for B-stage (Wan et al., 2016).

3.8 Water Reclamation

Worldwide water consumption is gradually increasing, as a result of increased population and the development of certain activities, such as agriculture and industry expansion. Additionally, water scarcity is a serious concern that affects a big percentage of the world's population (Roccaro, 2018; UNESCO, 2018).

Global water consumption per sector is represented in the figure 3.8-1.

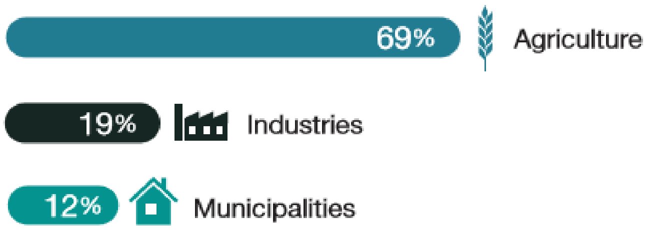


Figure 3.8-1: Global water withdrawal by sector (Food and Agriculture Organization, 2014).

In a smaller scale, Portugal water usage by sector is represented in the figure 3.8-2.

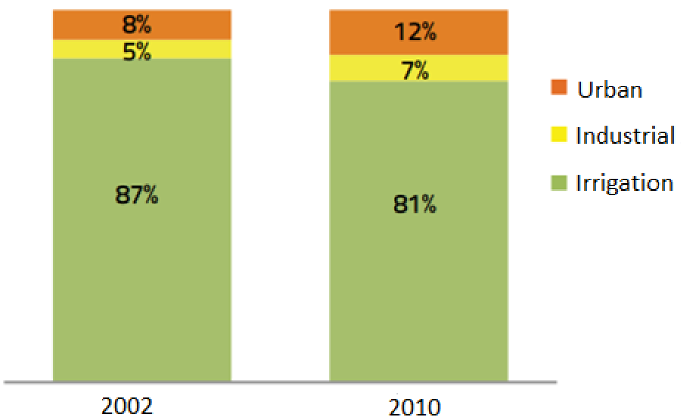


Figure 3.8-2: Water consumption by sector in Portugal in the year of 2002 and 2010 (Baptista et al., 2012).

In comparison with global average, Portugal stands out as a more agricultural focused country with reduced industrial water consumption. As stated previously water consumption is increasing, as seen in figure 3.8-2, in urban and industrial sectors.

India uses around 80% of the available water resources in the agricultural sector. The water comes predominantly from groundwater wells. Maintaining this constant water withdrawal rate, it is estimated that by 2050 India available water supplies will be depleted (Eslamian & Eslamian, 2016).

In Spain, 87% of total fresh water usage, could be replaced by reclaimed water, accounting for irrigation (68 %), refrigeration (14 %) and industrial applications (5 %). Urban consumption only amounts to 13% of total water consumption (Meneses et al., 2010)

Tertiary treatment of wastewater in conventional WWTP post biological treatment and settling, is being considered for water reuse in agriculture. This is standard practice in Spain, in the Valencian region due to water scarcity (Illueca-Muñoz et al., 2008). Wastewater can be reclaimed by membrane treatments, ion exchanges and electrolysis processes (Degrémont, 1991).

In figure 3.8-3, it is presented the difficulties associated with the removal or treatment of certain pollutants found in wastewater.

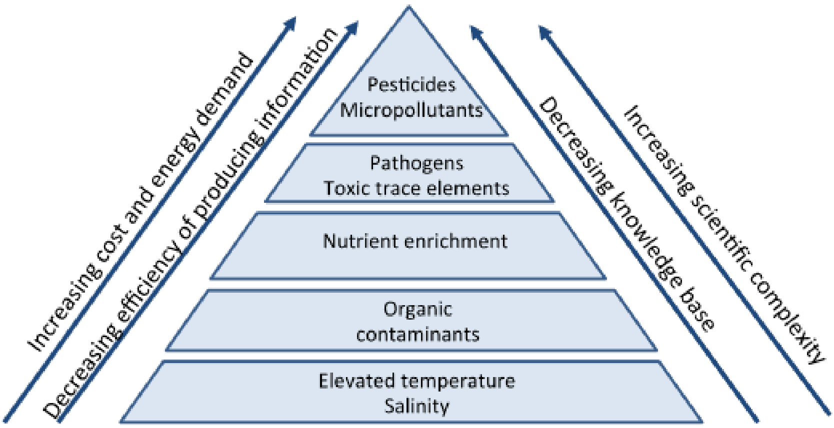


Figure 3.8-3: Wastewater treatment complexity for obtaining high-quality water (Lema & Suarez, 2017).

The monitoring and subsequent treatment of emerging pollutants is limited by scientific complexity (Lema & Suarez, 2017).

Tertiary treatment processes can be divided into two: methods that remove salts (nanofiltration, reverse osmosis, electrodialysis), which if water reclamation is intended for irrigation may be essential for crop development; and processes that do not remove salts, not interfering with conductivity of the wastewater. Salts removal in tertiary treatment is usually not required unless the WWTP is localized in coastal areas where sea water infiltration occurs (Illueca-Muñoz et al., 2008).

When reclaiming wastewater to provide turfgrass or landscape irrigation, there are benefits of using recycled water as turfgrass has high tolerance for some water components such as salinity and nutrients. In Tunisia, golf courses have been irrigated with secondary effluents of WWTP for more than 20 years and the turfgrass maintains its high-quality, with no adverse effects (Lazarova & Bahri, 2005).

3.8.1 Membrane Filtration

Membrane processes for water and wastewater reclamation are advantageous in comparison with conventional physicochemical treatments, since the production of high quality water is not dependent on the characteristics on the affluent feed water (Ordóñez et al., 2014). Membrane separation processes are pressure driven systems classified by pore size (Wintgens et al., 2005). Membrane filtration methods and respective substance removals are represented in figure 3.8-4.

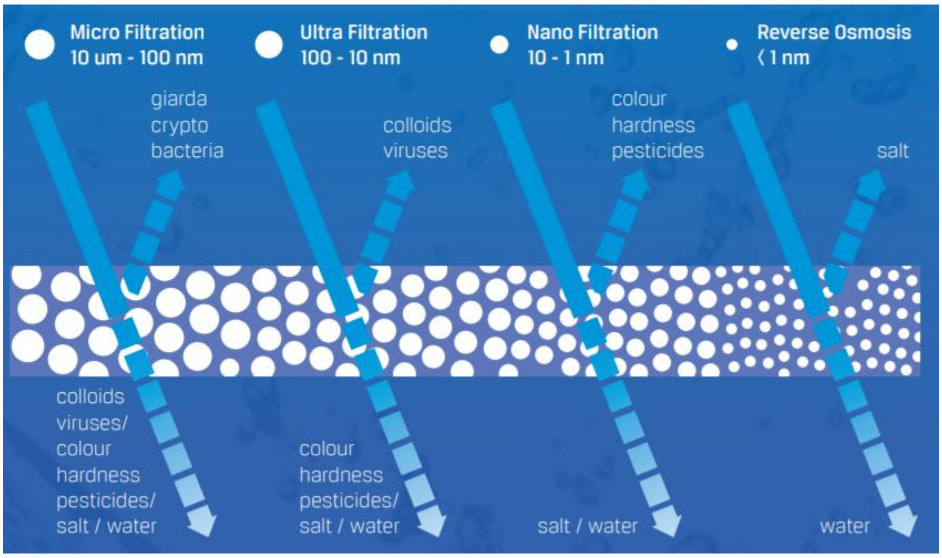


Figure 3.8-4: Membrane filtration types and applications (European Commission, 2010).

Microfiltration removes bacteria and suspended solids while ultrafiltration provides an additional removal of viruses. Nanofiltration contributes to the retention of multivalent ions (example: Ca^{2+}, Mg^{2+}) while reverse osmosis is also responsible for removing monovalent ions (example: Na^{+}, Cl^{-} ,NO_3^{-}) (Vaseghi et al., 2016).

3.8.1.1 Microfiltration

Microfiltration (MF) being the coarsest membrane is still capable of removing particulate matter (Judd, 2010). Microfiltration as well as ultrafiltration are the desired methods for the elimination of suspended solids and microorganisms (Ordóñez et al., 2014).

MF does not retain most viruses due to the large pore size. Nevertheless, viruses tend to bind to other solids. Increments to membrane filter clogging also increase bacteria retention factor to 10^4 and 10^2 for viruses. An additional disinfection stage is still required for constant anti-septic water quality (Wintgens et al., 2005).

3.8.1.2 Ultrafiltration

Ultrafiltration (UF) applied to the secondary clarifier effluent is presented as a feasible option for water reclamation and to provide irrigation. This process achieves removal efficiencies of 50% COD, 60% TOC, 30% TP and 100% for turbidity and phenols. Ammonia removal is not significant (Căilean et al., 2015).

Most of the natural organic matter (NOM), turbidity, manganese, iron and bacteria can be removed in UF. Nonetheless, depending on its usage, it is recommended a follow up process of disinfection for a more complete removal of contaminants (Eslamian, 2016).

An agri-food industry WWTP in Italy, whose affluent wastewater was produced mainly from vegetables processing, with only 5 to 10% being relative to faecal pollution, comprehended a tertiary treatment comprised of ultrafiltration followed by UV disinfection. This WWTP was evaluated for water reuse in irrigation and the results in crop development were similar with reclaimed water and conventional well water (Vergine et al., 2017).

3.8.1.3 Nanofiltration and Reverse Osmosis

Reverse osmosis (RO) and nanofiltration (NF) are membrane processes utilized in tertiary treatment for obtaining high water quality to meet the most stringent reuse water quality requirements (Jacob et al., 2010).

NF removes organic and inorganic constituents, bacteria and viruses. Its effluent only demands slight disinfection. RO portrays superior removal rates (including the elimination of salts) and is used for obtaining potable water from wastewater for groundwater recharge as well as to perform water desalination. Both NF and RO effluents due to the elevated dissolved solids removal, may require treatment to adjust the stability of the treated water prior to its reuse (Tchobanoglous et al., 2014).

3.8.1.4 Membrane Bioreactors (MBR)

MBR technology combines biological activated sludge processes with membrane separation methods, resulting in a process suitable for water reclamation (Eslamian, 2016).

MBR systems can achieve high pathogen removal rates as well as elevated efficiency in heavy metals elimination. The effluent of an MBR system is capable of meeting the requirements for water reuse in irrigation (Norton-Brandão et al., 2013). Nonetheless MBR effluents carry a significant amount of aerobic bacteria and coliphages with both ultrafiltration and microfiltration membranes (Arévalo et al., 2012). UV disinfection of the MBR effluent only exhibited an efficient removal of one type of organisms which were the somatic coliphage (Francy et al., 2012).

3.8.2 Microbial Fuel Cells

Microbial fuel cells (MFC) is currently at an experimental testing level but its technology is promising, providing gains in water reclamation and energy recovery, by converting biochemical energy of organic matter into electricity. This technology is feasible for treating low to medium-strength wastewater, while anaerobic digestion is advantageous in treating high-strength wastewater (Aelterman et al., 2006; He et al., 2017).

MFC has shown potential for water reclamation with elevated removal efficiencies for COD, N, P, heavy metals and other elements (Abourached et al., 2016). It has been demonstrated a removal of more than 90 % of COD of a low-strength affluent wastewater with only 20 mg/L COD (He et al., 2017).

This process was estimated in the San Joaquin Valley of California where water prices were evaluated at 440 dollars per 43,560 cubic meters and electricity costs at 15.5 cents per kWh accounting to 7,1 million dollars of net profit. Some authors claim that MFC (its membranes and separators) is extremely costly and the energy gains may not be high enough to offset the costs (Abourached et al., 2016).

MFC have been limited to laboratory testing and more development is required in order to reduce the cost of its components with the intent of a more cost-effective process (Gude, 2016).

3.8.3 Disinfection

Disinfection is generally performed by two methods, either by: chemical agents, such as chlorine (and its compounds) or ozone; or by non-ionizing radiation like ultraviolet (UV) light or pasteurization. (Tchobanoglous et al., 2014). In the table 3.8-1, it is presented a summary of usual disinfectants.

Table 3.8-1: Summary of common disinfectants (Tchobanoglous et al., 2014).

Characteristics	Free and combined Chlorine species	Chlorine dioxide	Ozone	UV light
Hazardous chemicals that threat surrounding population	Yes, if Chlorine gas	No	No	No
Energy Intensive	No	No	Yes	Yes
High contact time	Yes	Yes	No	No
Effective in the destruction of resistant organic constituents (such as NDMA)	No	No	No, but reduces the concentration at higher dosages	Yes, at very high dosages
Residual Disinfectant	Yes	Yes	No	No

All four approaches convey effective disinfection although free and combined chlorine species are less effective in terms of viruses, spores and cysts inactivation.

Chlorine (free and combined chlorine species) is the most common approach for disinfection of water and wastewater, in the form of gaseous chlorine, chloramines and sodium hypochlorite (Collivignarelli et al., 2018). However, this reagent leads to the formation of disinfection byproducts (DBPs) which are carcinogenic (Liberti et al., 2003) and gaseous chlorine portray an unsafe operation.

Ultraviolet (UV) treatment is an optimal disinfection technology for reclaimed water. This disinfection method, however, does not provide treatment to ascaris egg, which are the most UV-resistant water related pathogen. Additionally, UV disinfection does not grant disinfection residual when applied. A filtration stage prior to UV is required to eliminate helminth eggs (Norton-Brandão et al., 2013). UV filtration is very effective in the elimination pathogens such as *giardia lamblia* and *cryptosporidium* (Collivignarelli et al., 2018).

Peracetic acid (PAA) is also a possible reagent for disinfection, with proven effectiveness (Dell'Erba et al., 2004). Doses of 1.5 to 2 mg/L of PAA with a contact time of 15 to 20 minutes, proved to be sufficient for bacteria removal of tertiary effluent wastewater, while a contact time of 60 minutes, provides coliphage virus removal (Luukkonen et al., 2014).

For secondary effluents a dose of 2 to 7 mg/L and a contact time of 30 minutes are required for a 3-log reduction in total coliform number. Nonetheless, the desired dose and contact time depends on wastewater quality (Luukkonen, Heyninck, Rämö, & Lassi, 2015). PAA disinfection can occur even in the presence of organic matter with no formation of undesirable byproducts, as the byproducts originated are not toxic for aquatic life (Kitis, 2004).

The disadvantage of PAA is the reagent price, which costs twice as much as sodium hypochlorite (Chhetri et al., 2014). Furthermore, PAA offers low disinfection efficiency against some viruses and parasites such as giardia lamblia cysts. Adding to this, PAA increases the organic content of the effluent due to the formation of acetic acid. A dose of 5 mg/L of PAA, results in the formation of 13 mg/L of acetic acid contributing to an increase of approximately 14 mg/L in COD (Kitis, 2004).

Performic acid (PFA) and perpropionic acids (PPA) are also chlorine disinfection alternatives, which in comparison with PAA portray a slightly more effective disinfection but also suffer from superior costs in comparison with sodium hypochlorite (Luukkonen et al., 2015).

4 Methodology

4.1 Case Study

4.1.1 Introduction

For the development of the present study, a conventional operating WWTP was chosen as a model for its hypothetical conversion to a WWTP with possibly neutral or even positive outputs in terms of energy and water for recovery and reuse.

4.1.2 WWTP of Espinho

The case study is the WWTP of Espinho which is located in Paramos Beach in the municipality of Espinho, belonging to the district of Aveiro. The figures 4.1-1 an 4.1-2, display the WWTP location in a nation-wide scale and in a more local scale, respectively.

Figure 4.1-1: Iberian Peninsula with Espinho WWTP marked location (Google Maps, 2018).

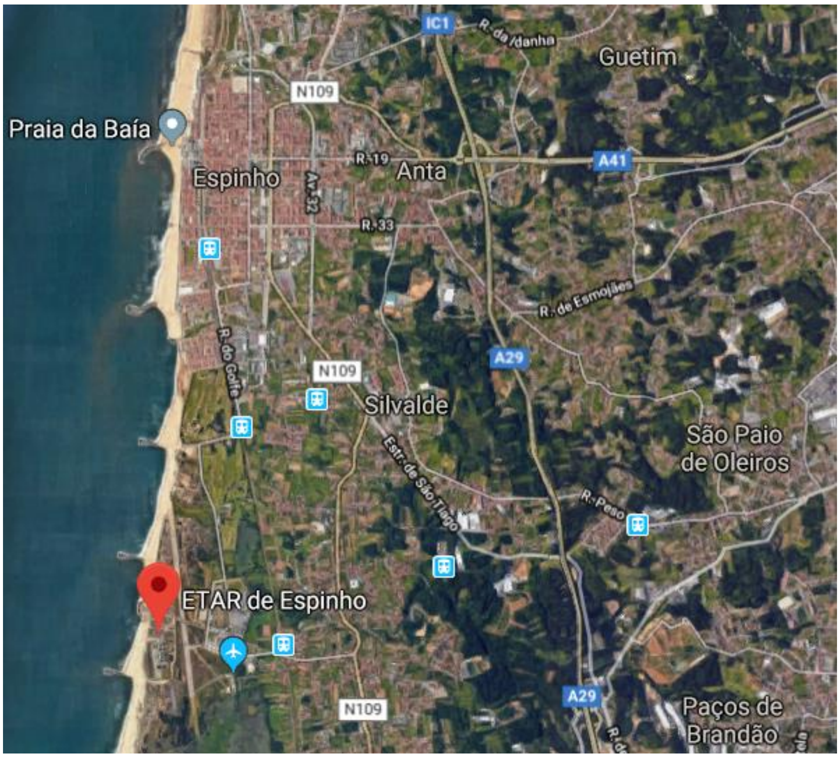


Figure 4.1-2: Espinho municipality with Espinho WWTP marked location (Google Maps, 2018).

Espinho WWTP was designed to treat the sewage of 194,232 population equivalent (PE), which represents the PE of the project's horizon year of 2030 during the high season ("Águas de Portugal," 2018). In figure 4.1-3, it is presented the satellite view of the WWTP.

Figure 4.1-3: Espinho WWTP satellite view (Google Maps, 2018).

The WWTP of Espinho possesses a liquid and a solid treatment phase and was designed solely to remove carbon from the influent wastewater. Both treatment phases can be observed in figure 4.1-3, in which the primary and secondary clarifiers, the aeration tank, the digesters, the gas holders and the gravity thickeners are visibly distinguishable.

The liquid phase is responsible for receiving the influent wastewater to the WWTP and to converting it by means of treatment to dischargeable water to the environment. The liquid treatment phase consists of a preliminary treatment, primary treatment and a secondary or biological treatment.

The liquid treatment phase is described below by sequential order of treatment.

1. Screening comprised of two treatment lines with a sequence of two screens: a coarse screen (40 mm screen) followed by a fine screen (6 mm, step-screen equipment);
2. Two rectangular aerated grit chambers in two separate treatment lines. Each chamber is air insufflated, to promote flotation, and has a surface skimmer for the removal of grease/oil/scum and a bottom scrapper blade for the removal of grit/sand.

Each aerated grit chamber has a usable volume of 81 m^3 and an area of 37 m^2.

The sand/grit settles at the bottom of the chamber and is removed by a specific sand extraction centrifugal pump and containerized for posterior transport and disposal.

The grease/oil/scum is removed by flotation in combination with the surface skimmer and is conducted by an eccentric screw pump to the anaerobic digesters;

3. The primary treatment is materialized in three equal circular conventional clarifiers. Each clarifier has a diameter of 22 meters, an area of 380 m^2 and a total usable volume of 1,020 m^3. Each clarifier possesses a surface skimmer for scum removal and a scraper blade, in the bottom, for the sludge to collect in the middle of the clarifier to facilitate its removal.

   Possibility of chemically enhanced primary treatment, due to the existence of a previous stage of coagulation and flocculation, which occurs in two different chambers with distinct mixing velocity gradients (G). It comprehends one sequential line, starting with one rapid mixing chamber for coagulation, followed by three slow mixing chambers in a series for flocculation preceding the primary sedimentation;

   CTGA has recently been assigned the management and operation of Espinho WWTP and started to implement CEPT by utilizing the coagulant PAX18 with a dosage of 7 mg/L. This dosage was obtained according to the initial jar tests realized before the beginning of this work.

4. Complete-mix conventional activated sludge reactor, with the purpose of carbon removal. The biological reactor is composed of three separate complete-mix treatment lines. Each line comprehends a usable volume of 2,124 m^3 and a maximum liquid depth of 3.91 m. The aeration tank has a maximum usable volume of 6,372 m^3, with all treatment lines being utilized.

   The biological reactor is aerated via 9 surface turbines, 3 in each line, with individual 45 kW of power.

   The biological reactor has the possibility of functioning with plug-flow configuration.

   The biological sludge is periodically withdrawn from the biological reactor and conducted to the solid treatment phase to maintain the set sludge age;

5. Secondary sedimentation occurs in four circular clarifiers. Two of the clarifiers have a diameter of 24 m, an area of 452 m^2 and a usable volume of 754 m^3. Conversely the other two clarifiers present a diameter of 34 m, an area of 908 m^2 and a usable volume of 2,698 m^3 each. The clarifiers comprehend a total surface area of 2,721 m^2 and a usable volume of 7,086 m^3. Each clarifier possesses a surface skimmer for scum removal and a scraper blade, in the bottom, for sludge collection in the middle of the clarifier to facilitate its removal.

   All the biological sludge that settles in the secondary clarifier is recirculated to the biological reactor with the purpose of maintaining the MLSS in the biological reactor.

The sludge only exits the biological treatment by being withdrawn from the biological reactor as there is no possibility of removing and conducting the biological sludge from the secondary clarifiers to the solid treatment phase;

6. Service water is obtained via mechanical filter, which functions with a maximum affluent flow of 40 m^3/h and a TSS concentration of 35 mg/L.

   The mechanical filter is followed by an UV unit, which will provide disinfection of the treated water.

In figure 4.1-4, it is represented the primary clarifier, with the biological reactor and secondary clarifier in the background.

Figure 4.1-4: Espinho WWTP primary clarifier, biological reactor and secondary clarifier in the back. The black tank on the left is the PAX 18, the coagulant used in CEPT, storage unit.

The solid treatment line is responsible for the sludge treatment, which incorporates thickening, stabilization and biogas production with subsequent conversion to electrical and thermal energy, dewatering and storage.

Sludge treatment phase is described below by sequential order of treatment.

1. Thickening: primary sludge in two circular gravity thickeners; secondary sludge in 2 (+1) mechanical thickeners.

Each individual gravity thickener has a diameter of 10 m, a peripheric depth of 3.5 m, a superficial area of 78.5 m^2 and a usable volume of 275 m^3.

The mechanical thickeners utilized are rotary drum filters, which are designed to handle 15 to 45 m^3/h of incoming sludge and to operate with 95% of solids capture. The equipment provides a thickened sludge with 3 to 6% solids concentration of waste activated sludge, while consuming 2 to 5 kg of polyelectrolyte per ton of affluent TSS load.

The mechanical thickener equipment installed has two motors, one that provides thickening, while the other one aids the sludge to flocculate. The total absorbed power of the equipment is 1.5 kW (Alfa Laval, n.d.).

2. Homogenization tank, which receives both thickened sludges, primary and secondary. It has a usable volume of 45 m^3 (8x2x2.8 meters).
3. Stabilization of the combined thickened sludge in two mesophilic anaerobic digesters (AD). Each anaerobic digester has a diameter of 18.5 m, a total depth of 12.35 m and a usable volume of 2,585 m^3. The total anaerobic digestion volume is 5,032 m^3.

   Possible addition of calcium hydroxide ($Ca(OH)_2$), most commonly known as hydrated lime, for the chemical stabilization of the sludge by increasing its pH.

   The produced biogas, from the anaerobic digestion process, is stored in two biogas holders with 1,040 m^3.

   The biogas is utilized in a CHP unit, which has 45% efficiency in producing thermal energy and 35% efficiency for producing electrical energy.

   WWTP with installed CHP units can either produce energy to be utilized in the plant or produce energy to sell to the local energy entities by introducing it directly in the local energy grid. The WWTP cannot do both. In Espinho WWTP, the current contract implies that the produced energy is entirely introduced in the local energy grid.
4. Equalization tank that receives the stabilized sludge before its dewatering, designed for an HRT of 1 day, presenting a usable volume of 280 m^3 (12.5x8x2.8 meters).
5. Sludge dewatering is materialized in 2 (+1) centrifuges with a unitary capacity of 10 m^3/h and 239 kg TSS/h. They are intended to work 24 h, 7 d/week. Each centrifuge has 22 kW of power and requires 0.008 kg of polyelectrolyte per kg of affluent TSS.

In figure 4.1-5, it is briefly exhibited the current treatment line of both the liquid and solid treatment phases of Espinho WWTP.

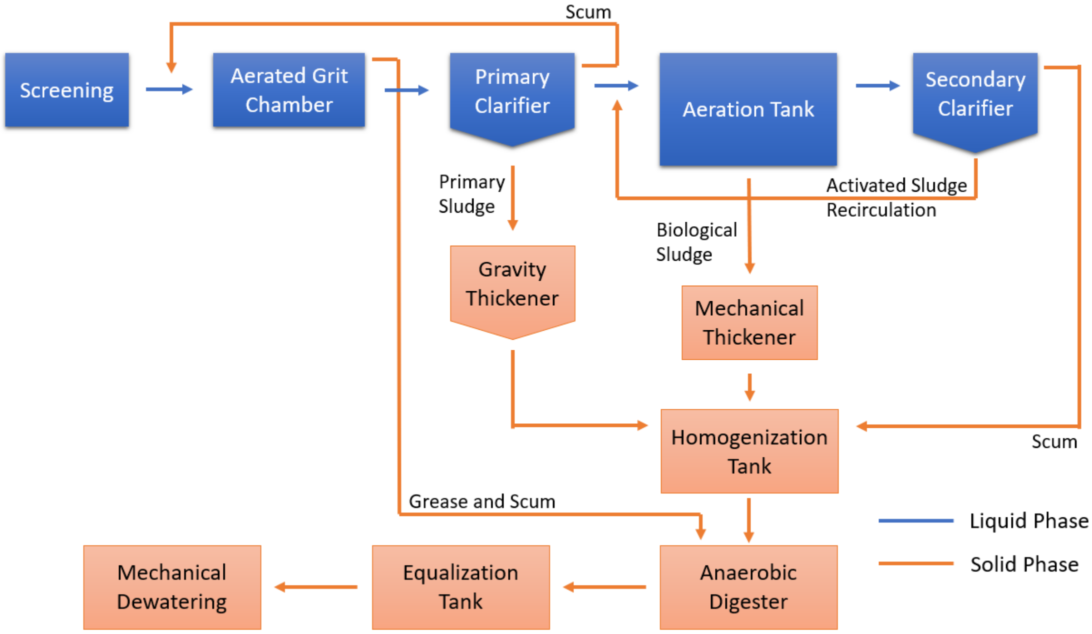


Figure 4.1-5: Espinho WWTP current treatment train.

The accessory equipment installed in Espinho WWTP is described below:

1. Sludge pumps:
   - 2 (+1) primary sludge pumps with an absorbed power of 3.8 kW, each pump can operate with a maximum flow rate of 30 m^3/h;
   - 2 (+1) excess WAS pumps with an absorbed power of 1.0 kW, each pump can operate with a maximum flow rate of 33 m^3/h;
   - WAS recirculation pumps, 8 pumps, 1 (+1) in each sedimentation tank, that operate 24 h/d. The pumps of the two original, smaller sized sedimentation tanks have an absorbed power of 3.7 kW and can operate with a flow rate of 182 m^3/h each. The two more recent, bigger sedimentation tanks, which were installed in the rehabilitation of 2008, have pumps with an absorbed power of 7.3 kW and operate with a flow rate of 366 m^3/h;
   - 2 (+1) thickened primary sludge pumps with an absorbed power of 1.1 kW, each pump can operate with a maximum flow rate of 6.8 m^3/h;
   - 2 (+1) thickened mixed sludge pumps with an absorbed power of 1.34 kW, each pump can operate with a maximum flow rate of 8.4 m^3/h;
   - 2 (+1) digested sludge pumps with an absorbed power of 1.1 kW, each pump can operate with a maximum flow rate of 5.8 m^3/h;
   - 2 (+1) AD sludge recirculation (heating) pumps with an absorbed power of 15 kW, each pump can operate with a maximum flow rate of 157 m^3/h;
   - 1 (+1) dewatered sludge pumps with an absorbed power of 4.7 kW, each pump can operate with a maximum flow rate of 3.5 m^3/h;
2. Biogas compressors, two units with an absorbed power of 22.7 kW each that operate 24 h/d.

3. Ventilators for deodorization, 4 ventilators, 1 (+1) in each facility, with an absorbed power of 18.5 kW, that operate 24 h/d. The ventilators are installed in the preliminary treatment and initial pumping of the affluent raw wastewater building and in the thickening and dewatering facility;
4. Initial and final pumps accounting for (Data of 2016 records):

- Scenario 1: 811,899 kWh/y;
- Scenario 2: 728,772 kWh/y;
- Scenario 3: 1,118,880 kWh/y.

Espinho WWTP underwent its last beneficial rehabilitation and expansion in the year of 2008.

Data records from the WWTP *in situ* energy meter show that in the year of 2016, the energy consumption/produced was the following:

- Total energy consumption: 2,805,956 kWh/y;
- Turbine aeration energy consumption: 898,678 kWh/y;
- CHP unit energy produced: 900,058 kWh/y.

It is unknown if in the year of 2016, CEPT was being performed in Espinho WWTP.

Photographs of the WWTP are displayed in Annex A.

4.1.3 Baseline Data

The data for the study of the influent wastewater was measured *in situ,* at the WWTP, in the beginning of the treatment train, in the tank that receives the raw influent wastewater serving as an equalization basin. The flow rate was measured with an electromagnetic flowmeter. The load parameters (COD, BOD, TSS, TN, TP) were determined via the collection of composite wastewater samples and further laboratory testing.

The data was retrieved from the Espinho WWTP Operation Reports of Luságua, the previous company responsible for the WWTP. The available data is from the year of 2016, with the seventh month, July, being disregarded due to errors in the measurements that compromised its usage, exhibiting unrealistic loading values. Data from previous years, as well as data from 2017, was very incomplete thus was rejected and not included in the study of the affluent wastewater to the treatment plant.

The table 4.1-1, shows the measurements of the wastewater at the entrance of the treatment plant prior to any treatment process in the year of 2016, representing the baseline data.

Table 4.1-1: Affluent wastewater baseline data – 2016.

