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Membrane Bioreactors in Wastewater Treatment: Future is Now

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Declaration

I declare that this document is an original work of my own authorship and that it fulfills all the requirements of the Code of Conduct and Good Practices of the Universidade de Lisboa.

Resumo

A geração de um ciclo de água sustentável baseado na reutilização de águas residuais tratadas é essencial a longo prazo para resolver o estresse dos recursos de água disponíveis e evitar o aumento da contaminação do planeta. Muitos tratamentos avançados foram desenvolvidos durante as últimas décadas para melhorar a qualidade do efluente oferecido pelos processos tradicionais em estações de tratamento de águas residuais (ETAR), a fim de alcançar os mínimos de reutilização de efluentes tratados exigidos pela regulamentação atual. Como consequência, o design de tratamento avançado provavelmente se tornou o aspecto mais desafiador em uma ETAR.

Os biorreatores de membrana (MBR) estão ganhando cada vez mais atenção dia após dia. Capazes de substituir toda uma linha de tratamentos secundários e terciários, esses sistemas compactos obtêm excelentes efluentes prontos para serem reutilizados na maioria dos usos catalogados, sendo especialmente interessantes quando o espaço é um fator limitante e para melhorias graças à sua adaptabilidade aos processos convencionais de lamas ativadas. A expansão do mercado desta tecnologia nos últimos anos induziu uma redução nos custos de capital e operacionais, superando uma das principais desvantagens clássicas da MBR e tornando-se uma alternativa atraente em termos econômicos quando se deseja efluentes de alta qualidade. Critérios operacionais fundamentais são revisados a fim de alcançar um desempenho ótimo do sistema, dando atenção especial às estratégias de manutenção, controle e prevenção de entupimentos. Finalmente, um modelo foi desenvolvido para projetar uma planta MBR compacta com base em todas as informações apresentadas e discutidas durante esta dissertaçao.

Keywords

MBR, filtraçao por membrana, lamas ctivadas, reciclar, ETAR.

Abstract

Generating a sustainable water cycle based in reusing treated wastewater is essential in the long term to solve the stress of the available freshwater resources and avoid increasing the planet contamination. Many advanced treatments have been developed during the last decades to improve the effluent quality offered by the traditional processes in wastewater treatment plants (WWTP) in order to achieve the treated wastewater reuse minimums demanded by the current regulation. As a consequence, advanced treatment design has probably become the most challenging aspect in a WWTP.

Membrane bioreactors (MBR), an advanced treatment which combines the activated sludge process with the membrane filtration, are gaining more attention day by day. Able to substitute an entire line of secondary and tertiary treatments, these compact systems obtain excellent effluents ready to be reused in most of the cataloged uses, being especially interesting when space is a limiting factor and for upgradings thanks to their adaptability to conventional activated sludge processes. The market expansion of this technology during the last years induced a reduction in membrane capital costs and an energy demand optimization search, overcoming one of the MBR major classic drawbacks and becoming an attractive alternative in economical terms when high quality effluents are desired. Fundamental operational criteria are reviewed in order to achieve an optimal system performance, giving special attention to fouling maintenance, control and prevention strategies. Finally, a model has been developed to design a compact MBR plant based on all the information presented and discussed during this thesis.

Keywords

MBR, Membrane filtration, Activated sludge, Recycle, WWTP.

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Acronyms and Symbols

Acronyms

AEC-MBR – Airlift MBR

BOD – Biochemical oxygen demand

CAPEX – Capital expenditures

CAS – Conventional activated sludge process

CEB – Chemically enhanced backwash

CEC – Contaminants of emerging concern

CFU – Colony-forming unit

CIP – Clean-in-place

COD – Chemical oxygen demand

COP – Clean-out-place

CW – Chemical washing

DO – Dissolved oxygen

EDC – Endocrine-disrupting chemicals

EPA – Environmental Protection Agency

EPS – Extracellular polymeric substances

F/M – Food to microorganisms ratio

GAC – Granular activated carbon

HDPE – High-density polyethylene

HRT – Hydraulic retention time

MBR – Membrane bioreactor

MF – Microfiltration

MLE – Modified Ludzack-Ettinger

MLSS – Mixed liquor suspended solids

NF – Nanofiltration

NTU – Nephelometric turbidity unit

OPEX – Operational expenditures

PAC – Powdered activated carbon

PAO – Phosphate accumulating organisms

PES – Polyethersulfone

PE – Equivalent persons

POP – Persistent organic pollutants

PPCP – Pharmaceuticals and personal care products

PVDF – Polyvinylidene fluoride

QS – Quorum sensing

RO – Reverse osmosis

SAD_p – Specific aeration demand per unit of permeate produced

SRT – Solid retention time

SS – Suspended solids

TDS – Total dissolved solids

TMP – Transmembrane pressure

TOC – Total organic carbon

TOD – Total oxygen demand

TSS – Total suspended solids

TSS – Total suspended solids

UF – Ultrafiltration

USGS – United States Geological Survey

UV – Ultraviolet

VOC – Volatile organic compounds

VSS – Volatile suspended solids

WHO – World Health Organization

wt% - Weight percentage

WWTP – Wastewater treatment plant

Symbols

$Ca(OCl)_2$ – Calcium hypochlorite

Cl_2 – Chlorine

ClO_2 – Chlorine dioxide

CO_2H – Carboxylic acid

Hg – Mercury

HO_2 – Hydrogen peroxy

NaCOl – Sodium hypochlorite

N – Total Nitrogen

O_2 – Oxygen

O_3 – Ozone

OH – Hydroxyl

pH – Hydrogen potential

P – Total Phosphorus

$–SO_3$ – Sulfur trioxide

α – Rate of oxygen transfer in clean water divided by the rate of mixed liquor

Chapter 1

The Wastewater Importance

Traditionally water had been mistakenly considered as an unlimited resource. Water covers 70,8% from the Earth's surface, from that only 2,5% is considered to be freshwater. Taking into account that most of this freshwater is located in glaciers and the ice caps of Antarctica and Greenland, water available for human consumption is reduced to 0,5% of the total water.

Our society has undergone a great development at all levels, allowing a global population grown from approximately 1,65 billion at the beginning of the XX century to more than 7,5 billion nowadays. In 2050, world's population is expected to be increased over 11 billion. As a consequence, water consumption is considered to be six times higher nowadays than at the beginning of the XX century. Moreover, our social model achieved a big evolution as well with several impacts on global environment because of contamination. Water contamination levels in most rivers of the planet continue getting worse as well with bigger seas or oceans deoxygenated death zones due to untreated or insufficiently treated wastewater discharges, affecting ecosystems with the consequent repercussions in the food chains or the fishing industry. Therefore, potable water demand is constantly growing while available freshwater resources are more stressed and limited day by day because of higher demand, contamination and the climate change.

Wastewaters had been viewed as a complication to be discharged into the environment or even ignored during the human history. During the last century an increasing importance to treating wastewaters before discharging them into the media was given in order to stop expanding contamination in the water cycle. For this reason, wastewater began being recollected and treated in wastewater treatment plants (WWTP), where the influent was usually submitted at what is known as preliminary, primary and secondary treatments before being discharged into the media. This conception has continued evolving due to an increasing lack of available freshwater resources in a lot of regions, recognizing the importance of reusing treated wastewater to solve the stress of the freshwater resources as well with a sustainable water cycle generation in the long term. Despite contamination is importantly reduced after a secondary treatment, the presence of many dangerous contaminants for the human health has been detected in those treated wastewaters. Therefore, it is necessary to add what are known as tertiary or advanced treatments in WWTP to achieve better effluents able to be reused for human purposes.

Nowadays wastewater treatment emphasis is focused, apart from constructing new WWTP to continue increasing rates of treated wastewater, on upgrading existing WWTP to provide advanced treatments that allow reusing more treated wastewater day by day. As a consequence of appearing many tertiary system alternatives and combinations to achieve different effluent levels required by the current legislation, advanced treatment design is probably the most challenging aspect in a WWTP. It is fundamental to know the incoming wastewater characteristics and the desired treated wastewater quality. Limitations in capital and operational costs must be taken into account, as well with the available physical space for the WWTP construction or expansion, the compatibility with existing facilities if an upgrading is being carried out, the equipment availability, the personnel or the energy and resource requirements. Conventional advanced treatments are presented and analyzed in those terms, also presenting conventional combinations between them used to achieve different treated wastewater quality levels.

Membrane bioreactors (MBR) are a less conventional advanced treatment that combines the biological process done in a conventional secondary treatment with membrane filtration to achieve great quality effluents. These systems are gaining more attention day by day because of their maturity, which has provided them performance reliability as well with an increased market competence between manufacturers that has produced an attractive drop in capital costs, one of their main disadvantages with respect to other alternatives. MBR systems are examined in detail, first presenting and discussing all the conditions in which their operation is related in order to achieve an optimal performance, and secondly providing information to the lector about when it may be interesting to use MBR over other advanced treatment alternatives.

During chapter 2 conventional contaminants present in wastewater are presented and explained to describe which kind of influents have to face a WWTP. Chapter 3 presents the wastewater treatment objectives and regulations nowadays in terms of discharge and of reuse, also analyzing future trends in those terms. Once the contaminants and regulations are presented, the conventional secondary and tertiary or advanced treatments are explained during chapters 4 and 5, respectively, to understand how a WWTP works to achieve those quality objectives. After that, membrane bioreactors (MBR) are presented and explained in detail during chapter 6, beginning with the membrane filtration process, how and where the membranes are configured, system operating conditions (with a special focus on fouling phenomena), performance, capital and operational costs and a system design model. Conclusions extracted from the development of this thesis are finally presented.

Chapter 2

Wastewater Contaminants

It is necessary to understand what characterizes the wastewater before applying the right treatment. Any water coming from the sewage network is considered wastewater. Potable water is used in a concrete way that produces physical, biological and chemical variations and, after that, is poured to the sewage network and driven to a wastewater treatment plant (WWTP). Then the wastewater is treated in order to reduce its contamination enough to achieve the legal legislative limits to return into to the environment.

Wastewater could be classified as: domestic wastewater (coming from residential or commercial areas, institutional and recreational facilities), industrial wastewater (used for any industrial activity) or municipal wastewater (domestic wastewater mixed with rain water or industrial wastewater).

Therefore, each wastewater composition may be completely different, as a wastewater flow could include domestic, industrial, municipal or even uncontrolled wastewater spills. Its composition depends on the area and year time. Domestic wastewater, for example, varies depending on the season and the day moment, while industrial wastewater has big variations depending on used processes and materials.

2.1. Wastewater Components

In order to apply the right wastewater treatment is mandatory knowing its physical, biological and chemical characteristics, determining its composition. It is important to firstly distinguish between organic and inorganic matter contained in the wastewater.

Organic Matter

This matter comes from animals, vegetables and human activities related with organic compounds formed by carbon, hydrogen, oxygen, and sometimes nitrogen, sulfur, phosphorus or iron combinations. The presence of organic matter in the wastewater negatively affects its oxygen quantity, as the organic compounds soluble in water with low molecular weights are biodegradable under oxygen consuming bacteria or fungi action. As more complex the organic molecules are, less soluble in water, being less biodegradable.

There are different parameters used to measure the organic matter quantity a wastewater has:

- Biochemical oxygen demand (BOD): Measures the oxygen amount necessary to oxidize the organic matter as a result of the aerobic biochemical oxidation action.

- Chemical oxygen demand (COD): Measures the organic compounds contained in the wastewater, being biodegradable or not, using the necessary oxygen to oxidize the organic matter introducing a strongly oxidizing chemical agent in acid media.

- Total oxygen demand (TOD): Measures the total oxygen amount necessary to oxidize all the oxidizable matter contained in the wastewater.

- Total organic carbon (TOC): Measures the carbon oxidation present in the organic matter into carbon dioxide. There are organic compounds that do not oxidize, therefore obtained TOC values will be less than real ones.

Inorganic Matter

Inorganic compounds are important in order to determine and control water quality, as they can affect very much at water uses. Their concentration in natural water could grow because of rock or mineral contact (dissolving them into the water) or because of a wastewater discharge, as conventional wastewater treatments are focused on eliminating organic matter.

Inorganic matter is usually more present in industrial than in domestic wastewaters. Domestic wastewater presents equilibrium between organic and inorganic compounds, as organic biodegradation requires nitrogen, phosphorus, iron and salt traces, all of them inorganic compounds.

Physical Characteristics

Table 2.1 Wastewater physical characteristics (Metcalf & Eddy, 1995).

Characteristic	Definition
Color	Wastewater color varies depending on the time the water is circulating through the sewage network, being darker as more time the wastewater has been circulating.
Odor	Odors appear when gases are released during organic matter decomposition processes. Industrial wastewaters could contain odorous products by themselves or products that may produce odors during treatments.

Solids	Total solids are the obtained matter after submitting wastewater an evaporation process between 103 and 105ºC. They can be found as: • Suspended Solids (SS). • Settleable Solids: Solids that precipitate and settle. Allow knowing the sludge that will be obtained by primary decantation. • Colloidal matter: Filterable particles, seized between 0,001 and 1 µm. • Dissolved matter: Formed by organic, inorganic molecules and dissolved ions in water.
Temperature	Important parameter because of its influence on aquatic life development, being the oxygen less soluble in hot than in cold water. Dissolved oxygen decrease produces the death of the aquatic ecosystem. Moreover, too high temperatures produce aquatic plants and fungi massive growth.
Turbidity	Parameter used to measure the light transmission capacity through the wastewater, showing its quality related with its suspended matter amount.

Organic Chemical Constituents

Table 2.2 Wastewater organic chemical constituents (Metcalf & Eddy, 1995).

Characteristic	Definition
Carbohydrates	Some of them are water soluble, like sugars, and others not, like starches. Cellulose is easily decomposed.
Fats, oils and grease	Formed by carbon, hydrogen and oxygen, they are one of the organic compounds hardest to be decomposed by bacteria. Oils usually float in the wastewater.
Pesticides	They are not common from wastewaters, usually appearing from water canals coming from agricultural uses. Toxic for the majority of life forms, they are dangerous contaminants, a lot of them considered as priority.
Proteins	Principal animal organisms compound, less present in vegetables. Containing carbon, hydrogen, oxygen, nitrogen and sometimes sulfur, phosphorus or iron, they have a complex and unstable chemical composition that makes only some of them water soluble. Wastewater with a high proteins presence issues strong odors due to decomposition processes.
Priority contaminants	EPA establishes pouring limits for this list of 129 contaminants, most of them volatile organic compounds (VOC). They can be eliminated, transformed or transported to the WWTP by processes of volatilization, degradation and adsorption in particles or sludge, among others.
Surfactants	Formed by big sized molecules low soluble in water that make squids appear in WWTP.

Volatile Organic Compounds (VOC)	Those organic compounds with a boiling point under 100ºC or a vapor pressure higher than 1 mm Hg at 25ºC. They are very important due their dangerousness to human health, as once they are in a gaseous state they have much more mobility and releasing them into the atmosphere becomes easier.

Inorganic Chemical Constituents

Table 2.3 Wastewater inorganic chemical constituents (Metcalf & Eddy, 1995).

Characteristic	Definition
Alkalinity	Hydroxides, carbonates and calcium, sodium, magnesium, potassium or ammonia bicarbonates are causes of alkalinity, which helps regulating pH changes produced by adding acids to the wastewater.
Chlorides	Conventional wastewater treatments do not remove large amount of chlorides.
Heavy metals	Usually coming from industrial or commercial activities, they may have to be removed if the wastewater is to be reused. Lead, cadmium, chromium and mercury are considered priority contaminants, as nickel, manganese, iron, zinc and copper are highlighted as well.
Nitrogen	Basic element for proteins synthesizing. It is mandatory knowing its wastewater quantity in order to know if it is possible treating it by biological processes (if there is not enough nitrogen, it must be added). Being an element that favors biological growth, it has to be reduced before discharging the wastewater to the environment in order to avoid massive plants growth.
pH	Hydrogen ion dissolved in water concentration value. Range of concentrations where it can be biological life is narrow, being harder to proceed with the biological treatment if you have a wastewater out of this range. It has to achieve pH values between 6 and 9. If pH value is less than 6, wastewater tends to be corrosive as it has excessive hydrogen ions, whereas if pH value is higher than 9, some metal ions precipitate as carbonates and hydroxides.
Phosphorus	Element that helps plants growth in water, therefore there is interest in controlling its amount entering to surface waters once the wastewater is discharge into the environment.
Sulfur	Sulfate ion is naturally found both in network waters and in wastewaters. During proteins synthesizing process it is reduced to sulfur and sulfur hydrogen under the bacteria action in anaerobic conditions. Sulfur hydrogen could be accumulated on pipes, suffering from biological oxidation that makes it becoming sulfuric acid that corrodes the pipes. Sludge digesters reduce sulfates to sulfurs, which can alter biological processes if its concentration is higher than 200 mg/l.

Gaseous Chemical Constituents

Table 2.4 Wastewater gaseous chemical constituents (Metcalf & Eddy, 1995).

Characteristic	Definition
Hydrogen sulfur	Uncolored, flammable and bad smelling gas, formed during containing sulfur organic matter decomposition processes.
Methane	Flammable, uncolored and odorless high energy valuable hydrocarbon, not found in large quantities in wastewaters.
Dissolved oxygen	Necessary for the aerobic microorganisms to breathe. In the same way, it is very important for aquatic life development. EPA established 5 mg/l as the DO minimum level in rivers.

Biological Constituents

Table 2.5. Wastewater biological constituents (Metcalf & Eddy, 1995).

Characteristic	Definition
Bacteria	Most common bacteria in wastewater are Escherichia Coli (E. Coli), found in human fecal matter. Bacteria are necessary during organic matter decomposition and stabilization processes.
Animals and plants	Worms and crustaceans must be specially taken into account as some of them are parasitic species and can be dangerous for human health.
Algae	Water containing high nitrogen values produces their fast growth. They cause bad smell and taste. They also consume dissolved oxygen.
Fungi	They are necessary to avoid organic matter accumulation, as they feed on death organic matter. Fungi can grow and be developed in low humidity or pH areas.
Protozoa	Feeding on bacteria and other microorganisms, they are important for the biological wastewater treatments and for rivers purification, as they can maintain the natural equilibrium between different microorganisms. There are some of them which are pathogen and cause infections, as the *Giarda lamblia* or the *Cryptosporidium*.
Viruses	Parasitic particles that invade cellules from a living body and they reproduce themselves. Ejected viruses with the human fecal matter could usually produce diarrhea.

2.2. Wastewater Composition

When we are talking about wastewater composition, we refer to a wastewater amount of the physical, chemical or biological constituents that have been previously explained. Wastewaters can be classified in high, medium or low concentrated depending on the quantity they have of this constituents. A table is shown below with values of these quantities a domestic wastewater could have. It can be used as a guide to evaluate an incoming wastewater.

Table 2.6 Typical domestic wastewater compositions (Metcalf & Eddy, 1995).

Contaminant	Concentration		
	Low	Medium	High
Total Solids (TS) [mg/L]	350	720	1200
Total Dissolved Solids (TDS)	250	500	850
Fixed	145	300	525
Volatile	105	200	325
Suspended Solids (SS)	100	220	350
Fixed	20	55	75
Volatile	80	165	275
Settleable Solids [mg/L]	5	10	20
Biochemical Oxygen Demand (BOD) at 20ºC [mg/L]	110	220	400
Total Organic Carbon (TOC) [mg/L]	80	160	290
Chemical Oxygen Demand (COD) [mg/L]	250	500	1000
Total Nitrogen [mg/L]	20	40	85
Organic	8	15	35
Free Ammonia	12	25	50
Total Phosphorus [mg/L]	4	8	15
Organic	1	3	5
Inorganic	3	5	10
Chlorides [mg/L]	30	50	100
Sulfate [mg/L]	20	30	50
Alkalinity [mg/L]	50	100	200
Grease [mg/L]	50	100	150
Total Coliform [nº/100mL]	10^6-10^7	10^7-10^8	10^8-10^9
Volatile Organic Compounds (VOC) [µg/L]	<100	100-400	>400

2.3. Contaminants of Emerging Concern (CEC)

In recent years, the presence of a group of contaminants, termed contaminants of emerging concern (CEC), have been recognized as significant water pollutants that have adverse effects on human and wildlife endocrine systems. The United States Geological Survey (USGS) provides a useful definition of CEC: “Any synthetic or naturally occurring chemical or any microorganism that is not commonly monitored in the environment but has the potential to enter the environment and cause known or suspected adverse ecological and/or human health effects”.

In some cases, release of emerging chemical or microbial contaminants to the environment has been probably occurring for a long time, but may not have been recognized until new detection methods were developed. In other cases, synthesis of new chemicals or changes in use and disposal of existing chemicals can create new sources of emerging contaminants. These contaminants are normally found at trace levels in a range of ng/L or µg/L, and unlike the priority contaminants, they are not regulated at current environmental regulations as in general terms it is not well-known how far their possible ecotoxicological effects can go.

CEC include several types of chemicals (EPA, 2008):

- Persistent organic pollutants (POP): Such as polybrominated diphenyl ethers (PBDE) used in flame retardants, furniture foam, plastics, etc., and other global organic contaminants such as perfluorinated organic acids.