Month	Flow Rate (m^3/month)	COD Loading (kg/month)	BOD Loading (kg/month)	TSS Loading (kg/month)	TN Loading (kg/month)	TP Loading (kg/month)
1	1,127,199	405,888	165,297	218,886	17,706	1,505
2	1,078,132	289,361	147,671	101,644	18,757	2,169
3	929,486	364,003	161,779	146,495	19,090	2,911
4	917,227	216,374	107,432	81,938	16,051	2,174

Month	Flow Rate (m^3/month)	COD Loading (kg/month)	BOD Loading (kg/month)	TSS Loading (kg/month)	TN Loading (kg/month)	TP Loading (kg/month)
5	829,972	224,483	113,898	93,453	21,016	2,306
6	617,344	529,922	218,445	199,431	26,875	4,715
7	-	-	-	-	-	-
8	438,649	492,759	216,849	224,131	32,433	5,167
9	432,953	332,901	165,074	132,148	25,111	3,405
10	517,098	344,524	160,437	119,775	20,335	2,748
11	523,424	390,172	197,980	141,655	23,537	3,142
12	479,881	434,452	209,785	196,998	23,935	3,584
Average	717,397	365,894	169,513	150,596	22,258	3,075

It can be noted that the treatment plant suffers from seasonality, being the first five months of the year (January, February, March, April and May), the ones which experience more affluent flow rate. Moreover, these months are characterized by inferior concentrations in the parameters being studied, as the affluent load does not show an increasing pattern with increments to the flow rate. The affluent TSS is considerably low, when compared with the average affluent wastewater reported by the literature.

The seasonality effect can be clearly noticed in figures 4.1-6 and 4.1-7, in which each month, represented by its number (e.g. January = month 1), is displayed by flow rate and COD/BOD loading.

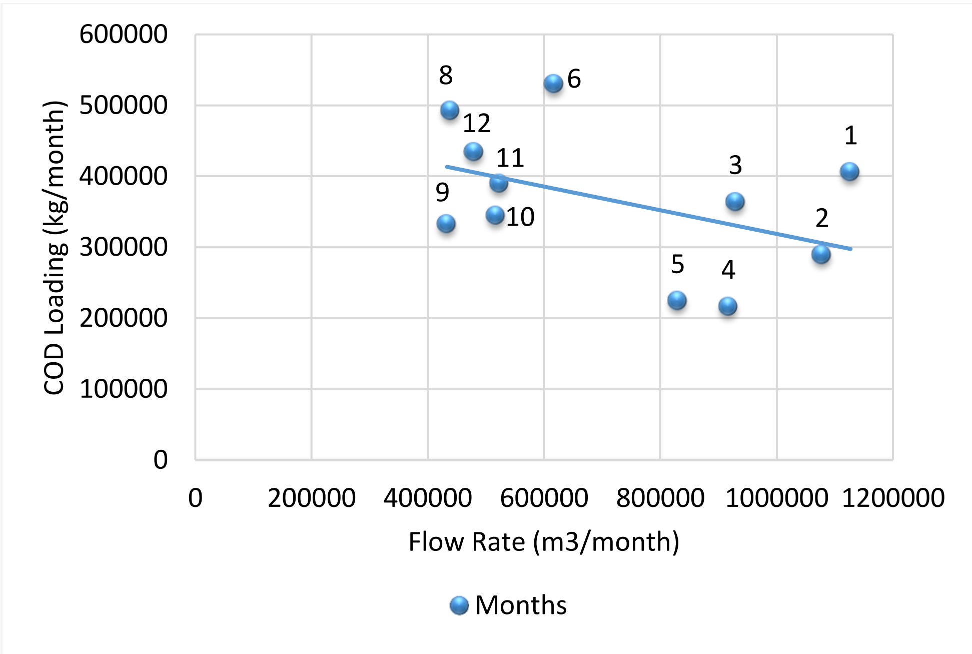


Figure 4.1-6: COD loadings and flow rate of each month in 2016.

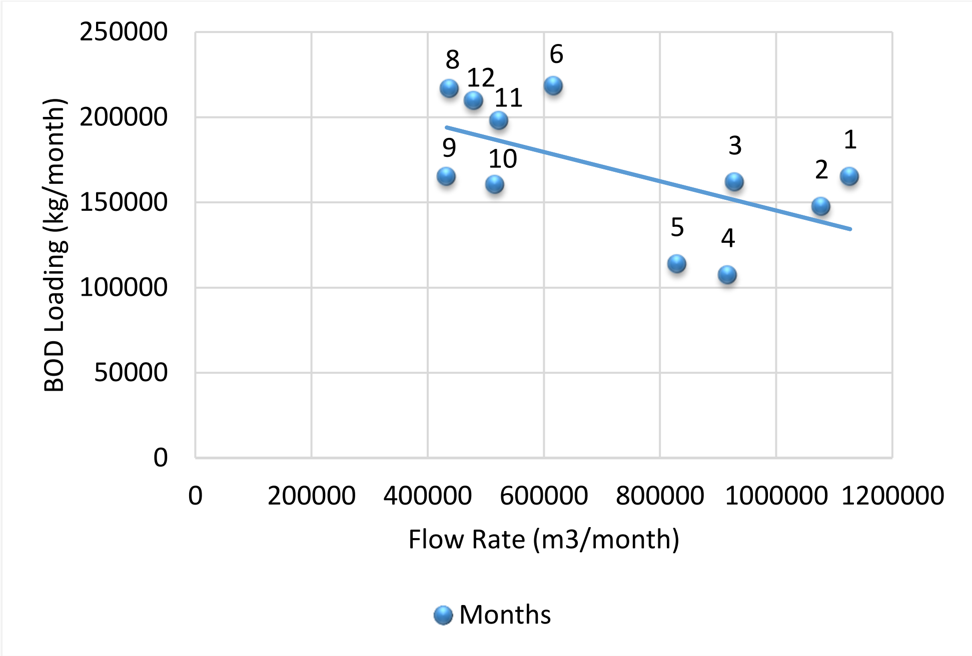


Figure 4.1-7: BOD loadings and flow rate of each month in 2016.

There is a noticeable pattern, evidencing the different conditions in the wastewater, that reaches the WWTP over the course of the year. Six months are characterized by lower flow rate and higher loading being June, August, September, October, November and December. The remaining five months are characterized by higher flow rate and lower loading, that is January, February, March, April and May.

4.1.4 Design Data

4.1.4.1 Design flow-rate

The design flow-rate is used in the design of critical treatment unit processes (e.g. primary/secondary clarifiers) and to validate the overflow rate and the detention time, as well as, the design of conduits, connecting pipes, channels and pumping stations.

The maximum flow rate pumped to the WWTP, via the screw pumping station installed in the beginning of the water treatment line, represents the design flow rate. The maximum flow rate value was observed in the 6 of May of 2016, according to the reports that include the flow meters operation details.

The design peak flow rate considered is 2,868 m^3/h.

4.1.4.2 Scenarios

Espinho WWTP design will be verified to operate in three different scenarios, that vary in terms of quantity and quality of the affluent wastewater to the WWTP.

The different scenarios were conceived to display a range of possible load concentrations in the affluent wastewater to the treatment plant. The scenarios were created via existent data corresponding to the year of 2016, as shown in the previous section, Baseline Data.

Scenario 1

Scenario 1 represents the average conditions of the affluent wastewater to the treatment plant. This scenario will be used to simulate the average WWTP operation throughout the year. It will be used verify that the desired treatment is accomplished and to determine the cost of operation of the WWTP, such as the chemical needs and the energy expenditure.

This scenario was obtained via averages of the existent information of the affluent wastewater to the WWTP.

In table 4.1-2, it is represented the scenario 1 affluent wastewater characteristics, as in flow rate, load and concentration of TSS, COD, BOD, TN and TP.

Table 4.1-2: Data of affluent wastewater conditions of scenario 1.

Parameters		Units	Value
Flow rate		m^3/d	23,627
		m^3/h	984
Load	COD	kg/d	12,000
	BOD	kg/d	5,562
	TSS	kg/d	4,935
	TN	kg/d	731
	PT	kg/d	101
Concentration	COD	mg/L	508
	BOD	mg/L	235
	TSS	mg/L	209
	NT	mg/L	31
	PT	mg/L	4

Scenario 2

Scenario 2 is the case scenario with the most demanding treatment. In this scenario the affluent wastewater is characterized by possessing higher concentrations, meaning higher load and lower affluent flow rate.

This is the case scenario which will be used for the verification of the design of many unit processes mainly in the solid treatment phase, as well as to determine chemical storage requirements and solid phase pumping.

The design project will have to verify that in this scenario the desired treatment is accomplished.

This scenario was obtained via data from the month of June (6), which according to the existent data represents the period of time during which, higher concentrations (of COD, BOD, TSS, TN, TP) were verified in the wastewater.

In table 4.1-3, it is represented the scenario 2 affluent wastewater characteristics, as in flow rate, load and concentration of TSS, COD, BOD, TN and TP.

Table 4.1-3: Data of affluent wastewater conditions of scenario 2.

Parameters		Units	Value
Flow rate		m^3/d	20,578
		m^3/h	857
Load	COD	kg/d	17,664
	BOD	kg/d	7,282
	TSS	kg/d	6,648
	TN	kg/d	896
	PT	kg/d	157
Concentration	COD	mg/L	858
	BOD	mg/L	354
	TSS	mg/L	323
	NT	mg/L	44
	PT	mg/L	8

Scenario 3

Scenario 3 represents the less demanding treatment conditions, as the affluent wastewater is characterized by lower load and higher affluent flow rate, which leads to lower load concentrations. This case scenario will be used to verify that the desired treatment is accomplished in the WWTP, when such affluent wastewater conditions occur.

This scenario was obtained via data from the month of January (1), which according to existent data represents the interval of time during which, lower concentrations in the wastewater were verified.

In table 4.1-4, it is represented the scenario 3 affluent wastewater characteristics, as in flow rate, load and concentration of TSS, COD, BOD, TN and TP.

Table 4.1-4: Data of affluent wastewater conditions of scenario 3.

Parameters		Units	Value
Flow rate		m^3/d	36,361
		m^3/h	1,515
Load	COD	kg/d	13,093
	BOD	kg/d	5,332
	TSS	kg/d	7,061
	TN	kg/d	571
	PT	kg/d	49
Concentration	COD	mg/L	360
	BOD	mg/L	147
	TSS	mg/L	194
	NT	mg/L	16
	PT	mg/L	1

In the figures 4.1-8 and 4.1-9, each month and the three projected scenarios are represented, by flow rate and COD/BOD loading.

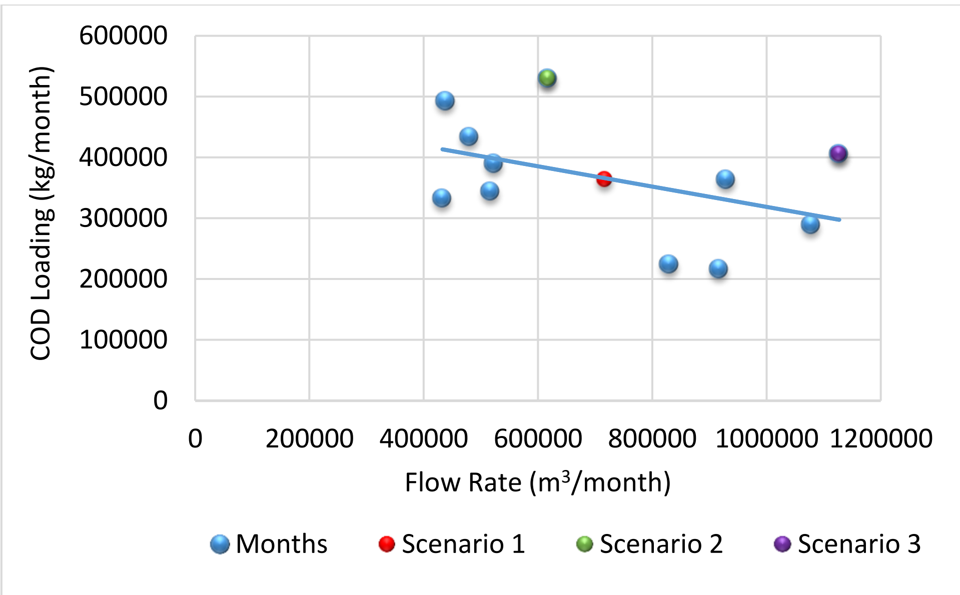


Figure 4.1-8: COD loadings and flow rate of each month in 2016.

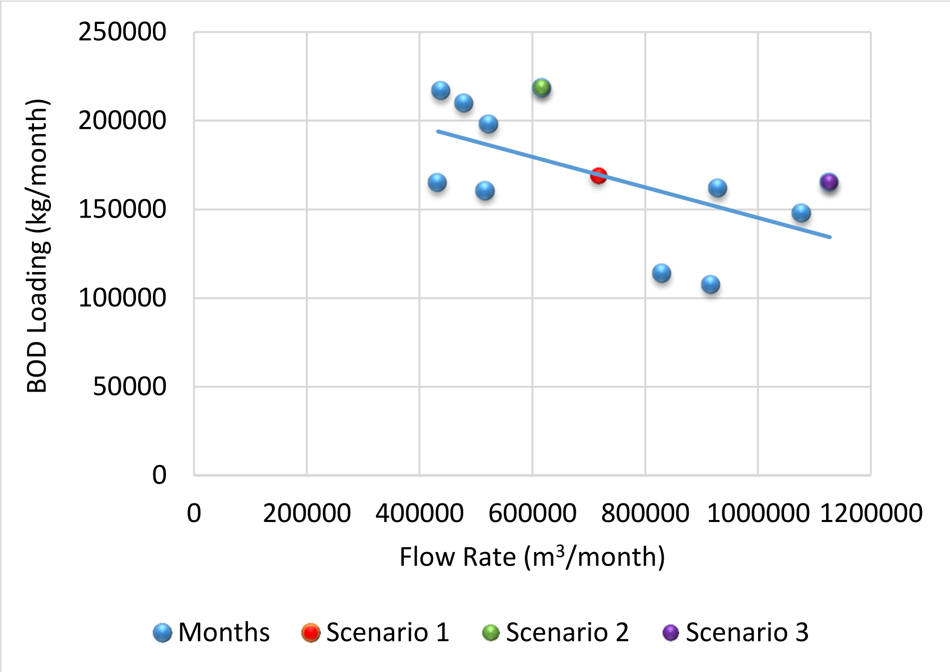


Figure 4.1-9: BOD loadings and flow rate of each month in 2016.

4.1.5 Treatment Objectives

The WWTP treatment efficiency and the discharge permit limits of the WWTP to the receiving environment, were defined by the Portuguese government by the Law-Decree No. 152/97 and are detailed in *Licensa de Utilização dos Recursos Hídricos – Rejeição de Águas Residuais (LURH).*

The discharge permit limits and the removal efficiencies, for each parameter evaluated, are shown in table 4.1-5.

Table 4.1-5: Espinho WWTP wastewater effluent discharge permit limits (Ministério do Ambiente, 1997; APA, 2017).

Parameter	Treated effluent concentration mg/L	Minimum removal efficiency %
COD	125	75
BOD	25	70 - 90
TSS	35	90

In this specific WWTP, nutrient removal is not considered as a requirement, for that reason only COD, BOD and TSS have discharge limits.

4.1.6 Methods for WWTP Optimization

Espinho WWTP optimization was conceived in the direction of closing the gap between being a high energy expenditure facility and a facility in which energy neutrality is not a remote reality. Additionally, water reclamation for reuse was also evaluated to provide irrigation for a golf course nearby.

The methods evaluated are:

- CEPT – benefits in energy efficiency and production;
- Aeration by diffusers – benefits in energy efficiency;
- Co-digestion – benefits in energy production;
- Photovoltaic solar panel modules – benefits in energy production;
- Ultrafiltration – provides water reclamation.

4.2 Determination of coagulant optimum dose in CEPT

The laboratory tests were conducted to determine the optimum dose of coagulant to maximize the removal of TSS, COD and BOD while setting the maximum elimination of phosphorus at 50 % in the primary clarifier. Coagulant optimum dose determination was conducted at the CTGA laboratory, in Coimbra.

Wastewater was retrieved from Espinho WWTP, directly from the raw wastewater tank, prior to sedimentation. The wastewater samples were collected in a 24-hour period, consisting of composite samples.

Jar tests were performed with different doses of the coagulant polyaluminium chloride (PAX18). The dosages tested were 15 mg/L, 30 mg/L, 45 mg/L, 60 mg/L.

A dose of 0.5 mg/L of anionic polymer, consisting of the flocculant, was also introduced in each jar test.

Afterwards TSS, COD, TP and orthophosphates determinations were carried out with the five samples of distinct coagulant doses.

4.2.1 Materials

The materials used in the jar test procedure and in the correspondent TSS, COD, TP and orthophosphates determinations were the following:

- Volumetric flasks (1,000 mL);
- Stirring machine with 4 paddles capable of variable speeds from 0 to 200 RPM (VELP FC 4S);
- Graduated pipettes of 10, 5, 2, 1 and 0.5 mL;
- Beaker of 200 mL and 50 mL;
- Analytical balance (SCALTEC SPB 42);
- Watch glass;
- Stainless steel laboratory spoon/spatula;
- Porcelain capsules;
- Tweezers;
- Glass fiber filter;
- Laboratory chronometer;
- Heating/drying oven (memmert);
- Thermoreactor (Spectroquant® TR 420);
- Desiccator;
- Filter system with vacuum pump;
- Filtering flask;
- Graduated cylinder;
- Spectrophotometer (Spectroquant® Prove 100);
- Polyaluminium chloride;
- Anionic polymer;
- Distilled water;

4.2.2 Jar Tests

Flocculant Preparation

Initially, the flocculant solution was prepared. As the anionic polymer utilized was in powder form, it was produced a solution with a concentration of 50 mg/L of anionic polymer. Subsequently, it was introduced 10.1 mL of this solution in each jar to obtain the 0.5 mg/L concentration of anionic polymer.

The method for the preparation of the anionic polymer was the following:

- A volumetric flask of 1,000 mL was filled with 200 mL of distilled water;
- An analytical balance was used to measure 10 mg of anionic polymer;
- The volumetric flask was placed in the stirring machine with the paddle at 200 RPM speed;
- The 10 mg of anionic polymer were gradually introduced in the volumetric flask;
- The solution was put in the stirring machine at 200 RPM speed for 45 min to 1 h, in order to obtain the anionic polymer solution.

Jar Tests

Ultimately, the Jar test procedure was conducted. The method utilized was of standard nature and is hereinafter delineated by sequential order.

1. Five transparent jars of 1,000 mL were filled with 1 L of wastewater.
2. The transparent jars of 1,000 mL were placed in the stirring machine.
3. The coagulant was introduced in each jar, via a pipet, to obtain the respective concentrations.
4. Rapid mixing was induced by setting the paddles rotation speed at 120 RPM for 2 minutes;
5. The mixing speed was then set at 45 RPM and the flocculant was added to each jar test simultaneously;
6. The mixing speed was fixed at 45 RPM for 20 minutes to promote floc formation.
7. Following the coagulation and flocculation processes, the mixing was turned off and the wastewater in each jar was allowed to sediment for 20 minutes;
8. A volume of supernatant was retrieved from each jar with the intent to further analyze and determine the necessary parameters (COD, TSS, TP, Orthophosphates) for the selection of the optimum coagulant dose.

4.2.3 TSS Determination

The procedure utilized for the TSS determination is described by sequential order.

1. Five already prepared and numbered glass fiber filters, one for each sample, were weighed in the analytical balance and their respective number and weight was recorded.
2. Each glass fiber filter was individually put in the filtering system with tweezers.
3. A filtering flask was assembled to the filter holder/system.

4. A graduated cylinder was utilized to add 100 mL of a wastewater sample to the filtering flask.
5. The vacuum pump of the filtering system was turned on, applying vacuum to the system until all the water was pulled through the filter.
6. The vacuum was slowly released, and the glass fiber filter was removed from the filter holder.
7. The glass fiber filter disk was introduced in a pre-heated drying oven at 103-105 °C, for 1 h.
8. The glass fiber filter was removed from the drying oven and immediately put in the desiccator for 1 h.
9. The glass fiber filter was withdrawn from the desiccator and put in the analytical balance for the weighing with the use of tweezers.
10. Step 7 to 9 were repeated until the weighing of the glass fiber filter containing the solids, stabilized, meaning the difference between measurements did not exceed 0.5 mg.

To obtain the TSS concentration present in the sample, it was required to proceed with the following equation:

$$\text{TSS (mg/L)} = \frac{(\text{B}' - \text{A}')}{\text{V}_{\text{Sample}}} \times 10^3$$

Where:

A' – weight of the glass fiber filter (g);

B' – weight of the glass fiber filter + TSS (g);

V_{Sample} – volume of wastewater introduced in the filtering flask to perform the TSS determination (mL).

4.2.4 COD Determination

The methodology utilized for COD determination was via Spectroquant® cell tests kits, which are exhibited in figure 4.2-1.

Figure 4.2-1: Spectroquant® cell tests kits on the left; Spectroquant® cell tests COD vials on the right.

The procedure is displayed in Annex B.

4.2.5 TP and Orthophosphates Determination

The methodology utilized for TP and Orthophosphates determination was via Spectroquant® cell tests kits, the procedures are displayed in Annex C and Annex D, respectively.

4.3 **WWTP Design Verification**

4.3.1 Assumptions

The energy price considered in this study, was fixed at 0.11 €/kWh, which is the average energy price in Espinho WWTP.

4.3.2 Mass Balances

The mass balances were conceived for all three scenarios of Espinho WWTP, with conventional primary treatment and with CEPT in the treatment line.

The mass balances were all verified via a calculus tool named DenikaPlus, which is a design and simulation software based on the "*University Work Group*" (HSG) of Germany, Switzerland and Austria, and in the Technical Rules of the German organism *Abwasser Technische Vereinigung e.v. (ATV)/ German Association of Wastewater*. This program was also utilized to determine the specific sludge production for each scenario with conventional primary treatment and with CEPT in the WWTP treatment line.

The mass balances were developed with assumptions and factors supported by the literature. For the determination of the biological sludge and its characteristics, it was considered the following:

- 0.5 kg BOD_5 / kg of TSS;
- 2 kg COD / kg BOD5;
- 0.1 kg N / kg TSS;
- 0.02 kg P / kg TSS.

For the removal efficiency of the conventional primary treatment, it was considered a removal of 50% TSS, 30% COD/BOD_5, 20% TN and 10% TP and it was projected a solids concentration of 2.5%.

In the chemically enhanced primary treatment (CEPT), it was considered a removal of 80% TSS, 60% COD/BOD_5, 20% TN and 39% TP, according to the jar tests performed in the laboratory. In this process, the primary sludge assumed a solids concentration of 1.5%.

The biological sludge TSS was determined by the specific sludge production, which was obtained via DenikaPlus. This parameter varies in all scenarios due to different affluent conditions. The solids concentration varied and was in line with the MLSS, in the biological reactor, considered for each scenario.

In the sludge thickening processes, gravity thickening of the primary sludge and mechanical thickening of the biological sludge, it was considered 90% solids capture and a solids concentration of 4%.

In the dewatering process with the existing equipment, it was considered 95% solids capture and a solids concentration of 20%.

4.3.3 Primary Treatment

The design of the existing primary clarifiers was verified via the formulas of surface overflow rate (1) and hydraulic retention time (2), presented in Annex E.

The hydraulic retention time and surface overflow rate was verified to meet the criteria already exhibited in the literature review, which is also displayed in table 4.3-1.

Table 4.3-1: Conventional primary treatment and CEPT design verification parameters.

Parameter	Unit	Conventional Primary Treatment	Chemically Enhanced Primary Treatment
Hydraulic retention time	h	1.5 – 2.5	1.5 – 2.5
Surface overflow rate	$m^3/(m^2.h)$	1.25 – 2.1	2.8 – 3.4

It was considered the usage of all (3) primary clarifiers in all scenarios.

4.3.4 Biological Treatment

The treatment design verification followed the guidelines and design criteria expressed in the document *ATV-A131E of May 2000*, of the *German Association of Wastewater*.

It was considered a temperature of 28°C for the endogenous respiration to calculate the aeration necessities. For the calculus of the specific sludge production it was considered a process temperature of 14°C.

It was considered the usage of all (4) secondary clarifiers in all scenarios.

4.3.5 Aeration

The peak factor for carbon removal considered is 1.3, which is due to the intended sludge age, as recommended by the *ATV-A131E* document.

4.3.5.1 Turbine Aeration

The coefficients considered for the design verification of the turbine aeration are described in table 4.3-1.

The design verification was performed via Metcalf & Eddy, Wastewater Engineering (Tchobanoglous et al., 2014), and Development of Standard Procedures for evaluating Oxygen Transfer Devices (U.S. EPA & American Society of Civil Engineers (ASCE), 1983).

Table 4.3-2: Auxiliary parameters utilized in the turbine aeration design verification.

Parameter	Unit	Value
α coefficient	-	0.75
β coefficient	-	0.95
Ɵ coefficient	-	1.024
Liquid density (γw)	m.c.a/m	1
Effective saturation depth (de)	% of the reactor usable depth	0.06
Vapor pressure of water at 20 ºC (pv20)	m.c.a	0.238
Atmospheric pressure at sea level (Ps)	m.c.a	10.34
Atmospheric pressure at the WWTP altitude (Pb)	m.c.a	10.34
Oxygen saturation concentration at 20 ºC and atmP (C*s20)	mg/l	9.092
Oxygen saturation concentration at 27 ºC and atmP (C*sT)	mg/l	7.83
Oxygen concentration (C)	mg/l	2
Oxygen transfer capability of surface slow speed mechanical aerators	kg O_2/kWh	1.8

4.3.5.2 Fine Bubble Aeration

The coefficients considered for the design verification of the diffused air aeration are described in table 4.3-3.

The design verification was performed via Metcalf & Eddy, Wastewater Engineering (Tchobanoglous et al., 2014).

Table 4.3-3: Auxiliary parameters utilized in the fine bubble aeration design verification.

Parameter	Unit	Value
Reactor water surface height	m	3.91
Design desired oxygen concentration	mg/L	2
Elevation	m	0
Temperature	ºC	28
Pw	m	3.69

Parameter	Unit	Value
atmP,h	m	10.34
Air Oxygen Concentration	%	19.00
Cst	mg/L	7.83
Pb/Pa	-	1.00
Csth	mg/L	7.83
Cŝth	mg/L	8.85
Cs20	mg/L	9.09
α	-	0.65
β	-	0.95
F	-	0.90
SOTE (fine bubble)	%/m	5.82

The value considered for standard oxygen transfer efficiency (SOTE) is 21.5%. This value takes into account the depth of the reactor and fine air bubble oxygen transfer efficiency, which was obtained via equipment supplier.

Treatment Line with Conventional Primary Treatment

The blower chosen was consulted via equipment suppliers and its characteristics are displayed in table 4.3-4.

Table 4.3-4: Blower characteristics.

Parameter	Unit	Value
Pressure	mbar	500
Air flow rate	m^3/min	90
Engine power	kW	110
Absorbed Power of each blower	kW	97
Absorbed Power of all 3 blowers	kW	292

Treatment Line with CEPT

The blower chosen was consulted via equipment suppliers and its characteristics are displayed in table 4.3-5.

Table 4.3-5: Blower characteristics.

Parameter	Unit	Value
Pressure	mbar	500
Air flow rate	m^3/min	79
Engine power	kW	110
Absorbed Power of each blower	kW	84
Absorbed Power of all 3 blowers	kW	251

4.3.6 Sludge Thickening

Gravity Thickening

Gravity thickening design verification was realized for two hypotheses of Espinho WWTP treatment line, either with conventional primary treatment or with CEPT.

It is considered the utilization of the two gravity thickeners.

Mechanical Thickening

Mechanical thickening design verification was realized for two hypotheses of Espinho WWTP treatment line, either with conventional primary treatment or with CEPT.

It was admitted a more cautious flow rate of 30 m^3/h per rotary drum filter.

The cost of the flocculant reagent considered for the thickening process was 2.89 €/kg of polyelectrolyte.

4.3.7 Anaerobic Digestion

In the AD process, it was considered that the volatile solids are 70% of the affluent TSS. Even though the Water Environment Federation. et al., (2012) provides a formula to determine the destruction of the volatile solids based on the SRT. For the destruction of the volatile solids it was considered 55% for all scenarios as a conservative measure.

The biogas production considered was 0.9 m^3/kg of volatile solids destroyed.

The biogas lower heating value (LHV) considered was 21,500 KJ/m^3 or 6 kWh/m^3.

It was considered 5% inoperability of the CHP units to provide realistic data by taking into account possible and uncertain shortcomings.

The CHP unit efficiency of producing electrical and thermal energy is 35% and 45% respectively.

Catalogues of current CHP units with the same characteristics have a superior efficiency in producing electrical energy, some of which, reaching efficiencies of around 42%.

For the calculus of the energy produced in the CHP unit, with the biogas produced in the AD/AcoD process, it was not considered the higher energy value of the primary sludge.

4.3.8 Co-Digestion

The purpose of the AcoD study, was to analyze at a surface level, the possible energy recuperation by implementing AcoD in Espinho WWTP while already performing CEPT. The AcoD analysis is done without the elaboration of an AcoD mass balance, meaning the AcoD additional returns are not taken into account, therefore the incremental costs of operation are not considered. The AcoD analysis is only realized to determine the possible energy production, that could be obtained if this process was implemented.

In this study it was pondered the introduction of co-substrate to the maximum capacity of the digesters. The digesters are oversized due to the affluent conditions and, as a result, are low loaded and possess plenty of unused volume. The AD heating system is designed to function when the digesters are working at full capacity, meaning they do not need replacement if co-digestion is implemented.

The co-digestion substrate considered for further application is a FW (food waste) from a food processing industry that produces baby food. This substrate is rich in carbohydrates and poor in fat. It possesses an elevated C/N ratio which will enhance the methane production in the AD. The AcoD substrate characteristics are: 50,000 mg/L of COD and 35,000 mg/L of VSS.

The anaerobic co-digestion performance was studied after the optimization of the two main criteria: volatile solids loading and digester hydraulic retention time. The volume of substrate added to the digesters, in each scenario, was achieved by adding as much substrate to enhance the methane production, while not dropping below the 15 days of hydraulic retention time and not surpassing the 2.4 kg VS/(m^3.d) of volatile solids loading.

4.3.9 Sludge Dewatering

Dewatering design verification was realized for the Espinho WWTP treatment line with conventional primary treatment and with CEPT.

The cost of the flocculant reagent utilized in the dewatering process was 2.89 €/kg of polyelectrolyte.

4.3.10 Photovoltaic Solar Panels

It was considered the installation of photovoltaic (PV) solar panels to maximize the energy production in Espinho WWTP.

In table 4.3-6, is displayed the individual PV solar panel module characteristics considered in this study.

Table 4.3-6: Photovoltaic solar panel module characteristics.

Parameter	Unit	Value
Length	m	1.60
Width	m	1.00
height	m	0.04
Power	kW	0.25

The model of the solar panels intended for installation was obtained via an equipment provider company. The modules expected for installation will be structured and unified in rows. It was considered a spacing of 2.5 m in between each module row.

It was utilized the website (https://pvwatts.nrel.gov/pvwatts.php), which via geographic localization estimates the monthly average number of hours of sun per day and provides the annual energy production for the PV panel modules power to be installed in the WWTP.

4.3.11 Water Reclamation by Ultrafiltration

It was considered the implementation of membrane ultrafiltration (UF) to provide tertiary treatment to the secondary effluent. UF is intended to produce sufficient water, to irrigate a golf course near Espinho WWTP.

The golf course has an area of 42 ha, requiring 200,000 m^3/y for irrigation. The average water consumption is 550 m^3/d. This value is possibly higher in the more irrigation demanding periods.

The water necessities by the golf course represent 2.3% of the treated water by the WWTP (scenario 1).

It was projected the reclamation of 1000 m^3/d by the membrane ultrafiltration unit. This unit will replace the previous method for obtaining service water in the WWTP. The reclaimed water will provide irrigation for the golf course and service water for various uses within the WWTP.

The membrane ultrafiltration unit was obtained in conjunction with an equipment provider. The unit considered has an energy consumption of 0.5 kWh/m^3 of treated water. The unit equipment price considered was 300,000 €, the electric installations were set at 50,000 € and the civil construction was stipulated to be 50,000 €.

The reclaimed water price was fixed at 0.5 €/m^3.

In this study it was also considered the costs of investment and installation of the reclaimed water pipeline from the WWTP to the golf course. The design results of the reclaimed water pipeline, as well as the description of the necessary investment costs are displayed in Annex H.

4.4 Energy Balance and Operation Costs

The total operation costs of a WWTP consist largely, but not only, of costs in energy, reagents, maintenance, staff, as in operators and the operation manager and costs of transport and disposal of coarse materials, grit and sludge to final destination. In this study the total operation costs are represented solely by the energy and reagent costs. The civil construction costs, maintenance costs and the staff expenses are similar in all the options studied, they are fixed costs, and for that reason not considered in this analysis.

The CEPT reagents were calculated from the affluent flow rate, according to the optimum coagulant and flocculant doses, which were determined via the jar tests performed. In contrast, the reagent costs for application in the thickening and in the dewatering processes, were calculated from the affluent TSS load to these sludge treatment phases.

The energy consumption and energy costs were calculated via the absorbed power and the time of operation of each individual equipment installed, for the following equipment:

- Deodorization ventilators;
- Sludge pumps;
- Surface aeration turbines;
- Rotary drum filters, that provide the mechanical thickening;
- Biogas compressors;
- Centrifuges, that provide the dewatering of the digested sludge.

From the records of 2016, solely the energy consumption of the initial and final wastewater pumping was utilized, as they are true to the projected WWTP conditions simulated in this study.

The energy balance was determined to better understand the WWTP standing in terms of consumed and produced energy and to better visualize how far it is from reaching energy neutrality. The energy balance/neutrality represents the percentage of the total energy needs, that the WWTP can produce. Therefore, energy neutrality is considered when the energy balance reaches 100%.

The standard aeration considered is performed via the surface aeration turbines. Fine bubble aeration is one of the optimization approaches considered.

The total costs of the WWTP with conventional primary treatment, as well as, the total costs of the WWTP with CEPT with turbines or fine bubble aeration, are calculated to better visualize the advantages of each treatment scheme by comparison. This analysis allows the determination of the monetary savings of implementing CEPT and CEPT with fine bubble aeration in the WWTP treatment line.

4.5 Cost-benefit Analysis

It was realized a cost-benefit analysis of the implementation of CEPT, fine bubble aeration, PV solar panel modules and ultrafiltration.

In this assessment it was considered the capital expenditure (CAPEX) and the operating expenditure (OPEX), as well as, the revenue provided by each option.

The CAPEX includes the civil construction costs, equipment costs and the cost of electrical installations. These costs were obtained via construction and equipment providers.

The OPEX is composed of the maintenance and replacement costs, energy costs and reagent costs. These costs were acquired via reagent and equipment providers.

The cost-benefit analysis is determined for a period of 15 years.

The net balance is calculated via the sum of the revenue minus the CAPEX and OPEX. To this net balance it is applied a depreciation factor that is calculated by the following formula, in which the *n* represents the year of operation.

$$\text{Depreciation factor} = \frac{1}{(1+0.015)^{\mathrm{n}}}$$

CEPT

The coagulant and flocculant reagent prices were obtained via reagent provider, the company Quimitécnica.

It was considered the cost of 285 €/ton of Kemira's PAX 18, more commonly known as polyaluminium chloride.

The flocculant expected for implementation to perform CEPT, is an anionic polymer, with a cost of 2,890 €/ton.

Fine Bubble Aeration

The civil construction costs correspond to the construction of the facility for the blower units and the installation of the pipelines.

It was considered a cost of 50,000 € for the building and 25,000 € for the pipeline.

For the diffusers network it was considered a price of 100,000 €.

It is required three blower units, which have an individual price of 25,000 €.

The price estimated to replace the current turbines, with new equipment, was 150,000 €.

Photovoltaic Solar Panels

The price of the solar panels intended for installation is 1,000 €/kW installed and it was considered a yearly maintenance price of 15 €/installed kW.

Ultrafiltration

The cost-benefit analysis was evaluated for the implementation of a Ultrafiltration unit, capable of producing a daily volume of 1,000 cubic meters in Espinho WWTP.

The reclaimed water volume was projected to provide irrigation for a golf course, located 1 km away from the plant. In Annex H, it is displayed the location of the golf course, as well as, the pipeline design.

The ultrafiltration unit is designed to provide 200,000 m^3/y to the golf course and to provide service water for Espinho WWTP.

The CAPEX and OPEX values considered are described below.

The civil construction costs are comprised of all the civil construction expenses of the ultrafiltration unit, which are estimated to be 50,000 €.

The equipment price is expected to be 300,000 €, consisting of the ultrafiltration membranes and the accessories.

For the pipeline and inherent civil construction costs, it was estimated a price of 83,500 €. In Annex H, this cost is described in more detail.

The membrane replacement costs were estimated to be 10,000 €/y.

5 Results

5.1 Determination of Coagulant Optimum Dosage in CEPT

Laboratory tests were conducted to determine the optimum dose of coagulant to maximize the removal of TSS, COD and BOD while setting the maximum elimination of phosphorus at 50 % in the primary clarifier.

In the figure 5.1-1, it is displayed the raw wastewater affluent to Espinho WWTP.

Figure 5.1-1: Four jars filled with raw wastewater from Espinho WWTP in the stirring machine where the Jar tests were performed.

In the figure 5.1-2, the rapid mixing (coagulation process) had already been materialized, the mixing speed was set at 45 RPM and the anionic polymer was introduced in all four jars. The jars are sorted by ascending order of coagulant doses from left to right (15, 30, 45, 60 mg polyaluminium chloride/L).

Figure 5.1-2: Jar tests with different doses of coagulant - beginning of the flocculation phase (slow mixing), post coagulation phase (rapid mixing).

In figure 5.1-3, the 20 minutes of flocculation had passed, the flocs were fully formed and aggregated. The paddle mixing halted, depicting the beginning of the sedimentation/clarification process during which solid-liquid separation is induced.

Figure 5.1-3: Jar tests with different doses of coagulant – beginning of the sedimentation phase.

The figure 5.1-4, portrays the clarified wastewater after 20 minutes of sedimentation.

Figure 5.1-4: Jar tests with different doses of coagulant – after 20 minutes of sedimentation.

In figure 5.1-4, it is visible the different coagulation doses and its effect on clarifying the wastewater. The jar with 60 mg/L of polyaluminium chloride (on the far right) visually demonstrates to achieve a higher removal of turbidity/TSS.

In the table 5.1-1, the jar test results are presented. Apart from the various coagulant dosages, the results of primary clarification/sedimentation without the addition of chemicals is also represented, as 0 mg/L of coagulant dosage.

Table 5.1-1: PAX 18 coagulant dosages and due removal with Espinho WWTP wastewater.

Parameters	Units	Prior to settling	Coagulant Dosages (mg/L)				
			0	15	30	45	60
Orthophosphates	mg PO4/L	15.0	14.9	13.5	10.0	5.0	1.8
Total Phosphorus (TP)	mg P/L	9.6	6.7	5.9	3.9	2.1	0.8
Chemical Oxygen Demand (COD)	mg O2/L	1,490	707	600	566	549	535
Total Suspended Solids (TSS)	mg/L	450	200	80	70	50	30

In the figure 5.1-5, the respective removal percentage for the various dosages of coagulant are represented.

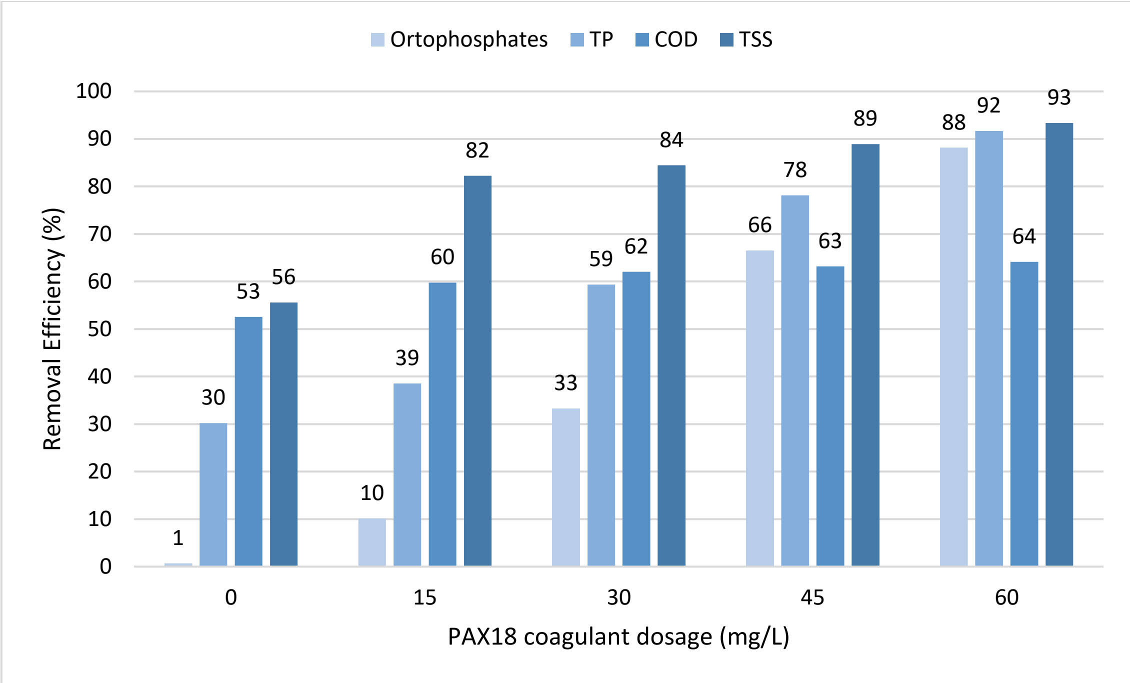


Figure 5.1-5: PAX 18 coagulant dosages and due removal percentage with Espinho WWTP wastewater.

The most significant difference was observed from the no chemicals added experiment to the 15 mg/L of PAX18, representing an increase of 27% of TSS removal, 7% of COD removal, 8% of TP removal and 9% of orthophosphates removal.

Higher dosages past the 15 mg/L of coagulant did not show much increased benefit mostly in terms of COD removal. Regarding the TSS, the increased coagulant dosage past the 15 mg/L did not significantly increase its removal. It was verified a 11% increase in TSS removal from 15 mg/L to 60 mg/L of PAX18.

The results demonstrate that from 15 mg/L to 30 mg/L of PAX18, COD and TSS removal increased solely by 2%; from 30 mg/L to 45 mg/L of PAX18, COD removal increased by 1% and TSS removal increased by 4%; from 45 mg/L to 60 mg/L of PAX18, the same trend was observed resulting in the increase of COD removal by 1% and TSS removal by 4%.

In contrast, it was demonstrated that the removal of TP and orthophosphates increased greatly in the higher dosages of PAX18, showing greater benefit even at the higher dosages tested. From 15 mg/L to 30 mg/L of PAX18, it was observed an increased removal of TP by 21% and orthophosphates removal by 23%; from 30 to 45 mg/L of PAX18, TP removal increased by 19% and orthophosphates removal increased by 33%; from 45 to 60 mg/L of PAX18, TP removal increased by 14% while orthophosphates removal increased by 22%.

5.2 Mass Balances

In this chapter it is presented the mass balances for Espinho WWTP with conventional primary treatment and with CEPT, for all the three scenarios considered in this study.

In figure 5.2-1, Espinho treatment line is displayed with the affluent/effluent of each process numbered to identify the components in the mass balances. Each individual component is described in table 5.2-1.

The mass balances of the WWTP with conventional primary treatment for scenario 1,2 and 3 are displayed in tables 5.2-2, 5.2-3 and 5.2-4. In scenario 3 there is not enough phosphorus for the biological treatment to occur. Therefore, the addition of 23 kg TP/d is a requirement for consistent and proper biological treatment. This phosphorus load increment is already considered in scenario 3 mass balance.

The mass balances with CEPT of scenario 1,2 and 3 are displayed in tables 5.2-5, 5.2-6 and 5.2-7. In scenario 3 with CEPT the phosphorus shortage does not occur. There is effectively less carbonated matter affluent to the biological reactor, which leads to a lower nutrient demand in the biological treatment, for the conversion of the organic matter.

The specific sludge production, obtained via the program DenikaPlus, utilized to determine the mass balances for each scenario considered is visible below.

Specific sludge production – WWTP treatment line with conventional primary treatment:

- Scenario 1: 0.890 kg TSS/kg BOD_5;
- Scenario 2: 0.899 kg TSS/kg BOD5;
- Scenario 3: 1.050 kg TSS/kg BOD5.

Specific sludge production – WWTP treatment line with CEPT:

- Scenario 1: 0.783 kg TSS/kg BOD5;
- Scenario 2: 0.790 kg TSS/kg BOD5;
- Scenario 3: 0.895 kg TSS/kg BOD5.

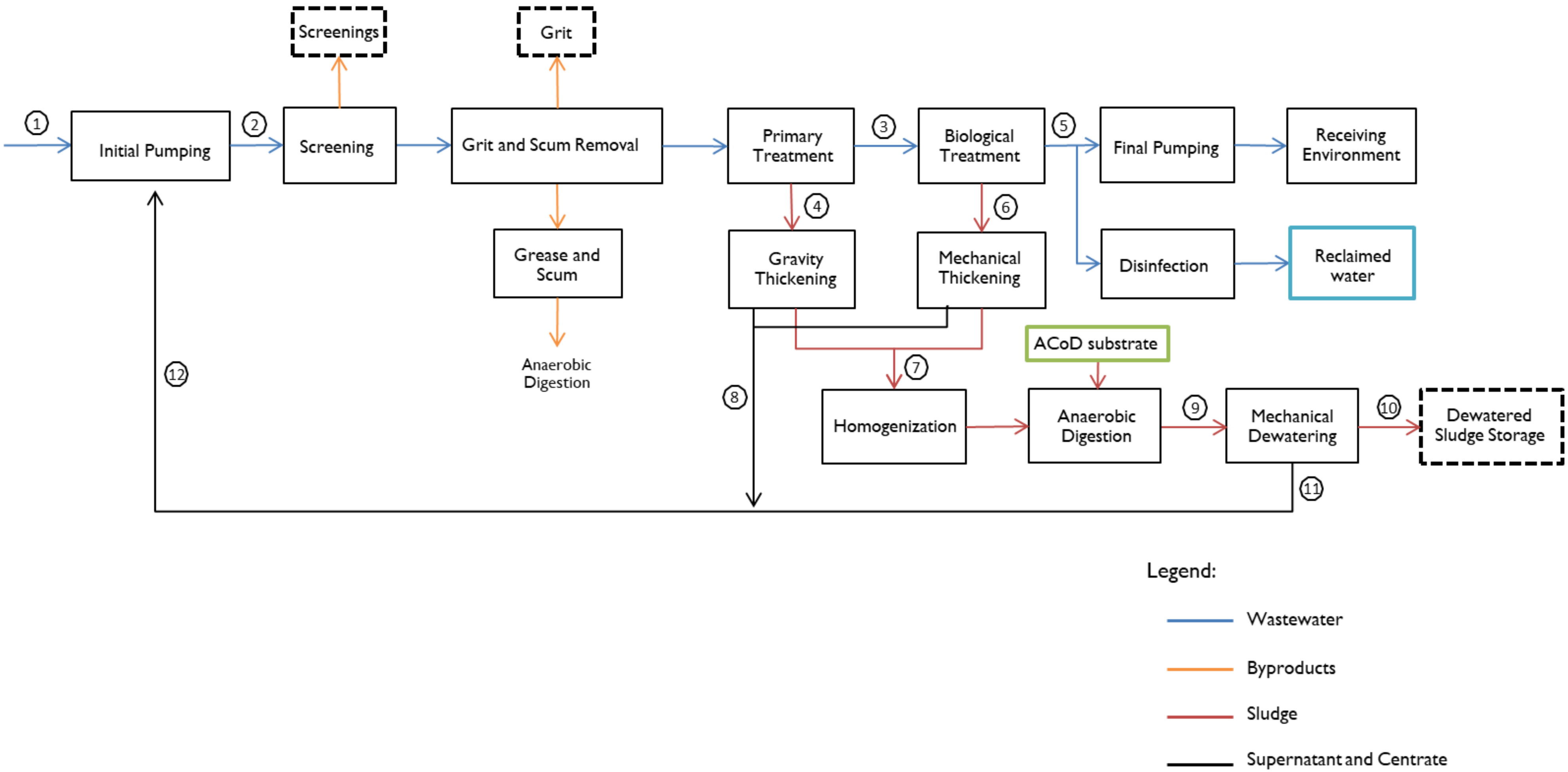


Figure 5.2-1: Espinho WWTP treatment line and components numbering.

Table 5.2-1: Number of the mass balance components in Espinho WWTP treatment line.

Number	Mass Balance components
1	Affluent wastewater
2	Affluent wastewater + returns
3	Primary effluent
4	Primary sludge or chemically enhanced primary sludge (*)
5	Secondary effluent
6	Biological sludge
7	Thickened sludge
8	Thickener supernatant
9	Digested sludge (digestate)
10	Dewatered sludge
11	Dewatering centrate
12	Total returns

*Chemically enhanced primary sludge when CEPT is considered.

Mass Balances with Conventional Primary Treatment

Table 5.2-2: Mass balance of Scenario 1 (with conventional primary treatment).

Parameter		Units	1	2	3	4	5	6	7	8	9	10	11	12
Average flow-rate		m^3/d	23,627.0	24,973.0	24,857.4	115.6	23,614.6	1,242.8	148.9	1,209.5	148.9	17.4	136.5	1,346.0
Loads	BOD_5	kg/d	5,562.0	5,984.5	4,189.1	1,795.3	590.4	-	-	-	-	-	-	422.5
	COD	kg/d	12,000.0	12,845.0	8,991.5	3,853.5	2,951.8	-	-	-	-	-	-	845.0
	TSS	kg/d	4,935.0	5,780.0	2,890.0	2,890.0	826.5	3,728.3	5,956.5	661.8	3,663.3	34,80.1	183.2	845.0
	VSS	kg/d	3,454.5	4,011.6	2,023.0	2,023.0	578.6	2,609.8	4,169.6	463.3	1,876.3	1,782.5	93.8	557.1
	TN	kg/d	731.0	1,039.5	926.1	113.5	525.6	400.5	420.4	93.6	420.4	205.4	214.9	308.5
	TP	kg/d	101.0	117.9	106.1	11.8	31.5	74.6	73.1	13.2	73.1	69.5	3.7	16.9
Concentrations	BOD_5	mg/L	235.4	239.6	168.5	15530.7	25.0	-	-	-	-	-	-	313.9
	COD	mg/L	507.9	514.4	361.7	33334.8	125.0	-	-	-	-	-	-	627.8
	TSS	mg/L	208.9	231.5	116.3	25000.0	35.0	3,000.0	40,000.0	547.2	24,600.0	200,000	1,341.7	627.8
	VSS	mg/L	146.2	160.6	81.4	17500.0	24.5	2,100.0	28,000.0	383.0	12,600.0	102,439	687.2	413.9
	TN	mg/L	30.9	41.6	37.3	981.8	22.3	322.3	2822.8	77.4	2,822.8	11,807	1,574.3	229.2
	TP	mg/L	4.3	4.7	4.3	102.0	1.3	60.0	491.0	10.9	491.0	3,992	26.8	12.6

Table 5.2-3: Mass balance of Scenario 2 (with conventional primary treatment).

Parameter		Units	1	2	3	4	5	6	7	8	9	10	11	12
Average flow-rate		m^3/d	20,578.0	22,361.0	22,205.5	155.5	20,559.8	1,645.7	198.5	1,602.6	198.5	23.2	180.3	1,783.0
Loads	BOD_5	kg/d	7,282.0	7,845.3	5,491.7	2,353.6	514.0	-	-	-	-	-	-	563.3
	COD	kg/d	17,664.0	18,790.7	13,153.5	5,637.2	2,570.0	-	-	-	-	-	-	1,126.7
	TSS	kg/d	6,648.0	7,774.7	3,887.3	3,887.3	719.6	4,937.1	7,942.0	882.4	4884.3	4,640.1	244.2	1,126.7
	VSS	kg/d	4,653.6	5,396.4	2,721.1	2,721.1	503.7	3,455.9	5,559.4	617.7	2501.7	2,376.6	125.1	742.8
	TN	kg/d	896.0	1,310.7	1,167.0	143.7	623.4	543.6	549.2	138.0	549.2	272.6	276.6	414.7
	TP	kg/d	156.0	178.5	160.7	17.9	61.9	98.7	98.9	17.6	98.9	94.1	4.9	22.5
Concentrations	BOD_5	mg/L	353.9	350.8	247.3	15,136.4	25.0	-	-	-	-	-	-	315.9
	COD	mg/L	858.4	840.3	592.4	36,253.7	125.0	-	-	-	-	-	-	631.9
	TSS	mg/L	323.1	347.7	175.1	25,000.0	35.0	3,000.0	40,000.0	550.6	24,600.0	200,000.0	1,354.1	631.9
	VSS	mg/L	226.1	241.3	122.5	17,500.0	24.5	2,100.0	28,000.0	385.4	12,600.0	10,2439	693.6	416.6
	TN	mg/L	43.5	58.6	52.6	923.9	30.3	330.3	2,766.2	86.1	2,766.2	1,1750	1,533.8	232.6
	TP	mg/L	7.6	8.0	7.2	114.8	3.0	60.0	498.3	11.0	498.3	4054	27.1	12.6

Table 5.2-4: Mass balance of Scenario 3 (with conventional primary treatment).

Parameters		Units	1	2	3	4	5	6	7	8	9	10	11	12
Average flow-rate		m^3/d	36,361.0	37,944.0	37,781.3	162.6	36,344.0	1,437.4	188.5	1,411.5	188.5	22.0	171.5	1,583.0
Loads	BOD_5	kg/d	5,332.0	5,866.8	4,106.8	1,760.0	908.6	-	-	-	-	-	-	534.8
	COD	kg/d	13,093.0	14,162.6	9,913.8	4,248.8	4,543.0	-	-	-	-	-	-	1,069.6
	TSS	kg/d	7,061.0	8,130.6	4,065.3	4,065.3	1,272.0	4,312.1	7,539.6	837.7	4,636.9	4405.0	231.8	1,069.6
	VSS	kg/d	4,942.7	5,647.9	2,845.7	2,845.7	890.4	3,018.5	5,277.8	586.4	2,375.0	2256.2	118.7	705.2
	TN	kg/d	571.0	864.4	764.9	99.5	321.0	443.9	446.3	97.1	446.3	250.0	196.3	293.4
	TP	kg/d	49.0	70.4	86.2(*)	7.0	0.0	86.2	76.5	16.8	76.5	71.9	4.6	21.4
Concentrations	BOD_5	mg/L	146.6	154.6	108.7	10,823.6	25.0	-	-	-	-	-	-	337.8
	COD	mg/L	360.1	373.3	2,62.4	26,128.4	125.0	-	-	-	-	-	-	675.7
	TSS	mg/L	194.2	214.3	107.6	25,000.0	35.0	3,000.0	40,000.0	593.5	24,600.0	200,000.0	1,352.1	675.7
	VSS	mg/L	135.9	148.8	75.3	17,500.0	24.5	2,100.0	28,000.0	415.5	12,600.0	102,439	692.6	445.5
	TN	mg/L	15.7	22.8	20.2	612.1	8.8	308.8	2,367.8	68.8	2,367.8	11,352	1,144.8	185.4
	TP	mg/L	1.3	1.9	2.3	43.3	0.0	60.0	406.0	11.9	406.0	3264	27.0	13.5

(*) Accounting for an introduction of 23 kg P/d.

Mass Balances with CEPT

Table 5.2-5: Mass balance of Scenario 1 (with CEPT).

Parameters		Units	1	2	3	4	5	6	7	8	9	10	11	12
Average flow-rate		m^3/d	23,627.0	24,877.4	24,552.7	324.7	23,614.3	938.5	151.8	1,111.4	151.8	17.7	139.1	1,250.4
Loads	BOD_5	kg/d	5,562.0	5,992.7	2,397.1	3,595.6	590.4	-	-	-	-	-	-	430.7
	COD	kg/d	12,000.0	12,861.5	5,144.6	7,716.9	2,951.8	-	-	-	-	-	-	861.5
	TSS	kg/d	4,935.0	5,796.5	1,159.3	4,870.7	826.5	1,876.9	6,072.8	674.8	3,734.8	3,548.1	186.7	861.5
	VSS	kg/d	3,454.5	4,057.5	811.5	3,409.5	578.5	1,313.9	4,251.0	472.3	1,912.9	2,483.6	130.7	603.0
	TN	kg/d	731.0	975.1	743.5	231.6	534.5	208.9	347.4	93.1	347.4	196.5	151.0	244.1
	TP	kg/d	101.0	118.2	72.1	46.1	34.6	37.5	70.2	13.5	70.2	66.4	3.7	17.2
Concentrations	BOD_5	mg/L	235.4	240.9	97.6	11,073.4	25.0	-	-	-	-	-	-	344.5
	COD	mg/L	507.9	517.0	209.5	23,765.4	125.0	-	-	-	-	-	-	689.0
	TSS	mg/L	208.9	233.0	47.2	15,000.0	35.0	2,000.0	40,000.0	607.2	24,600.0	200,000.0	1,342.7	689.0
	VSS	mg/L	146.2	163.1	33.1	10,500.0	24.5	1,400.0	28,000.0	425.0	12,600.0	0.0	939.9	482.3
	TN	mg/L	30.9	39.2	30.3	713.3	22.6	222.6	2,288.4	83.8	2,288.4	0.0	1,085.4	195.2
	TP	mg/L	4.3	4.8	2.9	142.0	1.5	40.0	462.1	12.1	462.1	0.0	26.9	13.8

Table 5.2-6: Mass balance of Scenario 2 (with CEPT).

Parameters		Units	1	2	3	4	5	6	7	8	9	10	11	12
Average flow-rate		m^3/d	20,578.0	21,980.7	21,551.9	428.8	20,559.6	992.4	200.5	1,220.6	200.5	23.4	182.1	1,402.7
Loads	BOD_5	kg/d	7,282.0	7,851.0	3,140.4	4,710.6	514.0	-	-	-	-	-	-	569.0
	COD	kg/d	17,664.0	18,802.0	7,520.8	11,281.2	2,569.9	-	-	-	-	-	-	1,138.0
	TSS	kg/d	6,648.0	7,786.0	1,557.2	6,432.1	719.6	2,480.9	8021.7	891.3	4,933.4	4,686.7	246.7	1,138.0
	VSS	kg/d	4,653.6	5,450.2	1,090.0	4,502.5	503.7	1,736.6	5615.2	623.9	2,526.8	3,280.7	172.7	796.6
	TN	kg/d	896.0	1,210.5	917.1	293.4	638.2	278.9	444.1	128.3	444.1	257.8	186.3	314.5
	TP	kg/d	156.0	178.8	109.0	69.7	59.4	49.6	101.5	17.8	101.5	96.6	4.9	22.8
Concentrations	BOD_5	mg/L	353.9	357.2	145.7	10,985.3	25.0	-	-	-	-	-	-	405.6
	COD	mg/L	858.4	855.4	349.0	26,308.3	125.0	-	-	-	-	-	-	811.2
	TSS	mg/L	323.1	354.2	72.3	15,000.0	35.0	2,500.0	40,000.0	730.2	24,600.0	200,000.0	1,354.5	811.2
	VSS	mg/L	226.1	248.0	50.6	10,500.0	24.5	1,750.0	28,000.0	511.1	12,600.0	0.0	948.2	567.9
	TN	mg/L	43.5	55.1	42.6	684.3	31.0	281.0	2,214.3	105.1	22,14.3	0.0	1,022.9	224.2
	TP	mg/L	7.6	8.1	5.1	162.6	2.9	50.0	506.2	14.6	506.2	0.0	27.1	16.2

Table 5.2-7: Mass balance of Scenario 3 (with CEPT).