- Pharmaceuticals and personal care products (PPCP): including a wide suite of human prescribed drugs (antidepressants, blood pressure, etc.), over-the-counter medications (such as ibuprofen), bactericides, sunscreens, or synthetic musks.

- Veterinary medicines: Such as antimicrobials, antibiotics, anti-fungals, growth promoters or hormones.

- Endocrine-disrupting chemicals (EDC): Including synthetic estrogens (such as 17β ethynylestradiol, which also is a PCPP) or androgens (such as trenbolone, a veterinary drug), naturally occurring estrogens (such as 17β-estradiol, testosterone), as well as many others (organochlorine pesticides, alkylphenols, etc.) capable of modulating normal hormonal functions and steroidal synthesis in aquatic organisms.

- Nanomaterials: Such as carbon nanotubes or nano-scale particulate titanium dioxide, of which little is known about either their environmental fate or effects.

Because CEC are found in many of the products we use every day, we are continuously releasing them into the environment where they can accumulate over time, as many CEC have chemical properties that make them resistant to natural environmental degradation processes. Conventional WWTP typically remove organics and pathogens, not being designed to all CEC removal. Therefore, some CEC can accumulate and persist in the environment, potentially causing adverse effects in the long term. However, some studies have shown that using additional advanced treatments, WWTP can substantially reduce the concentration of these contaminants in wastewaters.

Chapter 3

Wastewater Treatment Objectives and Regulations

In their beginnings, wastewater treatment objectives were concerned about removal of suspended solids and floatable material, treatment of biodegradable organics and elimination of pathogenic organisms. Unfortunately, these objectives were not met uniformly, as some wastewater was being discharged into the environment only partially treated or directly untreated.

During the decades of 70s and 80s, wastewater treatment objectives continue, on the one hand, focused on treating more of the wastewater production by ensuring BOD, SS and pathogenic organisms reduction but at higher level. On the other hand, nutrients such as nitrogen and phosphorus also began to be removed. The results of these efforts were a significant surface water quality improvement, understanding better the environmental effects caused by wastewater discharges.

From 90s on, apart from continue improving the results on previous objectives, wastewater treatment began to focus on the health concerns related to toxic and potentially toxic chemicals released to the environment, not only to continue getting a better surface water quality and protect the aquatic life, but also to regenerate wastewater with the proper conditions for being reused. As a consequence, while the early treatment objectives remain valid nowadays, treatment objectives and goals are continuously being added.

3.1. Current Treated Wastewater Discharge Regulation

In 1991, European Economic Community developed its 91/271/CEE directive where wastewater collection, treatment and discharge where regulated. In 2000, European Union commission established its 2000/60/EC directive, based on 91/271/CEE directive, where current requirements for contaminant levels in treated wastewaters where settled down in order to protect the environment from pollution generated by insufficiently treated wastewater discharges.

Table 3.1 Requirements for discharges from urban wastewater treatment plants (2000/60/EC directive).

Parameters	Concentration	Minimum percentage of reduction[1]
BOD[2]	25 mg/L O_2	70-90
COD	125 mg/L O_2	75
TSS	35 mg/L[3]	90[3]

1 Reduction in relation to the load of the influent.

2 BOD_5 at 20ºC without nitrification. The parameter can be replaced by TOC or TOD parameters if a relationship can be stablished between BOD_5 and the substitute.

3 This requirement is optional. 35 mg/L and 90% percentage (more than 10.000 p.e.); 60 mg/L and 70% percentage reduction (2.000-10.000 p.e.).

Table 3.2 Requirements for discharges from urban wastewater treatment plants to sensitive areas which are subject ti eutrophication (2000/60/EC directive).

Parameters	Concentration	Minimum percentage of reduction[1]
P	2 mg/L P (10.000 p.e. – 100.000 p.e.) 1 mg/L P (> 100.000 p.e.)	80
N	15 mg/L N (10.000 p.e. – 100.000 p.e.) 10 mg/L N (> 100.000 p.e.)[2]	70-80

1 Reduction in relation to the load of the influent.

2 Alternatively, the daily average must not exceed 20 mg/L N. This requirement refers to a water temperature of 12ºC or more during the biological reactor operation of the WWTP. As a substitute for the condition concerning the temperature, it is possible to apply a limited time of operation, which takes into account the regional climate conditions.

Wastewater discharging regulation is mainly focused on achieving two classical wastewater treatment objectives: suspended solids removal, indicated by the TSS concentration or reduction percentage, and treatment of biodegradable organic matter, indicated by the BOD and COD concentrations or reduction percentages. Traditionally suspended solids are removed in wastewater treatment plants (WWTP) by both preliminary and primary treatments. After that, a secondary treatment is introduced, where the organic matter biodegradation is produced by what is known as the activated sludge process. After those treatments, the effluent is ready to be discharged into the environment fulfilling the legislative criteria. Special discharge requirements are necessary to accomplish if the WWTP discharge point is a sensitive area subjected to eutrophication. This phenomenon implies an excessive nutrient contribution from the wastewater discharged into an environment where those nutrients may produce an uncontrolled massive plants growth, alternating the ecosystem natural balance of the area. Conventional secondary effluent does not achieve either nitrogen or phosphorus concentrations or percentage reductions demanded by the European regulation. Therefore, it is necessary to introduce advanced or tertiary treatment systems based on biological or chemical nutrients removal.

3.2. Current Treated Wastewater Reuse Regulation

European Union does not have a reused wastewater legislative framework at the moment. However, several European countries which experience at least temporary water stress conditions (mostly from the Mediterranean region) have developed their own regulation trying to reach a sustainable solution by reusing wastewater. Spain, the European country where more wastewater is reused, has a restrictive contaminant levels current regulation (RD 1600/2007) that has been taken as a guideline.

Table 3.3 Spanish wastewater reuses quality criteria (RD 1600/2007).

Intended Water Use	Maximum Admissible Values				
	Intestinal Nematodes	**Escherichia Coli**	**Suspended Solids**	**Turbidity**	**Other Criteria[1]**
1. Urban Uses					
1.1. Quality: Residential[2] a) Watering private gardens[3]. b) Sanity devices discharge[3].	1 egg/10 L	0 CFU/100 mL	10 mg/L	2 NTU	*Legionella spp.* 100 CFU/L (if exists aerosolization risk)
1.2. Quality: Services a) Watering green urban areas (parks, sport fields or similar)[4]. b) Street washing[4]. c) Fire protection systems[4]. d) Industrial vehicles washing[4].	1 egg/10 L	200 CFU/100 mL	20 mg/L	10 NTU	
2. Agricultural Uses[5]					
2.1. Quality[4] a) Crops watering with a water application system that allows regenerated water direct contact with edible parts for fresh human consumption.	1 egg/10 L	100 CFU/100 mL[6]	20 mg/L	10 NTU	*Legionella spp.* 1.000 CFU/L (if exists aerosolization risk)
2.2. Quality a) Crops watering with a water application system that allows regenerated water direct contact with edible parts, but human consumption is not fresh but after an industrial process. b) Producing for consumption meat or milk animals pastures watering. c) Aquaculture.	1 egg/10 L	1.000 CFU/100 mL[6]	35 mg/L	No limit is set	*Taenia saginata* and *Taenia solium*: 1 egg/L (if producing meat animal pastures are watered)
2.3. Quality a) Woody crops located watering that prevents the regenerated water contact with consumed fruits by humans. b) Ornamental flower or greenhouses crops watering without contact of regenerated water with final production. c) Non-industrial, silage fodder, cereal or oilseed crops watering.	1 egg/10 L	10.000 CFU/100 mL	35 mg/L	No limit is set	*Legionella spp.* 100 CFU/L

3. Industrial Uses					
3.1. Quality[4] a) Cleaning and process waters, excepting the food industry. b) Another industrial uses.	No limit is set	10.000 CFU/100 mL	35 mg/L	15 NTU	*Legionella spp.* 100 CFU/L
c) Cleaning and process waters for food industry use.	1 egg/10 L	1.000 CFU/100 mL[6]	35 mg/L	No limit is set	*Legionella spp.* 100 CFU/L
3.2. Quality a) Cooling towers and evaporative condensers.	1 egg/10 L	0 CFU/100 mL	5 mg/L	1 NTU	*Legionella spp.* 0 CFU/L It will be required for its authorization: - The approval, by the health authority, of the 4th July RD 965/2003 specific control program of the facilities, establishing the hygienic-sanitary criteria for legionellosis prevention and control. - Industrial exclusively use and non-located in urban areas or near places with public or commercial activity.
4. Recreational uses					
4.1. Quality[4] a) Gulf courses watering.	1 egg/10 L	200 CFU/100 mL	20 mg/L	10 NTU	If watering is directly applied to the soil area (drip, micro-sprinkler) 2.3. quality criteria are set. *Legionella spp.* 100 CFU/L (if exists aerosolization risk)
4.2. Quality a) Ponds, water masses and circulating ornamental flows, where public access to the water is impeded.	No limit is set	10.000 CFU/100 mL	35 mg/L	No limit is set	P_T: 2 mg P/L (in standing waters)
5. Environmental uses					
5.1. Quality a) Aquifer recharge by percolation located through the land.	No limit is set	1.000 CFU/100 mL	35 mg/L	No limit is set	N_T[7]: 10 mg N/L NO_3: 25 mg NO_3/L 257 to 259 RD 849/1986 articles
5.2. Quality a) Aquifer recharge by direct injection.	1 egg/10 L	0 CFU/100 mL	10 mg/L	2 NTU	
5.3 Quality a) Forests, green areas and other public non-accessible areas. b) Forestry.	No limit is set	No limit is set	35 mg/L	No limit is set	
5.4 Quality a) Other environmental uses (wetlands, minimum flows or similar maintenance).	Minimum required quality will be studied on a case-by-case basis.				

1 Other Contaminants (See 11th April RD 849/1986 Annex II) contained in wastewater discharge authorization: Contaminants entrance in the environment must be limited. In case of dealing with dangerous substances (See 6th July 907/2007 Annex IV) compliance with EQS (Environmental Quality Standard, see 11th April RD 849/1986 245.5.a article, modified by 23th May RD 606/2003) must be ensured.

2 They must undergo controls that ensure correct facilities maintenance.

3 Their authorization will be conditioned to double circuit obligatory presence for its use.

4 When existing an aerosolization possibility water use, it is essential to follow the conditions stipulated, for each case, by the health authority.

5 Regenerated wastewater additional information characteristics: Conductivity = 3 dS/m; Sodium Adsorption Ratio (SAR) = 6 meq/L; Boron = 0,5 mg/L; Arsenic = 0,1 mg/L; Beryllium = 0,1 mg/L; Cadmium = 0,01 mg/L; Cobalt = 0,05 mg/L; Chrome = 0,1 mg/L; Copper = 0,2 mg/L; Manganese = 0,2 mg/L; Molybdenum = 0,01 mg/L; Nickel = 0,2 mg/L; Selenium = 0,02 mg/L; Vanadium = 0,1 mg/L.

6 Taking into account a 3 classes sampling plan with the following values: n (sample units number) = 10; m (bacterial counting admissible limit value) = 100 CFU/100 mL; M (bacterial counting maximum permissible limit value) = 1.000 CFU/100 mL; c (Sample units maximum number whose bacteria number is situated between m and M) = 3. It is mandatory to carry out the presence/absence pathogens detection (salmonella, etc.) when it c = 3 for M = 1.000 is habitually repeated.

7 Total Nitrogen, organic and inorganic nitrogen sum present in the sample.

Spanish treated wastewater reuse is divided in 5 groups depending on its future use: urban, agricultural, industrial, recreational or environmental. All of the groups are subdivided in different categories, from more restrictive to less restrictive possible reuses. Restriction is mainly focused on ensuring human health protection. Therefore, as further away the future treated wastewater reuse is from human contact in all directions, more permissive the legislation is.

In this case obviously the legislation is more restrictive than for treated wastewater discharge. Firstly, lower suspended solid concentrations are demanded, given again by TSS parameter, and now turbidity is also introduced as a demanded parameter, given by the colour of the wastewater and therefore harder to fulfil than BOD or COD criteria as it not only depends on the biodegradable organic matter, but also on the suspended particles present in the wastewater. To achieve those requirements, additional advanced or tertiary treatment systems have to been introduced after secondary treatment to achieve higher suspended particles removal percentages. Secondly, intestinal nematodes, *Escherichia Coli* and most of times *Legionella spp.* are also used as an indicators to control pathogens. Extra advanced treatment systems will have to be introduced, commonly consisting in disinfection methods. Environmental uses often are specifically restricted by nutrient concentrations, so again it will be necessary to remove them by same tertiary treatment techniques used for discharging treated wastewater into sensitive eutrophication areas. Finally, agricultural uses specifically require maximum concentrations of a list of heavy metals that have been considered noxious. For this case, less conventional and more specific advanced treatment alternatives, usually chemical precipitation or ion exchange, will have to be used to achieve metal removal.

3.3. Future Trends in Treated Wastewater Quality Criteria

As can be seen by comparing both regulations, treated wastewater quality criteria established by the Spanish legislative framework in order to reuse it in different ways are, in any case, more restrictive than the treated wastewater quality criteria established by the European Union legislative framework in order to discharge it into the environment. This is logic on the one hand as is mandatory to guarantee that it will not appear any human health problem because of reusing treated wastewater, but on the other hand, it is also denoted that the European Union treated wastewater discharge regulation allows pouring wastewaters into the environment which continue having contaminants. It happens in the same way with the Spanish treated wastewater reuse regulation.

It is important to remind that apart from the traditional contaminants contained in the wastewater that both regulations take into account when treating wastewater, contaminants of emerging concern (CEC) have been detected during the last years thanks to new detection methods were developed or created by new sources related with human activities, water uses, etc. They have the potential to enter the environment by wastewaters and cause known or suspected adverse ecological and/or human health effects.

Therefore, if we want to reach a long term sustainable water cycle both with the environment and human health based on treated wastewater reusing, it is necessary that actual existing discharge and reuse regulations continue evolving taking more steps forward, becoming more restrictive in the future not only with allowed contaminant levels wastewater could have, but also restricting more contaminants ensuring a better treated wastewater quality.

Chapter 4

Conventional Secondary Treatment

The wastewater treatment plant (WWTP) is the place where the wastewater process is done in order to convert the incoming wastewater, which could be used water from any combination of domestic, industrial, commercial or agricultural activities, into an effluent that can be returned to the water cycle minimizing its impact to the environment or being directly reused.

Treatment consists of subjecting the influent to a series of physical, chemical and biological processes:

- Physical processes: They remove substances by use of naturally occurring forces, such as gravity or electrical attraction, as well as by using physical barriers. The mechanisms involved in this kind of treatments do not result in changes in chemical structure of the influent by themselves. Physical methods include sedimentation or flotation, as well as barriers such as bar racks, screens, deep bed filters or membranes.

- Chemical processes: Chemical products are added to the wastewater in order to react with a portion of the undesired contaminants and removing them.

- Biological processes: Removal of suspended solids by microorganisms such as algae, fungi or bacteria under aerobic or anaerobic conditions during which organic matter in wastewater is oxidized or incorporated into cells that can be eliminated by removal process or sedimentation.

All those processes follow a typical order in a conventional WWTP:

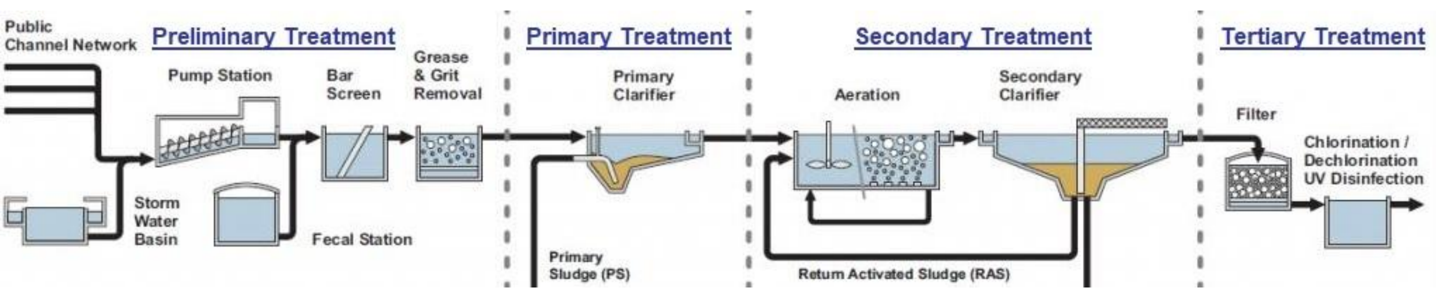

Figure 4.1 Conventional wastewater treatment processes in a WWTP. Source: Sustainable Sanitation and Water Management Toolbox (SSWM).

4.1. Preliminary Treatment

The purpose of preliminary treatment is to ensure a satisfactory quality of final effluent and final sludge product and to protect the treatment process from malfunction associated with accumulation of screenings, debris, inorganic grit, excessive scum formation or loss of efficiency associated with grease, oil films or fat accumulation.

Wastewater preliminary treatment is essentially made of physical processes required to ensure that the treatment plant can process satisfactorily the incoming wastewater flow, enabling the WWTP to produce the required final quality of effluent and a treated sludge suitable for recovery or for the specified disposal objectives. Preliminary treatment is able to remove between 5 – 15% of total suspended solids (TSS), 5 – 15% of biochemical oxygen demand (BOD) and 10 – 25% of fecal coliforms (Metcalf & Eddy, 1995).

4.2. Primary Treatment

After the wastewater has been preliminary treated, there are treatment units commonly called primary sedimentation or settlement tanks, responsible for removing SS that have not been eliminated by the previous treatment. Their purpose is to reduce the velocity of the incoming wastewater stream, allowing the settleable solids to physically fall down at the bottom of the tank.

After the wastewater is pumped into the sedimentation tank, it is detained some hours until the suspended material has either sunk to the bottom or risen to the top of the tank. Sludge is collected on the bottom and is scraped and pumped out of the tank, oils and grease end up floating on top of the water (having lower density) and then the wastewater is sent for undergo secondary treatment.

Main primary treatment objective focuses on suspended solids elimination, also achieving a certain biodegradable contaminants reduction, as a part of the eliminated solids are constituted by organic matter. Primary treatment is able to eliminate between 25 - 40% of BOD, 40 - 70% of TSS and 25 - 70% of fecal coliforms (Metcalf & Eddy, 1995).

4.3. Secondary Treatment

Also known as the conventional activated sludge process (CAS), the secondary treatment stage involves the production of an activated mass of microorganisms capable of aerobically stabilizing the organic content remaining in the influent. Therefore, secondary treatments are focused on eliminating the remaining organic matter.

Wastewater is commonly introduced into an aerated tank of microorganisms which are collectively referred to as activated sludge or mixed liquor. Aeration is achieved by the use of submerged diffused or surface mechanical aeration (or combinations between them), which maintain the activated sludge in suspension. Following a period of contact between the wastewater and the activated sludge, the outflow is separated from the sludge in a secondary settlement tank.

The basic unit of operation of the process is the floc. The floc is suspended in the aeration tank and consists of millions of aerobic microorganisms (bacteria, fungi, yeast, protozoa and worms), particles, coagulants and impurities that have come together and formed a mass. This mass helps to collect pollutants from the wastewater. To maintain the desired microbiological mass in the aeration tank, sludge is returned to the aeration tank while an excess due to biological growth is periodically or continuously wasted. The concentration at which the mixed liquor is maintained in the aeration tank affects the treatment efficiency.

CAS eliminates between 80 – 95% of the BOD, 80 – 90% of TSS and 90 – 98% of fecal coliforms the original wastewater had (Metcalf & Eddy, 1995).. Although the amount of principal contaminants present in the WWTP effluent has been considerably reduced at this point and wastewater fulfills with the European Union treated wastewater quality general criteria in order for being discharged into the environment, this water continues being contaminated by several components that need to be taken into account from the environmental, human health, water reuse and sustainability points of view. Nitrogen and phosphorus, principal nutrients, are only removed between 10 – 30% and 10 – 25% at this point, respectively (Metcalf & Eddy, 1995). Moreover, percentages of removed organic matter, suspended solids or pathogens do not reach most of treated wastewater reuses quality requirements. Therefore, many different kinds of tertiary or advanced treatments, most common analyzed in the following chapter, have appeared at the end of the WWTP treatment processes in order to reach better treated wastewater qualities.

4.4. Sludge Treatment

Wastewater treatment manages to extract pollution from water, generating sludge during the process. Sludge is mostly water with lesser amount of solid material removed from the wastewater. Sludge treatment is focused on reducing sludge weight and volume to reduce disposal costs, and on reducing potential health risks of disposal options. Water removal is the first step in means of weight and volume reduction, while pathogen destruction is frequently accomplished through healing during thermophilic digestion, composting or incineration. The choice of a sludge treatment method depends on the volume of sludge generated, and comparison of treatment costs required for available disposal options (composting, agriculture, etc.). Treating the generated sludge could account for about 50-60% of the total cost of the wastewater treatment (Radjenovic, 2008).