Parameters		Units	1	2	3	4	5	6	7	8	9	10	11	12
Average flow-rate		m^3/d	36,361.0	37,862.2	37,400.0	462.2	36,342.2	1,057.8	203.6	1,316.4	203.6	23.8	184.8	1501.2
Loads	BOD_5	kg/d	5,332.0	5,909.6	2,363.8	3,545.8	908.6	-	-	-	-	-	-	577.6
	COD	kg/d	13,093.0	14,248.2	5,699.3	8,548.9	4,542.8	-	-	-	-	-	-	1155.2
	TSS	kg/d	7,061.0	8,216.2	1,643.2	6,932.4	1,272.0	2,115.6	8,143.3	904.8	5,008.1	4,757.7	250.4	1155.2
	VSS	kg/d	4,942.7	5,751.3	1,150.3	4,852.7	890.4	1,480.9	5,700.3	633.4	2,565.1	3,330.4	175.3	808.6
	TN	kg/d	571.0	752.0	545.0	206.9	324.0	221.0	324.7	103.2	324.7	247.0	77.8	181.0
	TP	kg/d	49.0	72.1	44.0	28.1	1.7	42.3	52.3	18.1	52.3	47.3	5.0	23.1
Concentrations	BOD_5	mg/L	146.6	156.1	63.2	7,672.1	25.0	-	-	-	-	-	-	384.8
	COD	mg/L	360.1	376.3	152.4	18,497.7	125.0	-	-	-	-	-	-	769.5
	TSS	mg/L	194.2	217.0	43.9	15,000.0	35.0	2,000.0	40,000.0	687.3	24,600.0	200,000.0	1,355.1	769.5
	VSS	mg/L	135.9	151.9	30.8	10,500.0	24.5	1,400.0	28,000.0	481.1	12,600.0	0.0	948.5	538.7
	TN	mg/L	15.7	19.9	14.6	447.7	8.9	208.9	1,595.2	78.4	1,595.2	0.0	420.9	120.5
	TP	mg/L	1.3	1.9	1.2	60.8	0.0	40.0	257.1	13.7	257.1	0.0	27.1	15.4

5.3 WWTP Performance Verification

This section is dedicated to the verification of the WWTP performance with conventional primary treatment, prior to the upgrades evaluated in this work.

5.3.1 Conventional Primary Treatment

In table 5.3-1, it is represented the performance results of the primary treatment when faced with the affluent conditions of the studied scenarios.

It was considered the usage of the 3 clarifiers in all scenarios.

Table 5.3-1: Conventional primary treatment design verification results for the scenarios considered.

Parameters	Units	Scenario 1	Scenario 2	Scenario 3	Design Flowrate
Peak hourly flowrate	m^3/h	1,041	932	1,581	2,868
Overflow rate	$m^3/(m^2.h)$	0.9	0.8	1.4	2.5
Detention time	h	2.9	3.3	1.9	1.1

5.3.2 Biological Treatment

Espinho WWTP aeration tank has 3 available lines of 2,124 m^3 each. For optimal process performance, as well as, to provide enough aeration (with the mechanical aeration equipment installed) it was considered the usage of two treatment lines in scenario 1 and 3 and the usage of three treatment lines in scenario 2, by manipulating the MLSS in the aeration tank.

In table 5.3-2, it is displayed the biological treatment design verification results for the scenarios considered.

Table 5.3-2: Biological reactor design performance verification results.

Parameters	Units	Scenario 1	Scenario 2	Scenario 3
Affluent organic load (BOD_5)	kg/d	4,189	5,492	4,107
MLSS	kg/m^3	2.67	2.7	3.15
Mass of suspended solids	kg	11,342	17,204	13,381
Vat required	m^3	4,248	6,372	4,248
Nº zones required in the AT	nº	2	3	2

Parameters	Units	Scenario 1	Scenario 2	Scenario 3
Sludge Age	d	3.0	3.5	3.1
F/M	kg BOD_5/(kg MLVSS.d)	0.37	0.32	0.31
BOD_5 Volume Loading Rate (BR)	kg/(m^3.d)	0.99	0.86	0.97
Daily excess waste activated sludge	kg TSS/d	3,728	4,937	4,312
OUdc/AOR	kg O_2/d	4,419	5,952	4,350

Secondary Clarifier

It is considered the usage of the 4 secondary clarifiers in all scenarios.

In the table 5.3-3, it is displayed the secondary sedimentation results.

Table 5.3-3: Secondary sedimentation results for the scenarios considered.

Parameters	Units	Scenario 1	Scenario 2	Scenario 3	Design flow rate
Flow rate	m^3/h	1,036	925	1,574	2,868
Overflow rate	m^3/(m^2.h)	0.4	0.3	0.6	1.1
Hydraulic retention time	h	6.8	7.7	4.5	2.5

5.3.3 Aeration

The biological reactor is aerated via turbines, in table 5.3-4 it is displayed the aeration design verification results.

Table 5.3-4: Turbine aeration results for the scenarios considered.

Parameter	Units	Scenario 1	Scenario 2	Scenario 3
AOR	kg O_2/d	4,419	5,952	4,350
Peak factor for carbon	-	1.3	1.3	1.3
AOR with peak factor	kg O_2/h	239	322	236
Reactor surface water height	m	3.91	3.91	3.91
Oxygen concentration required	mg/l	2	2	2
Effective saturation depth (de)	m	0.23	0.23	0.23
Temperature correction (τ)	-	0.86	0.86	0.86
SOTR	kg O_2/h	452	609	445
Installed power	kW	270	405	270

Parameter	Units	Scenario 1	Scenario 2	Scenario 3
Energy consumption	kWh	251	338	247
Turbine operation time	h/d	22.3	20.1	22.0

The WWTP aeration represents: 50% of the total energy needs in scenario 1; 58% of the total energy needs in scenario 2; 46% of the total energy needs in scenario 3.

5.3.4 Sludge Thickening

Gravity Thickening

In table 5.3-5, it is displayed the gravity thickening design verification results.

Table 5.3-5: Gravity thickening results for the scenarios considered.

Parameters	Units	Scenario 1	Scenario 2	Scenario 3
Affluent sludge flow rate	m^3/d	115.6	155.5	162.6
Affluent sludge TSS load	kg/d	2,890	3,887	4,065
Solids loading	$kg/(m^2.d)$	18.4	24.7	25.9
Overflow rate	$m^3/(m^2.d)$	0.7	1.0	1.0

Mechanical Thickening

In table 5.3-6, it is displayed the mechanical thickening design verification results.

Table 5.3-6: Mechanical thickening results for the scenarios considered.

Parameters	Units	Scenario 1	Scenario 2	Scenario 3
TSS affluent load	kg TSS/d	3,728	4,937	4,312
Number of rotary drum thickeners required	-	2	2	2
Minimum flow rate per thickening drum	m^3/h	29.6	32.7	29.8
Polyelectrolyte required	kg/d	19	25	22
Thickening operation time	h/d	20	24	23

5.3.5 Anaerobic Digestion

The literature review shows that the most probable VS destruction value is in between 50 to 60% but, according to the WEF formula, the destruction of VS is superior than 55% in every scenario. The results of the destruction of VS by the WEF formula, show the maximum possible destruction of VS and, as a result, were not utilized in the CHP energy production simulation. They are only shown to provide data of the maximum energy production if the VS destruction percentage values, of the WEF formula, apply. The WEF formula, VS destruction results are shown below.

- Scenario 1: 67%;
- Scenario 2: 63%;
- Scenario 3: 64%.

In table 5.3-7, it is displayed the anaerobic digestion design verification results.

Table 5.3-7: Anaerobic digestion design verification results.

Parameters	Unit	Scenario 1	Scenario 2	Scenario 3
Thickened sludge load	kg TSS/d	5,956.5	7,941.9	7,539.6
Thickened sludge flow rate	m^3/d	148.9	198.5	188.5
Volatile solids loading	kg VS/(m^3.d)	0.83	1.1	1.05
Hydraulic retention time	d	33.8	25.3	26.7
Required thermal energy (digester heat losses and sludge heating)	J/d	1.4E+10	1.88E+10	1.79E+10
Produced biogas volume	m^3	2,063.9	2,751.9	2,612.5
CHP functioning time	h	22	22	22
Thermal energy produced	J/h	9.08E+08	1.21E+09	1.15E+09
Electric energy produced	kWh/d	4,098	5,465	5,188

The combustion of the biogas produced in the CHP unit leads to an electrical energy production of:

- Scenario 1: 4,098 kWh/d (1,495,944 kWh/y);
- Scenario 2: 5,465 kWh/d (1,994,578 kWh/y);
- Scenario 3: 5,188 kWh/d (1,893,540 kWh/y).

Considering the VS destruction from the WEF formula, the combustion of the biogas in the CHP unit leads to an electrical energy production of:

- Scenario 1: 4,993 kWh/d (22% increase from the 55% VS destruction);
- Scenario 2: 6,259 kWh/d (15% increase from the 55% VS destruction);
- Scenario 3: 6,037 kWh/d (16% increase from the 55% VS destruction).

Considering a CHP unit with an efficiency of 42% (instead of 35%) in producing electrical energy, the combustion of the biogas in the CHP unit leads to an electrical energy production of:

- Scenario 1: 4,918 kWh/d (20% increase from the CHP unit with 35% efficiency);
- Scenario 2: 6,558 kWh/d (20% increase from the CHP unit with 35% efficiency);
- Scenario 3: 6,225 kWh/d (20% increase from the CHP unit with 35% efficiency).

5.3.6 Sludge Dewatering

In table 5.3-8, it is displayed the sludge dewatering design verification results.

Table 5.3-8: Sludge dewatering design verification results.

Parameters	Units	Scenario 1	Scenario 2	Scenario 3
Digested sludge load	kg/d	3,663	4,884	4,637
Digested sludge flow rate	m^3/d	149	199	186
Centrifuge operation time	h/d	7.7	10.2	9.7
Polyelectrolyte required	kg/d	29	39	37

5.4 WWTP Performance Verification with CEPT

This section is dedicated to the verification of the WWTP design and performance following the CEPT implementation.

5.4.1 Chemically Enhanced Primary Treatment

The primary treatment design principle is the flow rate, which does not change when chemicals are applied. The circumstances (flow rate and the number of clarifiers required) when CEPT is performed are similar to the conventional primary treatment. The difference is observed in the produced sludge and in the clarified wastewater conditions. Consequently, the wastewater returns, to the beginning of the WWTP, are different, leading to different wastewater conditions in terms of flow rate and load in the preliminary treatment and in downstream processes.

In table 5.4-1, it can be observed the CEPT design verification results.

It was considered the usage of the three clarifiers in all scenarios.

Table 5.4-1: Chemically Enhanced primary treatment design verification results for the scenarios considered.

Parameters	Units	Scenario 1	Scenario 2	Scenario 3	Design Flowrate
Peak hourly flowrate	m^3/h	1,037	916	1,578	2,868
Overflow rate	$m^3/(m^2.h)$	0.9	0.8	1.4	2.5
Detention time	h	3.0	3.3	1.9	1.1
PAX required	kg/d	373	330	568	-
Polyelectrolyte required	kg/d	12	11	19	-

5.4.2 Biological Treatment

Espinho WWTP aeration tank has 3 available lines of 2,124 m^3 each. For optimal process performance as well as aeration cost efficiency it was considered the usage of 2 treatment lines in all scenarios by manipulating the MLSS in the aeration tank.

In the table 5.4-2, it is represented the performance results of the biological treatment. The OUdc, or more commonly known as AOR (actual oxygen requirement), represents the daily oxygen uptake for carbon removal, this value lacks the peak factor for carbon respiration.

Table 5.4-2: Biological reactor design performance verification results.

Parameters	Units	Scenario 1	Scenario 2	Scenario 3
Affluent organic load (BOD_5)	kg/d	2,397	3,140	2,364
MLSS	kg/m^3	1.57	1.98	1.79
Mass of suspended solids	kg	6,648	8,389	7,604
Vat required	m^3	4,248	4,248	4,248
Nº zones required in the AT	nº	2	2	2
Sludge Age	d	3.5	3.4	3.6
F/M	kg BOD_5/(kg MLVSS.d)	0.36	0.37	0.31
BOD_5 Volume Loading Rate (BR)	kg/(m^3.d)	0.56	0.74	0.56
Daily excess waste activated sludge	kg TSS/d	1,877	2,481	2,116
OUdc/AOR	kg O_2/d	2,607	3,384	2,578

According to the ATV guidelines the return sludge pumps (including reserve equipment) should be designed for a flow rate of 2,868 m^3/h.

Secondary Clarifier

In table 5.4-3, it is displayed the secondary sedimentation design verification results.

Table 5.4-3: Secondary sedimentation results for the scenarios considered.

Parameters	Units	Scenario 1	Scenario 2	Scenario 3	Design flow rate
Flow rate	m^3/h	1,023	898	1,558	2,868
Overflow rate	m^3/(m^2.h)	0.4	0.3	0.6	1.1
Hydraulic retention time	h	6.9	7.9	4.5	2.5

5.4.3 Aeration

The biological reactor is aerated via turbines, in table 5.4-4 it is displayed the aeration design verification results.

Table 5.4-4: Turbine aeration results for the scenarios considered.

Parameter	Units	Scenario 1	Scenario 2	Scenario 3
AOR	kg O_2/d	2,607	3,384	2,578
Peak factor for carbon	-	1.3	1.3	1.3
AOR with peak factor	kg O_2/h	141	183	139
Reactor surface water height	m	3.91	3.91	3.91
Oxygen concentration required	mg/l	2	2	2
Effective saturation depth (de)	m	0.23	0.23	0.23
Temperature correction (τ)	-	0.86	0.86	0.86
SOTR	kg O_2/h	267	346	264
Installed power	kW	270	270	270
Energy consumption	kWh	148	192	147
Turbine operation time	h/d	13.2	17.1	13.0

The aeration energy consumption, when performing CEPT, represents: 37% of the total energy needs in scenario 1; 44% of the total energy needs in scenario 2; 34% of the total energy needs in scenario 3.

5.4.4 Sludge Thickening

Gravity Thickening

In table 5.4-5, it is displayed the gravity thickening design verification results.

Table 5.4-5: Gravity thickening results for the scenarios considered.

Parameters	Units	Scenario 1	Scenario 2	Scenario 3
Affluent sludge flow rate	m^3/d	324.7	428.8	462.2
Affluent sludge TSS load	kg/d	4,871	6,432	6,932
Solids loading	kg/(m^2.d)	31.0	40.9	44.1
Overflow rate	m^3/(m^2.d)	2.1	2.8	2.9

Mechanical Thickening

In table 5.4-6, it is displayed the mechanical thickening design verification results.

Table 5.4-6: Mechanical thickening results for the scenarios considered.

Parameters	Units	Scenario 1	Scenario 2	Scenario 3
TSS affluent load	kg TSS/d	1,877	2,481	2,116
Number of rotary drum thickeners required	-	2	2	2
Minimum flow rate per thickening drum	m^3/h	29.8	29.5	29.6
Polyelectrolyte required	kg/d	9	12	11
Thickening operation time	h/d	15	16	17

5.4.5 Anaerobic Digestion

The literature review shows that the most probable VS destruction value is in between 50 to 60% but according to the WEF formula, the destruction of VS was superior than 55% in every scenario. The results of the destruction of VS by the WEF formula, show the maximum possible destruction of VS and as a result were not utilized in the CHP energy production simulation. They are only shown to provide data of the maximum energy production if the VS destruction percentage values of the WEF formula apply. The WEF formula, VS destruction results, with CEPT in the treatment line, are shown below.

- Scenario 1: 67%;
- Scenario 2: 63%;
- Scenario 3: 63%.

In the CHP unit, the chemically produced sludge from CEPT, was not considered as it does not contribute to the system energy production. In the table 5.4-7, it is displayed the anaerobic digestion design verification results.

Table 5.4-7: Anaerobic digestion design verification results.

Parameters	Unit	Scenario 1	Scenario 2	Scenario 3
Thickened sludge load	kg TSS/d	6,072.8	8,021.7	8,143.3
Thickened sludge flow rate	m^3/d	151.8	200.5	203.6
Volatile solids loading	kg VS/(m^3.d)	0.81	1.09	1.08
Hydraulic retention time	d	33.1	25.1	24.7

Parameters	Unit	Scenario 1	Scenario 2	Scenario 3
Required thermal energy (digester heat losses and sludge heating)	J/d	1.44E+10	1.90E+10	1.93E+10
Produced biogas volume	m^3	2,023.3	2,709.1	2,697.1
CHP functioning time	h	22	22	22
Thermal energy produced	J/h	8.90E+08	1.19E+09	1.19E+09
Electric energy produced	kWh/d	4,018	5,380	5,356

The energy produced via AD, in Espinho WWTP with CEPT, is underestimated as the primary sludge possesses a higher energy value that was not considered. In this study the primary and secondary sludge have the same energy value.

The combustion of the biogas produced in the CHP unit leads to an electrical energy production of:

- Scenario 1: 4,018 kWh/d (1,466,526 kWh/y);
- Scenario 2: 5,380 kWh/d (1,963,545 kWh/y);
- Scenario 3: 5,356 kWh/d (1,954,859 kWh/y).

Considering the VS destruction from the WEF formula, the combustion of the biogas in the CHP unit leads to an electrical energy production of:

- Scenario 1: 4,895 kWh/d (22% increase from the 55% VS destruction);
- Scenario 2: 6,162 kWh/d (15% increase from the 55% VS destruction);
- Scenario 3: 6,135 kWh/d (15% increase from the 55% VS destruction).

Considering a CHP unit with an efficiency of 42% (instead of 35%) in producing electrical energy, the combustion of the biogas in the CHP unit leads to an electrical energy production of:

- Scenario 1: 4,821 kWh/d (20% increase from the CHP unit with 35% efficiency);
- Scenario 2: 6,455 kWh/d (20% increase from the CHP unit with 35% efficiency);
- Scenario 3: 6,427 kWh/d (20% increase from the CHP unit with 35% efficiency).

5.4.6 Sludge Dewatering

The sludge dewatering design verification results are expressed in table 5.4-8.

Table 5.4-8: Sludge dewatering design verification results.

Parameters	Units	Scenario 1	Scenario 2	Scenario 3
Digested sludge load	kg/d	3,735	4,933	5,008
Digested sludge flow rate	m^3/d	152	201	204
Centrifuge operation time	h/d	7.8	10.3	10.5
Polyelectrolyte required	kg/d	30	39	40

5.5 Fine Bubble Aeration

5.5.1 Treatment Line with Conventional Primary Treatment

In table 5.5-1, it is displayed the diffused air aeration design results for the scenarios considered with conventional primary treatment.

Table 5.5-1: Fine bubble aeration results for the scenarios considered.

Parameter	Unit	Scenario 1	Scenario 2	Scenario 3
AOR	kg O_2/d	4,419	5,952	4,350
peak AOR	kg O_2/h	239	322	236
SOTR	kg O_2/h	702	945	691
Number of reactors	-	2	3	2
SOTR per reactor	kg O_2/h	351	315	345
Air flow rate per reactor	m^3/h	5,898	5,296	5,806
Required air flow rate	m^3/h	11,796	15,889	11,612
	m^3/min	197	265	194
Blower air flow rate	m^3/min	90		
Number of required blowers	-	3	3	3
Blowers oxygen transfer	kg O_2/kWh	2.41	3.24	2.37

The air blower operation characteristics are displayed in table 5.5-2.

Table 5.5-2: Air blowers functioning characteristics.

Parameters	Units	Scenario 1	Scenario 2	Scenario 3
Required air flow rate	m^3/d	283,111	381,344	278,677
Blowers daily operation time	h/d	17.5	23.5	17.2

The oxygen transfer capability takes the value of 2.41 kg O_2/kWh, 3.24 kg O_2/kWh and 2.37 kg O_2/kWh for scenario 1,2 and 3 respectively.

5.5.2 Treatment Line with CEPT

In table 5.5-3, it is displayed the diffused air aeration design results for the scenarios considered with CEPT.

Table 5.5-3: Fine bubble aeration results for the scenarios considered.

Parameter	Unit	Scenario 1	Scenario 2	Scenario 3
AOR	kg O_2/d	2,607	3,384	2,578
peak AOR	kg O_2/h	141	183	140
SOTR	kg O_2/h	414	537	409
Number of reactors	-	2	2	2
SOTR per reactor	kg O_2/h	207	269	205
Air flow rate per reactor	m^3/h	3,480	4,517	3,441
Required air flow rate	m^3/h	6,960	9,034	6,882
	m^3/min	116	151	115
Blower air flow rate	m^3/min	79		
Number of required blowers	-	2	2	2
Blowers oxygen transfer	kg O_2/kWh	2.47	3.21	2.45

The air blower operation characteristics are displayed in table 5.5-4.

Table 5.5-4: Air blowers functioning characteristics.

Parameters	Units	Scenario 1	Scenario 2	Scenario 3
Required air flow rate	m^3/d	167,043	216,827	165,174
Blowers daily operation time	h/d	17.6	22.8	17.4

The oxygen transfer capability takes the value of 2.47 kg O_2/kWh, 3.21 kg O_2/kWh and 2.45 kg O_2/kWh for scenario 1,2 and 3 respectively.

5.6 Co-Digestion

Espinho WWTP with CEPT was studied, for the application of co-substrate in the anaerobic digesters, to perform AcoD, to enhance the methane/biogas production and consequent energy production.

AcoD is feasible to materialize due to unutilized available digesters volume.

5.6.1 Anaerobic Co-digestion

The AcoD substrate to be applied varies in all three scenarios. In table 5.6-1, it is displayed the amount of FW which is expected for application in the anaerobic digesters to perform co-digestion.

Table 5.6-1: FW substrate applied in the anaerobic digesters in each scenario.

Parameter	Unit	Scenario 1	Scenario 2	Scenario 3
Flow rate	m^3/d	180	135	130
TSS load	kg/d	9,000	6,750	6,500
VSS load	kg/d	6,300	4,725	4,550
Total TSS derived from the substrate	%	61	46	46
Total flow rate derived from the substrate	%	54	40	14

The literature review shows that the most probable VS destruction value is in between 50 to 60% but according to the WEF formula, the destruction of VS was superior than 55% in every scenario. The results of the destruction of VS by the WEF formula, show the maximum possible destruction of VS, and as a result, were not utilized in the CHP energy production simulation. They are only shown to provide data of the maximum energy production if the VS destruction percentage values of the WEF formula apply. The WEF formula, VS destruction results are shown below.

- Scenario 1: 56%;
- Scenario 2: 56%;
- Scenario 3: 56%.

The design verification results, as well as the energy production when performing co-digestion in the WWTP with CEPT scheme, are shown in table 5.6-2.

Table 5.6-2: AcoD design verification results.

Parameters	Unit	Scenario 1	Scenario 2	Scenario 3
Thickened sludge load	kg TSS/d	15,073	14,772	14,643
Thickened sludge flow rate	m^3/d	332	336	334
Volatile solids loading	kg VS/(m^3.d)	2.06	2.03	1.99
Hydraulic retention time	d	15	15	15
Required thermal energy (digester heat losses and sludge heating)	J/d	3.14E+10	3.17 E+10	3.16E+10
Produced biogas volume	m^3	5,142	5,048	4,949
CHP functioning time	h	22	22	22
Thermal energy produced	J/h	2.26E+09	2.22E+09	2.18E+09
Electric energy produced	kWh/d	10,210	10,024	9,828

Espinho WWTP anaerobic digesters allow the introduction of a maximum flow rate of 336 m^3/d, in both thickened sludge and co-substrate. This flow rate leads to the minimum hydraulic retention time of 15 days.

The produced biogas combustion in the CHP unit leads to an electrical energy production of:

- Scenario 1: 10,210 kWh/d (3,726,826 kWh/y);
- Scenario 2: 10,024 kWh/d (3,658,770 kWh/y);
- Scenario 3: 9,828 kWh/d (3,587,299 kWh/y).

The implementation of AcoD increases the biogas production by: 3,119 m^3/d in scenario 1; 2,339 m^3/d in scenario 2; 2,252 m^3/d in scenario 3.

Considering the VS destruction from the WEF formula, the combustion of the biogas in the CHP unit leads to an electrical energy production of:

- Scenario 1: 10,396 kWh/d (2% increase from the 55% VS destruction);
- Scenario 2: 10,206 kWh/d (2% increase from the 55% VS destruction);
- Scenario 3: 10,007 kWh/d (2% increase from the 55% VS destruction).

Considering a CHP unit with an efficiency of 42% (instead of 35%) in producing electrical energy, the combustion of the biogas in the CHP unit leads to an electrical energy production of:

- Scenario 1: 12,253 kWh/d (20% increase in energy production);
- Scenario 2: 12,029 kWh/d (20% increase in energy production);
- Scenario 3: 11,794 kWh/d (20% increase in energy production).

5.7 Photovoltaic Solar Panels

The areas intended for the installation of the PV solar panels are displayed in yellow, in figure 5.7-1, which represents a google earth aerial view of Espinho WWTP.

Figure 5.7-1: Photovoltaic solar panel installation areas considered in Espinho WWTP (Google Maps, 2018).

The areas considered for installation are described in table 5.7-1.

Table 5.7-1: Usable area and number of PV solar panel modules.

Parameters	Units	Area 1	Area 2	Area 3	Total
Usable Length	m	160	71	40	-
Usable Width	m	14	44	11	-
Area	m^2	2,240	3,124	440	5,804
Nº of modules	-	480	710	80	1,270

The usable area for module installation is 5,804 m^2. In this area it is projected the installation of photovoltaic solar panel modules totaling 315 kW of power.

According to the website utilized, Espinho WWTP solar radiation exposure results, show a photovoltaic (PV) solar energy production of 434,959 kWh/y with the modules projected for installation. The results of the website are displayed in Annex F.

5.8 Energy Balance and Operation Costs

The total energy consumption in the WWTP was calculated at 4,363,250 kWh/y, considering the WWTP treatment line with conventional primary treatment (scenario 1). When CEPT is considered, the energy consumption decreases to 3,467,508 kWh/y (scenario 1). These values consider the aeration is done via turbines, with fine bubble aeration these values would be inferior.

In tables 5.8-1, 5.8-2, 5.8-3, it is displayed the energy/reagent consumption as well as the energy balance in the WWTP with conventional primary treatment and with CEPT.

The sludge pumping costs are described in Annex G, in which it is displayed the power installed, time of operation , accepted flow rate and the correspondent energy expenses.

.Table 5.8-1: WWTP energy costs.

Considerations	Scenarios	Biogas compressor	Deodorization	WAS Recirculation Pumping	Sludge Pumping (excluding WAS recirculation)	Initial and Final wastewater Pumping
		Energy €/y	Energy €/y	Energy €/y	Energy €/y	Energy €/y
Conventional Primary Treatment	1	43,747	35,653	21,199	32,047	89,308
	2	43,747	35,653	21,199	33,894	80,164
	3	43,747	35,653	21,199	33,498	123,076
CEPT	1	43,747	35,653	21,199	33,088	89,308
	2	43,747	35,653	21,199	34,902	80,164
	3	43,747	35,653	21,199	35,286	123,076
CEPT + Fine Bubble Aeration	1	43,747	35,653	21,199	35,951	89,308
	2	43,747	35,653	21,199	37,049	80,164
	3	43,747	35,653	21,199	37,353	123,076

Table 5.8-2: WWTP energy and reagent costs of treatment processes.

Considerations	Scenarios	Primary Treatment	Aeration	Thickening		Dewatering	
		Reagents €/y	Energy €/y	Energy €/y	Reagents €/y	Energy €/y	Reagents €/y
Conventional Primary Treatment	1	0	242,053	2,409	19,664	13,538	30,913
	2	0	326,041	2,890	26,039	18,051	41,217
	3	0	238,263	2,770	22,743	17,137	39,129
CEPT	1	51,939	142,818	1,806	9,899	13,803	31,517
	2	45,891	185,383	1,927	13,084	18,232	41,631
	3	79,049	141,220	2,047	11,158	18,509	42,262
CEPT + Fine Bubble Aeration	1	51,939	118,131	1,806	9,899	13,803	31,517
	2	45,891	153,338	1,927	13,084	18,232	41,631
	3	79,049	116,809	2,047	11,158	18,509	42,262

Table 5.8-3: WWTP total costs and energy balance.

Considerations	Scenarios	Total Energy Costs €/y	Digestion Produced Energy €/y	Total Costs €/y	Total Savings (comparison with Conventional Primary Treatment) %	Energy Neutrality %	Increment to Energy Neutrality (comparison with Conventional Primary Treatment) %
Conventional Primary Treatment	1	479,958	164,554	365,981	-	34%	0%
	2	561,643	219,404	409,496	-	39%	0%
	3	515,346	208,289	368,929	-	40%	0%
CEPT	1	381,426	161,318	313,464	14%	42%	8%
	2	421,210	215,990	305,828	25%	51%	12%
	3	420,740	215,035	338,175	8%	51%	11%
CEPT + Fine Bubble Aeration	1	356,738	161,318	288,776	21%	45%	11%
	2	389,164	215,990	273,782	33%	56%	16%
	3	396,328	215,035	313,763	15%	54%	14%

In figure 5.8-1, it is displayed the energy demand percentage of each process in the total energy consumption of Espinho WWTP, with conventional primary treatment, according to the design verification.

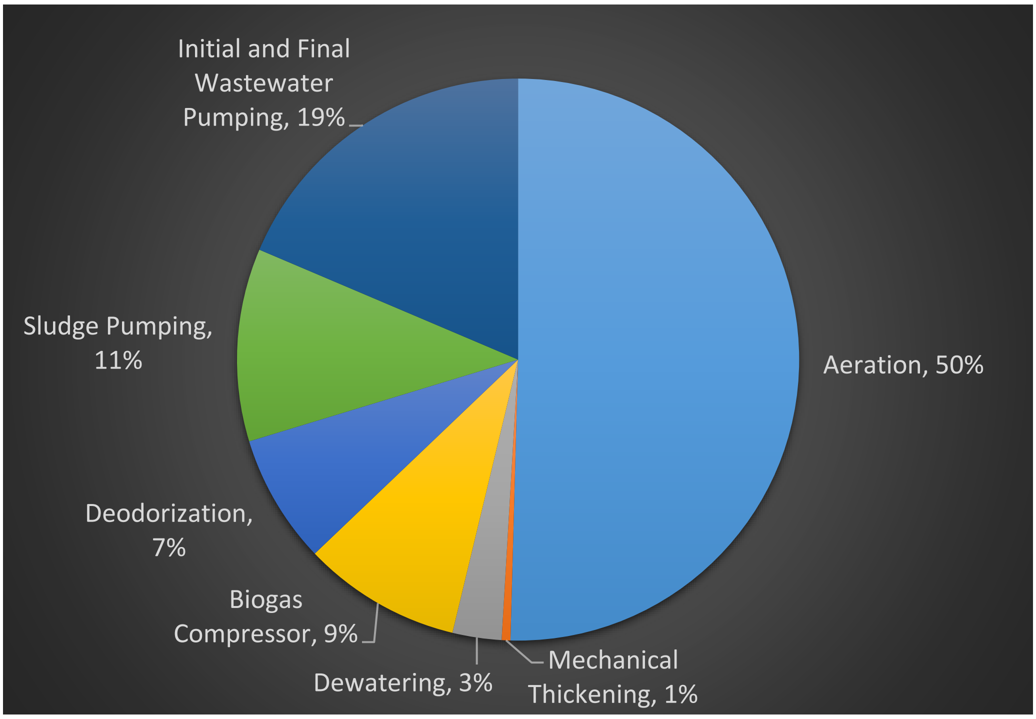


Figure 5.8-1: Energy expenditure distribution in Espinho WWTP with conventional primary treatment.

In figure 5.8-2, it is displayed the energy demand percentage of each process in the total energy consumption of Espinho WWTP, with CEPT, according to the design verification.

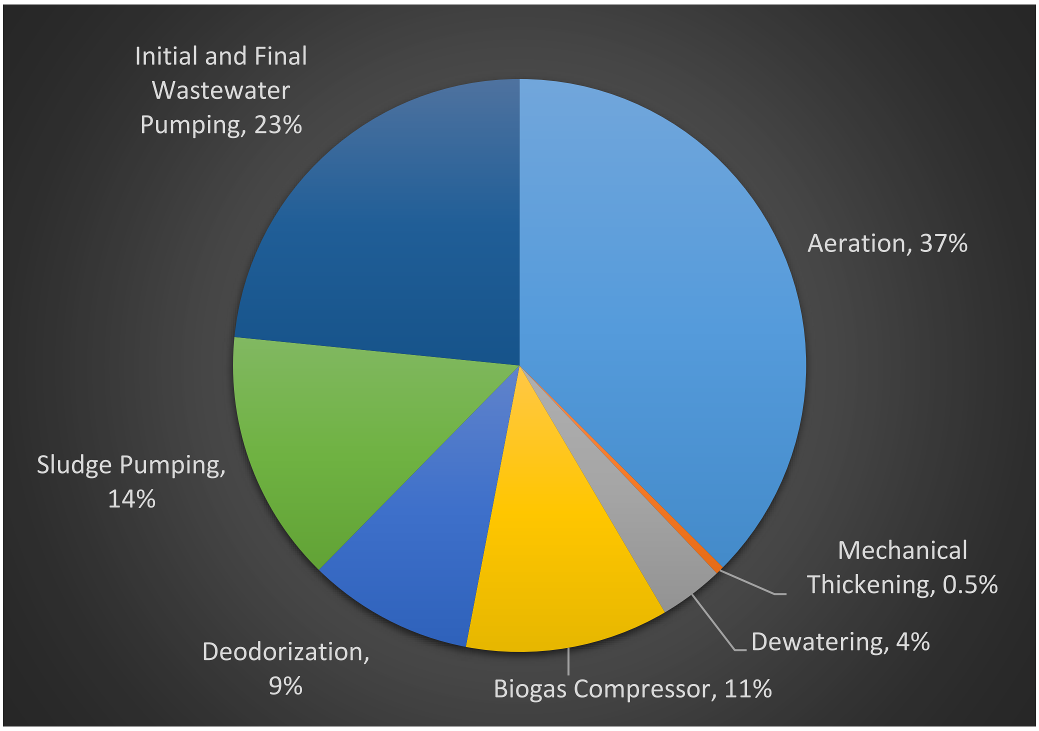


Figure 5.8-2: Energy expenditure distribution in Espinho WWTP with CEPT.

The implementation of CEPT in Espinho WWTP leads to energy savings and total energy and operation savings, in comparison with the treatment line with conventional primary treatment, of:

- Scenario 1: 895,742 kWh/y (52,518 €/y) or 2,454 kWh/d (144 €/d);
- Scenario 2: 1,276,665 kWh/y (103,668 €/y) or 3,498 kWh/d (284 €/d);
- Scenario 3: 860,058 kWh/y (30,755 €/y) or 2,356 kWh/d (84 €/d).

The implementation of CEPT and fine bubble aeration in Espinho WWTP leads to energy savings and total energy and operation savings, in comparison with the treatment line with conventional primary treatment and turbine aeration, of:

- Scenario 1: 1,120,176 kWh/y (77,205 €/y) or 3,069 kWh/d (212 €/d);
- Scenario 2: 1,567,987 kWh/y (135,714 €/y) or 4,296 kWh/d (372 €/d);
- Scenario 3: 1,081,980 kWh/y (55,166 €/y) or 2,964 kWh/d (151 €/d).

Espinho WWTP with conventional primary treatment, in scenario 1, has an energy balance of 34%, which means that it is missing 66%, in terms of energy, to reach energy neutrality.

Espinho WWTP with CEPT, in scenario 1, has an energy balance of 42%, which means that the WWTP cannot produce 58% of its total energy needs.

Espinho WWTP with CEPT and by performing fine bubble aeration, in scenario 1, has an energy balance of 45%, which means that it is missing 55%, in terms of energy, to reach energy neutrality.

In table 5.8-4, it is displayed the energy produced by implementing AcoD in the Espinho WWTP with CEPT.

Table 5.8-4: AcoD CHP produced energy.

Considerations	Scenarios	Anaerobic Co-Digestion	
		Produced Energy kWh/y	Produced Energy €/y
Performing AcoD in the WWTP with CEPT	1	3,726,826	409,951
	2	3,658,770	402,465
	3	3,587,299	394,603

The produced energy with ACOD in comparison with the AD, performed in Espinho WWTP with CEPT, increases by:

- Scenario 1: 154%;
- Scenario 2: 86%;
- Scenario 3: 84%.

The main WWTP cost indicators/benchmarks, such as the total energy costs of Espinho WWTP per m^3 of treated wastewater and the total energy costs of Espinho WWTP per kg of affluent COD are displayed in the 5.8-5.

Table 5.8-5: Espinho WWTP cost indicators.

Considerations	Scenarios	Total Energy Costs per m^3 of treated wastewater	Total Energy Costs per kg of affluent COD
		kWh/m^3	kWh/kg COD
Conventional Primary Treatment	1	0.48	0.93
	2	0.63	0.74
	3	0.34	0.91
CEPT	1	0.38	0.74
	2	0.48	0.56
	3	0.28	0.74

Considerations	Scenarios	Total Energy Costs per m^3 of treated wastewater	Total Energy Costs per kg of affluent COD
		kWh/m^3	kWh/kg COD
CEPT + Fine Bubble Aeration	1	0.36	0.69
	2	0.44	0.52
	3	0.26	0.69

The results demonstrate that Espinho WWTP is more energy efficient when performing CEPT and fine bubble aeration.

In table 5.8-6, it is displayed the energy balance following the implementation of the PV solar panel modules.

Table 5.8-6: WWTP energy balance with the implementation of the photovoltaic solar panels.

Considerations	Scenarios	Photovoltaic solar panels	Energy Neutrality	Produced Photovoltaic Energy (comparison with Total Energy Costs)
		Produced Energy €/y	%	%
Conventional Primary Treatment	1	47,845	44%	10.0%
	2	47,845	48%	8.5%
	3	47,845	50%	9.3%
CEPT	1	47,845	55%	12.5%
	2	47,845	63%	11.4%
	3	47,845	62%	11.4%
CEPT + Fine Bubble Aeration	1	47,845	59%	13.4%
	2	47,845	68%	12.3%
	3	47,845	66%	12.1%

All the methods considered to improve the energy balance, with the respective benefit, in terms of the energy produced in comparison with the total energy needs of the WWTP, are displayed in figure 5.8-3, for scenario 1.

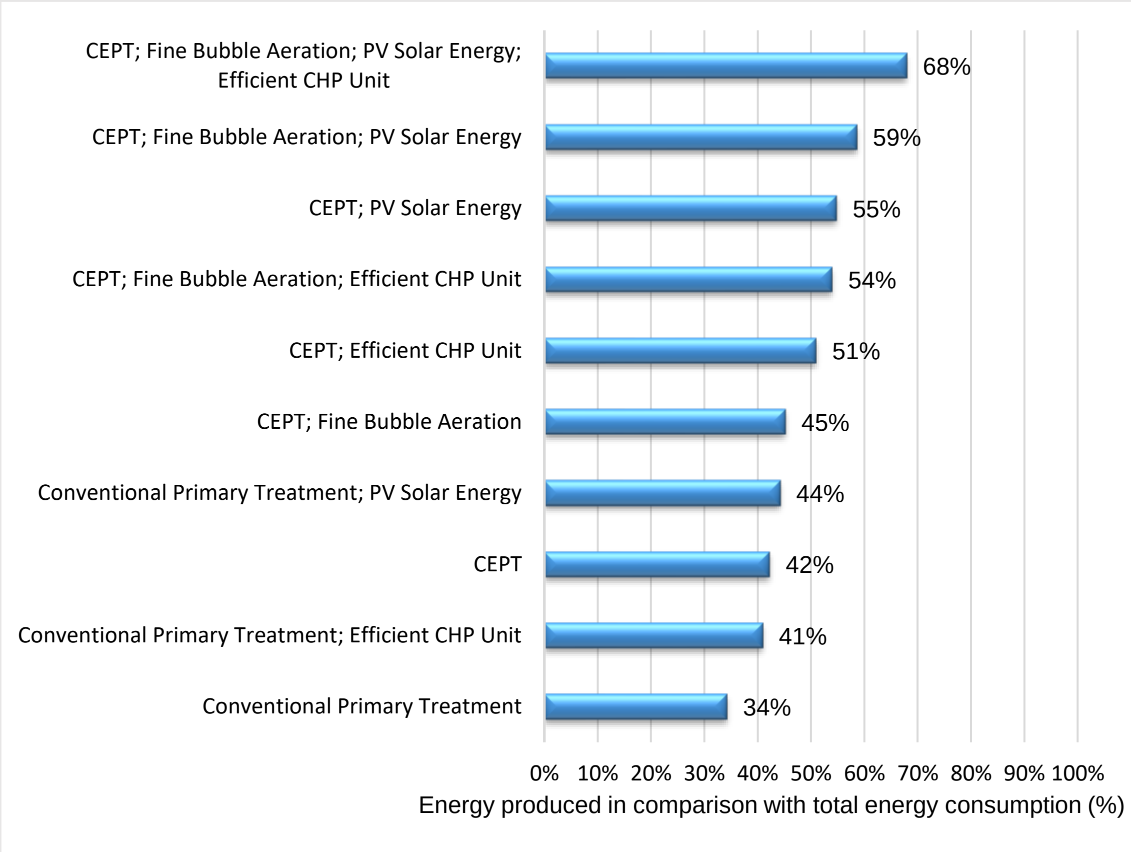


Figure 5.8-3: Methods to improve the energy balance, amount of energy produced in comparison with total energy needs.

Espinho WWTP with CEPT, considering the aeration is done via fine bubble aeration and by implementing the photovoltaic solar panel modules and a more efficient CHP unit (42% efficiency in producing electrical energy), in scenario 1, reaches an energy balance of 68%, which means that it is 32% deficient in producing its total energy needs to reach energy neutrality. This is the approach that renders the best results in setting the WWTP in the direction of energy neutrality.

5.9 Cost-benefit Analysis of Improvement

5.9.1 CEPT

The cost-benefit of the CEPT implementation and operation was evaluated and is displayed in table 5.9-1, for scenario 1. This analysis was done with the current Espinho WWTP treatment line, meaning the aeration is performed via surface turbines.

CEPT proves to be beneficial right from the start of its implementation. The payback period is inferior to 1 year, as it occurs as soon as it is performed.

Table 5.9-1: CEPT cost-benefit analysis – scenario 1.

Operation Year	CAPEX	OPEX			Total Costs	Savings				Updated Net Balance
		CEPT	Additional Dewatering Expenses			Aeration	Mechanical Thickening		Total	
	Civil Construction and other	Reagents	Reagents	Energy		Energy	Reagent	Energy		
Year	(€/y)	(€/y)	(€/y)	(€/y)	(€/y)	(€/y)	(€/y)	(€/y)	(€/y)	(€/y)
0	0	51,939	604	264	52,807	99,236	9,765	602	109,603	55,956
1	-	51,939	604	264	52,807	99,236	9,765	602	109,603	110,258
2	-	51,939	604	264	52,807	99,236	9,765	602	109,603	162,943
3	-	51,939	604	264	52,807	99,236	9,765	602	109,603	214,047
4	-	51,939	604	264	52,807	99,236	9,765	602	109,603	263,604
5	-	51,939	604	264	52,807	99,236	9,765	602	109,603	311,650
6	-	51,939	604	264	52,807	99,236	9,765	602	109,603	358,219
7	-	51,939	604	264	52,807	99,236	9,765	602	109,603	403,343
8	-	51,939	604	264	52,807	99,236	9,765	602	109,603	447,055
9	-	51,939	604	264	52,807	99,236	9,765	602	109,603	489,387
10	-	51,939	604	264	52,807	99,236	9,765	602	109,603	530,370
11	-	51,939	604	264	52,807	99,236	9,765	602	109,603	570,035
12	-	51,939	604	264	52,807	99,236	9,765	602	109,603	608,412
13	-	51,939	604	264	52,807	99,236	9,765	602	109,603	645,530
14	-	51,939	604	264	52,807	99,236	9,765	602	109,603	681,418
15	-	51,939	604	264	52,807	99,236	9,765	602	109,603	716,104
Total	**0**									**716,104**

5.9.2 Fine Bubble Aeration

Diffused air aeration was evaluated for Espinho WWTP with CEPT already being performed. WWTP equipment requires replacement at least every 15 years and since the rehabilitation of the WWTP it has been 10 years. In this analysis it was considered the replacement cost of the turbines to more accurately evaluate if the investment in fine bubble aeration is beneficial even with only 3.91 meters of depth in the biological reactor.

In table 5.9-2, it is displayed the cost-benefit analysis, as well as, the payback period of the installation/operation of fine bubble aeration, in scenario 1, in Espinho WWTP with CEPT.

Scenario 1 represents the average affluent wastewater conditions and the payback period is 5 years.

Considering Espinho WWTP with CEPT, performing fine bubble aeration provides energy savings in the aeration process of:

- Scenario 1: 224,434 kWh/y or 24,688 €/y;
- Scenario 2: 291,322 kWh/y or 32,045 €/y;
- Scenario 3: 221,922 kWh/y or 24,411 €/y.

The optimization of the aeration process by investing in fine bubble aeration, in the treatment line with CEPT, provides a reduction of 6.5% of the WWTP total energy consumption.

The implementation of fine bubble aeration to Espinho WWTP with conventional primary treatment would provide the following energy savings in the aeration process:

- Scenario 1: 340,453 kWh/y or 37,450 €/y;
- Scenario 2: 458,583 kWh/y or 50,444 €/y;
- Scenario 3: 335,121 kWh/y or 36,863 €/y.

Table 5.9-2: Fine bubble aeration cost-benefit analysis – scenario 1.

Operation Year	CAPEX				OPEX	Total Costs	Savings	Updated Net Balance
	Civil Construction and other	Diffusers network	Blower units	Turbine replacement	Energy		Energy	
Year	(€)	(€)	(€)	(€)	(€/y)	(€/y)	(€/y)	(€/y)
0	75,000	100,000	75,000	-150,000	118,131	218,131	24,688	-74,199
1	-	-	-	-	118,131	118,131	24,688	-49,139
2	-	-	-	-	118,131	118,131	24,688	-24,804
3	-	-	-	-	118,131	118,131	24,688	-1,177
4	-	-	-	-	118,131	118,131	24,688	21,757
5	-	-	-	-	118,131	118,131	24,688	44,013
6	-	-	-	-	118,131	118,131	24,688	65,607
7	-	-	-	-	118,131	118,131	24,688	86,553
8	-	-	-	-	118,131	118,131	24,688	106,866
9	-	-	-	-	118,131	118,131	24,688	126,559
10	-	-	-	-	118,131	118,131	24,688	145,647
11	-	-	-	-	118,131	118,131	24,688	164,143
12	-	-	-	-	118,131	118,131	24,688	182,061
13	-	-	-	-	118,131	118,131	24,688	199,413
14	-	-	-	-	118,131	118,131	24,688	216,212
15	-	-	-	-	118,131	118,131	24,688	232,472
Total	**75,000**	**100,000**	**75,000**	**-150,000**				**232,472**

5.9.3 Photovoltaic Solar Panels

Photovoltaic solar panels were considered in this work, to maximize the energy production in the WWTP. In table 5.9-3, it is displayed the cost-benefit analysis, which revealed a payback period of 8 years.

Table 5.9-3: Photovoltaic Solar Panels cost-benefit analysis.

Operation Year	CAPEX	OPEX	Total Costs	Income	Updated Net Balance
	Equipment and installation	Maintenance		Produced Energy	
Year	(€)	(€/y)	(€/y)	(€/y)	(€/y)
0	315,000	4,725	319,725	47,845	-267,862
1	-	4,725	4,725	47,845	-222,048
2	-	4,725	4,725	47,845	-177,529
3	-	4,725	4,725	47,845	-134,278
4	-	4,725	4,725	47,845	-92,267
5	-	4,725	4,725	47,845	-51,468
6	-	4,725	4,725	47,845	-11,854
7	-	4,725	4,725	47,845	26,599
8	-	4,725	4,725	47,845	63,919
9	-	4,725	4,725	47,845	100,130
10	-	4,725	4,725	47,845	135,257
11	-	4,725	4,725	47,845	169,323
12	-	4,725	4,725	47,845	202,353
13	-	4,725	4,725	47,845	234,370
14	-	4,725	4,725	47,845	265,397
15	-	4,725	4,725	47,845	295,455
Total	**315,000**				**295,455**

5.9.4 Ultrafiltration

The ultrafiltration cost-benefit analysis, as well as the payback period is displayed in table 5.9-4.

Considering the reclaimed water is sold at 0.5 €/m^3, the payback period is verified in the 7^{th} year of operation.

The reclaimed water cost of production to the WWTP is 0.15 €/m^3.

Table 5.9-4: Ultrafiltration cost-benefit analysis.

Operation Year	CAPEX				OPEX		Total Costs	Income	Updated Net Balance
	Civil Construction	UF Equipment	Electric Installations and other	Pipeline and other inherent civil construction costs	Membrane Replacement	Energy		Reclaimed Water	
Year	(€)	(€)	(€)	(€)	(€/y)	(€/y)	(€/y)	(€/y)	(€/y)
0	50,000	300,000	50,000	83,500	10,000	20,075	513,575	100,000	-407,463
1	-	-	-	-	10,000	20,075	30,075	100,000	-333,568
2	-	-	-	-	10,000	20,075	30,075	100,000	-261,758
3	-	-	-	-	10,000	20,075	30,075	100,000	-192,017
4	-	-	-	-	10,000	20,075	30,075	100,000	-124,271
5	-	-	-	-	10,000	20,075	30,075	100,000	-58,485
6	-	-	-	-	10,000	20,075	30,075	100,000	5,384
7	-	-	-	-	10,000	20,075	30,075	100,000	67,377
8	-	-	-	-	10,000	20,075	30,075	100,000	127,537
9	-	-	-	-	10,000	20,075	30,075	100,000	185,905
10	-	-	-	-	10,000	20,075	30,075	100,000	242,519
11	-	-	-	-	10,000	20,075	30,075	100,000	297,419
12	-	-	-	-	10,000	20,075	30,075	100,000	350,644
13	-	-	-	-	10,000	20,075	30,075	100,000	402,231
14	-	-	-	-	10,000	20,075	30,075	100,000	452,216
15	-	-	-	-	10,000	20,075	30,075	100,000	500,636
Total	**50,000**	**300,000**	**50,000**	**83,500**					**500,636**

6 Discussion

6.1 Coagulant Optimum Dosage in CEPT

In 2008, Espinho WWTP was rehabilitated to operate with CEPT. According to the existing data, the aluminum coagulant dosage that is being performed is 7 mg/L, which seems low. Based on the jar tests conducted in this study, the CEPT process could be optimized. Generally, the WWTP affluent wastewater contains TP in short supply. Considering this, the optimum aluminum coagulant dosage achieved was 15 mg/L with a dosage of 0.5 mg/L of anionic polymer as flocculant. This dosage provides the maximum removal of TSS, COD and BOD_5, while not causing the excessive removal of TP that occurs in the higher dosages of coagulant.

The results obtained with the selected coagulant dosage of 15 mg/L, to be applied, are in accordance with the literature review.

6.2 WWTP Optimization

6.2.1 Primary Treatment

The primary clarifiers of Espinho WWTP, when conventional primary treatment is considered, are not fit for the design flow-rate in terms of surface overflow rate. The surface overflow rate obtained was 2.5 $m^3/(m^2.h)$, which is superior to the upper limit of 2.1 $m^3/(m^2.h)$ warranted by Tchobanoglous et al., (2014). When CEPT is performed the surface overflow rate accepted range increases to 2.8 to 3.4 $m^3/(m^2.h)$, allowing for a stable and sound clarifying operation.

The WWTP was designed, in its previous beneficial rehabilitation in the year of 2008, for a design flow rate of 2435 m^3/h. In this regard, the design flow rate, with conventional primary treatment, obtains a surface overflow rate value of 2.1 $m^3/(m^2.h)$, which is suitable for proper clarifying operation. However, the design flow rate considered in this study, obtained via data of one year of *in-situ* measurements, is 2868 m^3/h which is superior, meaning the suspended solids in the sedimentation tank will not settle satisfactorily with conventional primary treatment when the design flow rate occurs.

The design verification results reveal a detention time of 1.1 h in the primary clarifiers, when considering the design flow rate. This detention time value is inferior to the warranted range for adequate performance provided by the literature review, as a result, when Espinho WWTP operates at the design flow rate, the removal of TSS in the primary treatment should be inferior and far from ideal.

In scenario 3, Espinho WWTP while performing conventional primary treatment, lacks phosphorus for the secondary treatment to occur. In fact, the conventional primary treatment allows a big parcel of the affluent carbonated matter to reach the biological reactor and the affluent phosphorus, one of the main inorganic nutrients required by the microorganisms, is not enough to allow them to grow and carry out the oxidation-reduction reactions. The storage of phosphorus to introduce in the treatment process is required if Espinho WWTP is being operated with conventional primary treatment.

When considering Espinho WWTP with CEPT, the lack of phosphorus in scenario 3 does not occur even with a higher removal of phosphorus, from 10% to 39%, because the removal of carbonated matter affluent to the biological treatment is much higher, in comparison, and the phosphorus needs are as a result inferior.

CEPT materializes the capture of 60% of the carbonated matter, from the previous 25% with conventional primary treatment, resulting in a lower affluent organic matter load to the biological treatment. Therefore, less organic matter requires biological treatment, hence fewer nutrients are required by the microorganisms to develop and accomplish the biodegradation of the wastewater substrate in the biological reactor.

The chemical reagents are the sole investment, when considering the implementation of CEPT in Espinho WWTP. In this regard, CEPT implementation proves to be beneficial from the start of its execution, as reported, in table 5.9-1, as the profit is higher than the costs.

The coagulation and flocculation chambers are already constructed and inserted in the treatment line, so no civil construction work is required, resulting in an inexistent payback period, as the returns are immediate.

The dewatering costs increase due to the formation of chemical sludge, nevertheless this cost is outweighed by the energy savings that occur in the aeration of the biological reactor and in the mechanical thickening. In the primary treatment, more TSS, COD and BOD_5 is removed, therefore less organic matter proceeds to the biological treatment phase and less aeration is required.

6.2.2 Biological Treatment and Aeration

Scenario 2 with conventional primary treatment is the most demanding scenario, in terms of aeration required, because of the considerable affluent organic load that reaches the aeration tank, resulting in an energy consumption of 8,121 kWh/d in the aeration process.

Espinho WWTP when performing conventional primary treatment, in scenario 2, requires three treatment lines in the biological reactor as solely with two treatment lines, the biological reactor would have to be operated with a MLSS of 4 kg/m^3, the literature upper limit for a complete-mix CAS system, and the 6 installed turbines would not provide enough aeration.

Scenario 2 with conventional primary treatment is the most energy intensive scenario, as the aeration represents half of the total energy consumption. This percentage is, in accordance with, the ordinary CAS systems provided by the literature.

In the WWTP with CEPT, the affluent organic load to the biological reactor is significantly lower, for that reason the biological reactor can operate in all scenarios with only two treatment lines as the aeration necessities are inferior. Considering this, the biological reactor is projected to operate with two treatment lines to maintain stability during its operation throughout the year, this is done by manipulating the MLSS.

During the WWTP operation, the control of the MLSS in the biological reactor is required in order to keep the sludge age in the ideal range to promote adequate treatment. MLSS can be measured via an MLSS analyzer or by retrieving samples from the biological reactor and then conducting laboratory tests to determine the samples concentration. Ultimately, knowing the affluent wastewater characteristics, the WAS recirculation percentage and the WAS removal, the MLSS can be calculated.

In theory by increasing the MLSS in the biological reactor it would be possible to utilize solely one treatment line and maintain an adequate sludge age, but the 3 available turbines installed would not provide enough aeration.

The MLSS was manipulated to allow sufficient sludge age in the range of 3 to 4 days, to promote the growth of the microorganisms and stable biological treatment, while inhibiting nitrification and consequent denitrification in the secondary settling tank, that occurs if sludge age is too high. Denitrification causes the sludge to float, preventing it from settling and possibly allowing it to exit the system by the secondary clarifier weirs along with the treated effluent.

Considering the excess WAS is removed from the process from the aeration tank, the MLSS control is done by managing the removal of excess WAS and controlling the recirculation of WAS from the secondary clarifier to the aeration tank. Increasing the removal of excess WAS and/or decreasing the recirculation of WAS lowers the MLSS. On the contrary decreasing the removal of excess WAS and/or increasing the recirculation of WAS boosts the MLSS.

The present affluent conditions may not be completely identical to the ones utilized in this study, therefore the operation parameters such as the MLSS and sludge age may need to be properly adjusted during the operation of the WWTP.

Assuming the primary treatment phase of the WWTP is performed without chemical addition, the aeration results obtained demonstrate that 50% of the total WWTP energy costs derive from the aeration of the biological reactor. This value is in accordance with the literature for a CAS system. In scenario 2 with conventional primary treatment, the most energy demanding scenario, the aeration represents 58% of the total energy costs.

Espinho WWTP when performing CEPT, the particulate matter capture in the primary treatment increases, resulting in the reduction of downstream loading to the secondary treatment, which leads to energy savings in the biological reactor as less aeration is required. The implementation of CEPT is important in reducing the oxygen necessities and the cost of aeration. According to the design verification results the energy necessities in the aeration process, in Espinho WWTP, with CEPT are approximately half of the energy necessities in the aeration process with conventional primary treatment. The results demonstrate an energy reduction from 6,029 kWh/d to 3,557 kWh/d, when considering the aeration is done via surface turbines.

The aeration of the biological reactor, while performing CEPT, represents 37% of the total WWTP energy costs. This value is much lower due to the aeration savings obtained with CEPT.

Espinho WWTP is a particular plant because it requires the pumping of the entire raw affluent wastewater and the pumping of the entire treated secondary effluent for its discharge. The initial and final pumping stations have a big impact in the energy consumption of the WWTP, representing approximately 20% of the total energy costs. By disregarding the initial or the final pumping of the wastewater, the aeration would represent a higher percentage of the total energy costs and probably would be situated on the higher spectrum of a CAS system according to the literature review.

The most energy demanding scenario with CEPT, in terms of aeration, is scenario 2 with an energy consumption of 4,617 kWh/d. In this scenario, the aeration represents 44% of the total energy costs, a much lower percentage from the 58% with conventional primary treatment.

Generally, the temperature utilized in WWTP aeration design projects in this region ranges from 24°C to 28°C. For conservative reasons the design verification temperature considered for the endogenous respiration was 28°C, as higher temperatures lead to higher aeration needs due to promoting the metabolic activity of the microorganisms, as well as, reducing the solubility of gases, such as oxygen, in water, leading to a decreased transfer rate. The aeration necessities simulated in this study are conservatively overestimated and represent the maximum aeration demands. During the WWTP operation the aeration necessities could possibly be inferior.

According to the Espinho WWTP Operation Reports of 2016, the energy consumed in the turbine aeration process was 898,678 kWh/y, which averages at 2,462 kWh/d. The available data shows that turbine aeration operation time was 9 h/d, considering the utilization of only two lines of the biological reactor. This seems low as the design verification results exhibit a minimum necessary time of aeration of 13 h/d in scenario 1, when CEPT is performed, to maintain the 2 mg O_2/L in the biological reactor. The lack of oxygen in the biological reactor, may lead to insufficient treatment and ultimately to not fulfill the discharge permit limits. In the other CEPT scenarios (2 and 3) and when conventional primary treatment is performed, the aeration becomes more demanding, therefore the time of operation required by the aeration equipment increases.

Fine bubble aeration, when performed in Espinho WWTP with CEPT, leads to 6% reduction in total energy consumption in comparison with Espinho WWTP with CEPT and turbines.

The optimization of the aeration process with the investment in fine bubble aeration and the replacement of the turbines, improves the energy balance by 3% in scenario 1 and by 5% in scenario 2, the most demanding scenario.

Considering the implementation of fine bubble aeration in Espinho WWTP with conventional primary treatment, the energy savings would be higher, because fine bubble aeration is a more energy efficient process and the aeration needs are more demanding since more organic matter reaches the biological reactor.

The maximum upper limit of oxygen transfer capability of turbine aeration, according to the literature review is 2.1 kg O_2/kWh. Considering the design verification results obtained, the fine bubble aeration could reach an oxygen transfer capability of 3.2 kg O_2/kWh in scenario 2 and a minimum value of 2.4 kg O_2/kWh in scenario 3, which is still more efficient than turbine aeration even with just a water surface height of 3.91 meters in the aeration tank. When comparing fine bubble aeration aeration and turbine aeration, the first is increasingly more beneficial, the more affluent carbonated matter reaches the aeration tank and the higher the available depth of the aeration tank.

Fine bubble aeration proves to be more valuable if the investment and implementation is done right from the start of the WWTP construction or during the 5 years period prior to the replacement of the aeration equipment. The equipment replacement should take place every 15 years.

With this in consideration, fine bubble aeration leads to a payback period of 5 years. In Espinho WWTP, the investment in fine bubble aeration over turbine aeration is beneficial as the aeration equipment should probably need replacement in the future 5 years.

If the aeration equipment solely demanded replacement in a future longer than 5 years, the payback period would increase. The maximum payback period is 11 years, when fine bubble aeration is implemented to replace newly installed turbines or turbines with 10-years, or more, of life expectancy.

In a company's business perspective, a complete WWTP beneficial rehabilitation would prove beneficial in the replacement of surface turbine aeration by fine bubble aeration as this rehabilitation is designed for a period of 20 years, in which after 5 to 11 years the fine bubble aeration would prove advantageous. Considering a partial WWTP beneficial rehabilitation for equipment replacement, the replacement of the turbine aeration equipment by fine bubble aeration is only beneficial if the company is responsible for the operation of the WWTP for a period longer than 5 years.

In general, the secondary treatment equipment installed in Espinho WWTP is well designed. However, the installed WAS recirculation pumps, including reserve equipment, were designed for a maximum flow rate of 2192 m^3/h. According to the ATV guidelines, this is not sufficient as the WAS recirculation pumps need to be able to lift one time the WWTP design flow rate of 2868 m^3/h. This issue is simple to overcome with a rather inexpensive investment in the acquisition of two additional WAS recirculation pumps.