Chapter 5

Conventional Advanced Treatments

Tertiary or advanced wastewater treatments are additional processes needed to remove suspended and dissolved substances remaining after conventional secondary treatment, achieving better wastewater qualities for environmental sustainability purposes.

Tertiary or advanced treatment design is probably the most challenging aspect in a WWTP. Firstly, it important to define the incoming wastewater characteristics at the entrance of the WWTP, the characteristics that will have after being secondary treated, and for which purpose you want to continue treating the water, as the current regulation is restricting different contaminants and at different concentrations depending on the wastewater reuse (see chapter 3.2). Municipality owner of the WWTP needs also must be taken into account: limitations in capital costs (CAPEX) investment and the ability to pay for the project, annual operation and maintenance costs (OPEX), available physical space for the WWTP construction or expansion, compatibility with existing facilities if an upgrading is being carried out, equipment availability, personnel requirements (as more facilities are being added in a treatment plant more highly skilled people are needed) or the energy and resource requirements. Past experience allows dealing with information about known performance, maintenance problems or adaptability to changing conditions, giving reliability to the wastewater treatment system. Finally, but as important as CAPEX and OPEX costs, environmental impacts of the tertiary or advanced treatment should be also studied.

In all these points of view, commonly used advanced treatment systems and their combinations for different purposes have been analyzed along this chapter.

5.1. Nutrients Control

Nitrogen and phosphorus are the principal nutrients of concern in treated wastewater. Discharging or reusing treated wastewater containing those nutrients in sensitive areas subjected to eutrophication, such as lakes, may stimulate an uncontrolled plant growth and therefore alternating the ecosystem natural balance of the area. Moreover, the presence of a big amount of aquatic plants may interfere with beneficial uses of the water resources and exhaust dissolved oxygen in the receiving media, what would present toxicity to aquatic life. Therefore, control of nitrogen and phosphorus became an important issue in treated wastewater quality and nutrient removal is a regulated parameter both for discharging treated wastewater in sensitive areas or for reusing it in environmental uses.

Various treatment methods have been used employed chemical, physical or biological systems to remove both nitrogen and phosphorus. In selecting a nutrient control strategy, it is important to judge the characteristics of the incoming wastewater, the level of nutrient control required, type of existing treatments and their flexibility of operation and the nutrient removal advanced treatment system costs. Conventional nitrogen and phosphorus removal solutions are considered in the following discussion.

5.1.1. Nitrogen Removal

Nitrogen is usually removed from wastewater thanks to a biological process known as nitrification-denitrification. Most of nitrogen is present in wastewaters in form of ammonia. First phase of the process, nitrification, consists in an oxidization from ammonia to nitrite and then from nitrite to nitrate made by specific nitrifying bacteria, known as *Nitrosomonas* and *Nitrobacter*, respectively (EPA, 2007). It is possible to promote this kind of bacteria growth in the tank where the conventional activated sludge process of the secondary treatment takes place by modifying some operational criteria, mostly related with providing a higher and extended aeration able to maintain a minimum level of 2 mg/L of dissolved oxygen in all the tank and also extending the age of the sludge, given by the applied solid retention time (SRT), to allow these slow-growing bacteria appear and perform the nitrification process (EPA, 2007).

Denitrification is the second step of the process to achieve complete nitrogen removal. It consists on reducing nitrate obtained by nitrification into nitrogen under anoxic conditions (in the absence of oxygen), which will be used by microorganisms to grow. It will be fundamental for denitrification, apart from operating under anoxic conditions with the ammonia transformed into nitrate, to have enough organic carbon or matter to provide the energy source for the conversion of nitrate into nitrogen by the bacteria. Depending on the system organization, denitrification is usually performed in three different ways:

- Modified Ludzack-Ettinger process (MLE): An anoxic zone where denitrification is achieved is installed before the aerobic zone where the activated sludge process takes place with nitrification. Although the logical order of the complete nitrogen removal process is altered, pre-denitrification is advantageous as the biodegradable matter from the incoming wastewater is available at the anoxic zone, proportioning higher organic carbon source for the conversion of nitrate into nitrogen, achieving higher denitrification rates which reduce the required tank volume. Moreover, the oxidation capacity of nitrate degrades part of the organic matter, reducing oxygen demand and therefore the energy demand during the aeration process. However, nitrogen removal is limited in these systems because of its dependence on the recycling ratio of nitrate produced by nitrification transported from the aerated tank back to the anoxic zone. Effluent nitrogen concentrations between 5 – 10 mg/L are usually achieved by this configuration (EPA, 2007).

- Bardenpho process: Four stages are used during this process, which allow achieving higher nitrogen removal rates, proportioning effluents with usually between 3 – 7 mg/L nitrogen concentration. The influent enters into an anoxic stage where denitrification occurs and after that both nitrification and conventional activated sludge take place in a second aerated tank. Nitrate produced in this second zone is recirculated to the anoxic zone at the entrance as it happened in MLE process. After that, there is a third anoxic stage where a carbon dose can optionally be added to feed the bacteria, achieving nitrogen concentrations under 3 mg/L, and a final aerated zone of smaller volume (EPA, 2007). So, the denitrification-nitrification process is made two times during the system, that is why the Bardenpho process achieves higher nitrogen removals than the MLE process. On the other hand, the main disadvantage of this process is the higher space required.

5.1.2. Phosphorus Removal

As it happens with nitrogen, applying longer SRT to conventional activated sludge process increase removal rates of the other main nutrient, phosphorus. However, achieved reductions would continue being under both requirements for discharging treated wastewater in sensitive areas or reusing it in environmental uses. The key to biologically remove phosphorus by an improved activated sludge process is to expose the phosphate-accumulating organisms (PAO) to alternating anaerobic and aerobic conditions (EPA, 2007). These alternating conditions produce a stress that makes them consume more phosphorus than on a simply aerated condition. Those conditions are usually achieved by introducing an anaerobic stage during the activated sludge process. Therefore, biological nutrient removal systems are commonly composed by an anaerobic zone to achieve phosphorus removal, an anoxic zone where the denitrification is produced and an aerobic zone where both nitrification and the rest of the conventional activated sludge process are carried out. Complete biological nutrients removal is usually performed in three different configurations:

- A^2/O process: Modification of the conventional nitrogen removal MLE process by adding an anaerobic stage at the entrance of the process, where phosphorus removal is achieved. It will be followed by an anoxic stage where denitrification occurs and a final aerobic stage where nitrification and the conventional activated sludge process take place. A nitrate recirculation from the aerobic stage to the anoxic stage is also present as it happened during the conventional MLE process. Effluent concentrations between 5 – 10 mg/L of nitrogen and 0,5 – 1 mg/L of phosphorus are usually achieved (EPA, 2007).

- Modified Bardenpho Process: As it happened before, this configuration modifies the original Bardenpho process used to remove nitrogen by adding an anaerobic stage at the entrance of the scheme, where phosphorus removal is achieved. The rest of the configuration remains equal as it was during the original Bardenpho process, having two anoxic/aerobic stages after the new first anaerobic stage where denitrification and nitrification are produced as well with the conventional activated sludge process. Effluent concentrations between 3 – 7 mg/L of nitrogen and 0,5 – 1 mg/L of phosphorus are usually achieved (EPA, 2007). As it happened with the original Bardenpho process, it is possible to add a carbon dose at the second anoxic/aerobic cycle to achieve nitrogen concentrations under 3mg/L.

- Oxidation Ditch: This configuration consists is an oval shaped tank where all three stages, anoxic, anaerobic and aerobic, sequentially take place thanks to horizontal or vertical aerators that provide circulation, oxygen transfer and aeration to the system. The velocity at which the wastewater is circulating inside the system is a key parameter, generally recommended to be over 0,3 m/s. Without the need of recirculate nitrates to other stages or introducing carbon dose to promote higher denitrification rates as the anoxic stage is the first one produced in the sequential cycle, it is seen as an efficient system in terms of operational costs. However, suspended solids concentrations are relatively higher than other systems. Effluent concentrations between 5 – 10 mg/L of nitrogen and 0,5 – 1 mg/L of phosphorus are generally achieved (EPA, 2007).

Phosphorus is also usually chemically removed by the addition of certain chemicals that produce low-solubility salts when combined with phosphate. Metal salts, most commonly ferric chloride or aluminum sulfate, or lime are typically used for phosphorus removal of wastewater. Chemical can be applied at the end of the activated sludge process just before the secondary clarifier or in two points, both before the first clarifier at the entrance of the primary treatment and before the secondary clarifier at the end of the activated sludge process. Two-point addition is popular for many applications because it achieves better efficient uses of chemicals for phosphorus precipitation. The required chemical dose is related to concentration of phosphorus in the incoming wastewater. For low phosphorus concentrations between 0,3 – 1 mg/L the dose can be between 1,2 – 4 moles of aluminum sulfate or ferric chloride per mole of phosphorus (Minnesota Pollution Control Agency, 2006). Generally, the capital costs of chemical removal systems are lower than biological phosphorus removal systems. However, it must be taken into account these chemicals do not remove nitrogen. Moreover, sludge production will increase in the wastewater treatment unit process where the chemical is applied, approximately by 40% in the primary treatment and 26% in the secondary.

5.2. Suspended Solids removal

Despite at the end of a conventional secondary treatment enough biodegradable matter and suspended solids have been removed to discharge the treated wastewater into the environment, higher requirements in terms of suspended solids removal are demanded to reuse the treated wastewater. For this reason, many advanced or tertiary systems have appeared to continue reducing suspended solids present at the end of a secondary treatment in order to achieve better effluent qualities in terms both of suspended solid concentrations and turbidity. Most of these methods consist on physical or chemical treatments, sometimes combined between them. Most conventional advanced suspended solid removal systems have been discussed.

5.2.1. Coagulation-Flocculation

Size range of contaminant particles present in wastewater is very varied. From the biggest ones, previously removed in a secondary effluent because of their settleability, to the lowest ones, which present better stabilities in wastewaters because of static electricity, what makes impossible removing them by decantation or flotation. Some of them are known as colloids, constituting an important part of the wastewater contamination, being the principal turbidity cause. Generally sized between 0,001 and 1 µm, it is also impossible to separate them from the wastewater by some of the existing filtration systems.

Coagulation-Flocculation treatment has as its main objective electrically destabilize this kind of particles by using certain chemical products and allow them the potential to collide and stick together, forming flocs which can be removed by a decantation process. The process is divided in three steps:

- Coagulation: The goal of this first process is, as it has been previously said, electrically destabilize colloid particles by adding a coagulant, usually ferric chloride or aluminum sulfate in a rapid mixing compartment where wastewater spends an average time between 1 and 3 minutes before moving onto the flocculation compartment. The effectiveness of coagulation depends on the coagulant type and dose, velocity in which the coagulant is mixed and wastewater pH (there is always a pH range in which any coagulant is more effective).

- Flocculation: Once the colloids are electrically destabilized, they have the potential to collide and stick together, forming flocs. Flocculation consists on a wastewater agitating process, usually between 10 and 30 minutes, causing particles to be continuously moving to encourage the most collisions between them as possible, not only forming the flocs, but also increasing their volume and weight in order to became settleable. Flocculation effectiveness depends on the previous coagulation quality, a slow and homogeneous agitation (fast mixing would destroy formed flocs) and wastewater characteristics.

- Sedimentation: Last step consists in a decantation process, where wastewater flow is slowed to achieve a calmed environment where the previously formed flocs can settle down to the bottom of the sedimentation tank, usually being a lamella clarifier, which uses a series of inclined plates called lamellas to increase the settlement velocity, being possible to reduce the size of the sedimentation tank and, therefore, its price.

One of the principal disadvantages of this treatment is associated with the toxic or highly putrescible amount of sludge that may be generated, increasing OPEX costs which are already high due to the use of chemical products (not only coagulant, but also polymers may be added to achieve adequate settling of solids or dissolved salts to a pH adjustment).

By the addition of a coagulation-flocculation treatment after a conventional secondary treatment the treatment line is typically able to achieve suspended solid concentrations between 10 – 20 mg/L, BOD between 10 – 20 mg/L and turbidities between 5 – 10 NTU. Pathogens are also removed by this process. It achieves between 2 -3 logs (99-99%) of *Giardia lamblia* or *Cryptosporidium parvum* (protozoa), 97,7 to 99,9% of viruses, 80% of intestinal nematodes and 1 – 2 log (90-99%) of *Escherichia Coli* (bacteria) removal (International Water Association, 2016). Moreover, some inorganic matter is also eliminated, such as arsenic, fluoride or phosphorus (between 60-95%), being especially interesting to be controlled to avoid aquatic plants over-growth, although higher dosages are required in order to achieve low phosphorus effluent concentrations between 0,3 – 1 mg/L (International Water Association, 2016)..

Suspended solid concentrations and turbidities obtained would allow reusing treated wastewater in some of the less demanding reuses, like most of industrial or recreational reuses. However, pathogen concentrations remaining are above of the limits demanded by the legislation, especially in terms of intestinal nematodes, where the treatment does not offer excellent results. Therefore, more advanced treatments are required after a coagulation-flocculation treatment, usually to achieve better suspended solids and pathogens removals.

5.2.2. Granular Filtration

Granular filtration is used commonly after coagulation-flocculation processes in order to achieve superior removals of suspended solids. Treatment consists in constructed beds of sand or other suitable granular material, such as anthracite, contained in a liner made of an impermeable material. Influent wastewater, introduced at the surface of the bed, passes one single time through the granular material to its bottom. Suspended material is trapped between the grains of the filter media by a combination of physical and chemical mechanisms, mostly because of adsorption (particles stick onto the surface of the individual filter grains).

The complete filtration process is followed by a second phase, called cleaning or backwashing, which must be done when presence of suspended solids in the effluent increases over an acceptable level. It consists on cleaning the filter media to remove the trapped solids that have been accumulating during the filtration. This is done by reversing the flow through the filter, often using air in conjunction with water. In most WWTP, used wastewater for the backwashing process is returned to primary or secondary treatments to be processed again. This second phase is especially important in terms of maintenance, as problems may occur (mud balls, filter bed shrinkage or excessive media loss) that may be impossible to correct without totally replacing the filter media. However, there are some filter systems that operate continuously, both phases taking place simultaneously.

Sand bed performances depend on wastewater characteristics (as less treated the incoming wastewater is, more often the filter will have to be backwashed, losing efficiency by increasing OPEX costs), environmental factors inside the filter (media reaeration, making oxygen available for aerobic decomposition, and temperature) and the design characteristics of the filter. The main factors that determine CAPEX costs are land (the required land area may be a limiting factor) and media used for filtering (if appropriate filter media are not available locally, costs could be higher). However, being an energy-efficient treatment with low energy costs, without using any chemicals and requiring regular but low maintenance make it an attractive choice in the long run.

By the addition of a granular filter treatment after a coagulation-flocculation and a conventional secondary treatment, the treatment line is typically able to achieve suspended solid concentrations between 4 – 10 mg/L, BOD between 4 – 10 mg/L and turbidities between 0,3 – 3 NTU (Minnesota Rural Water Association, 2009). These levels would offer the possibility of reusing the treated wastewater in all the possible reuses in terms of SS and turbidity, but reductions on fecal coliforms and intestinal nematodes may continue being insufficient. Therefore, it continues being necessary a final advanced treatment step to reduce pathogens.

5.3. Disinfection

Disinfection has become one of the primary mechanisms for the inactivation or destruction of pathogenic organisms. Chlorination, ozone and ultraviolet radiation are the principal disinfectants used to treat wastewater. The World Health Organization (WHO) compiled a risk ranking of pathogens, reflecting the potential concentrations in wastewaters, their resistance to treatment and infectious doses, defining the helminths (interstinal nematodes, such as *Ascaris lumbricoides* or *Taenia solium*) as high risk pathogens, bacteria (diarrhea is the most prevalent type of infection, with colera the worst form) and protozoa (such as *Cryptosporidium parvum* or *Giardia lamblia,* causing diarrhea or dysentery) as lower risk pathogens, and viruses (most of them causing diarrhea) as the least risk.

In general terms, bacteria are highly susceptible to all three treatments, helminth eggs and protozoan cysts are more resistant to chlorine, and certain viruses are more resistant to UV disinfection. Depending on the contact time, chlorination inactivates 1–4 log units of viruses (90-99,99%), 2-5 log units of bacteria (99-99,999%), 0–3 log units of protozoan cysts (0-99,9%), but almost no helminth eggs without large contact times. Similar results are found with the other disinfectants, but UV radiation is much more efficient at inactivating protozoa and ozonation results in better inactivation of protozoa as well as less inactivation of viruses (Jiménez, B., 2010; Jacangelo J.G., 2002; Medeiros R., 2015).

5.3.1. Chlorination

Chlorine is the most used disinfectant for treating wastewater because it destroys target organisms by oxidizing cellular material, but also because it is applied for a wide variety of objectives other than disinfection in wastewater treatment field, including activated sludge bulking control or odor control. Therefore, is a well-established technology.

Chlorine can be supplied in many forms, the principal ones used at wastewater treatment plants are (EPA, 1999): chlorine (Cl_2), chlorine dioxide (ClO_2), calcium hypochlorite [$Ca(OCl)_2$] and sodium hypochlorite ($NaOCl$). All forms of chlorine are highly corrosive and toxic. Thus, storage, shipping and handling means a risk, requiring increased safety regulations.

The required degree of disinfection can be achieved by varying the dose and the contact time depending on chloride demand (chloride residual is unstable in the presence of high concentrations of chloride-demanding materials, requiring higher doses to effect adequate disinfection), wastewater characteristics and discharge requirements. The dose usually ranges from 5 to 20 mg/L, having a flexible dosing control (EPA, 1999).

After disinfection, residual chlorine can persist in the effluent for many hours. This is a good point on the one hand because it can prolong disinfection (not allowing pathogenic re-growth), but on the other hand, chloride content of the wastewater is increased as well as the total dissolved solids level by a toxic component to aquatic life even at low concentrations. To minimize the effect, chlorinated wastewater is often being dechlorinated, removing the free and combined chlorine residuals to reduce residual toxicity after chlorination and before discharge. Sulfur dioxide, sodium bisulfite and sodium metabisulfite are the commonly used dechlorinating chemicals. Chlorination/dechlorination systems are more complex to operate and maintain than chlorination systems.

Apart from pathogens, chlorination is also effective in oxidizing certain organic and inorganic compounds, eliminating certain noxious odors during the process as well. However, oxidizing certain types of organic matter more hazardous compounds such as trihalomethanes are created that have to be taken into account from the human health point of view. Because of this reason, chlorine disinfections are usually the last step of the advanced wastewater treatments.

Chlorine is the most cost-effective disinfection, except when dechlorination is required (total cost of chlorination will be increased approximately 30 to 50% with the addition of dechlorination). Chlorination system CAPEX costs are dependent on the manufacturer, the site, the capacity of the plant and the characteristics of the wastewater to be disinfected. Annual OPEX costs include power consumption, cleaning chemicals and supplies, miscellaneous equipment repairs and personnel costs (EPA, 1999).

5.3.2. Ultraviolet Radiation

The disinfection of treated wastewater via ultraviolet radiation (UV) involves a passing film of wastewater within close proximity of a UV lamp. UV radiation is generated by a physical process consisting on a electrical discharge through mercury vapor, eliminating the need to generate, transport or store corrosive or toxic chemicals, being user-friendly with operators.

Low-pressure mercury arc lamps are the principal mean of generating UV energy used for disinfection. The mercury lamp is favored because about 85% of the light output is monochromatic at a wavelength of 253,7 nm, which is the optimal range for germicidal effects (250 to 270 nm). The lamps are typically about 0,75 to 1,5 m in length and about 15 to 20 mm in diameter. The ideal wall lamp temperature is between 35 to 50 ºC. Operationally, the lamps are either suspended outside of the liquid to be treated or submerged in the liquid. When they are submerged, the lamps are encased in quartz tubes to prevent cooling effects. These equipments require less space than chlorination. A preventive maintenance program is necessary to control fouling of tubes (EPA, 1999).

The effectiveness of a UV disinfection system depends on the wastewater characteristics (as lower quality the wastewater has, less efficient the treatment is, specially losing efficiency for secondary treated effluents with SS levels above 30 mg/L), the intensity of UV radiation, the amount of time the microorganisms are exposed to radiation (approximately 20 to 30 seconds with low-pressure lamps, being shorter than other disinfectants) and the reactor configuration (EPA, 1999).

UV disinfection does not result in a lasting residual in the wastewater. This is an advantage on the one hand, as there is no residual effect that can be harmful for humans or aquatic life, but on the other hand is a disadvantage when wastewater must be piped or stored over significant distances and times (particularly relevant to reuse schemes) as re-growth of the microbial population is considered a risk. Because of this, a residual quantity of chlorine is added after UV disinfection, especially for reused wastewaters.

Costs have decreased in recent years due to improvements in lamp and system design, increased competition and improvements in the system's reliability. However, UV disinfection is not as cost-effective as chlorination, unless when dechlorination is used. CAPEX costs of UV disinfection systems depend on the manufacturer, the site, the capacity of the plant and the characteristics of the wastewater to be treated. In terms of OPEX, UV disinfection is a little cheaper than chlorination to operate and maintain because is not necessary to be using chemical products to disinfect the wastewater, including power consumption, cleaning chemicals and supplies, miscellaneous equipment repairs, lamps replacement, ballast and sleeves and staffing requirements (EPA, 1999).

5.3.3. Ozonation

Ozone is produced when oxygen (O_2) is dissociated by an energy source into oxygen atoms and subsequently collides with an oxygen molecule to form an unstable gas, ozone (O_3), which is used to disinfect wastewater.

Ozone is an unstable element that decomposes to elemental oxygen in a short amount of time. Because of that, most of the WWTP that use this system as a disinfection method generate their own ozone onsite by imposing a high voltage alternating current across a dielectric discharge gap that contains extremely dry air or pure oxygen. This is advantageous as they are fewer safety problems associated with shipping and handling, but on the other hand it is a more complex technology than chlorination or UV disinfection, requiring more complicated equipment made of corrosion-resistant materials such as stainless steel as ozone is very corrosive and reactive.

After being generated, ozone is fed into a contact chamber containing the wastewater to be disinfected. When ozone decomposes in water, the free radicals hydrogen peroxy (HO_2) and hydroxyl (OH) that are formed have a great oxidizing capacity, needing a short contact time between 10 and 30 minutes to disinfect the wastewater (EPA, 1999). No harmful residuals are needed to be removed after the process is done, as the ozone is rapidly decomposed. This system elevates the dissolved oxygen concentration of the effluent, what could be interesting or even mandatory as it has been explained during chapter 2.1. As it happens with UV disinfection, ozone does not result in a lasting residual in the wastewater, with the consequent advantages and disadvantages with respect to chlorination. A residual amount of chlorine is therefore also added after ozonation, especially for reused wastewaters.

The off-gases from the contact chamber, irritating and toxic, must be treated to destroy any remaining ozone before being released into the atmosphere. Therefore, it is essential to maintain an optimal ozone dosage for better efficiency. On the other hand, if the ozone dosage during the disinfection process is too low they process may not effectively inactivate some viruses, spores and cysts. When pure oxygen is used as the feeding gas, the off-gases from the contact chamber can be recycled to generate more ozone.

Despite ozone disinfection is considered the most efficient inactivation system, especially in terms of viruses removal, its elevated CAPEX costs related with the necessary complex equipment to generate the ozone onsite and also elevated OPEX costs related with the use of energy during the ozone generation makes it the less cost-effective of the three kinds of disinfection, being the less used because of this. Nevertheless, ozone does not only have a disinfection capacity on wastewaters, but also the capacity to remove a large amount of contaminants of emerging concern known nowadays. As it has been explained during chapter 2.3, CEC are gaining more importance day by day and therefore ozonation could also gain more importance because of its disinfection and CEC removal combination of capacities.

5.4. Proposed Advanced Treatment Lines

Different advanced treatment lines have been proposed taking into account all the information about conventionally used advanced treatments and their uses discussed along this chapter. They consist on combinations of those systems added after a conventional secondary treatment in order to achieve an effluent wastewater that fulfills with the contaminant requirements regulated by both treated wastewater European Union discharge legislation or Spanish reuse legislation, depending on the treated wastewater destination.

Discharge to Sensitive Area

During the previous chapter we have seen that a conventional secondary treatment achieves enough SS, BOD and COD removals to discharge the treated wastewater into a general point. However, special requirements related with both main nutrients, nitrogen and phosphorus, are asked in order to discharge the treated wastewater in zones sensitive to eutrophication. Those special requirements can be achieved by different modifications of the conventional activated sludge process, allowing both nitrogen and phosphorus removal by introducing anoxic and anaerobic stages during the process, respectively, in different configurations. Choosing the optimal biological nutrient removal configuration will depend on the incoming wastewater characteristics, the wanted effluent quality in terms of nutrients, the available space and both CAPEX and OPEX costs.

Reuse Quality B

Being the less quality scenario of treated wastewater to be reused, this treatment line consists on adding granular filtration and disinfection after the conventional secondary treatment. It is important to remark that if UV radiation or ozone is chosen as the disinfection method it will be necessary to add final maintenance disinfection, where a residual amount of chlorine is added to the effluent in order to avoid microbial population re-growth. This treatment line is able to generate an effluent of concentrations below 10 mg/L of SS, 10 mg/L of BOD, 5 NTU of turbidity and between 200 to 10.000 CFU/100 mL of *Escherichia Coli*. The obtained treated wastewater will be able to be reused for 2.2 or 2.3 qualities of the agricultural uses, 3.1 quality of the industrial uses, 4.2 quality of the recreational uses or 5.1 quality of the environmental uses (see chapter 3.2). Note that to reuse the treated wastewater to 5.1 environmental quality use it will be necessary to modify the conventional activated sludge process of the line in order to achieve the required nutrient removal.

Reuse Quality A

A better quality of treated wastewater is achieved by this last proposed advanced treatment line, consisting in on adding coagulation-flocculation, granular filtration and disinfection after the conventional secondary treatment. As it happened with the previous proposed treatment line, if UV radiation or ozone is chosen as the disinfection method it will be necessary to introduce final maintenance disinfection. This treatment is able to generate an effluent of concentrations below 10 mg/L of SS, 10 mg/L of BOD, 3 NTU of turbidity and less than 200 CFU/100 mL of *Escherichia Coli*. The obtained treated wastewater will be able to be reused for more restringing 1.2 quality of the urban uses, 2.1 quality of the agricultural uses or 4.1 quality of the recreational uses (see chapter 3.2).

Although this treatment line offers excellent effluents in terms of SS, BOD or turbidity, is not able to provide an absence of *Escherichia Coli,* required by different treated wastewater possible reuses. Therefore, extra advanced treatments are needed for those reuses. Usually membrane filtration is used to achieve those requirements, technology that will be discussed during the following chapter.

Chapter 6

Membrane Bioreactor (MBR)

Development carried out to obtain better filtration membrane generations, more productive and less expensive, have led to the emergence of a new concept of biological wastewater treatment: membrane bioreactors (MBR). This technology consists in an activated sludge process where separation between solids and liquid is made by membrane filtration instead of settlement in a decantation tank, combining both systems to achieve important advantages with respect to conventional activated sludge processes:

- As the secondary settlement tanks are replaced by more compact filtration membrane modules, they require less space, not only allowing more compact WWTP construction, but also being especially interesting when a WWTP needs an upgrading and available space is a limiting factor.

- The total retention of activated sludge in the bioreactor allows operating under high mixed liquor suspended solids concentrations (MLSS) and elevated solid retention time (SRT). Higher volumetric loading could therefore be applied to MBR systems, resulting in lower hydraulic retention times. The high MLSS concentration allows the reduction of the bioreactor's size. Operating at long SRT allows, on the one hand, leading to a excessive sludge minimization and, on the other hand, the development of slow-growing microorganisms responsible for the degradation of specific organic pollutants such as nitrogen.

- The use of MF or UF membranes improves the quality of the wastewater effluent, allowing the complete physical retention of most of suspended solids, bacteria and some viruses, offering an excellent disinfection capacity. Wastewater produced is able to be reused in most of possible reuses established by current legislation. Therefore, MBR not only can substitute conventional secondary treatment, but also most of the tertiary treatment lines.

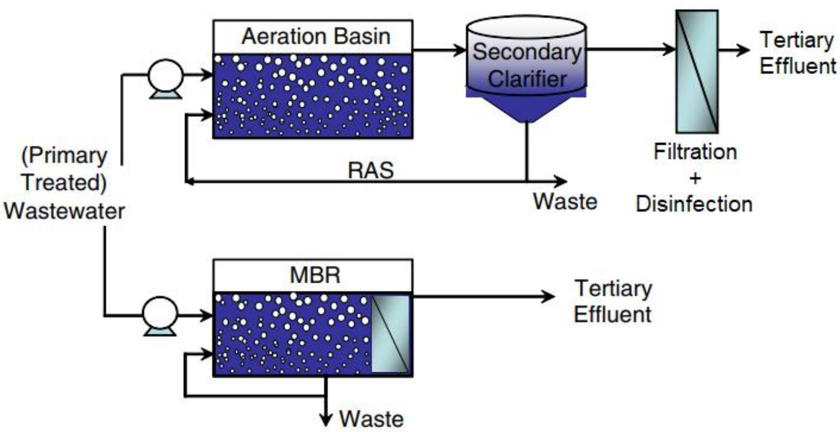

Figure 6.1 Comparison between a conventional secondary and tertiary treatment line and MBR layouts (Le-Clech, P., 2010).

Although MBR systems are presented as a new form of wastewater treatment, research and commercialization of this technology began during the 1960s, when the combination of filtrating membranes with a biological process was considered as an optimal wastewater treatment system. However, MBR were not representing a viable economic option because of their primary disadvantage: expensive CAPEX and OPEX costs. Because of this reason, membrane bioreactors were used for small, decentralized treatment applications.

During the last decades, the interest on applying MBR on treating wastewater has significantly increased. On the one hand, because of the need of reusing wastewater and how stringent the current legislations that define the reuse of treated wastewater have become. On the other hand, because of the market expansion these systems have achieved thanks to technological advancements in terms of more productive and less expensive filtration membranes or new reactor configurations that optimize energy consumptions, reducing OPEX costs as well.

Given the large number of MBR plants in operation and under construction, confidence in this technology keeps increasing, making MBR a system of choice for wastewater treatment and reuse.

6.1. Membrane Filtration

Filtration by membranes has been evolving to become one of the most advanced wastewater treatments during the last decades, being widely used when high effluent qualities have to be achieved in order to reuse wastewater. A membrane is a thin layer of semi-permeable material that allows the passage of water and blocks contaminants when a driving force is applied, providing the filtered solvent on one side of the membrane and the removed solute on the other side. Pressure-driven membrane technologies use hydraulic pressure to force water molecules through the membranes. As they are most commonly used for wastewater treatment over electrically-driven membrane technologies (electrodialysis and reversal electrodialysis, which use electric current to move ions across the membranes leaving purified water behind), discussion will be focused on the first ones. Pressure-driven membranes include, in order of decreasing permeability: microfiltration (MF), Ultrafiltration (UF), nanofiltration (NF) and reverse osmosis (RO) (EPRI, 1997).

- Microfiltration (MF): MF membranes are generally considered to have a pore range from 10 µm to 0,1 µm, usually between 0,1 – 0,2 µm. They operate at relatively low incoming wastewater pressures, between 100 - 400 kPa (Minnesota Rural Water Association, 2009). Contaminants removed by MF include macromolecules, sand, silt, clays, *Giardia lamblia* and *Cryptosporidium* cysts, algae or some bacterial species. MF does not remove most of viruses present in wastewater, but it can control those microorganisms being combined with disinfection. MF could be also used as a pretreatment to NF or RO, removing natural synthetic organic matter to reduce fouling potential in those future treatments.

- Ultrafiltration (UF): For UF pore sizes range from 0,05 µm to 0,005 µm approximately, usually 0,01 µm being used. They operate with pressures between 200 - 700 kPa (Minnesota Rural Water Association, 2009). UF will remove all the contaminants removed by MF, as well as humic material and some viruses, not being an absolute barrier for them. Disinfection is recommended after an UF process as it can provide a second barrier to pathogen contaminants. As MF, is it often used as a pretreatment to NF or RO in order to provide a better wastewater quality by particle contaminants removal, reducing the fouling potential at the following treatment step.

- Nanofiltration (NF): This filtration, as RO, is designed to remove dissolved substances utilizing semi-permeable membranes that do not have definable pores. Maximum removed sizes range from 0,01 µm to 0,001 µm or even a little less. Pushing water through these membranes require higher operation pressures, usually between 600 – 1.000 kPa (Minnesota Rural Water Association, 2009). At this level, all the particles are virtually completely removed, including all cysts, bacteria, viruses and humic material, providing excellent protection to disinfection by products formation (such as trihalomethanes). NF differs from RO in terms of its lower removal efficiency for dissolved substances, particularly for monovalent ions, resulting in unique applications such as hardness ions removal at lower pressures that would use RO (often being called softening membranes because of that). As NF also reduces alkalinity, some measures to add alkalinity may be required to reduce corrosivity of the water produced.

- Reverse Osmosis (RO): These membranes can effectively remove nearly all inorganic contaminants from wastewater by using operation pressures over 1.000 kPa (Minnesota Rural Water Association, 2009). RO, as NF, removes dissolved solids through the process of reverse osmosis. Osmosis is the natural flow of a solvent, such as water, through a semi-permeable membrane from a less concentrated solution to a more concentrated solution. This flow will continue until concentrations on both sides of the membrane are equal. The amount of pressure that must be applied to the more concentrated solution to stop this water flow is called osmotic pressure. Reverse osmosis is the reversal of the natural osmotic process, achieved by applying pressure in excess of the osmotic pressure to the more concentrated solution. RO can effectively remove all the particles and dissolved substances the previous membrane filtrations were able to eliminate, as well as monovalent ions. However, RO membranes are not absolute barriers and some passage of particulate or dissolved matter may occur, which can be attributed to slight manufacturing imperfections. Disinfection is recommended to ensure the safety of the water.

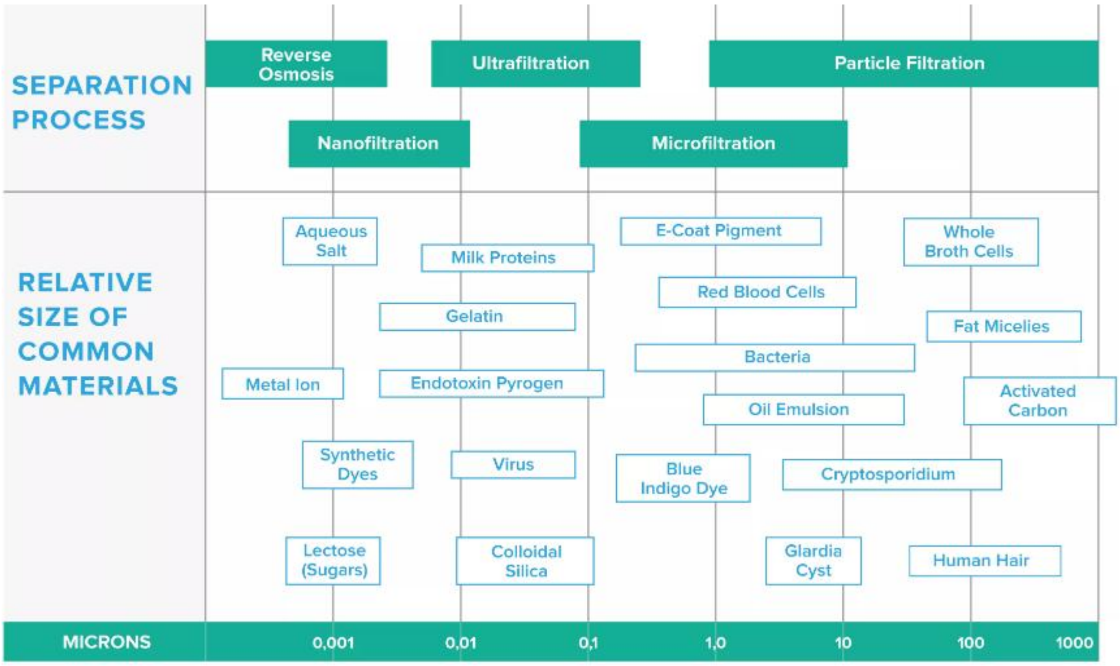

Figure 6.2 Membrane filtration and common materials size. Source: Enviroconcepts.

Typically MF and UF membranes are used in MBR systems. Choosing an optimal material of which the membranes will be made of is fundamental in order to achieve good performances. MBR membranes are usually made of polymeric materials. Although there are also ceramic and metal oxide membranes, they are more expensive and traditionally used for industrial processes because of their efficiency at very high temperatures. Cellulose polymers are inexpensive and widely used. Being susceptible to biodegradation, they must be operated with a pH from 4 to 8, having some resistance to continuous low-level oxidants (chlorine doses of 0,5 mg/L or less may control biodegradation and biological fouling without damaging the membrane). Polymers most used in MBR membranes are constituted by polyvinylidene fluoride (PVDF), followed by high-density polyethylene (HDPE) and polyethersulfone (PES) (Ribera, G., 2013).

6.2. Membrane Modules

Membranes are generally mounted in modules or cassettes, which include aeration ports, permeate flow connections and supporting frames. Aeration is used at the bottom of the membrane module to provide extra oxygen, keep the biomass in suspension and producing a turbulent two-phase flow velocity (around 0,2 - 0,4 m/s) on the membrane surface to limit fouling deposition. Membrane media is usually manufactured as flat sheets, hollow fibers or tubular fibers, then configured into hollow-fiber, spiral-wound, multi-tubular or flat-plate membrane modules.

- Hollow-fiber modules: In a hollow-fiber element, fibers are rolled together and sealed in a pressure container. A hollow-fiber module consists on several thousands to even millions of hollow fiber elements. They are usually disposed vertically, with some space in between allowing them to have some flexibility to move mostly laterally due to an aeration system, responsible for moving up accumulated sludge through the membrane agglomeration. Some types of hollow-fiber modules present a reinforced core to provide greater structural robustness. Filtration occurs from the exterior to the interior of the fibers in MBR. Effluent is collected at their end by a conduit system, either on only one side or both sides (if fibers are solidified in both ends). These modules are generally cheap to manufacture, are very compact (as their membrane surface per unit of volume is high) and can tolerate strong backwashing. However, being packed in high density modules make fluid dynamics and distributions harder to control than other modules as multi-tubular or flat-plate ones, requiring more frequent washing and cleaning.

Figure 6.3 Typical hollow-fiber module. Source: Xiner-Membrane Material Technology Co., Ltd.

- Spiral-wound modules: The spiral-wound element consists in two flat membrane sheets placed back to back and separated by a mesh-like porous support called permeate carrier, being sealed on three of their sides like an envelope. The fourth side is fixed onto a perforated plastic tube that collects the product water at the center of the unit. The membranes are rolled up around the tube in the form of a spiral. Incoming wastewater enters at the end of the spiral-wound element in a parallel path to the central tube. Then wastewater passes through the membranes and follows the spiral configuration to the central perforated tube, leaving behind all the dissolved and particulate contaminants rejected by the membranes. Wastewater that does not penetrate membranes continues flowing across the membranes surface, becoming increasingly contaminated, and exits the spiral-wound element through an opposite end from the incoming wastewater. Spiral-wound modules are cheap to manufacture, competing with hollow-fiber ones in terms of costs. However, they require well-treated influent wastewater to be operating at optimal levels. They are generally associated to NF or RO and therefore not usually used in MBR.

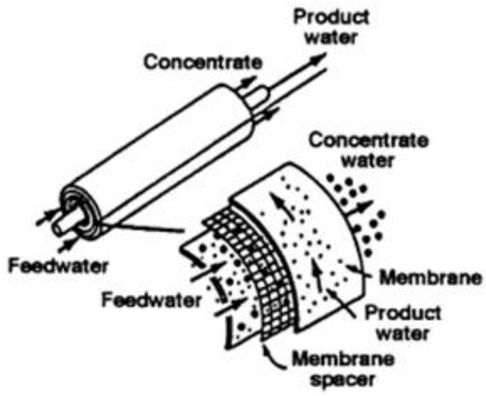

Figure 6.4 Spiral-wound element
(EPRI Community Environmental Center, 1997).

- Multi-tubular modules: They are currently the only standardized MBR product, comprising a bundle of cylindrical tubes, usually 200 mm in diameter. Membranes are going inside those tubes, which are constructed of a microporous substrate material which provides mechanical strength. Wastewater flows from inside to outside the membrane tubes. Tubular membranes are known for their robustness (they can be subjected to higher pressures in demanding situations), long membrane life and high flux rates. However, they are usually the most expensive modules to manufacture, also requiring more energy and space than other options. Multi-tubular modules are usually used in external MBR configurations.

Figure 6.5 Tubular elements which form multi-tubular modules.
Source: The MBR site.

- Flat-Plate Modules: They are formed by flat panels vertically installed in series and parallel between them, with their ends sealed to avoid contact with the influent and the permeate wastewater. Each panel, usually rectangular, consists of two membranes with their active part facing the incoming wastewater direction. Wastewater flows between panels and effluent goes into collection tubes by tangential filtration. Space between panels, typically between 2 to 3 mm, should minimize fouling tendency and allow adequate aeration for cleaning process. Aerators are placed below panels to move up accumulated sludge. As it happens with multi-tubular modules, having well defined widths allow these modules controlling fluid dynamics and distribution, requiring less frequent washing or cleaning. Moreover, these modules usually have the lowest energy demands. However, these membrane configuration makes them being easily damaged.

Figure 6.6 Panels which form flat-plate modules.
Source: RisingSun Membrane Technology Co., Ltd.

Table 6.1 Comparison between different module configurations (clear advantage: +++; clear disadvantage: -) (Gili, C, 2008).