Considering Espinho WWTP with conventional primary treatment, the excess WAS pumps installed need replacement/investment as they are not adequate, as shown in Annex G, as they do not pump the required flow rate. Each individual excess WAS pump installed is able to lift 33 m^3/h, totaling 66 m^3/h as there are 2 excess WAS pumps in operation. Provided that the installed pumps work 24 h/d, they would still not be able to remove the necessary sludge, 1646 m^3/d. A pumping equipment capable of lifting 69 m^3/h or more is required. This does not happen in the treatment line with CEPT, as a more substantial fraction of the suspended solids is eliminated in the primary treatment and consequently does not reach the biological treatment.

In the secondary sedimentation the overflow rate, for all scenarios, is below the optimal range, this does not pose a problem as with low overflow rate the sedimentation tank operates adequately. When the design flow rate is considered, the overflow rate takes the value of 1.1 $m^3/(m^2.h)$, which is in the optimal range. The detention time for the design flow rate is 2.5 h, which is slightly above the ideal design range obtained via literature review, nevertheless this is not a problem and it will not impact the treatment process. However, a long detention time may cause the sludge to float, which is problematic. During the operation of the WWTP, close attention to the possible occurrence of these events is necessary. If this occurs, the recirculation of sludge, from the secondary clarifier to the biological reactor, should be increased or one of the secondary clarifiers should be put out of service, during the lower affluent flow rate season. The management and the selection of correct procedure needs to be evaluated during the WWTP operation as it depends on the affluent flow rate to the plant.

6.2.3 Thickening

The hydraulic loading of the gravity thickeners is in the optimal range of 10 to 50 kg/(m^2.d), when the upstream primary treatment considered is CEPT. However, if conventional primary treatment is considered, the ideal hydraulic loading value ranges from 100 to 150 kg/(m^2.d) and the gravity thickeners operate with low hydraulic loadings with the minimum value being 18.4 kg/(m^2.d) in scenario 1. The gravity thickener overflow rate ranges from: 2 to 3 m^3/(m^2.d) in the scenarios with CEPT; 0.7 to 1 m^3/(m^2.d) in the scenarios with conventional primary treatment. These values are below the ideal design range provided by the literature review, nevertheless they do not constitute a problem to the treatment process. Espinho WWTP with conventional primary treatment reveals inferior hydraulic loadings that may lead to septic conditions, odors and to ultimately cause the sludge to float. The gravity thickeners are enclosed and possess a deodorization unit, therefore the odors produced are not considered an issue.

The rotary drum filters, that provide the mechanical thickening of the secondary sludge, in Espinho WWTP, are sufficient to provide thickening for the treatment line with conventional primary treatment and CEPT. The more demanding scenario is scenario 2, with conventional primary treatment, in which the equipment is required to operate 24 h/d. Espinho WWTP with CEPT, the maximum thickening operation time is observed in scenario 3, with 17 h/d.

In Espinho WWTP with conventional primary treatment, the mechanical WAS thickening process is overloaded due to the higher carbonated matter that reaches the biological reactor, consequently in scenario 2, the mechanical thickening equipment is operating at its maximum capacity.

6.2.4 Anaerobic Digestion

According to the literature the primary sludge has a higher energy value, that was not accounted for in the design verification results. The existent literature does not express the benefit in numbers of the additional energy production, of capturing and conducting more primary sludge, when performing CEPT, to the AD process. Consequently, in this study it was considered the same energy value for the primary and secondary sludge, therefore the energy production through biogas combustion, when CEPT is performed in Espinho WWTP, is possibly underestimated.

The affluent TSS load to the AD process is higher with CEPT, so presumably the VS load would be greater, but this does not occur. CEPT sludge contains mainly primary sludge, but also chemically produced sludge, which is not biodegradable.

The TSS provided by the chemical sludge does not contribute to the VS load, and as a result it does not contribute to the production of biogas.

It should be emphasized that the main reason for slightly lower energy production with CEPT is because the higher energy potential of the primary sludge was not considered, otherwise the energy production with CEPT could possibly be substantially higher. The energy production in Espinho WWTP with CEPT, could potentially be much higher and, as a result, the treatment plant could be even closer to energy neutrality, more than demonstrated in this study.

In scenario 2, with conventional primary treatment and CEPT, the energy production in the AD process was the highest because of the heavy affluent load to the WWTP, in terms of TSS, COD and BOD_5.

In this study, the VS destruction percentage values from the WEF formula were not utilized hence the energy produced, via CHP equipment associated with the anaerobic digestion, is very conservative and could be considerably higher. Considering the utilization of the VS destruction values from the WEF formula in the AD process, the energy production could be enhanced by: 22% in scenario 1; 15% in scenario 2; 15% with CEPT or 16% with conventional primary treatment in scenario 3. In the treatment line with CEPT and AcoD being performed, the energy production, with the VS destruction value provided by the WEF formula, would merely improve by 2% in all scenarios.

The data of the installed CHP unit was not available, therefore the electrical energy production efficiency considered was 35%, which is the same value that was considered in the base project of 2008. This efficiency value utilized, in comparison with current equivalent equipment, is particularly low. The average electrical energy production efficiency from CHP units with the same characteristics is 42%, meaning the electrical energy retrieved from Espinho WWTP AD process could be superior. Considering a CHP unit with an electrical energy production efficiency of 42%, the energy production would increase by 20% in all scenarios.

In scenario 1, when considering CEPT and two other methods studied for improving the energy balance, such as fine bubble aeration and photovoltaic solar panels, a 20% increase in energy production would lead to an improvement in the energy balance of 9%, from 59% to 68%. Espinho WWTP would solely be 32% deficient in producing its total energy needs to reach energy neutrality.

The increase in energy production via a more efficient CHP unit is definitely a solid procedure to enhance and optimize Espinho WWTP energy balance because it is not reliant on an excellent AD process, in stability, mixing nor HRT. On the contrary, the VS destruction percentage is dependent on the occurrence of ideal conditions in the AD process, that may not occur.

A more efficient CHP unit appears to bring more benefits in the long run than a higher VS destruction because it leads to a permanent energy increase even though, in scenario 1 the maximum VS destruction achieved an increase of 22% in energy production and a more efficient CHP unit could only achieve 20% more energy production. The ideal approach is to implement a more efficient CHP unit, while simultaneously striving to maximize the VS destruction in the AD process.

To maximize the energy production in the AD process, a pre-treatment of the sludge prior to the digesters could be studied for its application in Espinho WWTP. This was not evaluated in this study and would require a detailed analysis of its viability, but according to the literature a thermal sludge pre-treatment could lead to surplus biogas production and improved sludge dewaterability. As reported by the literature, the implementation of thermophilic pre-treatment prior to AD, has shown increases on the methane production and solids destruction by 25 %. This method would improve the energy balance significantly and reduce the sludge volume for disposal.

According to the Espinho WWTP Operation Reports of 2016, the energy obtained via CHP unit was 900,058 kWh/y, which is lower than it was expected in comparison with the design verification results obtained. The minimum values obtained were 1,495,944 kWh/y and 1,466,526 kWh/y in scenario 1, with conventional primary treatment in the treatment line and CEPT in the treatment line, respectively. The 2016 reports show that in certain months the produced energy by the CHP unit was limited, which can be due to problems in the digestion process, CHP unit malfunctions or issues in the removal and pumping of the sludge from upstream processes to the AD process. A defective CHP unit may prevent the heating of the digesters content, leading to a steep decline in biogas and energy production.

6.2.5 Co-Digestion

AcoD is usually implemented in some WWTP with unutilized digester volume. Espinho WWTP could follow the example, as the digesters can handle the implementation of co-digestion.

In scenario 1, the application of co-substrate achieved the maximum value as the digesters could receive an additional 180 m^3/d, as a result this was the scenario which obtained the highest energy production via AcoD.

It was introduced a volume of co-substrate that represented 54% of additional flow rate and 61% of additional TSS load and the digesters still maintained a satisfactory hydraulic retention time of 15 days.

It was demonstrated that Espinho WWTP anaerobic digesters are designed and capable of receiving a maximum affluent flow rate of 336 m^3/d of both thickened sludge and external residues to perform co-digestion.

The residue chosen to simulate the performance of co-digestion was a FW. It was not considered the increased methane production rates, nor the increased methane yield, provided by the enhanced C/N ratio. As a result, the estimated produced energy obtained with AcoD, could possibly be underestimated.

According to the literature, the introduction of 10% of FW in the anaerobic digesters can lead to a maximum increase in energy production of 78%. In the present study, the maximum increase in energy production via AcoD was obtained in scenario 1. In scenario 1, it was simulated a substantial application of co-substrate (FW), accounting to an introduction of 54% of FW, which resulted in an increase of 154% in biogas and energy production. In comparison with the literature review, the previewed biogas production did not increase as expected with the volume of co-substrate introduced.

Espinho WWTP with CEPT, in scenario 1, simulated the introduction of 1.25 kg VS/m^3 resulting in an energy production increase of 154%. The Rovereto WWTP, in Italy, after implementing AcoD for a year, obtained a daily increase in the energy production of 100% with the addition of 0.65 kg VS/m^3 (Mattioli et al., 2017). The AcoD results obtained in this study seem to be on the conservative side of the literature review, possibly due to the increased methane yield of FW not being considered.

All things considered, the AcoD is the process which may improve the energy balance the most, due to the increase in biogas production achieved and its combustion via the installed CHP unit. Adding to this, the implementation of co-digestion would create a surplus revenue to the WWTP, due to receiving and treating external residue. AcoD provided that it is feasible, it would greatly benefit the WWTP by substantially improving the energy production and by creating an additional revenue stream.

Considering all the energy improvements studied for implementation, including CEPT, fine bubble aeration, PV solar panels and AcoD, Espinho WWTP if adequately operated, could possibly become within reach of energy neutrality or even energy self-sufficient, like the WWTP of Zirl or the WWTP of Strass im Zillertal, two Austrian WWTP that demonstrate 110% and 160% energy self-sufficiency because of co-digestion (Insam & Markt, 2016).

Improving the CHP unit, from an equipment with 35% efficiency in producing electrical energy to 42%, would result in a 20% energy production increase, allowing the AcoD process to provide an even greater energy production, and allowing the WWTP to move even closer in the direction of energy neutrality. The investment in a more efficient CHP unit is essential, when trying to maximize the energy production of a treatment plant or when the implementation of AcoD is considered.

In Portugal, there is no legislation that allows the performance of co-digestion in municipal WWTP, nor legislation concerning the quality of the residues that can be applied to perform co-digestion. Legislation in this matter is necessary, as the residues introduce undesirable impurities, chemicals or other components, that reach the beginning of the WWTP via returns. These impurities may be unusual or even absent from domestic wastewater. If the residue contains heavy concentration of TP and TN, this is also something to take into consideration, as the WWTP may not be able to treat the heavy affluent nutrient load that is introduced. This presents a concern as the WWTP needs to be able to fully treat the co-substrates and the nutrients/impurities that return to the beginning of the WWTP satisfactorily. Additionally, the introduction of co-substrates with undesirable suspended impurities, like glass, metal or sediments can cause operational failures and additional maintenance in the digesters. In theory AcoD is an alluring method, nevertheless it has its constraints and the residue to be applied should be extensively studied as well as the WWTP that receives it.

The implementation of AcoD, depending on the volume of co-substrate that is intended to be introduced, can lead to an increase in the returns of a WWTP. The increment in returns may demand the investment in new pumping equipment or process equipment, such as blowers, mechanical thickeners or dewatering centrifuges.

In Espinho WWTP, further research is required to analyze the AcoD implementation and investigate the influence of the returns in the WWTP treatment line to obtain the correct operation costs and possible investment costs for a more accurate and global evaluation. The results of the produced energy, when performing AcoD, are reasonably accurate but its impact in the energy balance and the additional operation/investment costs need to be calculated.

6.2.6 Dewatering

The dewatering equipment installed in Espinho WWTP are centrifuges, which according to the literature have a high capital costs but lower operation/life cycle costs in comparison with a belt filter press because of the lower polyelectrolyte demands and cleaning required even though centrifuges require more energy to operate.

The centrifuges installed are suitable to provide the dewatering of the sludge following the processes of AD. The dewatering costs increase when performing CEPT, due to the formation of chemical sludge.

6.2.7 Photovoltaic Solar panels

In Portugal, the installation of photovoltaic solar panels in large scale WWTP, has currently been a requisite by the main contractors, such as Águas de Portugal (AdP group), during the design or rehabilitation phase of sizeable WWTP.

The photovoltaic solar panels are presented as a necessary condition, that the project/design companies need to comply with. An example of this is the Choupal WWTP, a 200,000 PE plant located in Coimbra, which is currently being evaluated for rehabilitation.

The implementation of photovoltaic solar panel modules is an option to consider when trying to improve the energy balance. In general, the installation of the photovoltaic solar panels has a minimum payback period of around 8 years. Considering a complete beneficial rehabilitation of Espinho WWTP, the photovoltaic solar panels prove to be profitable as the WWTP is designed to operate for 20 years.

The photovoltaic solar panels provide, an average of, 10% of energy savings in all scenarios.

6.3 Energy Balance

The trend of moving towards energy neutrality occurs with the implementation of CEPT, and more so, when also performing AcoD, fine bubble aeration and by installing PV solar panels.

The implementation of CEPT improves the energy balance, mainly due to the energy reduction on the aeration process, by 8%, 12% and 11% in scenario 1, 2 and 3 respectively. The more the affluent load to the WWTP, the more energy savings, due to aeration, CEPT provides.

As an immediate measure Espinho WWTP could optimize the primary treatment by performing CEPT. CEPT, apart from the AcoD which was not fully studied, is the only method with an immediate payback as it lacks investment. The implementation of CEPT alone provides total energy savings of 2,454 kWh/d, 3,498 kWh/d and 2,356 kWh/d in scenario 1, 2 and 3 respectively. All things considered CEPT allows for total savings, in terms of energy and costs of operation, of 144 €/d, 284 €/d and 84 €/d in scenario 1, 2 and 3 respectively.

According to the design verification results, in Espinho WWTP, the implementation of CEPT and the proposed photovoltaic solar panels, the energy balance improves, in scenario 1, from 34% to 55%, which accounts for an improvement of 21% in the energy balance in the direction of energy neutrality. Considering the aeration was done via fine bubble aeration the improvement would be greater, from 34% to 59%, meaning an improvement of 25% in the energy balance in the direction of energy neutrality. This is the most substantial improvement that is demonstrated in this work.

The methods considered lead to an increase in the energy efficiency of the WWTP. Considering all the methods studied in this work (excluding AcoD), in scenario 1, the scenario which represents the average affluent conditions of Espinho WWTP, the plant can supply 59% of its energy needs, while only lacking the production of 41% of its total energy consumption to reach energy neutrality. Considering a more efficient CHP unit (42% in producing electrical energy), the energy balance could increase to 68% and Espinho WWTP would solely be 32% deficient in producing its total energy needs to reach energy neutrality. This approach is the closest to energy neutrality, that could be confirmed in this project.

In general, the majority of the WWTP do not require the pumping of the initial and final effluent. Espinho WWTP is particular, as it performs the total pumping of the initial and final wastewater, which represents an average of 20% of the total energy consumption. This pumping operation is not a treatment procedure, but instead one that allows for the treatment and subsequent discharge to occur, and in Espinho WWTP it is inevitable due to the location of the WWTP. However, assuming Espinho WWTP lacked the need to perform the pumping of the initial and final wastewater, and considering the implementation of the studied methods for improving the energy balance, the treatment plant could be very close to energy neutrality.

Energy neutrality could possibly be reached when AcoD is performed in conjunction with the other methods studied, namely CEPT, fine bubble aeration, the installation of PV solar panel modules and a more efficient CHP unit.

AcoD when implemented with CEPT leads to an increase in biogas/energy production of 154% in scenario 1. The energy production via biogas combustion more than doubles, therefore it is probable that energy neutrality could be achieved by implementing AcoD in conjunction with all the other methods studied.

The total energy consumption of Espinho WWTP in kWh per m^3 of treated wastewater is consistent with the literature range of 0.3 to 0.6 kWh/m^3 for a CAS system (Wan et al., 2016). When CEPT is implemented the values of kWh/m^3 obtained decrease to the literature inferior limit due to the reduced energy expenditure in aeration.

The total energy consumption of Espinho WWTP in kWh per kg of affluent COD is also in conformity with the literature value of 0.85 kWh/kg COD (Guerrini et al., 2017). In scenario 1, the value obtained was 0.93 kWh/kg COD. With the implementation of CEPT, the treatment plant becomes more energy efficient and this value decreases to 0.74 kWh/kg COD and 0.69 kWh/kg COD when also performing fine bubble aeration.

This study demonstrates that the implementation of inexpensive methods can lead to substantial improvements in the optimization of Espinho WWTP energy balance and operation costs, while maintaining the same adequate level of treatment.

The total WWTP energy consumption of 2016, retrieved from the existing data records, seems to be lower than expected, accounting for 2,805,956 kWh/y. According to the design verification results the total energy consumption of the WWTP should be close to 4,363,250 kWh/y (scenario 1, conventional primary treatment) or 3,467,508 kWh/y (scenario 1, CEPT).

The processes, which contribute the most for the energy consumption verified, in Espinho WWTP, described by descending order, are the aeration, the initial and final pumping, the total sludge pumping costs, the biogas compressor and the deodorization.

The affluent wastewater characteristics to the WWTP are exactly the same. This indicates that the initial and final pumping, along with the sludge pumping, had to operate for the same duration as projected here, resulting in equivalent or identical energy consumption. The same applies to the deodorization, which should be working continuously throughout the WWTP operation.

The main factor that differentiates the energy consumption of 2016 from the design verification results, is the energy spent to provide aeration and the biogas compressor operation.

In the year of 2016, the aeration time was much lower. Adding to this, the energy produced with the AD was also inferior, meaning the biogas compressor possibly did not operate so intensively.

The design verification results reveal, mainly that more aeration time is required, but also that more biogas production is expected to be achieved. In this context, the biogas compressor, as well as the aeration, are expected a longer operation time, so it makes sense, that the energy consumption is superior to the energy consumption verified in 2016.

6.4 Water Reclamation by Ultrafiltration

In Portugal there is no specific water quality legislation for water reuse. In the year of 2018 there has been a nation-wide growing intent to move in the direction of the reclamation of treated wastewater. The Environment State Secretary aspires to create a nation-wide strategy, legislation to determine the treatment efficiency based on the reclaimed wastewater usage, action procedures for the water management entities, legislation and regulation of the wastewater network (LUSA & PÚBLICO, 2018).

In the year of 2018, in one of Lisbon's most popular music festival, Rock in Rio, the irrigation of a parcel of its enclosure was accomplished with reclaimed water from Beirolas WWTP (ADP, 2018).

As for the European Union, the commission has stated that the proposal of legislation on minimum treatment requirements, for water reuse in irrigation and aquifer recharge, will occur in 2018 (European Commission, 2018).

In this study the reclaimed water was theoretically projected to provide irrigation for a golf course nearby Espinho WWTP. According to the literature review, ultrafiltration is adequate to provide landscape/turf grass irrigation without no follow up process of disinfection. In Tunisia, secondary effluents have been utilized for irrigation of golf courses for more than 20 years.

The projected ultrafiltration unit for water reclamation has its payback period in the 7th year of operation, when considering that the reclaimed water is sold at 0.5 €/m^3 and that all the produced water is sold. If the price of the reclaimed water diminishes or the produced reclaimed water is not sold in its totality, the payback period increases.

An investigation to determine the reasonable price of the treated wastewater, as in how much the nearby industries/activities are willing to pay, as well as, the payback period that the company is willing to accept is necessary.

Moreover, the implementation of UF is only viable if the cost of the treated wastewater outweighs the collective cost of both the supplied potable drinking water and the groundwater derived from the water wells.

The golf club is located in the coast, 200 meters away from the sea, and for that reason it is likely that the groundwater wells might be contaminated from saltwater intrusion, which would ultimately terminate the possibility of irrigation via this source.

Nevertheless, in the event of the golf club, deciding to fulfill their irrigation necessities solely via groundwater, admitting that this is a possibility, the reclaimed water ceases to be a competitive option. If the golf club does not capture groundwater from the surrounding water wells, or assuming that they do but most of the water comes from the supplied potable drinking water network, the treated wastewater becomes a competitive and cheaper alternative.

The investment in water reclamation is only practicable and viable if the golf course's water consumption mostly comes from the supplied potable water network. Therefore, the bigger the parcel of water that comes from the supplied potable water network, the more the treated wastewater becomes a better option in a monetary perspective.

Water reclamation for irrigation is not a viable option in a monetary perspective, in comparison with water provided by groundwater wells, even though it is the most reasonable in an environmental perspective.

Water scarcity is a serious problem worldwide and the fresh water sources, which require minimum to no-treatment should be utilized as potable drinking water supplies.

Enforcing a slightly stricter treatment to the effluent of a WWTP, allows the production of water with satisfactory conditions for several usages, such as irrigation. Water reclamation, if properly exploited, increases the available drinking water supplies.

7 Conclusion

According to the design verification results obtained in this study, and considering its limitations, it can be concluded that Espinho WWTP is better designed for CEPT. Consequently, when performing conventional primary treatment:

- The primary sedimentation tanks do not meet the overflow rate, nor the detention time requirements with the design flow rate;
- The storage of phosphorus is required if Espinho WWTP is operated with conventional primary treatment. The introduction of phosphorus, in the treatment process, is necessary for the biological treatment to occur.
- The biological reactor requires three operating lines (scenario 2). On the contrary the WWTP with CEPT can operate with only two treatment lines in the biological reactor.
- The excess WAS pumps are not adequate, and cannot remove the daily excess WAS sludge from the biological reactor in scenario 2;
- The gravity thickeners have inferior hydraulic loadings, which may lead to septic conditions and to ultimately cause the sludge to float.
- In scenario 2, the mechanical thickening equipment is operating at its maximum capacity.
- The operation costs are higher, mainly due to the energy spent in the aeration process.

This work considered four approaches to optimize Espinho WWTP energy balance and reduce the plant operation costs. The methods studied are CEPT, fine bubble aeration, PV solar panel modules and AcoD. The main objective of this work, reaching energy neutrality, could not be confirmed. This study, however, proves that the energy balance of Espinho WWTP could be significantly improved in a cost-effective manner.

CEPT implementation proves to be beneficial from the start of its implementation, as the oxygen necessities and the cost of aeration are practically reduced in half. According to the jar tests conducted, the optimum PAX18 coagulant dosage is 15 mg/L in conjunction with a flocculant dosage of 0.5 mg/L. This coagulant dosage provides the highest removal in suspended matter while not causing an excessive phosphorus elimination.

Fine bubble aeration proves to be beneficial over surface turbine aeration in Espinho WWTP after 4 to 5 years by providing energy savings, even with just 3.91 meters of depth in the aeration tank. When comparing fine bubble aeration with surface turbine aeration, the first is increasingly more beneficial, the more affluent carbonated matter reaches the aeration tank and the higher the available depth of the aeration tank.

In Espinho WWTP it is possible to install 315 kW of photovoltaic solar panel modules, which cover an area of 5804 m^2. The projected photovoltaic solar panels are profitable after 8 years, improving Espinho WWTP energy balance by 10%.

The anaerobic digesters have unutilized available volume that allow the introduction of 130 to 180 m^3/d of co-substrate to perform AcoD. The energy boost provided by AcoD is underestimated, nevertheless it increases the energy production in the AD process by 84% to 154%.

The AD process is the method that improves the energy balance the most, therefore it is ideal to implement a more efficient CHP unit, while simultaneously striving to maximize the VS destruction in the AD process. The primary sludge higher energy potential was not considered in this study, therefore the energy production with CEPT could probably be higher and the treatment plant could be even closer to energy neutrality, more than demonstrated in this study.

As an immediate measure Espinho WWTP could optimize the primary treatment by performing CEPT. CEPT, apart from the AcoD which was not fully studied, is the only method with an immediate payback. Espinho WWTP with CEPT reaches an energy balance of 42% (scenario 1), which means the plant could supply 42% of its energy needs and would solely be 58% deficient in producing its total energy needs to reach energy neutrality.

The implementation of CEPT in comparison with conventional primary treatment or possibly in comparison with CEPT with a non-optimal coagulant dosage, leads to total energy savings of 895,742 kWh/y, mainly in the aeration and total operation savings of 52,518 €/y. CEPT reduces the energy consumption of the aeration phase by 40%.

Considering the implementation of CEPT, fine bubble aeration, PV solar panel modules and a more efficient CHP unit it is demonstrated in this work that Espinho WWTP reaches an energy balance of 68%, meaning the plant could supply 68% of its energy needs and would solely be 32% deficient in producing its total energy needs to reach energy neutrality. This indicates that energy neutrality could possibly be achieved when AcoD is performed in conjunction with the other methods. However, energy neutrality was not verified in this study, as the incremental costs of operation/investment with AcoD were not determined, and for that reason, the energy balance with the implementation of AcoD was not estimated. Nevertheless, AcoD increases the biogas/energy production by 154% (scenario 1), therefore, energy neutrality is a possibility.

Regarding the UF unit, the supply of treated wastewater for irrigation is only feasible, when competing with the supplied potable water network, because the price is lower, achieving a payback period of 7 years. When competing with groundwater, water reclamation ceases to be a viable option in a monetary perspective. The bigger the parcel of water that comes from the supplied potable water network, the more the treated wastewater becomes a better option in a monetary perspective.

This study demonstrates that, the implementation of inexpensive methods can lead to substantial improvements in the optimization of Espinho WWTP energy balance and operation costs, while maintaining the same adequate level of treatment.

8 Final Considerations

This study is theoretical and comprehends many safety factors, thus the results are not overestimated but reasonable and possibly quite consistent with reality. In this context, an analysis of the practical implementation of the studied options, would be interesting as a complement, by verifying the results obtained in improving Espinho WWTP energy balance.

The elaboration of AcoD mass balances would bring more complexity to this study and since the concentration of TP and TN of the chosen residue were lacking, it was decided not to proceed in this direction. AcoD implementation requires an in-depth attentive study. It demands an incremental application of co-substrate and a week-to-week close analysis of its effects in the digester content stability and performance. I would suggest as a starting point, the increment of 10 % of VS per week.

The realization of the AcoD mass balances, would verify that the volume of co-substrate expected for introduction in the anaerobic digesters, would have to be inferior due to the returns, as more affluent sludge reaches the reactors, resulting in less unutilized volume.

It would be interesting to determine the energy balance of Espinho WWTP with AcoD, to verify the possibility of reaching energy neutrality, when taking the effects in the treatment line of the co-substrate introduced and the produced returns into account. An evaluation of the design of Espinho WWTP, in terms of civil construction and equipment (pumps, rotary drum mechanical thickeners, dewatering centrifuges), when receiving the additional affluent load and flow rate provided by the co-substrate returns is necessary.

Bibliography

Aarhus Vand. (2018). Marselisborg WWTP - turning wastewater into green energy - Aarhus Vand. Retrieved September 21, 2018, from https://www.aarhusvand.dk/en/international/new_solutions/marselisborg-wwtp---turning-wastewater-into-green-energy2/

Abourached, C., English, M. J., & Liu, H. (2016). Wastewater treatment by Microbial Fuel Cell (MFC) prior irrigation water reuse. *Journal of Cleaner Production*, *137*, 144–149. https://doi.org/10.1016/j.jclepro.2016.07.048

ADP. (2018). Rock In Rio pioneiro na utilização de água reciclada | Águas do Tejo Atlântico. Retrieved September 15, 2018, from https://www.aguasdotejoatlantico.adp.pt/noticias/rock-rio-pioneiro-na-utilizacao-de-agua-reciclada

Aelterman, P., Rabaey, K., Clauwaert, P., & Verstraete, W. (2006). Microbial fuel cells for wastewater treatment. *Water Science & Technology*, *54*(8), 9. https://doi.org/10.2166/wst.2006.702

Águas de Portugal. (2018). Retrieved March 26, 2018, from http://www.adp.pt/pt//?id=61&img=28&bl=6

Ahmed, M. B., Zhou, J. L., Ngo, H. H., Guo, W., Thomaidis, N. S., & Xu, J. (2017). Progress in the biological and chemical treatment technologies for emerging contaminant removal from wastewater: A critical review. *Journal of Hazardous Materials*, *323*, 274–298. https://doi.org/10.1016/j.jhazmat.2016.04.045

Aichinger, P., Wadhawan, T., Kuprian, M., Higgins, M., Ebner, C., Fimml, C., … Wett, B. (2015). Synergistic co-digestion of solid-organic-waste and municipal-sewage-sludge: 1 plus 1 equals more than 2 in terms of biogas production and solids reduction. *Water Research*, *87*, 416–423. https://doi.org/10.1016/J.WATRES.2015.07.033

Alfa Laval. (n.d.). *ALDRUM G3 – a revolution in sludge thickening – Rotary drum filter for medium to high capacity sludge thickening*.

Alley, E. R. (2007). *Water Quality Control Handbook*. *Environment* (Vol. 1). McGraw-Hill. https://doi.org/10.1036/0071467602

Andreoli, C. V., Von Sperling, M., & Fernandes, F. (2007). *Biological Wastewater Treatment Series (Volume 6: Sludge Treatment and Disposal)*. London, UK: IWA Publishing.

APA. LURH - Licença de Utilização dos Recursos Hídricos - Rejeição de Águas Residuais (2017). Portugal.

Appels, L., Baeyens, J., Degrève, J., & Dewil, R. (2008). Principles and potential of the anaerobic digestion of waste-activated sludge. *Progress in Energy and Combustion Science*, *34*(6), 755–781. https://doi.org/10.1016/j.pecs.2008.06.002

Arévalo, J., Ruiz, L. M., Parada-Albarracín, J. A., González-Pérez, D. M., Pérez, J., Moreno, B., & Gómez, M. A. (2012). Wastewater reuse after treatment by MBR. Microfiltration or ultrafiltration? *Desalination*, *299*, 22–27. https://doi.org/10.1016/j.desal.2012.05.008

Arnaiz, C., Gutierrez, J. C., & Lebrato, J. (2006). Biomass stabilization in the anaerobic digestion of wastewater sludges. *Bioresource Technology*, *97*(10), 1179–1184. https://doi.org/10.1016/j.biortech.2005.05.010

ATV-DVWK Specialist Committees. (2000). *ATV-DVWK Standard A 131 E. Dimensioning of Single-Stage Activated Sludge Plants*.