Characteristic	Hollow-fiber	Spiral-wound	Multi-tubular	Flat-Plate
Compaction Density	+++	++	-	+
Energy Demand	++	+	-	+++
Pressure Drop	++	++	+++	-
Extracted Volume	++	+	-	+
Pretreatment Quality	++	-	+++	+
SS Removal	+	-	+++	++
Economical Cost	+++	+++	-	+
Ease of cleaning (in situ)	-	-	++	+
Ease of cleaning (aeration)	+++	-	-	-

6.3. MBR Configuration

Materials and modules in which membranes are manufactured in MBR have been previously reviewed and discussed. These systems combine membrane filtration with an activated sludge process in different configurations depending on the membrane location.

6.3.1. External/Sidestream MBR

First MBR generation is based on a sidestream configuration, for which the membranes are located outside the bioreactor. Influent wastewater enters into the aerated bioreactor tank where it remains in contact with the activated sludge. When the organic content of the wastewater has been aerobically stabilized after a defined period of time, mixed liquor is pumped to the external dry membrane modules at high cross-flow velocity, usually around 2 – 4 m/s (Le-Clech, P, 2010), where is filtered thanks to the created pressure by the flow velocity. Filtered wastewater is extracted from the system cycle, while biomass is returned to the bioreactor. Excessive sludge is pumped out of the system in order to maintain a constant biomass age.

Multi-tubular modules have been typically used in this configuration, which permeation is usually operated from inside to outside the membranes. External MBR are accessible for maintenance operations, have low aeration costs and allow high operational fluxes (between 40 – 120 $L/m^2·h$ (Gili, C, 2008)), featuring high permeate fluxes and relative simplicity in adapting to existing wastewater processes. However, external MBR have very high OPEX costs mainly due to their high energy demand, also requiring frequent cleaning operations. Because of this reason, this configuration rarely has been developed in large scale.

Airlift MBR (AEC-MBR) have been recently developed as an innovative external MBR configuration. This reactor has lower energy consumption compared with conventional cross-flow MBR, using air to recirculate liquid and control membrane fouling (Kojic, P.S., 2017). Being the continuous aeration the main force for the circulation of the activated sludge, feed pump is only used to overcome the hydraulic losses. Turbulence created by the air injection in membranes surface provides continuous mixing that prevents from the biofilm layer accumulation over the membrane walls, reducing fouling phenomena. This increases membranes hydraulic permeability, resulting in higher flux rates of more quality at low transmembrane pressures (pressure at which wastewater is filtered through the membranes, describing how much force is needed to push wastewater, known as TMP) and therefore lower energy consumptions.

Although AEC-MBR are very recent and must continue gaining market strength based on more knowledge and experience, it is proposed as a real alternative to submerged MBR configuration in the long term.

6.3.2. Internal/Submerged MBR

The introduction of the submerged membrane configuration allowed a significant reduction of CAPEX and OPEX costs with respect to the external membrane configuration, resulting in the development of the mostly used MBR second generation. With the membrane directly immersed into the bioreactor, a small negative pressure imposed on the permeate site acts as the driving force for the effluent wastewater to be permeated through the membranes, once the organic content of the wastewater has been aerobically stabilized after a period of time being in contact with the biomass. This way of moving the wastewater inside the system considerably reduces the energy demand, simplifying the system itself. However, submerged MBR operate at lower TMP, achieving lower permeation fluxes. Aeration is used under membranes to provide extra oxygen, keep the biomass in suspension and produce a turbulent two-phase flow velocity (estimated around 0,2 to 0,4 m/s) on the membrane surface in some systems (Le-Clech, P, 2010), which creates a surface shear stress that prevents the biofilm layer accumulation over the membrane walls, reducing fouling phenomena (same principle as the applied in external airlift MBR).

In submerged MBR, usually membranes are configured as vertical flat-plate modules, vertical or horizontal hollow-fiber modules, filtration going from outside to inside the membranes. Membranes can be located inside the bioreactor tank or submerged on a separated tank dimensioned for the membranes accommodation. Unique tank configuration is more efficient for small treatment systems but it not adapts very well to larger treatment systems or more nutrient removal demanding systems, where separating membranes in a second tank allows better performances (Oliveira, F, 2017). Two submerged tanks configuration allows individual optimization for specific requirements of the biological activated sludge and membrane filtration processes, being possible, for example, to optimally apply air in fine bubbles in the bioreactor tank and in thick bubbles in the membrane tank, which allows better membrane fouling control.

Conventional submerged MBR have been much more used than other existing or studied MBR configurations, what has allowed a commercial market growth with a consequent drop in their CAPEX costs. Moreover, they offer more reliability as there is more information about their performances due to both technical and experimental experiences. Because of this maturity, conventional submerged MBR are seen as the most recommended option to nowadays install in a WWTP. Therefore, further discussion about operational criteria and costs will be mainly focused on this kind of systems.

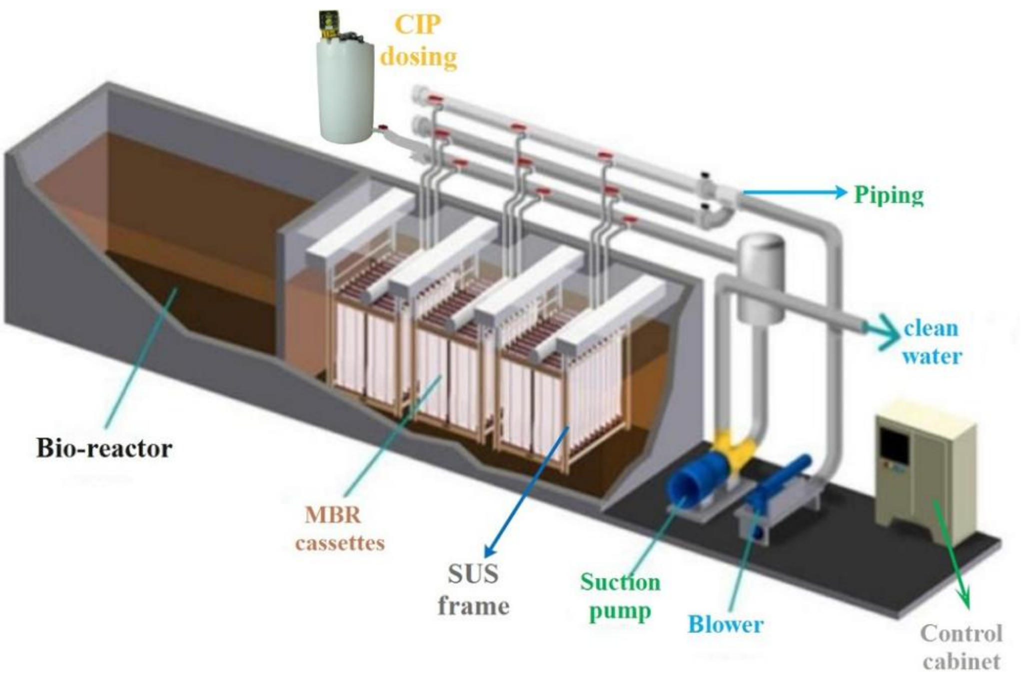

Figure 6.7 Scheme of a submerged MBR separated in two different tanks. Source: Alibaba.

6.4. MBR Operating Conditions

MBR effectiveness depends on the incoming wastewater characteristics, desired effluent characteristics and system designing. Designing an optimal operating MBR system could make it a desirable option over other wastewater treatment options. Operating conditions and variables of which an MBR performance depends on are shown up next. This section is focused on defining those parameters for conventional submerged MBR as it is the most used configuration at the moment and the most recommended nowadays.

6.4.1. Influent Wastewater Pretreatment

All the system design has to consider the incoming wastewater characteristics. The importance of receiving well-treated wastewater into MBR from the membrane filtration performance point of view has been previously discussed. Because of this reason, MBR are usually situated after conventional pretreatment and primary treatment stages in WWTP. However, MBR influent requires proper and efficient pre-screening to remove materials which can tangle around the membrane fibers such as hair, lint or other fibrous materials, by the installation of fine screen size of 0,5 mm. As a result, the increased amount of waste sludge produced has to be considered.

6.4.2. Wastewater Flux

Flux ($L/m^2 \cdot h$) is the velocity at which the wastewater passes through a spatial unit of membrane. As it happens with conventional membrane filtration, is a fundamental design parameter, indicating productivity. Since fouling rate increases exponentially with the flux, to achieve operational sustainability conventional submerged MBR should have modest net fluxes (flux based on a performance over a complete cleaning cycle) between 15 and 25 $L/m^2 \cdot h$, preferably below what is known as the critical flux, resulting in TMP of generally less than 50 kPa (Le-Clech, P., 2010).

Traditionally, there have been two ways to operate MBR regarding operating flux, which determine the cleaning needs and therefore the net flux. On the one hand, operating conditions can be chosen to maintain a sustainable permeability operation (little or negligible TMP increase at constant flux) during a long period of time with only moderate preventing and control measures of fouling (generally relaxation). Most submerged flat-plate MBR and external MBR have traditionally operated under these conditions. On the other hand, operating conditions can be chosen to have intermittent operations, what allows a higher operational flux which can be sustained by filtration cycle operations and, as a result, employing intermittent measures as relaxation supplemented by backwashing and usually some kind of chemical cleaning procedure. Traditionally, submerged hollow-fiber MBR operate intermittently.

Anyway, as higher the flux is, higher the rate of contaminants blocked by membranes, requiring maintenance processes more usually, what decreases productivity. Therefore, contamination in incoming wastewater highly affects the optimal flux of the MBR installation, what shows the need of pre-treating well the influent. Temperature affects not only the viscosity of the wastewater and therefore the flux, but also the bacterial activity performance during the activated sludge process. Membranes pore size is also an important element determining flux. However, given the initial permeability obtained with MF on one hand and more easily recoverable fouling in UF on the other hand, optimization of the pore size remains a difficult task in MBR (Le-Clech, P., 2010). Conventional submerged MBR are generally proposed with a pore size between 0,02 and 0,5 µm (Le-Clech, P., 2010).

The greatest impact on operating versus net flux is the peak loading, often from storm waters if no flow balance is provided. Most of the MBR suppliers allow their systems to be operated at high fluxes (up to twice the normal value) to face with potential peak loadings. However, these periods of high permeability are usually limited to 1 or 2 hours, requiring increased aeration and extended relaxation periods at lower flux to allow the accumulated fouling to be physically removed (EPA, 2007).

6.4.3. Solid and Hydraulic Retention Times (SRT; HRT)

Main operating conditions in MBR systems include solid retention time (SRT) and hydraulic retention time (HRT). They indicate, on the one hand, the average time the activated sludge solids are in the system and, on the other hand, the average time the influent wastewater is in the system. They are related to the quantity and quality of the wastewater to be treated and have a strong influence on the nature of the activated sludge. One of the main advantages of the MBR systems is that they can decouple SRT and HRT. This intricate relationship is still not totally understood as the biomass characteristics are extremely complex to fully quantify and qualify. Both parameters are directly related to the bioreactor volume and MBR OPEX costs (Le-Clech, P., 2010)..

Long SRT are applied in conventional submerged MBR (usually between 20 and 40 days). This results in higher mixed liquor suspended solids (MLSS) concentration (optimal between 9 g/L to 15 g/L in a conventional submerged MBR) and food to microorganisms ratio (F/M) can be between 0,05 - 0,15 day^{-1} (Le-Clech, P., 2010). As a consequence, much less wasting sludge is produced during the biological process with respect to a conventional activated sludge treatment, eliminating important operational costs (see chapter 4.4). As higher the SRT is, higher MLSS is, demanding higher aeration. However, treatment efficiency is not linearly dependent on biomass concentration, because the specific biological activity can be reduced at substrate deficient states (Giordano, C., 2007). Long SRT avoid nitrifying bacteria from being washed out from the bioreactor, improving the nitrification capability of the activated sludge. A minimum SRT of 5 days is necessary to ensure complete nitrification.

Increasing HRT also favors the occurrence of nitrification, but in the other hand, as lower HRT is, less sedimentable the sludge is, increasing membranes resistance which produces higher solid retentions and, consequently, better removal efficiencies, despite flux reduction. Therefore, operating with low HRT is desirable as the permeate quality increases and the space needed for the system is reduced. However, the membrane resistance is increased because of higher fouling rates. Another important parameters related with HRT are the food to microorganisms ratio (F/M) and the stabilization of the volatile suspended solids (VSS). MBR can be operated at relatively low HRT, generally between 2 to 8 hours in conventional submerged MBR (Viero, A., 2008).

6.4.4. Nutrients Removal

Nutrient removal is seen as an important issue in wastewater treatment depending on where the wastewater will be reused. It has been previously seen that operating a conventional submerged MBR with long solid retention times (SRT) allow nitrifying bacteria not being recirculated out of the bioreactor, achieving the nitrification process. Denitrification can be done by the use of intermittent aeration during the activated sludge process, alternating aerobic and anoxic time phases. Intermittent aeration allows complete nitrogen removal in one tank, optimizing space needed with respect to other nitrogen removal possibilities. In these systems, nitrogen elimination depends on the aeration control and can reach up more than 90% of removal (Kraume, M., 2005). Therefore, designs of aeration strategies to optimize nitrogen elimination are needed.

Pre-denitrification is another way to achieve denitrification in conventional submerged MBR, achieved by installing an anoxic tank before the aerated tank, following the same principle of the Ludzack-Ettinger modification of a conventional activated sludge process (see chapter 5.1.1). Many MBR denitrification is done by pre-denitrification because of two main reasons: firstly, biodegradable organic matter available in the anoxic zone improves denitrification rates, reducing the required aerated tank volume, and secondly, the oxidation capacity of nitrate degrades part of the organic matter, reducing oxygen demand and therefore the energy demand during the aeration process. However, nitrogen removal is limited to 75-90% (Kraume, M., 2005) in these systems because of its dependence on the recycling ratio of nitrate produced by nitrification transported from the aerated tank back to the anoxic zone.

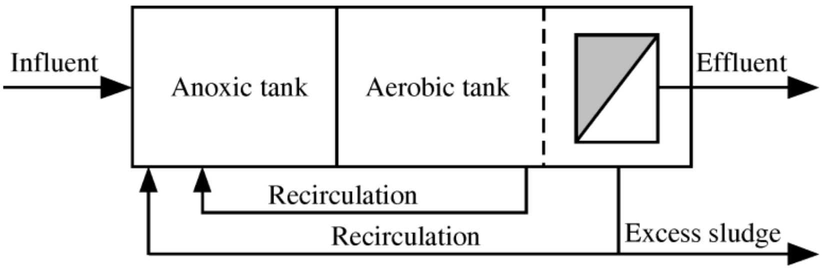

Figure 6.8 MBR pre-denitrification process scheme.

In Post-denitrification, the obvious order of nitrification and denitrification takes place. The elimination rate is therefore not limited by the recycling ratio. Since most organic matter has been degraded already in the aerated zone, in most of these systems a carbon dose is added into the anoxic tank to achieve higher denitrification rates. Despite pre-denitrification have been mainly used over intermittent denitrification or post-denitrification, some characteristics can make post-denitrification an attractive alternative over pre-denitrification: higher nitrification rates (over 90%), savings in the energy and equipment used for the nitrate recycling loop, and better biomass redistribution due to the sludge recirculation pattern over the entire reactor volume (less sludge in contact with the membranes, while more sludge is present in the anoxic zone). For these reasons, post-nitrification is identified as a promising configuration in conventional submerged MBR when high percentages of nitrogen removal are required. However, higher space is demanded with respect to pre-denitrification or intermittent denitrification.

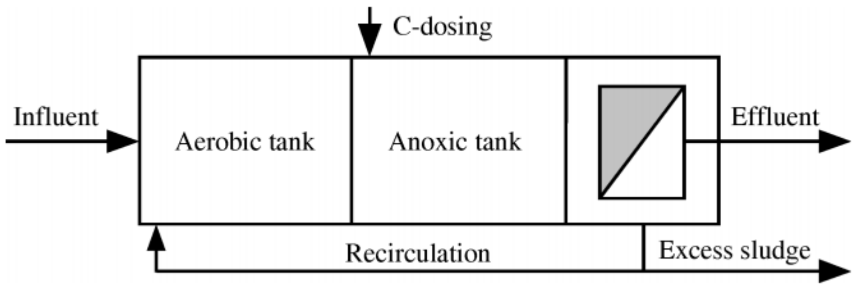

Figure 6.9 MBR post-denitrification process scheme.

As it happens with nitrogen, longer SRT increase removal rates of the other main nutrient, phosphorus. However, several studies and experiences have shown difficulties to achieve high phosphorus removals (usually between 50 – 70%) in conventional submerged MBR operating with long SRT, even with intermittent aeration. In order to achieve removal levels over 90%, it is necessary to add a chemical coagulator, usually aluminum sulfate, during the activated sludge process that will transform phosphate into a hardly soluble precipitating salt. Another possibility is to remove phosphorus biologically by installing an anaerobic reactor tank in the MBR system, usually before the aerobic bioreactor, which will serve as an environment for fermentation and survival of the phosphate accumulating organisms (PAO) built under aerobic conditions. The phosphate is removed by extracting the excessive sludge of the system, so a constant sludge extraction is necessary. Under controlled conditions, phosphate concentration could remain below 0,1 mg/L in the MBR effluent (Le-Clech, P., 2010).

6.4.4. Aeration

Oxygen is required in aerated MBR to maintain the existing biomass alive and to degrade biodegradable contaminants contained in the wastewater. More oxygen is used for the membranes scouring, a necessary process to prevent and control fouling, which is the main problem in membrane filtration. Air scouring for the membranes and aeration of activated sludge are the major components of energy consumption, accounting for 60-80% of the energy consumption of the biological treatment step in a conventional submerged MBR system (Barillon, B., 2013). As an important point, it has been shown by several studies and experiences that the size of the bubbles has a great impact in aeration performance: the activated sludge process performs better with small bubbles, while fouling is better prevented with big bubbles. Therefore, the size of bubbles must be considered and operated in two different sizes if the chosen MBR system allows doing so.

Air scouring appears to be an interesting option for an energy consumption reduction, especially for flat membrane technology, for which specific energy demand is approximately 30% higher than in hollow fiber systems (Barillon, B., 2013). Reducing SRT would reduce the biomass concentration in the bioreactor, what would reduce fouling and, therefore, the energy demand in terms of aeration in membranes to prevent it. However, one of the main advantages of MBR is the ability to operate with high SRT, which offers several advantages discussed before. So, long SRT should be used and air scouring optimized under this conditions. It will be important in optimization terms to properly adjust membrane aeration proportional to flux, taking into account both small and large aeration intensities have a negative impact on permeability, as low aeration intensities do not effectively remove membrane foulants, while excessive aerations dissolve sludge flocs, which increases pore blocking. Maintaining an equilibrated air distribution over the membrane modules is also an important issue, achieved by installing well distributed air diffusers and controlling clogging phenomena on them. Installing variable speed drives on the electromechanical equipment would enable to implement new control strategies to reduce energy costs not only in air scouring, but also on aeration for the biological process or in recirculation, another significant energy consumer (comparable to air scouring, accounting 20-22% of the energy consumption in MBR (Barillon, B., 2013)).

Properly aerating the bioreactor tank during the activated sludge process is a fundamental issue in MBR performance. The floc size and concentration are two of the main factors influencing the optimal quantity of oxygen that has to be introduced for the biological degradation. The α factor (rate of oxygen transfer in clean water divided by the rate in mixed liquor) is generally used to determine the level of oxygen transfer in MBR. It is known that the level of MLSS has a negative impact on the α factor, limiting the operation of MBR under elevated SRT. An average α value of 0,5 is considered acceptable (Barillon, B., 2013). On the other hand, despite the impact of floc size on oxygen transfer is not so well defined, higher volumetric removal rates are observed when the floc aggregations are smaller.

Apart from being an option that allows occurring nitrification and denitrification simultaneously in the same tank, the introduction of intermittent aeration has also been one of the main improvements in energy consumption. Specifically, limiting aeration for 10 seconds every 20 seconds (“10:10” aeration) or every 40 seconds (“10:30” aeration) are two of the common intermittent aeration systems. Some studies have used intermittent aerations of “10:10” and “10:30” operating a conventional submerged MBR system under same flux and temperature conditions, showing a decrease in fouling rates as lower the limitation of aeration is. Using intermittent aeration results in lower fouling rates compared to continuous aeration when working under the same overall airflow rate. A study shown that a flux of 23,3 $L/m^2 \cdot h$ was sustained operating under “10:30” conditions with a low specific aeration demand per unit of permeate produced (SAD_p) as 4,6 without significant impacts on fouling rate, with associated membrane aeration energy savings of up to 75% compared with continuous aeration (Bart, V., 2011). However, higher intermittent aeration cycles of “10:60” are not able to sustain high flux rates, what would increase CAPEX costs of the system by demanding more membrane filtration area. Therefore, an intermittent aeration of “10:30” is seen as an interesting operating strategy for conventional submerged MBR in order to reduce OPEX costs coming from an important reduction in aeration energy demand. However, it must be studied if denitrification is produced taking into account the incoming wastewater characteristics if a complete nitrogen removal is desired.

6.4.5. Clogging

Clogging is the agglomeration of solids within or at the entrance to the membrane channels. Although this phenomenon impact is identical as the fouling one (decreasing membrane permeability), clogging must be distinguished from membrane surface fouling with respect to its mechanisms, control and prevention, which may be useless to prevent clogging. Severe clogging is generally only fixed up by removal of the membrane from the tank and cleaning the membrane modules individually with a low pressure hose. This kind of manual interventions represent a risk compromising the integrity of the membranes.

The major measure to prevent clogging consists in the installation of a fine screen to introduce an incoming wastewater pretreatment before entering into the MBR system. As it has been previously explained, this measure prevents the entrance of solids that would produce an increase not only in membrane surface fouling, but also in clogging. Limiting the MLSS concentration is another way to control clogging, especially in hollow-fiber MBR. It can also be normally attributed to local region of high fluxes (> 40 $L/m^2 \cdot h$) or inadequate air scouring (Gkotsis, P., 2014). It has been shown that reduced aeration increases clogging. The air distribution is an important concern, being important for this phenomena prevention to install some of the air diffusers at the lower part of the membranes, where it is especially important to provide a satisfactory aeration. Fouling or clogging may also be produced in the air diffusers, what would decrease the air distribution and therefore increase clogging. Strategies used to limit clogging of aerators include regular air or liquid flushing and good monitoring of the MLSS concentration.

6.5. Fouling

Fouling is the phenomena that imply a reduction of flux at a constant operational pressure because of undesirable deposition of materials onto the membranes surface. Apart from reducing system productivity, it affects pretreatment needs, cleaning requirements, operating conditions, costs and performance. It is considered as the main operational problem in MBR systems. For this reason, fouling phenomena is going to be analyzed deeply in a separate section of the other operating conditions.

The major effect of membrane fouling, pore constriction, occurs when particles small enough to enter the pore but, once inside, they chemically adhere to the membrane material becoming deposited on the walls of the membrane pores, reducing the overall pore volume and increasing the resistance to flow. Pores can also be blocked by particles which have diameters approximately equal to the pore, disabling the water flow through the pore. Moreover, as the wastewater permeates through the membrane, there is a gradual increase of non-permeating or slow-permeating particles on the feed side of the membrane, generating a biofilm layer that decreases membrane effectiveness. Consequently, as more contaminated the incoming wastewater is, faster the membranes will become fouled. Fouling formation in MBR has been divided in three stages (Judd, S., 2005):

- Conditioning fouling: The initial conditioning stage is defined by strong interactions between the membrane surface and possible foulants. Colloid adsorption or pore blocking onto new or cleaned membranes is produced. Contribution of conditioning fouling to membrane resistance has been found negligible when filtration takes place. However, as some flocs roll slide across the membranes surface, biomass which gets closer to the membrane surface is then able to attach more easily, contributing to second stage.

- Steady fouling: Since adsorption can take place across the whole membrane surface (not just on the membrane pores) biological flocs may initiate an irregular biofilm layer formation without directly affecting flux. Complete or partial pore blocking takes place.

- Transmembrane pressure (TMP) jump: With regions more fouled than others, permeability is significantly less in those locations. As a result, permeation is promoted in less fouled areas, exceeding a critical flux in those locations. Under such conditions, fouling rapidly increases and therefore TMP suddenly increases as well.

Key parameters influencing fouling in a continuous submerged MBR are the influent wastewater properties, membrane characteristics and operating conditions (flux, SRT, HRT, aeration, etc.). All those parameters are discussed in terms of fouling prevention and reduction. Fouling prevention and control strategies are also analyzed.

6.5.1. Influent Wastewater Properties

The importance for MBR to receive a well-pretreated incoming wastewater has been previously discussed. For this reason, these systems are located after conventional preliminary and primary treatment in WWTP. Moreover, it has also been seen the need of installing a fine screen at the entrance of the MBR system to remove materials which can tangle around the membrane fibers and other particles that may increase fouling.

6.5.2. Membrane Characteristics

Membranes Surface

Several of the surface characteristics of the membranes are known to be strongly related to fouling, determining the interaction between the membranes and foulants. Membranes with hydrophilic surfaces are less susceptible to fouling. Due to the formation of hydrogen bonds, a thin layer of bounded water exists on the surface of a hydrophilic membrane, reducing foulants adsorption or adhesion on the membrane surface (Gkotsis, P., 2014). As a consequence, commercial membranes are usually made of hydrophilic polymers. Therefore, increase hydrophilicity of the membrane surface is a key goal to prevent fouling. Coating a thin layer of water-soluble polymers or surfactants from solution is a flexible technique to optimize hydrophilicity, smoothness and charge of the membrane surface (Gkotsis, P., 2014). Coating can be done by physical adsorption, by filtration or by casting. Ultraviolet-initiated graft polymerization is another technique to modify membranes surface by adding different hydrophilic monomers to develop low-fouling polymer membranes. Graft polymerization can also be done by a redox system composed of potassium persulfate and potassium metabisulfate, what gives the possibility of modifying the polymer membranes in situ.

The charge of the membrane is also an important characteristic. As the foulants are usually charged, it is appropriate to use a membrane carrying the same electrical charge as the foulants. Various chemical reactions may be used for creating different charged organic compounds on the membrane surface, such as $–SO_3$ or CO_2H (Gkotsis, P., 2014).

Membranes with a patterned surface morphology (such as pyramid or prism-patterned) have become interesting to mitigate membrane fouling, as several studies have shown that the patterns induce secondary flows at the membrane feed interface, creating higher local shear stresses near the apex of the forms, reducing fouling.

Membrane surface modification by adding nanoparticles have received special attention during the last few years. Depending on the type of nanoparticle added, hydrophilicity, charge or roughness membrane surface characteristics can be modified and potentiated. Two different methods are used for introducing nanoparticles in membranes: by depositing nanoparticles on the membrane surface by dipping the porous support in an aqueous suspension of nanoparticles, or by entrapping nanoparticles in a polymer matrix thanks to a phase inversion method that adds the nanoparticles to a casting solution (Gkotsis, P., 2014). Despite having big effects on fouling reduction shown by several studies, further research is needed to investigate the combined effects of the wastewater chemistry and the nature of the nanoparticles. Also, careful control and monitoring of the nanoparticles released from the modified membranes are necessary to minimize potential environment toxicity effects.

Modules Disposal

Regardless the membrane element and therefore the kind of module being used, it has been experimentally established that packaging modules with lower densities produces less fouling. However, more space is going to be needed for the system, sacrificing one of the main advantages of the MBR. A balanced decision between both parameters must be chosen to decide membrane modules density. Modules designed to improve lateral flow or movement of the membranes are known to improve their performance with respect to fouling. Membrane modules position is also important. Some studies revealed that MBR with hollow-fiber modules in a vertical orientation experienced 25% more membrane surface shear in filtration zone than horizontally oriented hollow-fiber modules at same aeration intensity, also founding that the addition of baffles in membrane modules promotes turbulence and shear in the upper section of the module (Ebrahim, A., 2017).

Membranes Movement

Membranes movement, generally achieved by vibration, improves fouling reduction with a low energy consumption. Membrane elements which allow longitudinal, transversal or rotational vibrations are therefore advantageous in this sense. Key parameters are the amplitude and frequency of vibration. Vibration is more effective for the biofilm layer removal than for pore adsorption or blocking mitigation. Transverse vibration decreases more the fouling rate than longitudinal movement in hollow-fiber membranes. A study showed that adding transverse vibration to longitudinal vibration results in almost doubling critical fluxes at same frequency in hollow-fiber membranes, although floc breakup at higher frequencies tended to reduce the critical flux (Ebrahim, A., 2017).

Greater movement can be achieved by using looser membranes. However, a study shown that increasing the looseness from 0% to 1% decreased the fouling tendency, whereas increasing the looseness from 1% to 2% increased fouling tendency (Ebrahim, A., 2017). This suggests there is an optimum membrane looseness. It is also known that too much looseness can lead to membranes breakage.

6.5.3. Operating Conditions

Being the main operational problem in MBR, fouling directly or indirectly affects most of the other operational parameters of the system. Therefore, previous considerations in order to prevent fouling in terms of flux, SRT, HRT and aeration have to be taken into account in order to achieve an optimal system performance in terms of OPEX costs, productivity and effluent quality.

6.5.4. Fouling Maintenance, Prevention and Control Strategies

Numerous maintenance strategies are used to prevent and control fouling in MBR. The most common methods include the application of conventional physical or chemical cleaning, modifying chemically the mixed liquor by adding specific chemicals such coagulants, the application of the quorum sensing method (QS) or the application of ultrasound, electric field or ozone.

Backwashing

Backwash process is designed to remove contaminants accumulated on the membranes. It is similar in principle to granular filtration backwashing (see chapter 5.2.2). Each membrane unit is backwashed separately and in an order pattern to minimize the number of units in simultaneous backwash. It is used in MBR systems which operate intermittently. Two modes of operation are usual: using a fixed cycle time where backwashing is done after a designed filtration time, causing an increase of TMP with each cycle if residual fouling occurs, or operating to achieve a maximum TMP value which shows the end of the cycle and the need of backwash the system, where lower cycles are produced as long the system is operating if residual fouling occurs. Anyway, backwashing is effective reducing TMP, but some deposited foulants tend to remain attached to membranes surface producing residual fouling in subsequent cycles. Therefore, cycling between backwashing and chemical cleaning is a common practice to reduce residual fouling.

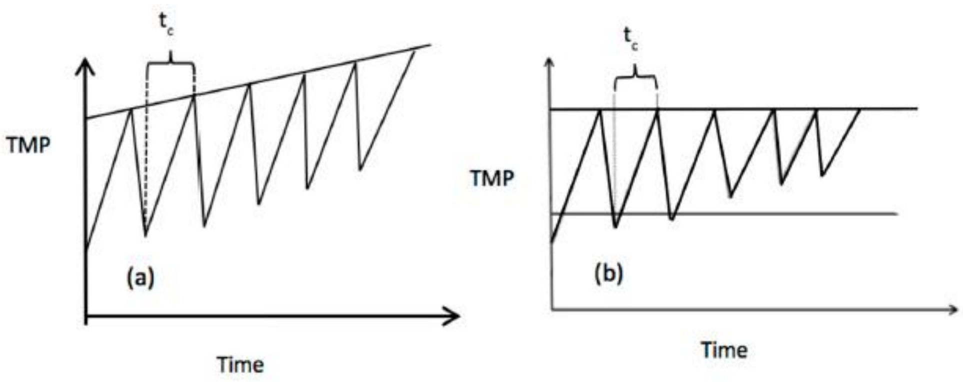

Figure 6.10 Typical profiles with intermittent backwashing (Ebrahim, A., 2017). (a) Operation to achieve a fixed cycling time; (b) Operation to achieve a maximum TMP value.

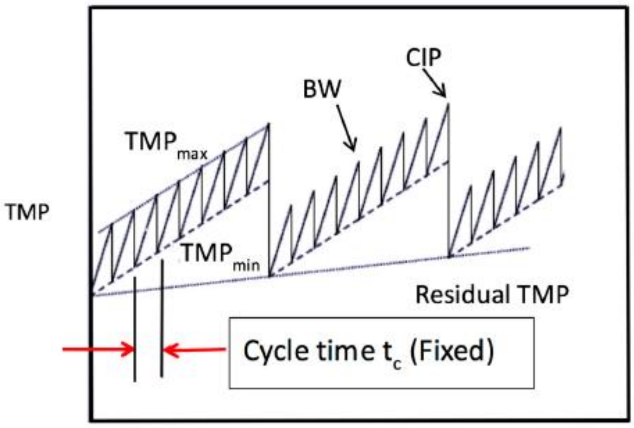

Figure 6.11 Typical profile of the TMP as a function of time showing the effect of intermittent backwash and chemical cleaning cycles both with different fixed cycling times (Ebrahim, A., 2017).

Key backwashing parameters are duration, frequency and backwash flux. Although more effective cleaning would generally seem to be achieved by more frequent, stronger and longer backwashing, all three parameters need to be optimally configured between them to avoid increasing the energy consumption as well as other problems that have been detected by several studies. Less frequent, longer backwashing (600 seconds of filtration and 45 seconds of backwash) have been found to be more efficient than more frequent but shorter backwashing (200 seconds of filtration and 15 second of backwash) (Jiang, T., 2005). However, excessive backwash duration and strength results in permeate loss, higher energy consumption and more irreversible fouling due to severe pore blocking and a more compact biofilm layer creation, suggesting there are both optimal backwash flux and duration for fouling mitigation (Ebrahim, A., 2017). Increased backwash flux is more effective than increased backwash duration using the same backwash volume (Ebrahim, A., 2017). Another study found that backwashing frequency (between 8 and 16 minutes) has more effect on fouling removal than either aeration (0,3 – 0,9 $m^3/m^2 \cdot h$) or backwash duration (25 – 45 seconds) for hollow-fiber submerged MBR (Gkotsis, P., 2014). Energy minimization can be achieved through the design of a generic control system with automatically optimized backwash duration according to the registered TMP value.

Air scouring can also be used to improve backwashing. Up to 400% increase in the flux over the achieved with continuous operation has been recorded using air backwashing, although in this case 15 minutes of backwashing were required every 15 minutes of filtration (Gkotsis, P., 2014). Even if air backwashing is undoubtedly effective, some evidences showed it can produce problems on membranes integrity because of partial drying. Moreover, it has been seen that high air flow rates during backwashing do not improved the fouling reduction, recommending moderate flow rates for optimal results.

Operation without backwashing, although increases the risk of accumulation of foulants on or within the membrane, inversely also preserves the biofilm layer on the membrane surface, less permeable and more selective than the membrane itself, which can become a measure of protection for the membrane surface if the resistance it offers does not become excessive.

Relaxation

The intermittent suspension of permeation has been incorporated in many MBR as a standard physical measure that promotes foulants transport back away from the membrane surface. This is further promoted by the shear created by air scouring. Even needing periods of higher fluxes to give the same permeate production as it provides a continuous operative mode, relaxation is still beneficial for the system as lower TMP are achieved, what allow higher permeations with lower energy demands and longer chemical cleaning cycles. Relaxation is almost present in modern full-scale submerged MBR and studies of maintenance protocols have tended to combine relaxation with backwashing to achieve optimal results. The protocol of intermittent filtration/relaxation has to be optimized in terms of the duration and interval ratio of each cycle to be more beneficial for fouling removal and retarding the TMP increase during filtration. Relaxation is typically applied for 1 – 2 minutes every 8 – 15 minutes of operation (Ebrahim, A., 2017).

Chemical Cleaning

Physical cleaning is supplemented with chemical cleaning to remove residual and irreversible fouling using acids, bases, oxidants or surfactants. Chemical cleaning can be classified into four categories (Ebrahim, A., 2017):

- Clean-in-place (CIP): Involves directly adding chemicals to the membranes.
- Clean-out-off-place (COP): Involves cleaning the membranes in a separate tank with a higher concentration of chemicals.
- Chemical washing (CW): Involves adding chemicals to the influent wastewater.
- Chemically enhanced backwash (CEB): Involves combining chemical and physical cleanings.

Key factors for chemical cleaning affecting fouling mitigation are the type of chemical agents, cleaning duration and interval, chemical concentration, cleaning temperature and flux. The type of chemical used depends mainly on the application, influent wastewater characteristics (pH, temperature and ionic strength) and the compatibility of the membrane material with chemicals. Sodium hypochlorite (NaCOl) and citric acid are the most common chemical cleaning agents because of their availability, relatively low price and high cleaning efficiency, although they are ineffective for removing iron species and less effective than coupling NaCOl and caustic soda for removing natural organic matter (Gkotsis, P., 2014). Unfortunately, such products are one of the main causes of deterioration in membrane integrity, and prolonged exposure causes oxidative damage to membranes. Therefore, it is important to smartly apply chemical cleaning to the MBR system to optimize its productivity and lifetime.

Chemical cleaning tends to be a combination between maintenance cleaning at moderate chemical concentrations of 200 – 500 mg/L NaOCl every 2 – 4 weeks in cycles of 30 – 120 minutes, designed to remove residual fouling usually in situ, and intensive or recovery chemical cleaning at high chemical concentrations of 0,2 – 0,3 wt% NaOCl (weight percentage) coupled with 0,2 – 0,3 wt% citric acid once or twice a year to remove irreversible fouling, usually ex situ or in the drained membrane tank to allow the membranes being soaked into the cleaning agent (Gkotsis, P., 2014).

Chemical Mixed Liquor Modification

Biomass quality can be controlled biochemically by adjusting the SRT or by the addition of chemicals. In practice, SRT is rarely chosen based on fouling control, more focused on achieved a desired effluent quality, sludge production rate, clogging propensity or biomass aeration efficiency.

Ferric chloride and aluminum sulfate (alum) are coagulants commonly used to reduce fouling in MBR. Although it is more expensive, dosing with ferric chloride was found to be more effective for fouling control than dosing with alum. Pre-coating MBR membranes with ferric chloride has also been shown to increase permeability and improve permeate quality.

Addition of adsorbents into MBR systems is another way to chemically modify the mixed liquor that achieves fouling reduction. Solid particles, commonly powdered activated carbon (PAC) or granular activated carbon (GAC), are dispersed and suspended by the mixed liquor in contact with the membrane surface, absorbing and degrading the present biofilm layer. Those particles also have a mechanical action that both vibrate and clean the membrane surface. Optimal PAC dosages have been identified between 1 – 2 g/L, as higher flux deterioration was performed during the later operational phase dosing 3 g/L (Gkotsis, P., 2014). Although PAC is more effective than GAC in terms of adsorption capability, studies have shown better efficiencies of GAC at higher concentrations in the long term (Ebrahim, A., 2017). Under the same operating conditions, adding a coagulant provided higher fouling mitigation than PAC addition. However, the use of both strategies operating together provided the greatest permeability improvement. The benefits of adsorbents include low costs and the possibility of operating continuously.

Quorum Sensing (QS)

Bacteria use the language of small signaling molecules called autoinducers to communicate and asses their population density in a process called quorum sensing (QS). QS has been shown to regulate the bacterial behavior and the production of extracellular polymeric substances (EPS), one of the main foulants. A study used acylase attached to magnetic carrier to inhibit QS in a MBR, showing a fouling reduction and improving the membrane permeability (Gkotsis, P., 2014). Since then, this fouling control technique has attracted a lot of attention and it is seen as a promising alternative for fouling control. However, it needs to be more studied to especially see its efficiency in full-scale MBR.

Ultrasound

Ultrasound proved to be able to improve membrane permeability by reducing fouling in a cross-flow filtration. The improvement of flux depends on the ultrasound intensity, irradiation radiation and direction. As higher the sludge concentration is, longer the ultrasound radiation time should be. However, some tests suggested that it can result in adverse impacts on membrane integrity .

Electric Field

Applying an electric field has shown significant effects on the membrane flux in MBR, allowing the sedimentation layer become thinner and a filtration resistance drop. A study demonstrated that employing a very low electric field (0,2 V/cm) resulted in improved performance both in terms of fouling control and effluent quality (Gkotsis, P., 2014). In another study of a MBR system filtrating in a cross-flow mode an electric field was applied intermittently to the membranes during the filtration progress, being very effective removing negatively charged foulants what allowed maintaining high permeate fluxes (Gkotsis, P., 2014).

Ozonation

Ozonation is also a technique that allows fouling reduction in MBR. A study carried out with two MBR systems operating in same conditions, one of them with ozone addition, showed ozonation to be able to effectively retard membrane fouling and prolong chemical cycle by 1 – 2 times in long term running MBR with dosage of 0,25 mg/g SS at 1 day intervals (Gkotsis, P., 2014).

Feedback Control Systems

Given the constant variations of the incoming wastewater, biomass nature and the temporal development of the fouling layer on the membrane surface, for any MBR system a pre-determined operating mode is going to be sub-optimal for at least some of the time. Feedback control systems have been proposed to optimize the use of fouling maintenance, prevention and control strategies in MBR by an adjustable control. Based on a polynomial model calibrated by consecutive cycles or on a permeability drop, control systems developed have resulted in a reduction of backwash durations between 25 to 50% (Gkotsis, P., 2014). Another relatively simple on-line method involved the combined monitoring of permeate flow rate, TMP and temperature to determine permeability and optimize the maintenance process. More complex systems, taking into account the impact of biofilm growth or the pressure drop in the permeation line have been successfully designed, although they still require extensive calibration. Successful application of control systems is only possible if inputs and outputs are properly defined. Outputs include control of the permeate pump, relaxation frequency, backwash frequency or membrane aeration rate.

6.6. MBR Performance

MBR systems have been presented as an alternative treatment that combine the activated sludge process with the membrane filtration in a technology able to achieve excellent effluent qualities, able to substitute both conventional secondary and tertiary treatment systems from most of wastewater treatment lines. Filtration membrane materials and modules and how they can be configured in MBR systems have been also presented and discussed, recommending the submerged MBR because of its lower OPEX costs with respect to external MBR. From that point of view, the fundamental operational criteria have been deeply discussed, focused on obtaining an optimal operation of a conventional submerged MBR. Different performances, both theoretical and experimental, of conventional submerged MBR are now presented as well with the selected operational criteria to achieve the obtained effluent quality in order to discuss when and how these systems can effectively substitute conventional advanced wastewater treatments.