Ayoub, M., Afify, H., & Abdelfattah, A. (2017). Chemically enhanced primary treatment of sewage using the recovered alum from water treatment sludge in a model of hydraulic clari-flocculator. *Journal of Water Process Engineering*, *19*(July), 133–138. https://doi.org/10.1016/j.jwpe.2017.07.014

Baptista, J. M., Almeida, M. D. C., Silva, A. C. M., Ribeiro, R., Fernando, R. M., Serafim, A., … Cameira, M. R. (2012). Programa Nacional para o Uso Eficiente da agua. *Lisboa: Instituto Superior de Agronomia*, 201.

Bastian, R., Cuttica, J., Fillmore, L., Bruce, H., Hornback, C., Levy, D., & Moskal, J. (2011). Opportunities for Combined Heat and Power at Wastewater Treatment Facilities : Market Analysis and Lessons from the Field Combined Heat and Power Partnership. *US EPA, CHP Partnership*, (October), 57. https://doi.org/430R11018

Billund BioRefinery. (2018). Billund BioRefinery - Resource Recovery for the Future. Retrieved September 21, 2018, from http://www.billundbiorefinery.dk/en/

Borges, N. B., Campos, J. R., & Pablos, J. M. (2015). Characterization of residual sand removed from the grit chambers of a wastewater treatment plant and its use as fine aggregate in the preparation of non-structural concrete. *Water Practice and Technology*, *10*(1), 164–171. https://doi.org/10.2166/wpt.2015.018

Bratby, J. (2006). *Coagulation and Flocculation in Water and Wastewater Treatment* (Second Edi). London: IWA Publishing. https://doi.org/10.2166/9781780407500

Budych-Gorzna, M., Smoczynski, M., & Oleskowicz-Popiel, P. (2016). Enhancement of biogas production at the municipal wastewater treatment plant by co-digestion with poultry industry waste. *Applied Energy*, *161*, 387–394. https://doi.org/10.1016/j.apenergy.2015.10.007

Căilean, D., Barjoveanu, G., Teodosiu, C., Pintilie, L., Dăscălescu, I. G., & Păduraru, C. (2015). Technical performances of ultrafiltration applied to municipal wastewater treatment plant effluents. *Desalination and Water Treatment*, *56*(6), 1476–1488. https://doi.org/10.1080/19443994.2014.951693

Cao, Y., & Pawłowski, A. (2012). Sewage sludge-to-energy approaches based on anaerobic digestion and pyrolysis: Brief overview and energy efficiency assessment. *Renewable and Sustainable Energy Reviews*, *16*(3), 1657–1665. https://doi.org/10.1016/j.rser.2011.12.014

Carlsson, M., Lagerkvist, A., & Morgan-Sagastume, F. (2016). Energy balance performance of municipal wastewater treatment systems considering sludge anaerobic biodegradability and biogas utilisation routes. *Journal of Environmental Chemical Engineering*, *4*(4). https://doi.org/10.1016/j.jece.2016.10.030

Carrère, H., Dumas, C., Battimelli, A., Batstone, D. J., Delgenès, J. P., Steyer, J. P., & Ferrer, I. (2010). Pretreatment methods to improve sludge anaerobic degradability: A review. *Journal of Hazardous Materials*, *183*(1–3), 1–15. https://doi.org/10.1016/j.jhazmat.2010.06.129

Chen, G., Yue, P. L., & Mujumdar, A. S. (2006). Sludge Dewatering and Drying. *Drying Technology*, *20*(4–5), 883–916. https://doi.org/10.1081/DRT-120003768

Cheremisinoff, N. P. (2002). *Handbook of water and wastewater Treatment Tecnhologies*. *Butterworth-Heinemann*. https://doi.org/10.1016/B978-075067498-0/50006-1

Chhetri, R. K., Thornberg, D., Berner, J., Gramstad, R., Öjstedt, U., Sharma, A. K., & Andersen, H. R. (2014). Chemical disinfection of combined sewer overflow waters using performic acid or peracetic acids. *Science of the Total Environment*, *490*, 1065–1072. https://doi.org/10.1016/j.scitotenv.2014.05.079

Christensen, M. L. ykkegaar., Keiding, K., Nielsen, P. H. alkjæ., & Jørgensen, M. K. oustru. (2015). Dewatering in biological wastewater treatment: A review. *Water Research*, *82*, 14–24. https://doi.org/10.1016/j.watres.2015.04.019

Collivignarelli, M. C., Abbà, A., Benigna, I., Sorlini, S., & Torretta, V. (2018). Overview of the main disinfection processes for wastewater and drinking water treatment plants. *Sustainability (Switzerland)*, *10*(1), 1–21. https://doi.org/10.3390/su10010086

Dai, W., Xu, X., Liu, B., & Yang, F. (2015). Toward energy-neutral wastewater treatment: A membrane combined process of anaerobic digestion and nitritation-anammox for biogas recovery and nitrogen removal. *Chemical Engineering Journal*, *279*, 725–734. https://doi.org/10.1016/j.cej.2015.05.036

Davies, P. (2005). The Biological Basis of Wastewater Treatment. *Strathkelvin Instruments Ltd. United Kingdom*, 20. Retrieved from http://bartec-benke.nl/media/1000154/thebiologicalbasisofwastewatertreatment.pdf

Davis, M. (2010). *Water and Wastewater Engineering: Design Principles and Practice*. McGraw-Hill. Retrieved from https://drive.google.com/open?id=0B544TwTVwnWkaXBoTVNtU3p2MFk

De Feo, G., De Gisi, S., & Galasso, M. (2008). Definition of a practical multi-criteria procedure for selecting the best coagulant in a chemically assisted primary sedimentation process for the treatment of urban wastewater. *Desalination*. https://doi.org/10.1016/j.desal.2007.12.003

De Feo, G., Galasso, M., Landi, R., Donnarumma, A., & De Gisi, S. (2013). A comparison of the efficacy of organic and mixed-organic polymers with polyaluminium chloride in chemically assisted primary sedimentation (CAPS). *Environmental Technology (United Kingdom)*, *34*(10), 1297–1305. https://doi.org/10.1080/09593330.2012.745622

Degrémont, G. (1991). *Mémento technique de l'eau* (Sixth Edit, Vol. 1 and 2). Paris: Lavoisier Publishing.

Dell'Erba, A., Falsanisi, D., Liberti, L., Notarnicola, M., & Santoro, D. (2004). Disinfecting behaviour of peracetic acid for municipal wastewater reuse. *Desalination*, *168*(1–3), 435–442. https://doi.org/10.1016/j.desal.2004.07.028

Demirbas, A., Edris, G., & Alalayah, W. M. (2017). Sludge production from municipal wastewater treatment in sewage treatment plant. *Energy Sources, Part A: Recovery, Utilization and Environmental Effects*, *39*(10), 999–1006. https://doi.org/10.1080/15567036.2017.1283551

Di Fraia, S., Massarotti, N., & Vanoli, L. (2018). A novel energy assessment of urban wastewater treatment plants. *Energy Conversion and Management*, *163*(November 2017), 304–313. https://doi.org/10.1016/j.enconman.2018.02.058

Diamantis, V., Verstraete, W., Eftaxias, A., Bundervoet, B., Vlaeminck, S. E., Melidis, P., & Aivasidis, A. (2013). Sewage pre-concentration for maximum recovery and reuse at decentralized level. *Water Science and Technology*, *67*(6), 1188–1193. https://doi.org/10.2166/wst.2013.639

Eslamian, S. (2016). *Urban Water Reuse Handbook*. New York: CRC Press. https://doi.org/10.1201/b19646

Eslamian, S., & Eslamian, F. (2016). *Handbook of Drought and Water Scarcity: Environmental Impacts and Analysis of Drought and Water*. New York: CRC Press.

European Commission. (2010). *Membrane technologies for water applications*. *Water*. https://doi.org/10.2777/25163

European Commission. (2018). Water Reuse - Environment - European Commission. Retrieved May 17, 2018, from http://ec.europa.eu/environment/water/reuse-actions.htm

Food and Agriculture Organization. (2014). Water Withdrawal Infographic, 2014. Retrieved from http://www.fao.org/nr/water/aquastat/infographics/Withdrawal_eng.pdf

Forster, C. F. (2003). *Wastewater Treatment and Technology*. Thomas Telford. Retrieved from https://books.google.pt/books?id=1QlvKJu6PRcC&pg=PA276&lpg=PA276&dq=optimum+c:n:p+ratio+of+aerobic+biological+treatment+100+5+1&source=bl&ots=EMEEcAr13-&sig=-SWMCtjqRfUVnGDKRdCn7hww5yw&hl=pt-PT&sa=X&ved=0ahUKEwi76NXu7oDcAhXIyKQKHcrdAU8Q6AEIejAJ#v=onepage&

Francy, D. S., Stelzer, E. A., Bushon, R. N., Brady, A. M. G., Williston, A. G., Riddell, K. R., … Gellner, T. M. (2012). Comparative effectiveness of membrane bioreactors, conventional secondary treatment, and chlorine and UV disinfection to remove microorganisms from municipal wastewaters. *Water Research*, *46*(13), 4164–4178. https://doi.org/10.1016/j.watres.2012.04.044

Gao, H., Scherson, Y. D., & Wells, G. F. (2014). Towards energy neutral wastewater treatment: methodology and state of the art. *Environ. Sci.: Processes Impacts*, *16*(6), 1223–1246.

https://doi.org/10.1039/C4EM00069B

Gebbie, P. (2001). Using Polyaluminium Coagulants in Water Treatment. *64th Annual Water Industry Engineers and Operators' Conference*, (39), 39–47.

Gikas, P. (2017). Towards energy positive wastewater treatment plants. *Journal of Environmental Management*, *203*, 621–629. https://doi.org/10.1016/j.jenvman.2016.05.061

Google Maps. (2018). Espinho WWTP. Retrieved July 19, 2018, from https://www.google.pt/maps/place/ETAR+de+Espinho/@40.9773363,-8.6441647,2417m/data=!3m1!1e3!4m5!3m4!1s0xd247817e15cfda3:0x2d584edba06c168a!8m2!3d40.9760901!4d-8.6479931

Gu, Y., Li, Y., Li, X., Luo, P., Wang, H., Robinson, Z. P., … Li, F. (2017). The feasibility and challenges of energy self-sufficient wastewater treatment plants. *Applied Energy*, *204*, 1463–1475. https://doi.org/10.1016/j.apenergy.2017.02.069

Gu, Y., Li, Y., Li, X., Luo, P., Wang, H., Wang, X., … Li, F. (2017). Energy Self-sufficient Wastewater Treatment Plants: Feasibilities and Challenges. *Energy Procedia*, *105*, 3741–3751. https://doi.org/10.1016/j.egypro.2017.03.868

Gude, V. G. (2015). Energy and water autarky of wastewater treatment and power generation systems. *Renewable and Sustainable Energy Reviews*, *45*, 52–68. https://doi.org/10.1016/j.rser.2015.01.055

Gude, V. G. (2016). Wastewater treatment in microbial fuel cells - An overview. *Journal of Cleaner Production*, *122*, 287–307. https://doi.org/10.1016/j.jclepro.2016.02.022

Guerrini, A., Romano, G., & Indipendenza, A. (2017). Energy efficiency drivers in wastewater treatment plants: A double bootstrap DEA analysis. *Sustainability (Switzerland)*, *9*(7), 1–13. https://doi.org/10.3390/su9071126

Han, D., Lee, C., Chang, S. W., & Kim, D. (2017). Enhanced methane production and wastewater sludge stabilization of a continuous full scale thermal pretreatment and thermophilic anaerobic digestion. *Bioresource Technology*. https://doi.org/10.1016/j.biortech.2017.08.108

Hao, X. D., Li, J., Van Loosdrecht, M. C. M., & Li, T. Y. (2018). A sustainability-based evaluation of membrane bioreactors over conventional activated sludge processes. *Journal of Environmental Chemical Engineering*, *6*(2), 2597–2605. https://doi.org/10.1016/j.jece.2018.03.050

Haydar, S., & Aziz, J. A. (2009). Characterization and treatability studies of tannery wastewater using chemically enhanced primary treatment (CEPT)-A case study of Saddiq Leather Works. *Journal of Hazardous Materials*, *163*(2–3), 1076–1083. https://doi.org/10.1016/j.jhazmat.2008.07.074

He, L., Du, P., Chen, Y., Lu, H., Cheng, X., Chang, B., & Wang, Z. (2017). Advances in microbial fuel cells for wastewater treatment. *Renewable and Sustainable Energy Reviews*, *71*(October), 388–403. https://doi.org/10.1016/j.rser.2016.12.069

Henze, M., Harremoës, P., Cour Jansen, J., & Arvin, E. (1997). *Wastewater Treatment: Biological and Chemical Processes*. Berlin, Heidelberg: Springer Berlin Heidelberg. https://doi.org/10.1007/978-3-662-22605-6

Illueca-Muñoz, J., Mendoza-Roca, J. A., Iborra-Clar, A., Bes-Piá, A., Fajardo-Montañana, V., Martínez-Francisco, F. J., & Bernácer-Bonora, I. (2008). Study of different alternatives of tertiary treatments for wastewater reclamation to optimize the water quality for irrigation reuse. *Desalination*, *222*(1–3), 222–229. https://doi.org/10.1016/j.desal.2007.01.157

Insam, H., & Markt, R. (2016). Synergistic co-digestion of solid-organic-waste and municipal-sewage-sludge: 1 plus 1 equals more than 2 in terms of biogas production and solids reduction [Water Research 87, 416-423]. *Water Research*, *95*, 392–393. https://doi.org/10.1016/j.watres.2016.02.053

Jacob, M., Guigui, C., Cabassud, C., Darras, H., Lavison, G., & Moulin, L. (2010). Performances of RO and NF processes for wastewater reuse: Tertiary treatment after a conventional activated sludge or a membrane bioreactor. *Desalination*, *250*(2), 833–839. https://doi.org/10.1016/j.desal.2008.11.052

Jover-Smet, M., Martín-Pascual, J., & Trapote, A. (2017). Model of suspended solids removal in the primary sedimentation tanks for the treatment of urban wastewater. *Water (Switzerland)*, *9*(6). https://doi.org/10.3390/w9060448

Judd, S. (2010). *The MBR book : principles and applications of membrane bioreactors for water and wastewater treatment*. Butterworth-Heinemann.

Kalogo, Y., & Monteith, H. (2013). *Energy and Resource Recovery From Sludge*. London, UK: IWA Publishing.

Kim, M., Chowdhury, M. M. I., Nakhla, G., & Keleman, M. (2017). Synergism of co-digestion of food wastes with municipal wastewater treatment biosolids. *Waste Management*, *61*, 473–483. https://doi.org/10.1016/j.wasman.2016.10.010

Kitis, M. (2004). Disinfection of wastewater with peracetic acid: A review. *Environment International*, *30*(1), 47–55. https://doi.org/10.1016/S0160-4120(03)00147-8

Koch, K., Helmreich, B., & Drewes, J. E. (2015). Co-digestion of food waste in municipal wastewater treatment plants: Effect of different mixtures on methane yield and hydrolysis rate constant. *Applied Energy*, *137*, 250–255. https://doi.org/10.1016/j.apenergy.2014.10.025

Koch, K., Plabst, M., Schmidt, A., Helmreich, B., & Drewes, J. E. (2016). Co-digestion of food waste in a municipal wastewater treatment plant: Comparison of batch tests and full-scale experiences. *Waste Management*, *47*, 28–33. https://doi.org/10.1016/j.wasman.2015.04.022

Krishnamurthy, S., & Viraraghavan, T. (2005). Chemical conditioning for dewatering municipal wastewater sludges. *Energy Sources*, *27*(1–2), 113–122. https://doi.org/10.1080/00908310490448154

Lazarova, V., & Bahri, A. (2005). *Water Reuse for Irrigation - Agriculture, Landscapes and Turf Grass*. *New York*. Boca Raton, Florida: CRC Press.

Lazarova, V., Choo, K.-H., & Cornel, P. (2012). *Water-energy Interactions in Water Reuse*. London, UK: IWA Publishing. Retrieved from https://books.google.com/books?hl=en&lr=&id=M_1kN8INayIC&pgis=1

Lee, C. S., Robinson, J., & Chong, M. F. (2014). A review on application of flocculants in wastewater treatment. *Process Safety and Environmental Protection*, *92*(6), 489–508. https://doi.org/10.1016/j.psep.2014.04.010

Lema, J. M., & Suarez, S. (2017). *Innovative Wastewater Treatment & Resource Recovery Technologies*. Retrieved from https://www.iwapublishing.com/books/9781780407869/innovative-wastewater-treatment-resource-recovery-technologies-impacts-energy

Lettinga, G. (1995). *Anaerobic Digestion and Wastewater Treatment Systems*. *Antonie van Leeuwenhoek* (Vol. 67). Wageningen, Netherlands. https://doi.org/10.1007/BF00872193

Li, W. W., & Yu, H. Q. (2016). Advances in Energy-Producing Anaerobic Biotechnologies for Municipal Wastewater Treatment. *Engineering*, *2*(4), 438–446. https://doi.org/10.1016/J.ENG.2016.04.017

Liberti, L., Notarnicola, M., & Petruzzelli, D. (2003). Advanced treatment for municipal wastewater reuse in agriculture. UV disinfection: Parasite removal and by-product formation. *Desalination*, *152*(1–3), 315–324. https://doi.org/10.1016/S0011-9164(02)01079-2

Lindmark, J., Thorin, E., Bel Fdhila, R., & Dahlquist, E. (2014). Effects of mixing on the result of anaerobic digestion: Review. *Renewable and Sustainable Energy Reviews*, *40*, 1030–1047. https://doi.org/10.1016/j.rser.2014.07.182

Ljunggren, M. (2006). Micro screening in wastewater treatMent – an overview. *Vatten*, *62*, 171–177.

LUSA, & PÚBLICO. (2018, August 25). Portugal reforça aposta no uso de águas residuais tratadas | Ambiente | PÚBLICO. Retrieved from https://www.publico.pt/2018/08/25/ecosfera/noticia/portugal-reforca-aposta-na-utilizacao-de-aguas-residuais-tratadas-1842030

Luukkonen, T., Heyninck, T., Rämö, J., & Lassi, U. (2015). Comparison of organic peracids in wastewater treatment: Disinfection, oxidation and corrosion. *Water Research*, *85*, 275–285. https://doi.org/10.1016/j.watres.2015.08.037

Luukkonen, T., Teeriniemi, J., Prokkola, H., Rämö, J., & Lassi, U. (2014). Chemical aspects of peracetic acid based wastewater disinfection. *Water SA*, *40*(1), 73–80. https://doi.org/10.4314/wsa.v40i1.9

Ma, X. (Cissy), Xue, X., González-Mejía, A., Garland, J., & Cashdollar, J. (2015). Sustainable water systems for the city of tomorrow-A conceptual framework. *Sustainability (Switzerland)*, *7*(9), 12071–12105. https://doi.org/10.3390/su70912071

Mamais, D., Tzimas, A., Efthimiadou, A., Kissandrakis, J., & Andreadakis, A. (2009). Evaluation of Different Sludge Mechanical Dewatering Technologies. *J Residuals Sci Tech.*, *6*(1), 27–34.

Mara, D. (2004). *Domestic wastewater treatment in developing countries*. London, UK: Earthscan Publications. https://doi.org/10.1017/CBO9781107415324.004

Mattioli, A., Gatti, G. B., Mattuzzi, G. P., Cecchi, F., & Bolzonella, D. (2017). Co-digestion of the organic fraction of municipal solid waste and sludge improves the energy balance of wastewater treatment plants: Rovereto case study. *Renewable Energy*, *113*, 980–988. https://doi.org/10.1016/j.renene.2017.06.079

McCarty, P. L., Bae, J., & Kim, J. (2011). Domestic wastewater treatment as a net energy producer - can this be achieved? *Environmental Science and Technology*, *45*(17), 7100–7106. https://doi.org/10.1021/es2014264

Meerburg, F. A., Boon, N., Van Winckel, T., Vercamer, J. A. R., Nopens, I., & Vlaeminck, S. E. (2015). Toward energy-neutral wastewater treatment: A high-rate contact stabilization process to maximally recover sewage organics. *Bioresource Technology*, *179*, 373–381. https://doi.org/10.1016/j.biortech.2014.12.018

Meneses, M., Pasqualino, J. C., & Castells, F. (2010). Chemosphere Environmental assessment of urban wastewater reuse : Treatment alternatives and applications. *Chemosphere*, *81*(2), 266–272. https://doi.org/10.1016/j.chemosphere.2010.05.053

Meroney, R. N., & Sheker, R. E. (2003). Removing Grit During Wastewater Treatment : CFD Analysis of HDVS Performance, *88*(5). https://doi.org/10.2175/106143015X14212658614478

Ministério do Ambiente. (1997). Decreto-Lei nº 152/97 de 19 de Junho de 1997. *Diário Da República - 1 Série-A*, *Nº 139*, 2959–2967.

Mo, W., & Zhang, Q. (2013). Energy-nutrients-water nexus: Integrated resource recovery in municipal wastewater treatment plants. *Journal of Environmental Management*, *127*, 255–267. https://doi.org/10.1016/j.jenvman.2013.05.007

Molinos-Senante, M., Sala-Garrido, R., & Iftimi, A. (2018). Energy intensity modeling for wastewater treatment technologies. *Science of the Total Environment*, *630*, 1565–1572. https://doi.org/10.1016/j.scitotenv.2018.02.327

Monte, M. M. (2010). *Contributo para o estudo da valorização energética de biogás em Estações de Tratamento de Águas Residuais*. Universidade Nova de Lisboa, Faculdade de Ciências e Tecnologia.

Murthy, S., Higgins, M., Maas, N., Chen, Y.-C., & Troxell, J. (2004). WERF Phase 2: Factors affecting conditioning during dewatering- Influence of dewatering equipment. *Proceedings* *of the Water Environment Federation*, *2004*(14), 367–375. https://doi.org/10.2175/193864704784137170

Ng, M., Liu, S., Chow, C. W. K., Drikas, M., Amal, R., & Lim, M. (2013). Understanding effects of water characteristics on natural organic matter treatability by PACl and a novel PACl-chitosan coagulants. *Journal of Hazardous Materials*, *263*, 718–725. https://doi.org/10.1016/j.jhazmat.2013.10.036

Norton-Brandão, D., Scherrenberg, S. M., & van Lier, J. B. (2013). Reclamation of used urban waters for irrigation purposes - A review of treatment technologies. *Journal of Environmental Management*, *122*, 85–98. https://doi.org/10.1016/j.jenvman.2013.03.012

Novak, J. T. (2006). Dewatering of sewage sludge. *Drying Technology*, *24*(10), 1257–1262. https://doi.org/10.1080/07373930600840419

Nowak, O., Enderle, P., & Varbanov, P. (2015). Ways to optimize the energy balance of municipal wastewater systems: Lessons learned from Austrian applications. *Journal of Cleaner Production*, *88*(February), 125–131. https://doi.org/10.1016/j.jclepro.2014.08.068

Olsson, G., K., N. M., Yuan, Z., Lynggaard-Jensen, A., & Steyer, J.-P. (2005). *Instrumentation, Control and Automation in Wastewater Systems*. *IWA Scientific and Technical Report Series* (Vol. No. 15). IWA Publishing. Retrieved from https://books.google.com/books?id=RW6aprXf7l0C&pgis=1

Ordóñez, R., Hermosilla, D., Merayo, N., Gascó, A., Negro, C., & Blanco, Á. (2014). Application of multi-barrier membrane filtration technologies to reclaim municipal wastewater for industrial use. *Separation and Purification Reviews*, *43*(4), 263–310. https://doi.org/10.1080/15422119.2012.758638

Papa, M., Foladori, P., Guglielmi, L., & Bertanza, G. (2017). How far are we from closing the loop of sewage resource recovery? A real picture of municipal wastewater treatment plants in Italy. *Journal of Environmental Management*, *198*, 9–15. https://doi.org/10.1016/j.jenvman.2017.04.061

Parra-Orobio, B. A., Torres-Lozada, P., & Marmolejo-Rebellón, L. F. (2016). Influencia de la proporción de mezcla en la codigestión anaerobia de biorresiduos municipales con lodos de aguas residuales domésticas sobre la producción de metano. *Dyna*, *83*(199), 86–93. https://doi.org/10.15446/dyna.v83n199.57382

Paulsrud, B., Rusten, B., & Aas, B. (2014). Increasing the sludge energy potential of wastewater treatment plants by introducing fine mesh sieves for primary treatment. *Water Science and Technology*, *69*(3), 560–565. https://doi.org/10.2166/wst.2013.737

Pilli, S., Yan, S., Tyagi, R. D., & Surampalli, R. Y. (2015). Thermal pretreatment of sewage sludge to enhance anaerobic digestion: A review. *Critical Reviews in Environmental Science and Technology*, *45*(6), 669–702. https://doi.org/10.1080/10643389.2013.876527

Poon, C. S., & Chu, C. W. (1999). The use of ferric chloride and anionic polymer in the Chemically Assisted Primary Sedimentation Process. *Chemosphere*, *39*(10), 1573–1582. https://doi.org/10.1016/S0045-6535(99)00055-7

Puchajda, B., & Oleszkiewicz, J. (2008). Impact of sludge thickening on energy recovery from anaerobic digestion. *Water Science and Technology*, *57*(3), 395–401. https://doi.org/10.2166/wst.2008.021

Qasim, S. R. (1999). *Wastewater Treatment Plants: Planning, Design and Operation* (Second Edi). Lancaster, Pa: Technomic Pub. Co.

Rieger, L., Takács, I., & Siegrist, H. (2012). Improving Nutrient Removal While Reducing Energy Use at Three Swiss WWTPs Using Advanced Control. *Water Environment Research*, *84*(2), 170–188. https://doi.org/10.2175/106143011X13233670703684

Roccaro, P. (2018). Treatment processes for municipal wastewater reclamation: The challenges of emerging contaminants and direct potable reuse. *Current Opinion in Environmental Science & Health*, *2*, 46–54. https://doi.org/10.1016/j.coesh.2018.02.003

Sepp, M., Kõiv, T., Nõges, P., & Nõges, T. (2018). Science of the Total Environment Do organic matter metrics included in lake surveillance monitoring in Europe provide a broad picture of browni fi cation and enrichment with oxygen consuming substances ? *Science of the Total Environment*, *610–611*, 1288–1297. https://doi.org/10.1016/j.scitotenv.2017.08.179

Sharma, S. K., & Sanghi, R. (2013). *Wastewater Reuse and Management*. *Wastewater Reuse and Management*. Dordrecht, Netherlands: Springer. https://doi.org/10.1007/978-94-007-4942-9

Spellman, F. R. (2010). *Spellman's Standard Handbook for Wastewater Operators Volume I*. New York: CRC Press. https://doi.org/10.1201/EBK1439818862

Sperling, M. von. (2007). *Biological Wastewater Treatment Series: Activated Sludge and Aerobic Biofilm Reactors*. London, UK: IWA Publishing. Retrieved from https://www.iwapublishing.com/sites/default/files/ebooks/9781780402123.pdf

Stamatelatou, K., & Tsagarakis, K. P. (2015). *Sewage Treatment Plants: Economic Evaluation of Innovative Technologies for Energy Efficiency*. London, UK: IWA Publishing.

Tandukar, M., & Pavlostathis, S. G. (2015). Co-digestion of municipal sludge and external organic wastes for enhanced biogas production under realistic plant constraints. *Water Research*, *87*, 432–445. https://doi.org/10.1016/j.watres.2015.04.031

Tchobanoglous, G., Burton, F. L., & Stensel, H. D. (2014). *Wastewater Engineering: Treatment and Resource Recovery*. *Metcalf & Eddy* (Fifth Edit). New York: McGraw-Hill Education. https://doi.org/10.1016/0309-1708(80)90067-6

Tuan, P. A., Mika, S., & Pirjo, I. (2012). Sewage Sludge Electro-Dewatering Treatment-A Review. *Drying Technology*, *30*(7), 691–706. https://doi.org/10.1080/07373937.2012.654874

Turovskiy, I. S., & Mathai, P. K. (2006). *Wastewater Sludge Processing*. New Jersey: John Wiley & Sons, Inc. https://doi.org/10.1002/047179161X

U.S. EPA. (2000). Wastewater Technology Fact Sheet Package Plants. *United States Environmental Protection Agency*, 1–7. https://doi.org/EPA 832-F-99-062

U.S. EPA, & American Society of Civil Engineers (ASCE). (1983). Development of Standard Procedures for Evaluating Oxygen Transfer Devices.

U.S.EPA. (2014). Energy Efficiency in Water and Wastewater Facilities. *U.S. Environmental Protection Agency*, 49.

UNESCO. (2012). *World Water Development Report Volume 4: Managing Water under Uncertainty and Risk*. *UN Water Report* (Vol. 1). https://doi.org/10.1608/FRJ-3.1.2

UNESCO. (2017). *The United Nations World Water Development Report 2017. Wastewater: The Untapped Resource. The United Nations World Water Development Report. Wastewater. The Untapped Resource.* Retrieved from http://unesdoc.unesco.org/images/0024/002471/247153e.pdf

UNESCO. (2018). *The United Nations World Water Development Report 2018: Nature-Based Solutions for Water*. Retrieved from http://unesdoc.unesco.org/images/0026/002614/261424e.pdf

Vaseghi, G., Ghassemi, A., & Loya, J. (2016). Characterization of reverse osmosis and nanofiltration membranes: effects of operating conditions and specific ion rejection. *Desalination and Water Treatment*, *57*(50), 23461–23472. https://doi.org/10.1080/19443994.2015.1135825

Vergine, P., Salerno, C., Libutti, A., Beneduce, L., Gatta, G., Berardi, G., & Pollice, A. (2017). Closing the water cycle in the agro-industrial sector by reusing treated wastewater for irrigation. *Journal of Cleaner Production*, *164*, 587–596. https://doi.org/10.1016/j.jclepro.2017.06.239

Vesilind, P. A. (2003). *Wastewater Treatment Plant Design*. London: Water Environment Federation; IWA Publishing.

Voutchkov, N. (2017). *Fundamentals of Clarifier Performance Monitoring and Control*. https://doi.org/10.13140/RG.2.2.16469.42725

Wan, J., Gu, J., Zhao, Q., & Liu, Y. (2016). COD capture: A feasible option towards energy self-sufficient domestic wastewater treatment. *Scientific Reports*, *6*(November). https://doi.org/10.1038/srep25054

Water Environment Federation., Water Environment Research Foundation., & United States. Environmental Protection Agency. (2012). *Solids process design and management.* McGraw-Hill. Retrieved from https://www.e-wef.org/Default.aspx?tabid=251&ProductId=18173

Water Environment Federation. (2005). *Clarifier Design, WEF Manual of Practice No. FD-8* (Second Edi). McGraw-Hill Professional. Retrieved from https://www.mheducation.ca/professional/products/9780071464161/clarifier+design:+wef+manual+of+practice+no.+fd-8/

Water Environment Federation. (2008). *Operation of Municipal Wastewater Treatment Plants*. *Environment*. https://doi.org/10.1036/0071543678

Water Environment Federation. (2011). *National Manual of Good Practice for Biosolids*.