6.6.1. Standard MBR Effluent Quality

Despite the effluent quality will depend on the incoming wastewater characteristics and the MBR operational design, a conservative standard MBR effluent quality should be achieved if the system is properly working installed after conventional preliminary and primary treatments. Note that to achieve both nitrogen and phosphorus removal rates it is necessary to apply any of the nutrient removal techniques reviewed during the 6.4.4 chapter.

Table 6.2 Conservative standard MBR effluent quality (own elaboration based on different bibliographic references).

Parameter	Concentration
BOD [mg/L]	< 5
SS [mg/L]	< 1
Turbidity [NTU]	< 0,2
Total Nitrogen [mg/L]	< 5
Total Phosphorus [mg/L]	< 0,5
Intestinal Nematodes [egg/10 L]	< 1
Fecal Coliforms [CFU/100 mL]	< 50

Comparing the conservative standard MBR effluent quality with the effluent qualities given by the proposed advanced treatment lines during the chapter 5.4, it can be seen that MBR obtains an effluent even better in terms of BOD, SS, turbidity and *Escherichia Coli* than the best of the proposed advanced treatment line that was obtaining the A quality of treated wastewater for reuse. That line consisted in a conventional secondary treatment followed by coagulation-flocculation, granular filtration, disinfection and maintenance disinfection if UV radiation was the chosen disinfection method, which would be all substituted by the MBR system installed after the primary treatment. Final maintenance disinfection is recommended after the MBR in order to avoid microbial population re-growth. Therefore, the obtained treated wastewater by an MBR system is able to be reused in all of the urban, agricultural, industrial, recreational or environmental reuses where the absence of *Escherichia Coli* is not required.

Fecal coliforms presence is the limiting factor to reuse MBR treated wastewaters in all possible reuses regulated by the spanish legislation. Despite the conservative standard MBR effluent quality provided shows obtained fecal coliforms concentrations under 50 CFU/100 mL, severe studies and experiences have shown conventional submerged MBR performances obtaining less than 2 CFU/100 mL and even the absence of *Escherichia Coli* in MBR effluents. Therefore, depending on the performance of the system in this term it will be possible to obtain the absence (or minimal values that may be considered as an absence) of fecal coliforms that allow reusing treated wastewater in all the possible uses. However, applying a disinfection technique after the MBR would add a second barrier for pathogens increasing the security in terms of fecal coliform removal, recommended if it is wanted to reuse the treated wastewater into a use that demands the absence of *Escherichia Coli.*

6.6.2. Performance Study of Four Commercial MBR systems

An interesting study of four commercial available MBR systems was developed by DeCarolis and Adham in 2007 operating at the pilot scale to investigate and compare their performances under the effect of various operating conditions during 16 months. Two types of influent where evaluated: first one, consisting in a typically medium concentrated wastewater (see chapter 2.2), and the second one consisting in a preliminary and primary treated effluent of the same wastewater. HRT ranged from 2 to 6 hours, SRT from 11 to 40 days, permeate flux from 26 to 41 $L/m^2 \cdot h$ and MLSS concentration from 9 to 14 g/L. The productivity of each MBR system was monitored by evaluating the decline in specific flux and increase in TMP or permeate vacuum pressure during operation at constant flux. To account for changes in viscosity caused by variations in the influent wastewater temperature, all specific values where corrected to 20ºC. The membrane modules used in each pilot were equivalent in size, configuration and material as those used in full-scale applications. The four commercial MBR systems where the following (DeCarolis, J., 2007).:

- Kubota MBR system: Two vertical cassettes of 100 MF flat sheet membranes of a 0,4 µm pore size made of chlorinated polyethylene for a total membrane area of 160 m^2 with a flow capacity of 66,5 L/min. This system consisted of a rotary brush prescreen of 3 mm, equalization storage tank and a process tank, divided in a denitrification anoxic zone of 6,4 m^3, a preaeration zone of 2,4 m^3 and a nitrification aerobic zone of 10,1 m^3 (where both cassettes were submerged in a double-deck configuration) for a total volume of 18,9 m^3. Influent entered into the anoxic zone, then pumped to the preaeration zone with fine bubble aeration, from where the mixed liquor flowed by gravity to the aerobic zone, where thick bubbles provided membrane scouring. MLSS was recycled from the aerobic zone to the anoxic zone at approximately 4 times the permeate rate to enable denitrification. At intervals of 9 minutes the permeate flow stopped to allow a period of 1 minute of membrane relaxation as a preventive fouling measure. Note that supplier recommends a design of 24,9 $L/m^2 \cdot h$ of flux, 1,6 m^3/min of membrane air scour flowrate, a vacuum pressure of less than 20 kPa and a pH range between 5,8 to 8,6.

- US Filter MBR system: Four vertical modules of approximately 2000 MF hollow-fiber membranes of a 0,2 µm pore size, 1,5 m length and made of PVDF for a total membrane area of 37 m^2 with a flow capacity of 15,1 L/min. The system included a rotary drum prescreen (wedge wire with openings of 1 mm), an aerobic tank of 9,8 m^3, a membrane tank of 0,3 m^3 and a permeate storage tank of 0,4 m^3. During the operation, air and MLSS were continuously injected to the bottom of the membrane tank to scrub and shake the fibers. Frequent backwashing every 12 minutes was performed, beginning with 45 seconds of a relaxation period followed by 15 seconds of backwash from inside to outside the membranes using effluent. Note that supplier recommends a design of 24,4 $L/m^2 \cdot h$ of flux, 0,24 m^3/min of membrane air scour flowrate, a vacuum pressure of less than 50 kPa and a pH range between 2 to 11.

- Zenon MBR system: Three vertical modules of approximately 2700 UF hollow fiber membranes of a 0,1 µm pore size, 1,7 m length and made of PVDF for a total membrane area of 67 m^2 with a flow capacity of 28,4 L/min. System consisted in rotary drum prescreen (0,8 mm perforation), an aerobic tank of 10 m^3 and a membrane unit of 0,8 m^3. Influent entered in the aerobic tank, where fine bubble aeration provided adequate dissolved oxygen for biological oxidation. Thick bubble aeration provided membrane scouring in the membrane tank, being cycled on an off at intervals of 10 seconds. Relaxation was done at intervals of 10 minutes as a preventive fouling measure. Note that supplier recommends a design of 25,4 $L/m^2 \cdot h$ of flux, 0,59 m^3/min of membrane air scour flowrate, a vacuum pressure of less than 80 kPa and a pH range between 5 to 9,5.

- Mitsubishi MBR system: Two horizontal cassettes of approximately 1829 MF hollow fiber membranes of a 0,5 µm pore size, 3,24 m length and made of polyethylene for a total membrane area of 100 m^2 and a flow capacity of 34,8 L/min. This system included a prescreen, an aerobic tank of 6 m^3 where the membrane cassettes were submerged and a permeate storage tank. Air diffusers, located at the bottom of the aerobic tank, continuously provided thick bubble aeration to move the membranes and aerate the biomass. To supply additional oxygen for the increasing oxygen demands during more aggressive conditions (low HRT, for example) the tank was retrofitted with fine bubble diffusers. Membranes were relaxed at intervals of 12 minutes as a preventive fouling measure. Note that supplier recommends a design of 20,8 $L/m^2 \cdot h$ of flux, 0,74 to 0,9 m^3/min of membrane air scour flowrate, a vacuum pressure of less than 40 kPa and a pH range between 2 to 12.

All of the different used MBR systems were recovery cleaned once during the study either cleaned in place (Us Filter and Zenon) or cleaned in-line (Mitsubishi and Kubota). Moreover, a maintenance cleaning was also done on a more frequent basis only on MBR systems using hollow-fiber membranes (US Filter, Zenon and Mitsubishi). For example, Zenon MBR were operated with maintenance cleanings three times per week, two of them applying 250 mg/L of NaCOl and the other applying 2 wt% of citric acid. Each maintenance cleaning lasted 10 minutes where the membranes were backwashed and relaxed in alternating intervals of 30 seconds. Backwashing was done with permeated wastewater where the desired chemical was added (DeCarolis, J., 2007)..

A summary of the effluent quality of the four tested MBR systems is presented in average +/- standard deviated concentrations during the 16 months of the study duration operating with different flux, HRT, SRT and MLSS.

Table 6.3 Summary of effluent qualities achieved during the four MBR systems study in average +/- standard deviation concentrations (DeCarolis, J., 2007).

Parameter	Kubota	US Filter	Zenon	Mitsubishi
BOD [mg/L]	<2	2 +/- 0,87	<2	<2
COD [mg/L]	18,4 +/- 9,6	20,5 +/- 13,4	17,3 +/- 8,6	23,2 +/- 5,3
TOC [mg/L]	6,5 +/- 1,4	5,8 +/- 1,2	6,8 +/- 1,1	6,9 +/- 1,1
Turbidity [NTU]	0,08 +/- 0,02	0,04 +/- 0,02	0,06 +/- 0,02	0,07 +/- 0,01
Ammonia-Nitrogen [mg/L]	0,6 +/- 1,5	0,25 +/- 0,36	0,71 +/- 1,3	3,1 +/- 5,3
Nitrate/Nitrite-Nitrogen [mg/L]	2,95 +/- 1,23	23,6 +/- 7,8	21,6 +/- 5,2	15,2 +/- 7,7
Phosphorus [mg/L]	0,15 +/- 0,13	0,41 +/- 0,20	0,66 +/- 0,29	0,67 +/- 0,35
Total Coliform [CFU/100 mL]	13 +/- 69	386 +/- 674	807 +/- 1314	7 +/- 7
Fecal Coliform [CFU/100 mL]	3 +/- 4	50 +/- 72	9 +/- 22	2 +/- 0

Talking about BOD and turbidity, in all four cases much lower concentrations were achieved than the ones previously given by the conservative standard MBR effluent quality, confirming that the proposed standard MBR effluent quality is on the security side and even better effluents are achieved in those terms.

In terms of nutrient removal, US Filter, Zenon and in less terms Mitsubishi systems achieved good performances on nitrification, but as they were not designed with a denitrification stage, nitrogen continues remaining in their effluents in form of nitrate or nitrite. On the other hand, Kubota system, designed with an anoxic zone to achieve denitrification, averagely performed effluent total nitrogen concentrations under 4 mg/L, value under the 5 mg/L given by the proposed conservative standard MBR effluent quality. Without a specific design for phosphorus removal, all four analyzed MBR systems provide phosphorus concentrations under 1 mg/L because the influent phosphorus concentration was already under 1 mg/L, allowing the discharge of those treated wastewaters in sensitive areas or reusing them in environmental uses.

Big differences are observed between all four systems in terms of coliforms removal. After some testing, it was determined that high bacteria levels achieved by Zenon system were attributed to contamination on the permeate side of the membranes, confirmed by disinfecting the permeate piping of the system, after which effluent total coliform concentrations were less than 2 CFU /100 mL. Evaluation of the US Filter system confirmed that total coliform removal may be affected by backwashing, as the permeate piping became contaminated from bacterial growth during the maintenance process, which occurred in the permeate storage tank. Therefore, backwashing is not recommended as a fouling prevention procedure if low fecal coliform concentrations are desired. Anyway, with all four systems operating in proper conditions (US Filter system without backwashing), average fecal coliform concentrations under 10 CFU/100 mL are achieved, under those 50 CFU/100 mL proposed by the conservative standard MBR effluent quality, confirming that values really close to the absence are achievable using MBR systems.

The study concluded announcing the potential benefits of operating MBR systems with primary effluent instead of raw wastewater, which are a reduction of process air requirements, less susceptibility to changes in the MBR effluent quality, lower prescreen maintenance requirements and a reduced space demand. Membrane fouling was observed to increase as operating flux increased. The calculated runtime between cleaning associated with the maximum allowable TMP of the membranes decreased with increasing flux: 69 days at 20 $L/m^2 \cdot h$, 58 days at 25 $L/m^2 \cdot h$, and 30 days between 30 and 41 $L/m^2 \cdot h$. MBR process can be optimized to operate under extreme conditions of HRT of 2 hours and flux of 37 $L/m^2 \cdot h$. MBR systems operated under those conditions during 75 days with minimal increase of TMP producing an effluent with less than 0,1 NTU of turbidity and less than 2 mg/L of BOD (DeCarolis, J., 2007).

6.6.3. Contaminants of Emerging Concern in MBR Effluents

Contaminants of emerging concern (CEC) were presented during the chapter 2.3 as recently monitored water pollutants that have adverse effects on human and wildlife endocrine systems. Conventional wastewater treatment systems are not designed to remove all CEC. Therefore, some of them may get accumulated in water cycle over time without well defined effects in the long term. Although legislative regulations of treated wastewater discharge or reuse do not take into account those contaminants, day by day are gaining more attention, so that a treatment providing good effluent qualities in term of CEC will become more attractive in the long run.

It is known that a conventional activated sludge process is able to achieve high removals of some CEC, but also have not practically any affection over other CEC concentrations. To introduce some examples, a conventional activated sludge process have reported ibuprofen or bispenhol-A removals over 80%, triclosan or 4-nonylphenol removals between 40 – 50% or carbamazepine removals of approximately 9% (Melo-Guimaraes, A., 2013).

Adding coagulation-flocculation, a conventional advanced treatment reviewed during chapter 5.2.1, after the activated sludge process may be interesting to increase CEC removal. However, some of them continue being not well removed from the wastewater at the end of the treatment. To continue with same examples, the addition of coagulation-flocculation have reported ibuprofen or bispenhol-A removals over 90%, triclosan removals again between 40 – 50% but 4-nonylphenol removals over 80%, or carbamazepine removals of approximately 12% (Melo-Guimaraes, A., 2013).

Membrane filtration is seen as an interesting advanced wastewater treatment to significantly reduce the presence of most of CEC in their effluents. Generally, as lower the pore size of the membrane is, higher will the CEC removal be. Therefore, RO systems are the most interesting ones, achieving removal percentages over 90% of most of CEC (Cazes, M. 2014). However, it has been discussed that, at lower the pore size of a membrane is, higher will be its CAPEX cost and OPEX costs, demanding higher vacuum pressures to properly operate.

MBR systems are seen as an interesting wastewater treatment to remove CEC as they combine membrane filtration with an activated sludge process, therefore combining both biological and physical CEC removal performances in one single system. Although MF or UF membranes are used in MBR, this technology achieves higher removal percentages than, for example, the addition of coagulation-flocculation after a conventional activated sludge process. However, some CEC continue presenting resistance to be removed by MBR systems. To continue with the examples provided before, MBR systems obtain effluents with ibuprofen or bisphenol-A removals of approximately 90%, triclosan or 4-nonylphenol removals over 90%, but again low carbamazepine removals of approximately 20% (Cazes, M., 2014).

MBR systems can be coupled with other complementary treatments with a positive impact on CEC not fully removed by biodegradation or by filtration. Figure 6.13 shows a removal comparison of some CEC between a conventional submerged MBR system and the same system operating with the addition of granulated activated carbon (GAC). Although the use of GAC would increase the OPEX costs of the system, it can be seen its efficiency in a MBR system to achieve high removals of CEC in general terms, even of carbamazepine, now removed over 90% from the effluent. Moreover, it has been seen during chapter 6.5.4 the advantage that supposes the addition of GAC in terms of fouling reduction in a MBR system. Therefore, use of GAC in MBR is seen as a promising solution to provide high CEC removals in those systems (Cazes, M. 2014)..

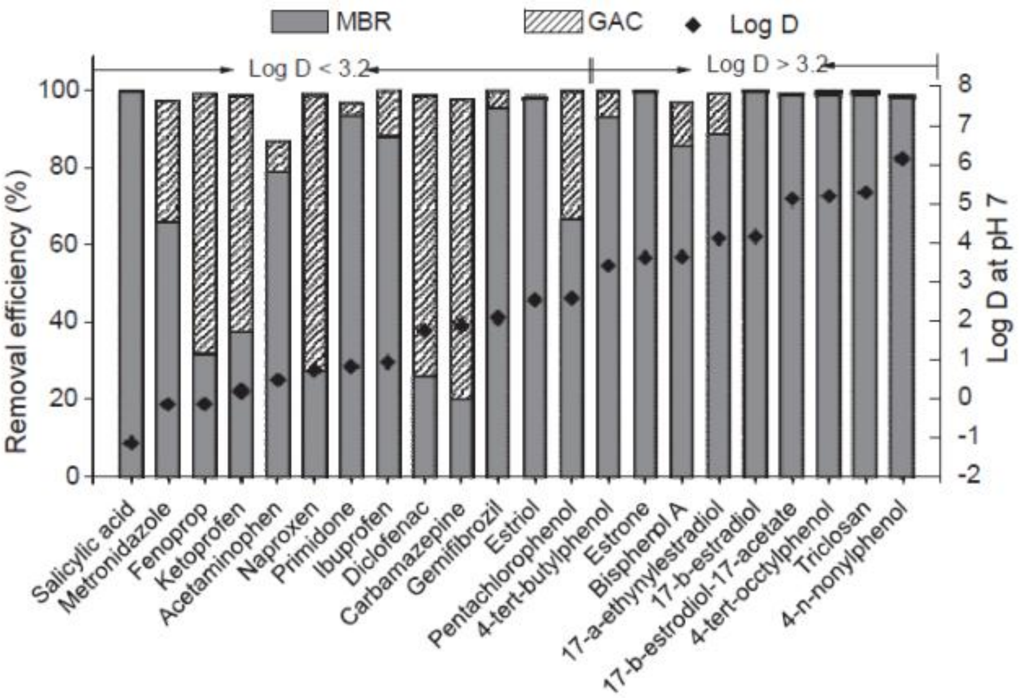

Figure 6.12 Comparison between a conventional submerged MBR removal efficiency on different CEC with respect to same system with a granular activated carbon (GAC) dose (Cazes, M., 2014).

Another interesting way to achieve high CEC removals combinable with a MBR system is disinfecting with ozone. As it has been previously seen during chapter 6.6.1., if a final disinfection is added as a second barrier to ensure the absence of pathogens in the effluent, ozonation is seen as an interesting process because of its double high capacity to remove not only pathogens, but also most of the CEC known nowadays.

6.6.4. A Real Case: MBR operating in Santo Tirso

A water treatment facility consisting on a MBR system was visited during the development of this thesis. The small-sized plant, with a maximum daily capacity of about 1.000 m^3 is used by three different textile industries to treat wastewaters produced by their industrial processes.

This solution was proposed, built and is actually managed by “Moinhos Água e Ambiente”, a Portuguese company from Santo Tirso whose activity consists on developing and applying technological environmental solutions, mostly related with the water management. The company operates in different economic sectors, going from domestic or urban to municipal or industrial applications. During the last years, “Moinhos Água e Ambiente” focused some of their efforts on applying the MBR technology in treating wastewaters to obtain treated effluents able to be safely reused. Nowadays they have a big presence in Angola, where they have been working along the past years in several projects to develop, build and manage solutions consisting in MBR systems able to provide treated wastewater ready for being reused in a country with high water shortage. They have an important presence in Portugal as well, especially in the northern part of the country, where most of the existing MBR systems are used to treat wastewaters coming from industrial companies. Nevertheless, more and more are the projects they have consisting in MBR systems to treat municipal wastewaters day by day in Portugal.

Specifically, the MBR plant visited was built to find a cheaper solution to treat the produced wastewater by the three textile companies than the price they would have to pay to send those wastewaters to the municipal WWTP. Taking advantage of their location, they decided to adopt a unique compact treatment system for all of them that is able to highly remove the contaminants present in the influent, obtaining an effluent ready to be reused for the industrial processes. Only the incoming wastewaters over 2.000 µS/cm of electrical conductivity are discarded from the treatment system and sent to the municipal WWTP, as it is cheaper for the industries to pay the municipal tax for treating those highly contaminated wastewaters than treat them by themselves using the MBR system.

Figure 6.13 MBR system tank, where both biological and filtration processes are done simultaneously.

The visited facilities consist in a pipe network that drives the wastewaters resulted from the textile processes done by the three different industries to the plant entrance, where a electrical conductivity measurer establishes if the wastewater is going to be treated inside the plant or will be discarded into a little deposit, where the wastewater over 2.000 µS/cm gets accumulated and is sent to a municipal WWTP. Once the influent has passed the initial barrier crosses a fine screen and arrives at an equalization tank, from where it is distributed inside the tank that is seen in figure 6.14. The MBR system is configured in a unique tank, where the biological and filtration process are done together. In the previous image they can be seen both different aerations of small bubbles used for the biological process on the top of the tank and of thick bubbles used for the membranes zone, respectively. A relaxation period of 1 minute every 10 minutes of operation is taken as another measure to fight against the fouling phenomena. Operating at a range of pressures close to 20 kPa and flows usually between 35 and 45 $L/m^2 \cdot h$, no relevant problems were been detected until the date during almost two years using vertical cassettes of Kubota flat sheet microfiltration membranes. The filtrated effluent is finally accumulated in another tank, from where is driven to the industries again ready for being reused.