Wintgens, T., Melin, T., Schäfer, A., Khan, S., Muston, M., Bixio, D., & Thoeye, C. (2005). The role of membrane processes in municipal wastewater reclamation and reuse. *Desalination*, *178*(1–3 SPEC. ISS.), 1–11. https://doi.org/10.1016/j.desal.2004.12.014

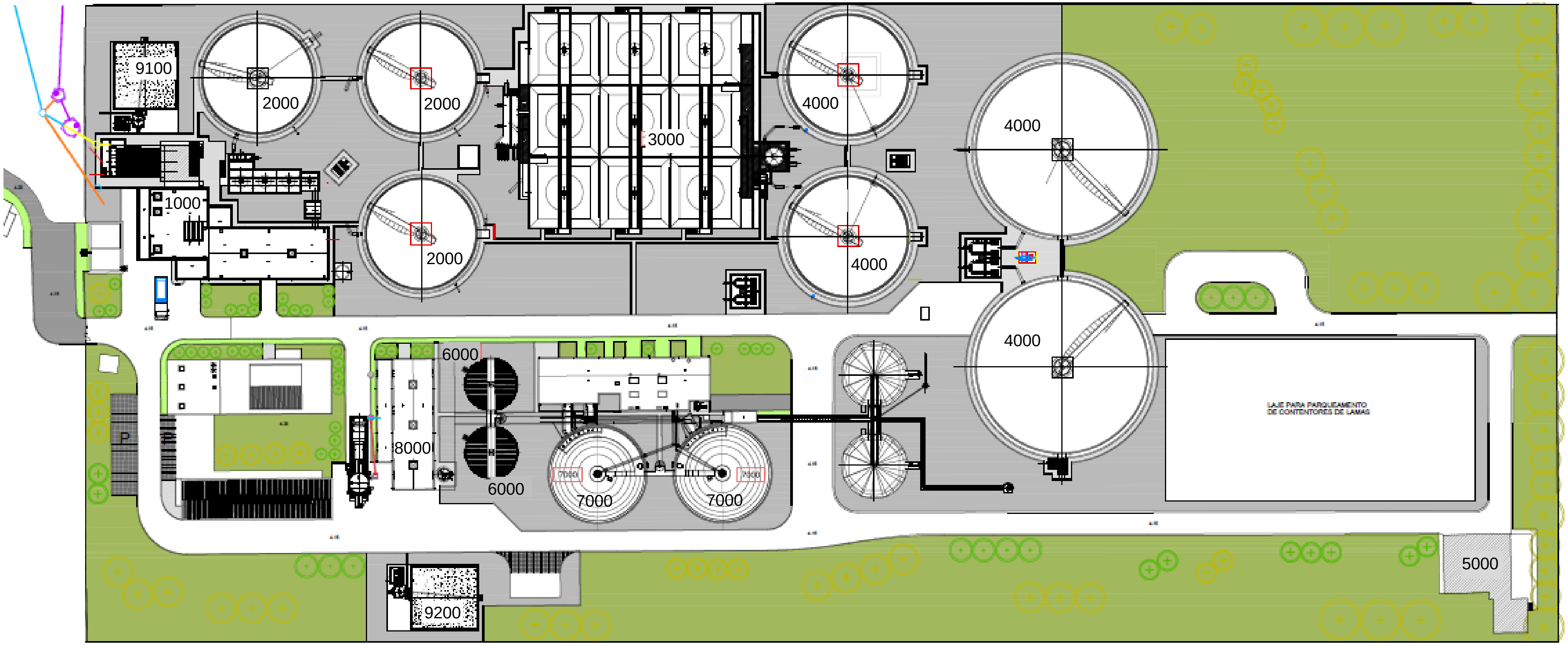
Zamanzadeh, M., Hagen, L. H., Svensson, K., Linjordet, R., & Horn, S. J. (2017). Biogas production from food waste via co-digestion and digestion- effects on performance and microbial ecology. *Scientific Reports*, *7*(1), 17664. https://doi.org/10.1038/s41598-017-15784-w

Zhang, X., Li, X., Zhang, Q., Peng, Q., Zhang, W., & Gao, F. (2014). New insight into the biological treatment by activated sludge: The role of adsorption process. *Bioresource Technology*, *153*, 160–164. https://doi.org/10.1016/j.biortech.2013.11.084

Zouboulis, A. I., & Tzoupanos, N. (2010). Alternative cost-effective preparation method of polyaluminium chloride (PAC) coagulant agent: Characterization and comparative application for water/wastewater treatment. *Desalination*, *250*(1), 339–344. https://doi.org/10.1016/j.desal.2009.09.053

Annexes

Annex A – Espinho WWTP Photographs



Espinho WWTP Layout – the numbered unit treatment processes are identified in the next table.

Nº	Unit processes
1000	Preliminary treatment
2000	Primary sedimentation
3000	Biological treatment
4000	Secondary sedimentation
5000	Pumping of the final effluent to the marine outfall
6000	Sludge thickening
7000	Anaerobic digestion of the thickened sludge
8000	Mechanical dewatering of the digested sludge
9100	Deodorization of the preliminary treatment
9200	Deodorization of the sludge treatment phase

Preliminary treatment facility

Fine screens

Aerated grit chamber

Empty aerated grit chamber (visible diffusers at the bottom of the tank)

Grit storage container for disposal

CEPT reagent storage container

Primary clarifier

Biological reactor during aeration

Secondary clarifier

Secondary clarifier

Secondary clarifier

Secondary clarifier surface skimmer arm and trough for scum removal

1 of 2 gravity thickeners – receives primary sludge

Mechanical thickeners (3) – receives biological sludge

1 of 2 Digesters

2 of 2 biogas holders

Biogas holders and biogas torch in the background (in the middle)

Dewatering/mechanical thickening facility

Polyelectrolyte preparation unit – for sludge conditioning of the dewatering and thickening processes

1 of 3 dewatering centrifuges

Dewatered sludge in the screw pump

Dewatered sludge silo

Annex B – COD Determination Method

1.14895.0001

Spectroquant®

COD Cell Test

COD

USEPA approved for wastewater

1. Definition

The COD (chemical oxygen demand) expresses the amount of oxygen originating from potassium dichromate that reacts with the oxidizable substances contained in 1 l of water under the working conditions of the specified procedure.

1 mol $K_2Cr_2O_7$ is equivalent to 1.5 mol O_2
Results are expressed as mg/l COD (= mg/l O_2)

2. Method

The water sample is oxidized with a hot sulfuric solution of potassium dichromate, with silver sulfate as the catalyst. Chloride is masked with mercury sulfate. The concentration of unconsumed yellow $Cr_2O_7^{2-}$ ions is then determined photometrically.
The method corresponds to DIN ISO 15705 and is analogous to EPA 410.4, APHA 5220 D, and ASTM D1252-06 B.

3. Measuring range and number of determinations

Measuring range	Number of determinations
15 - 300 mg/l COD	25

For programming data for selected photometers / spectrophotometers see www.service-test-kits.com.

4. Applications

This test measures organic and inorganic compounds oxidizable by dichromate. Exceptions: some heterocyclic compounds (e.g. pyridine), quaternary nitrogen compounds, and readily volatile hydrocarbons.

Sample material:
Groundwater and surface water
In-process controls
Wastewater

5. Influence of foreign substances

This was checked in solutions with a COD of 150 mg/l. The determination is not yet interfered with up to the concentrations of foreign substances given in the table.

Concentrations of foreign substances in mg/l or %					
Cl^-	2000	SO_3^{2-}	25	H_2O_2	10
Cr^{3+}	75			$NaNO_3$	10 %
CrO_4^{2-}	5			Na_2SO_4	10 %
NO_2^-	10			Na_3PO_4	10 %

6. Reagents and auxiliaries

Please note the warnings on the packaging materials!
Store the pack protected from light! The test reagents are stable up to the date stated on the pack when stored closed at +15 to +25 °C.

Package contents:
25 reaction cells
1 sheet of round stickers for numbering the cells

Other reagents and accessories:
MQuant™ Chloride Test, Cat. No. 110079, measuring range 500 - ≥3000 mg/l Cl^-
Spectroquant® CombiCheck 60, Cat. No. 114696
COD standard solution CRM, 100 mg/l COD, Cat. No. 125029
COD standard solution CRM, 200 mg/l COD, Cat. No. 125030
Water for chromatography LiChrosolv®, Cat. No. 115333

Pipette for a pipetting volume of 2.0 ml
Thermoreactor

7. Preparation

- Analyze immediately after sampling.
- Homogenize the samples.
- Check the chloride content with the MQuant™ Chloride Test. Samples containing more than 2000 mg/l Cl^- must be diluted with distilled water **prior to** determining the COD.

8. Procedure

Suspend the bottom sediment in the reaction cell by swirling.		
Pretreated sample	2.0 ml	**Carefully** allow to run from the pipette down the inside of the tilted reaction cell onto the reagent (**Wear eye protection! The cell becomes hot!**).
Tightly attach the screw cap to the cell. **In all subsequent steps the cell must be held only by the screw cap!** **Vigorously** mix the contents of the cell. Heat the cell at 148 °C in the preheated thermoreactor for 120 min. Remove the hot cell from the thermoreactor and allow to cool in a test-tube rack. **Do not cool with cold water!** Wait 10 min, swirl the cell, and return to the rack for complete cooling to room temperature (cooling time at least 30 min). Measure in the photometer.		

Notes on the measurement:

- For photometric measurement the cells must be clean. Wipe, if necessary, with a clean dry cloth.
- Measurement of turbid solutions yields false-low readings.
- The measurement value remains stable over a long term.
- **To increase the accuracy is recommended to measure against an own prepared blank sample (reaction cell + COD-free water [1]). Configure the photometer for blank measurement.**

[1] It is recommended to use Water for chromatography LiChrosolv®, Cat. No. 115333.

9. Analytical quality assurance

recommended before each measurement series
To check the photometric measurement system (test reagent, measurement device, handling) and the mode of working, the COD standard solutions CRM, 100 mg/l COD (Cat. No. 125029) and 200 mg/l COD (Cat. No. 125030) or Spectroquant® CombiCheck 60 can be used. Besides a **standard solution** with 250 mg/l COD, CombiCheck 60 also contains an **addition solution** for determining sample-dependent interferences (matrix effects).
Additional notes see under **www.qa-test-kits.com**.

Characteristic quality data:

In the production control, the following data were determined in accordance with ISO 8466-1 and DIN 38402 A51:

Standard deviation of the method (mg/l COD)	± 1.5
Coefficient of variation of the method (%)	± 0.94
Confidence interval (mg/l COD)	± 4
Number of lots	47

Characteristic data of the procedure:

Sensitivity: Absorbance 0.010 A corresponds to (mg/l COD)	2
Accuracy of a measurement value (mg/l COD)	max. ± 8

For quality and batch certificates for test kits see the website.

10. Note

The test reagents must not be run off with the wastewater!
Information on disposal can be obtained at www.disposal-test-kits.com.

Merck KGaA, 64271 Darmstadt, Germany,
Tel. +49(0)6151 72-2440
www.analytical-test-kits.com
EMD Millipore Corporation, 290 Concord Road, Billerica, MA 01821, USA, Tel. +1-978-715-4321

Annex C – Phosphorus Determination Method

1.14546.0001

Spectroquant®

Phosphate Cell Test

P

for the determination of orthophosphate

1. Method

In sulfuric solution orthophosphate ions react with ammonium vanadate and ammonium heptamolybdate to form orange-yellow molybdovanadophosphoric acid that is determined photometrically ("VM" method).
The method is analogous to APHA 4500-P C.

2. Measuring range and number of determinations

Measuring range	Number of determinations
0.5 - 25.0 mg/l PO_4-P	25
1.5 - 76.7 mg/l PO_4^{3-} 1.1 - 57.3 mg/l P_2O_5	

For programming data for selected photometers / spectrophotometers see www.service-test-kits.com.

3. Applications

This test measures only orthophosphate.

Sample material:
Groundwater and surface water, seawater
Wastewater
Industrial water
Boiler water
Nutrient solutions for fertilization
Soils after appropriate sample pretreatment

4. Influence of foreign substances

This was checked in solutions containing 10 and 0 mg/l PO_4-P. The determination is not yet interfered with up to the concentrations of foreign substances given in the table.

Concentrations of foreign substances in mg/l or %							
Ag^+	1000	Cu^{2+}	1000	**Pb^{2+}**	**10**	**EDTA**	**0.2 %**
AsO_4^{2-}	50	**Fe^{3+}**	**10**	**S^{2-}**	**10**	**Hydrazine**	**10**
Ca^{2+}	1000	Hg^{2+}	1000	SCN^-	1000	Na-acetate	10 %
Cd^{2+}	1000	Mg^{2+}	1000	SiO_3^{2-}	500	NaCl	20 %
CN^-	1000	Mn^{2+}	1000	SO_3^{2-}	1000	$NaNO_3$	20 %
Co^{2+}	100	NH_4^+	1000	Zn^{2+}	1000	Na_2SO_4	20 %
Cr^{3+}	50	Ni^{2+}	500				
$Cr_2O_7^{2-}$	**5**	NO_2^-	1000				

5. Reagents and auxiliaries

Please note the warnings on the packaging materials!
The test reagents are stable up to the date stated on the pack when stored closed at +15 to +25 °C.

Package contents:
25 reaction cells
1 sheet of round stickers for numbering the cells

Other reagents and accessories:
MQuant™ Phosphate Test, Cat. No. 110428,
measuring range 10 - 500 mg/l PO_4^{3-} (3.3 - 163 mg/l PO_4-P)
MColorpHast™ Universal indicator strips pH 0 - 14, Cat. No. 109535
Sulfuric acid 0.5 mol/l Titripur®, Cat. No. 109072
Phosphate standard solution Certipur®, 1000 mg/l PO_4^{3-}, Cat. No. 119898
Hydrochloric acid 25 % for analysis EMSURE®, Cat. No. 100316

Pipette for a pipetting volume of 5.0 ml

6. Preparation

- Use only phosphate-free detergents to rinse glassware. Otherwise fill with hydrochloric acid (approx. 10 %) and leave to stand for several hours.
- Analyze immediately after sampling.
- Check the phosphate content with the MQuant™ Phosphate Test. Samples containing more than 25.0 mg/l PO_4-P must be diluted with distilled water.
- **The pH must be within the range 0 - 10.** Adjust, if necessary, with sulfuric acid.
- Filter turbid samples.

7. Procedure

Pretreated sample (10 - 40 °C)	5.0 ml	Pipette into a reaction cell, close the cell tightly, and mix.
Measure the sample in the photometer.		

Notes on the measurement:

- For photometric measurement the cells must be clean. Wipe, if necessary, with a clean dry cloth.
- Measurement of turbid solutions yields false-high readings.
- The pH of the measurement solution must be within the range 0.5 - 1.0.
- The color of the measurement solution remains stable for at least 60 min after the end of the reaction time stated above.

8. Analytical quality assurance

recommended before each measurement series
To check the photometric measurement system (test reagents, measurement device, handling) and the mode of working, a dilute phosphate standard solution containing 15.0 mg/l PO_4-P (46.0 mg/l PO_4^{3-}) can be used.
Sample-dependent interferences (matrix effects) can be determined by means of standard addition.
Additional notes see under **www.qa-test-kits.com**.

Characteristic quality data:

In the production control, the following data were determined in accordance with ISO 8466-1 and DIN 38402 A51:

Standard deviation of the method (mg/l PO_4-P)	±0.10
Coefficient of variation of the method (%)	±0.75
Confidence interval (mg/l PO_4-P)	±0.2
Number of lots	22

Characteristic data of the procedure:

Sensitivity: Absorbance 0.010 A corresponds to (mg/l PO_4-P)	0.1
Accuracy of a measurement value (mg/l PO_4-P)	max. ±0.4

For quality and batch certificates for Spectroquant® test kits see the website.

9. Note

Information on disposal can be obtained at www.disposal-test-kits.com.

Merck KGaA, 64271 Darmstadt, Germany,
Tel. +49(0)6151 72-2440
www.analytical-test-kits.com

EMD Millipore Corporation, 290 Concord Road,
Billerica, MA 01821, USA, Tel. +1-978-715-4321

Annex D – Orthophosphates Determination Method

1.00474.0001
1.00474.0007

Spectroquant®

Phosphate Cell Test

P

for the determination of orthophosphate

1. Method

In sulfuric solution orthophosphate ions react with molybdate ions to form molybdophosphoric acid. Ascorbic acid reduces this to phosphomolybdenum blue (PMB) that is determined photometrically.
The method is analogous to EPA 365.2+3, APHA 4500-P E, and DIN EN ISO 6878.

2. Measuring range and number of determinations

Measuring range	Number of determinations
0.05 - 5.00 mg/l PO_4-P	25
0.2 - 15.3 mg/l PO_4^{3-} 0.11 - 11.46 mg/l P_2O_5	

For programming data for selected photometers / spectrophotometers see www.service-test-kits.com.

3. Applications

This test measures only orthophosphate.

Sample material:
Groundwater and surface water, seawater
Drinking water
Wastewater
Nutrient solutions for fertilization
Soils after appropriate sample pretreatment
Food after appropriate sample pretreatment

4. Influence of foreign substances

This was checked in solutions containing 2 and 0 mg/l PO_4-P. The determination is not yet interfered with up to the concentrations of foreign substances given in the table.

Concentrations of foreign substances in mg/l or %							
Ag^+	1000	Cu^{2+}	250	Ni^{2+}	500	EDTA	1000
AsO_4^{3-}	**0.2**	F^-	50	NO_2^-	1000	Surfactants[1]	100
Ca^{2+}	1000	Fe^{3+}	1000	Pb^{2+}	25	Na-acetate	1 %
Cd^{2+}	1000	**Hg^{2+}**	**10**	**S^{2-}**	**2.5**	NaCl	5 %
CN^-	1000	Mg^{2+}	1000	SiO_3^{2-}	1000	$NaNO_3$	10 %
Cr^{3+}	1000	Mn^{2+}	1000	SO_3^{2-}	1000	Na_2SO_4	10 %
$Cr_2O_7^{2-}$	**5**	NH_4^+	1000	Zn^{2+}	1000		

Reducing agents interfere with the determination.
[1] tested with nonionic, cationic, and anionic surfactants

5. Reagents and auxiliaries

Please note the warnings on the packaging materials!
The test reagents are stable up to the date stated on the pack when stored closed at +15 to +25 °C.

Package contents:
1 bottle of reagent P-1K
1 bottle of reagent P-2K
25 reaction cells
1 blue dose-metering cap
1 sheet of round stickers for numbering the cells

Other reagents and accessories:
MQuant™ Phosphate Test, Cat. No. 110428,
measuring range 10 - 500 mg/l PO_4^{3-} (3.3 - 163 mg/l PO_4-P)
MColorpHast™ Universal indicator strips pH 0 - 14, Cat. No. 109535
Sulfuric acid 0.5 mol/l Titripur®, Cat. No. 109072
Spectroquant® CombiCheck 10, Cat. No. 114676
Hydrochloric acid 25 % for analysis EMSURE®, Cat. No. 100316

Pipette for a pipetting volume of 5.0 ml

6. Preparation

- Use only phosphate-free detergents to rinse glassware. Otherwise fill with hydrochloric acid (approx. 10 %) and leave to stand for several hours.

At the first use **replace the screw cap of the reagent bottle P-2K by the blue dose-metering cap**.
Hold the reagent bottle **vertically** and, at each dosage, press the slide **all the way** into the dose-metering cap. **Before each dosage** ensure that the slide is **completely retracted**.
⚠ **Reclose the reagent bottle with the screw cap at the end of the measurement series, since the function of the reagent is impaired by the absorption of atmospheric moisture.**

- Analyze immediately after sampling.
- Check the phosphate content with the MQuant™ Phosphate Test. Samples containing more than 5.00 mg/l PO_4-P must be diluted with distilled water.
- **The pH must be within the range 0 - 10.** Adjust, if necessary, with sulfuric acid.
- Filter turbid samples.

7. Procedure

Pretreated sample (10 - 35 °C)	5.0 ml	Pipette into a reaction cell and mix.
Reagent P-1K	5 drops[1]	Add, close the cell tightly, and mix.
Reagent P-2K	1 dose	Add, close the cell tightly, and shake **vigorously until the reagent is completely dissolved**.
Leave to stand for 5 min (reaction time), then measure the sample in the photometer.		

[1] **Hold the bottle vertically while adding the reagent!**

Notes on the measurement:
- For photometric measurement the cells must be clean. Wipe, if necessary, with a clean dry cloth.
- Measurement of turbid solutions yields false-high readings.
- The pH of the measurement solution must be within the range 0.80 - 0.95.
- The color of the measurement solution remains stable for at least 60 min after the end of the reaction time stated above.

8. Analytical quality assurance

recommended before each measurement series
To check the photometric measurement system (test reagents, measurement device, handling) and the mode of working, Spectroquant® CombiCheck 10 can be used. Besides a **standard solution** with 0.80 mg/l PO_4-P, CombiCheck 10 also contains an **addition solution** for determining sample-dependent interferences (matrix effects).
Additional notes see under **www.qa-test-kits.com**.

Characteristic quality data:
In the production control, the following data were determined in accordance with ISO 8466-1 and DIN 38402 A51:

Standard deviation of the method (mg/l PO_4-P)	± 0.024
Coefficient of variation of the method (%)	± 0.97
Confidence interval (mg/l PO_4-P)	± 0.06
Number of lots	3

Characteristic data of the procedure:

Sensitivity: Absorbance 0.010 A corresponds to (mg/l PO_4-P)	0.02
Accuracy of a measurement value (mg/l PO_4-P)	max. ± 0.08

For quality and batch certificates for Spectroquant® test kits see the website.

9. Notes

- Reclose the reagent bottles immediately after use.
- **Information on disposal can be obtained at www.disposal-test-kits.com.**

Merck KGaA, 64271 Darmstadt, Germany,
Tel. +49(0)6151 72-2440
www.analytical-test-kits.com

EMD Millipore Corporation, 290 Concord Road,
Billerica, MA 01821, USA, Tel. +1-978-715-4321

Annex E – General Formulas

The formulas 1 and 2, are utilized in the design/verification of several treatment processes.

$$\text{Surface overflow rate } (\mathrm{m^3/m^2.h}) = \frac{\text{Flow rate } (\mathrm{m^3/\,h})}{\text{Surface area } (\mathrm{m^2})} \qquad (1)$$

$$\text{Hydraulic retention Time } (\mathrm{h}) = \frac{\text{Volume } (\mathrm{m^3})}{\text{Flow rate } (\mathrm{m^3/\,h})} \qquad (2)$$

Annex F – Espinho WWTP Solar Radiation

Caution: Photovoltaic system performance predictions calculated by PVWatts® include many inherent assumptions and uncertainties and do not reflect variations between PV technologies nor site-specific characteristics except as represented by PVWatts® inputs. For example, PV modules with better performance are not differentiated within PVWatts® from lesser performing modules. Both NREL and private companies provide more sophisticated PV modeling tools (such as the System Advisor Model at https://sam.nrel.gov) that allow for more precise and complex modeling of PV systems.

The expected range is based on 30 years of actual weather data at the given location and is intended to provide an indication of the variation you might see. For more information, please refer to this NREL report: The Error Report.

The energy output range is based on analysis of 30 years of historical weather data for nearby , and is intended to provide an indication of the possible interannual variability in generation for a Fixed (open rack) PV system at this location.

RESULTS

434,959 kWh/Year*

Month	Solar Radiation (kWh / m^2 / day)	AC Energy (kWh)	Value ($)
January	2.41	19,189	N/A
February	3.41	24,547	N/A
March	4.89	38,543	N/A
April	6.19	46,537	N/A
May	6.60	51,087	N/A
June	7.13	50,950	N/A
July	6.84	51,359	N/A
August	6.59	49,104	N/A
September	5.30	38,253	N/A
October	4.01	31,004	N/A
November	2.28	17,509	N/A
December	2.14	16,878	N/A
Annual	**4.82**	**434,960**	**0**

Location and Station Identification

Requested Location	ETAR de Espinho
Weather Data Source	(INTL) PORTO, PORTUGAL 18 mi
Latitude	41.23° N
Longitude	8.68° W

PV System Specifications *(Commercial)*

DC System Size	315 kW
Module Type	Standard
Array Type	Fixed (open rack)
Array Tilt	20°
Array Azimuth	180°
System Losses	14.08%
Inverter Efficiency	96%
DC to AC Size Ratio	1.2

Economics

Average Retail Electricity Rate	No utility data available

Performance Metrics

Capacity Factor	15.8%

Annex G – Sludge Pumping Costs

Sludge pumping costs in Espinho WWTP with conventional primary treatment.

Parameters	Units	Scenario 1	Scenario 2	Scenario 3
Primary sludge pumping	kW	7.6	7.6	7.6
	m3/h	60	60	60
	h/d	1.9	2.6	2.7
	€/y	587	790	827
Excess WAS pumping	kW	2	2	2
	m3/h	66	66	66
	h/d	18.8	24.9	21.8
	€/y	1,512	2,002	1,748
Thickened primary sludge pumping	kW	2.2	2.2	2.2
	m3/h	13.6	13.6	13.6
	h/d	4.8	6.4	6.7
	€/y	422	568	594
Thickened mixed sludge pumping	kW	2.68	2.68	2.68
	m3/h	16.8	16.8	16.8
	h/d	8.9	11.8	11.2
	€/y	953	1,271	1,207
Digested sludge pumping	kW	2.2	2.2	2.2
	m3/h	11.6	11.6	11.6
	h	12.8	17.1	16.2
	€/y	1,133.9	1,511.9	1,435.3
AD sludge recirculation heating pumps	kW	30	30	30
	m3/h	314	314	314
	h/d	22.0	22.0	22.0
	€/y	26,499	26,499	26,499
Dewatered sludge pumping	kW	4.7	4.7	4.7
	m3/h	3.5	3.5	3.5
	h/d	5.0	6.6	6.3
	€/y	938	1,250	1,187
TOTAL	€/y	32,047	33,895	33,499

Sludge pumping costs in Espinho WWTP with CEPT.

Parameters	Units	Scenario 1	Scenario 2	Scenario 3
Primary sludge pumping	kW	7.6	7.6	7.6
	m3/h	60	60	60
	h/d	5.4	7.1	7.7
	€/y	1,651	2,181	2,350
Excess WAS pumping	kW	2.0	2.0	2.0
	m3/h	66	66	66
	h/d	14.2	15.0	16.0
	€/y	1,142	1,207	1,287
Thickened primary sludge pumping	kW	2.2	2.2	2.2
	m3/h	14	14	14
	h/d	8.1	10.6	11.5
	€/y	712	940	1,013
Thickened mixed sludge pumping	kW	2.7	2.7	2.7
	m3/h	17	17	17
	h/d	9.0	11.9	12.1
	€/y	972	1,284	1,304
Digested sludge pumping	kW	2.2	2.2	2.2
	m3/h	12	12	12
	h/d	13.1	17.3	17.6
	€/y	1,156	1,527	1,550
AD sludge recirculation heating pumps	kW	30.0	30.0	30.0
	m3/h	314	314	314
	h/d	22.0	22.0	22.0
	€/y	26,499	26,499	26,499
Dewatered sludge pumping	kW	4.7	4.7	4.7
	m3/h	4	4	4
	h/d	5.1	6.7	6.8
	€/y	956	1,263	1,283
TOTAL	€/y	33,089	34,902	35,286

Sludge pumping costs in Espinho WWTP with CEPT and AcoD being performed.

Parameters	Units	Scenario 1	Scenario 2	Scenario 3
Primary sludge pumping	kW	7.6	7.6	7.6
	m3/h	60	60	60
	h/d	5.4	7.1	7.7
	€/y	1,651	2,181	2,350
Excess WAS pumping	kW	2.0	2.0	2.0
	m3/h	66	66	66
	h/d	14.2	15.0	16.0
	€/y	1,142	1,207	1,287
Thickened primary sludge pumping	kW	2.2	2.2	2.2
	m3/h	14	14	14
	h/d	8.1	10.6	11.5
	€/y	712	940	1,013
Thickened mixed sludge pumping	kW	2.7	2.7	2.7
	m3/h	17	17	17
	h/d	9.0	11.9	12.1
	€/y	972	1,284	1,304
Digested sludge pumping	kW	2.2	2.2	2.2
	m3/h	12	12	12
	h/d	28.6	28.9	28.8
	€/y	2,527	2,555	2,540
AD sludge recirculation heating pumps	kW	30.0	30.0	30.0
	m3/h	314	314	314
	h/d	22.0	22.0	22.0
	€/y	26,499	26,499	26,499
Dewatered sludge pumping	kW	4.7	4.7	4.7
	m3/h	4	4	4
	h/d	13.0	12.6	12.5
	€/y	2,449	2,383	2,360
TOTAL	€/y	35,952	37,049	37,354

Annex H – Reclaimed Water Pipeline

Reclaimed water pipeline (in white) from the WWTP to the Golf course

Reclaimed water pipeline characteristics.

Parameter	Unit	Value
k_s	$m^{(1/3)}/s$	110
Liquid specific weight	kN/m^3	9,810
Pipeline length	m	1,841
Pumping station flow rate	L/s	11.57
Geometric height	m	8.00

Reclaimed water pipeline design characteristics.

Parameter	Unit	Value
Material	-	PEAD
PN (Pressure Nominal)	-	10
DN (Diameter Nominal)	mm	125

Parameter	Unit	Value
ID (Interior Diameter)	mm	110
Water velocity	m/s	1.21
Pressure loss (J)	m/m	0.02
Pressure loss along the pipeline	m	30.96
Total pressure loss	m	38.96

Reclaimed water pipeline investment costs.

Article	Designation	Price
1	**GENERAL COMPLEMENTING WORK**	
TOTAL 1	GENERAL COMPLEMENTING WORK	7,300.00 €
2	**PIPELINE**	
2.1-TOTAL	SOIL EXCAVATION AND EMBANKMENT	25,529.49 €
2.2-TOTAL	PIPE AND ACESSORIES	27,615.00 €
2.3 - TOTAL	PAVEMENT	19,560.63 €
2.4 - TOTAL	COMPLEMENTARY WORK	3,500.00 €
TOTAL 2	PIPELINE	76,205.12 €
TOTAL		83,505.12 €