Figure 6.14 Treated effluent obtained by the MBR process.

6.7. MBR CAPEX and OPEX Costs

Traditionally both MBR excessive capital and operational costs had been the main reason of choosing other alternatives over these systems. Development carried out during the last decades to obtain better filtration membrane generations, more productive and less expensive, combined with the appearance and evolution of the submerged MBR, demanding less energy consumption than first generation of external MBR, have produced a market expansion of this technology, creating more competence between manufacturers that continued inducing a reduction in membrane capital costs and an energy demand optimization search. Actual CAPEX and OPEX costs of a conventional submerged MBR are presented and discussed in detail with respect to a conventional activated sludge process costs and same process combined with necessary conventional advanced treatments to obtain better effluent qualities, allowing reusing the treated wastewater as the MBR permit.

6.7.1. MBR CAPEX Costs

Calculating the CAPEX costs of a MBR system construction is not trivial, as it depends on many variables that may be different and therefore have different prices in any particular case. Those parameters can be fundamentally divided in four groups: the membrane filter modules; the mechanical equipment (pumps for influent and effluent transfers, sludge return cycle or discharge, air diffusers, mixers, a preescreaning system, etc.) consisting in pumps (influent and effluent transfers, sludge return cycle, sludge discharge, etc.); the electrical equipment necessary to make the whole system work; and the civil works necessary to build all the system, mostly dedicated on constructing tanks for the possible system treatment stages location, as well with the installation of all the mechanical and electrical equipment. It must be remarked that all the prices of those four groups have a big dependence on the size of the MBR plant: as bigger the plant is, proportionally lower the prices will be. Therefore, an economy of scale should be applied to calculate what will cost an MBR system depending on its size. Moreover, there are the costs associated with the land needed for the system construction that will depend on the land price in the country where the system is built, the industrial benefit of the building company and the administrative procedure to legally carry out the project, both of them usually applied in percentages over the total price of the project.

MBR plants began to be built in Europe at the beginning of the XXI century. From then on, number of constructed MBR systems has been increasing year by year. Several information about their CAPEX costs has been compiled, what can be used to get an idea of how much could cost a MBR system based on real experiences.

Three MBR plants were built by the Erfteverband German regional water management association between 1999 and 2008: Rödingen (1999), Nordkanal (2004) and Glessen (2008) with maximum capacities of 3.240 m^3/d, 45.000 m^3/d and 6.430 m^3/d, respectively. Note that Glessen plant was not a new construction, but the old conventional activated sludge process was adapted by transforming the secondary clarifier into a buffer tank and building a new pretreatment system and the membrane filtration tank, retaining the existing bioreactor and sludge storage tank. All three project CAPEX costs are presented in a percentage division between how much money was destined to membrane filter modules, mechanical equipment, electrical equipment and civil works, respectively (Brepols, C., 2010).

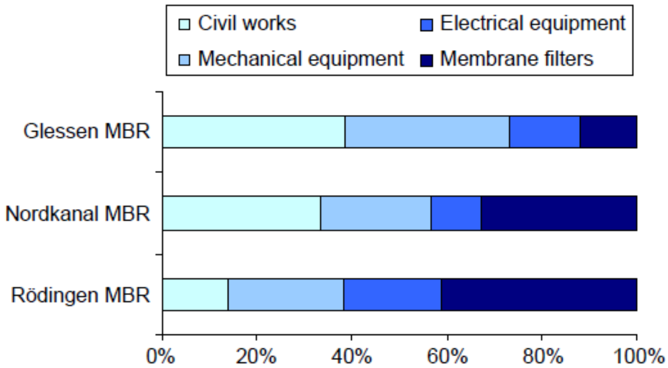

Figure 6.15 CAPEX costs percentage distribution of Rodingen, Nordkanal and Glessen MBR plants (Brepols, C., 2010).

A big difference between CAPEX costs percentage distributions of the three MBR plants is seen in figure 6.16, especially in terms of civil work and membrane filter modules costs. This can be explained by two main reasons. On the one hand, as it happened in rest of European countries, cost indices for industrial buildings were importantly increased in Germany between the period of construction of the first and the last of the three MBR plants evaluated. Therefore, MBR system compactness gains more value at this point as this technology requires less civil works during its construction compared with other alternatives. On the other hand, as new the construction of the MBR plant is, less percentage of the total CAPEX costs is destined to the membrane filter modules. This connection comes from something previously announced: new generations of membranes are cheaper day by day. As a consequence, membrane filter modules costs of the Rödingen MBR plant construction would have been approximately 66% lower in 2010 than they originally were in 1999 (Brepols, C., 2010).

Based in all the information available about the construction of the three German MBR plants, an estimation of how much a new MBR plant of a 10.000 PE of capacity (~ 2.500 m^3/d) would cost to be built in Germany in 2010. This information is presented in table 6.4 to have a first idea in terms of digits of the CAPEX costs of a MBR system.

Table 6.4 Estimated costs for a 2.500 m^3/d MBR plant construction in Germany in 2010 (Brepols, C., 2010).

Item	Investment Costs (€)
Membrane filter modules	558.000
Mechanical equipment	2.500.029
Electrical equipment	795.464
Civil Works	2.248.507
TOTAL CAPEX Costs	6.102.000
CAPEX Costs per m^3/d	2.440,8

To continue introducing digits, approximated capital costs of constructing six different MBR plants in Spain between 2007 and 2010 are presented in table 6.5 (Igleasias, R., 2017). El Valle, Mar Menor and Riquelme are small-sized plants consisting in flat-plate membrane modules with an extended aeration to achieve biological nitrogen removal. San Pedro, the biggest one with a capacity of 20.000 m^3/d, is a plant consisting in hollow-fiber membrane modules with an extended aeration to achieve biological nitrogen removal. Vacarisses and Vallvidrera, on the other hand, are small-sized plants consisting in hollow-fiber modules with biological nitrogen removal and chemical precipitation to remove phosphorus. All project prices are included: membrane filter modules, mechanical and electrical equipment, and civil works, but not the extra costs associated with the land price, the industrial benefit nor the administrative taxes.

Table 6.5 Approximated CAPEX costs of six MBR plants constructed in Spain (Iglesias, R., 2017).

MBR Plant (construction year)	Capacity (m^3/d)	CAPEX Costs (€)	CAPEX Costs per m^3/d (€/m^3/d)
El Valle (2007)	1.400	2.774.800	1.982
Mar Menor (2009)	1.880	3.763.760	2.002
Riquelme (2007)	1.575	3.266.550	2.074
San Pedro (2007)	20.000	12.500.000	625
Vacarisses (2010)	1.320	2.811.600	2.130
Vallvidrera (2008)	1.100	3.604.700	3.277

Two important conclusions are extracted from the six MBR plants approximated CAPEX costs.. On the one hand, it is confirmed by looking at their CAPEX costs per m^3/d the important dependence between the size of the plant and its costs. Constructing the biggest system, San Pedro, have cost more than 3 times less than the other five plants in terms of Euros invested per each m^3/d of the plant's capacity. As a consequence, as it happens with the rest of treatment systems, it will be interesting to do a good planning in the long term, constructing less and bigger plants than more and smaller plants to assimilate the wastewater treatment demand. On the other hand, comparing CAPEX costs per m^3/d of the small Spanish MBR plants with the estimated CAPEX costs per m^3/d shown in table 6.4, it can be seen that four of the five real cases have fewer costs per m^3/d than the estimated case, which is bigger in capacity. Applying the economy of scale, as bigger the plant is, fewer costs per m^3/d should have. Therefore, it is concluded that the estimated costs of the German MBR plant construction in 2010 were been probably overrated.

Another interesting estimation of two MBR plants construction was carried out during 2006, presented in table 6.6 (DeCarolis, J., 2007). This case was targeting not only to provide some digits of the CAPEX costs of a MBR system, but also to compare the proportional price reduction between a smaller and a bigger plant. Cost estimates were based on MBR plants designed to achieve complete nitrification, partial denitrification and biological phosphorus removal. Four different membrane filter suppliers were asked to estimate the costs of the membrane filter modules.

Table 6.6 Average estimated costs for two MBR plant constructions in 2006 (DeCarolis, J., 2007).

Item	Investment Costs	
	4.000 m^3/d	20.000 m^3/d
Membrane filter modules	1.590.000	5.750.000
Mechanical equipment	953.000	4.358.000
Electrical equipment	503.000	2.088.000
Civil Works	1.291.000	5.765.000
TOTAL CAPEX Costs	4.337.000	17.961.000
CAPEX Costs per m^3/d	1.084.25	898

Some conclusions are extracted from table 6.6. Firstly, the comparison between both estimated CAPEX costs per m^3/d shows again the existing economy of scale. However, comparing this scale with the scale shown by the real costs of the Spanish MBR plants, there was more advantage in constructing a big plant in real experiences than what DeCarolis estimated. Secondly, although the 20.000 m^3/d capacity plant estimation seems to be overrated with respect to the approximated real CAPEX costs of the Spanish San Pedro plant, DeCarolis estimations are more accurate than the estimation of a MBR plant construction in Germany in 2010. Membrane filter modules price has been identified as the main reason for which this estimation could be overrated. Comparing its costs distribution with respect Brepols estimation, membrane filter modules costs are proportionally much bigger in DeCarolis case. It is concluded that is because this estimation was made in 2006, while the German estimation was done in 2010, showing again the price reduction that the membrane filters are suffering year by year.

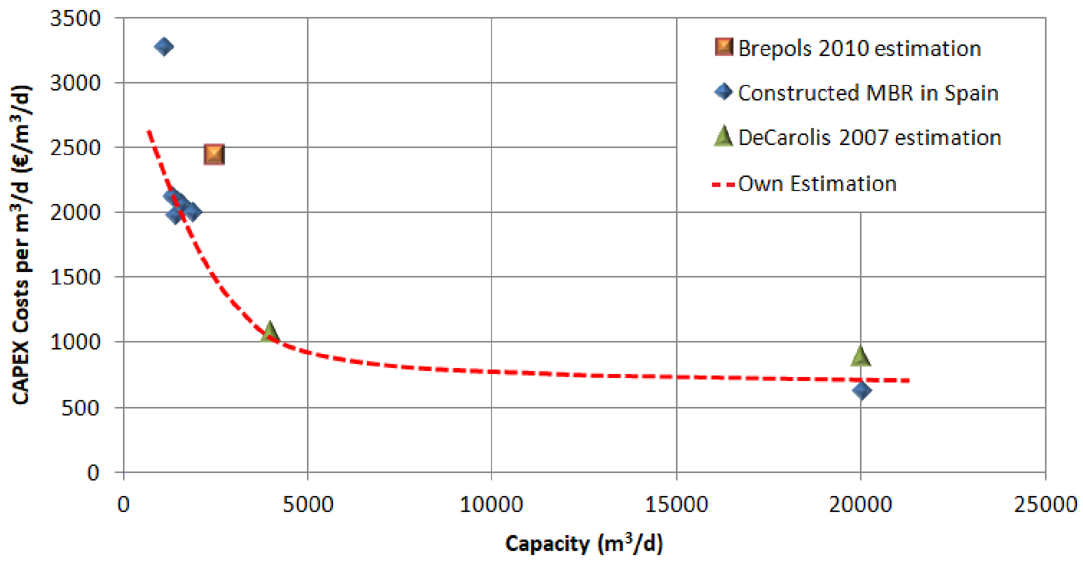

Figure 6.16 MBR CAPEX costs trend curve with respect to previously presented data (own elaboration).

An own estimated trend curve of CAPEX costs per m^3/d of a MBR plant according to the system capacity is presented in figure 6.17, taking into account previously presented real experience costs in constructed MBR systems in Spain and both Brepols 2010 and DeCarolis 2007 estimations. Note that the curve is based on estimations or real cases made between 2006 and 2010. Therefore, as it has been previously discussed, partial price destined to membrane filter modules of the system may be even lower nowadays.

6.7.2. MBR OPEX Costs

As it happens with CAPEX costs, estimating OPEX costs of a MBR plant depends on many parameters that may be considerably different case by case. Those variables can be grouped in five general categories: energy demand, personnel, maintenance, sludge and waste disposal and others (such as general expenses like the wastewater levy, effluent quality analysis or the possible addition of chemicals).

OPEX costs percentage distribution from German MBR plants of Rödingen and Nordkanal are presented in figure 6.16 (Brepols, C., 2010). Comparing both distributions gives an idea of how big the difference between each particular case could be. The main difference is seen in the operational costs related to maintenance, which can be explained taking into account the date where both plants began operating. Rödingen was built in 1999, operating during 5 more years than Nordkanal (2004), which was a relatively new plant when this data was compiled for the study in 2010. As a consequence, probably some of the original membrane filters from Rödingen had to be replaced, while in Nordkanal probably almost all the original membranes would continue being in an acceptable state after six years of operation. More examples could be also introduced to arrive at the conclusion that, as older the MBR plant is, more OPEX costs related with maintenance procedures will has. Differences between the other groups are explained firstly because, as Rödingen has higher maintenance costs, other groups are proportionally lower than in Nordkanal and secondly because of the big difference between both plants size (3.240 m^3/d and 45.000 m^3/d, respectively).

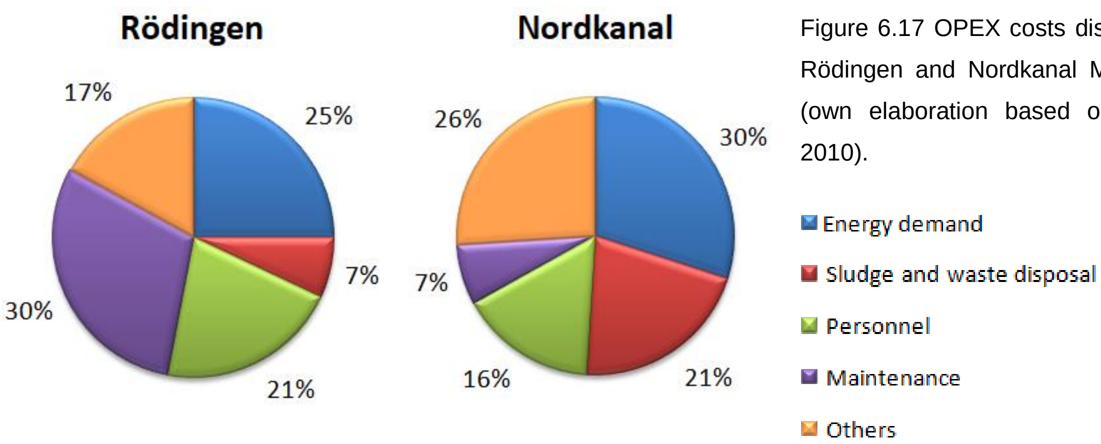

Figure 6.17 OPEX costs distribution in Rödingen and Nordkanal MBR plants (own elaboration based on Brepols, 2010).

An estimation of how much would cost to operate a hypothetical MBR plant of 10.000 PE capacity (~ 2.500 m^3/d) is presented in table 6.7 based in all the information available about the operation of those two MBR plants in Germany (Brepols, C., 2010). The installation is designed to be operating with a SRT of 25 days and a MLSS concentration of 12 g/L and with separate filtration tanks of hollow-fiber membrane modules with a lifetime from 5 to 10 years. Mechanical and electrical equipment lifetime is assumed to be 15 years. Lifetime of the civil works of the installation is fixed at 30 years.

Table 6.7 OPEX costs estimation of a 2.500 m^3/d MBR plant (Brepols, 2010).

Item	OPEX Costs (€/year)
Energy demand	82.782
Sludge and waste disposal	25.664
Personnel	70.000
Maintenance	96.420
Others	12.860
TOTAL OPEX Costs	287.726
OPEX Costs per treated m^3 (€)	0,315

Ten different MBR plants were analyzed in terms of OPEX costs in Spain (Iglesias, R., 2017). Their total OPEX costs and OPEX costs per m3 of treated wastewater are presented in table 6.8.

Table 6.8 OPEX costs of 10 different MBR plants operating in Spain (Iglesias, R., 2017).

MBR Plant (construction year)	Capacity (m^3/d)	Treated (m^3/d)	OPEX Costs per treated m^3 (€)
Calasparra (2006)	2.000	1.793	0,330
El Valle (2007)	1.400	219	1,600
La Bisbal (2003)	3.225	3.225	0,215
Los Cañares (2008)	3.750	161	2,180
Mar Menor (2009)	1.880	552	0,480
Riquelme (2007)	1.575	362	0,970
Riells Viabrea (2004)	2.116	1.116	0,397
San Pedro (2007)	20.000	8.247	0,241
Vacarisses (2010)	1.320	683	0,577
Vallvidrera (2008)	1.100	849	0,460

There is a big difference between OPEX costs per m^3 of treated wastewater of each Spanish plant. This can be explained: firstly because of an existing economy of scale as it happened with MBR systems CAPEX costs: as bigger the plant is, fewer OPEX costs per m^3 of treated wastewater. But the main reason of those differences presented is found on the percentage of wastewater the plant is treating with respect to its designed capacity. As lower this relationship is, much more will cost each treated m^3 of wastewater. This non-linear correlation is logic as the plant has a designed amount of equipment that is being used anyway despite is operating under its capacity. Therefore, only the data of Calasparra and La Bisbal plants, which operate at their maximum capacity or almost there, are considered reliable to be compared with other OPEX costs analysis, which assume the MBR plant to be working at its maximum capacity.

DeCarolis also performed an estimation of the OPEX costs his two hypothetical MBR plants would have in 2007, presented in table 6.9 (DeCarolis, J., 2007). MBR plants have capacities of 4.000 m^3/d and 20.000 m^3/d, respectively, operating with a SRT of 10 days, a HRT of 6 hours, a MLSS concentration of 8 g/L and a flux of 17 L/m^2·h. Membrane replacement is based on an 8 years lifetime. However, only OPEX costs related with the energy demand, maintenance and personnel were estimated by DeCarolis. Sludge and waste disposal and other OPEX costs are an own elaboration based on the average percentages they were representing in Rödingen and Norkandal MBR plants OPEX costs distribution.

Table 6.9 Average estimated OPEX costs for two different MBR plant sizes (DeCarolis, 2007). Sludge and waste disposal and other OPEX costs are an own estimation.

Item	Capacity	
	4.000 m^3/d	20.000 m^3/d
Energy demand	74.860	373.000
Personnel	29.770	83.370
Maintenance	113.000	499.000
Sludge and waste disposal	35.854	157.353
Others	66.566	292.227
TOTAL OPEX Costs (€/year)	320.040	1.404.950
OPEX Costs per treated m^3 (€)	0,219	0,192

Analyzing the OPEX costs per treated m^3 of wastewater it is confirmed the economy of scale also existing in OPEX costs of the MBR systems. Comparing this estimation with real OPEX costs from Calasparra and La Bisbal plants in Spain and with the estimation made by Brepols as well, it is concluded that, despite both estimations could be a little overrated again, they are quite accurate compared with real data obtained from MBR plants operating at full or almost full capacity.

Both DeCarolis and Brepols estimations were used by an interesting study that proposed the following general theoretical formula to have an initial idea of how much small sized MBR plants could cost in terms of OPEX depending on its capacity. Note that the original formula from the study published by Lo, McAdam and Judd in 2015 has been modified to adapt the obtained OPEX costs per treated m^3 in € instead of $.

$$OPEX\ costs\ \left(€/_{m^3}\right) = (-0{,}0509 \cdot lnQ + 0{,}664) \cdot 0{,}847$$

Where Q is the capacity of the MBR plant in m^3/d. To test the formula, capacities of Brepols and DeCarolis estimations have been used as well with the Calasparra and La Bisbal ones to calculate their OPEX costs and then compare them with the real cases or the original estimations. Results are presented in table 6.10.

Table 6.10 Comparison between calculated OPEX costs per treated m^3 of wastewater using the formula developed by Lo, McAdams and Judd with respect to real or estimated original costs.

MBR Plant	Capacity (m^3)	Original OPEX costs per treated m^3 (€)	Calculated OPEX costs per treated m^3 (€)
Calasparra	2.000	0,330	0,235
Brepols	2.500	0,315	0,225
La Bisbal	3.225	0,215	0,214
DeCarolis 1	4.000	0,219	0,205
DeCarolis 2	20.000	0,192	0,135

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Comparing the calculated OPEX costs by the formula with the original real or estimated plants, especially in La Bisbal and DeCarolis 1 cases the formula has been really accurate. However, the formula underrated Brepols and Calasparra cases, what again suggests that Brepols estimation is overrated again as it happened with its CAPEX costs. It must be taken into account that OPEX costs of the real case of Calasparra were obtained with the plant operating approximately at 90% of capacity. Therefore, operating at full capacity its original OPEX costs per treated m^3 would probably be lower. On the other hand, there is a big difference between the originally estimated and the calculated OPEX costs per treated m^3 for DeCarolis 2, confirming this formula can only be reliable on OPEX cost estimations for small-sized MBR plants, losing accuracy for big-sized systems. .

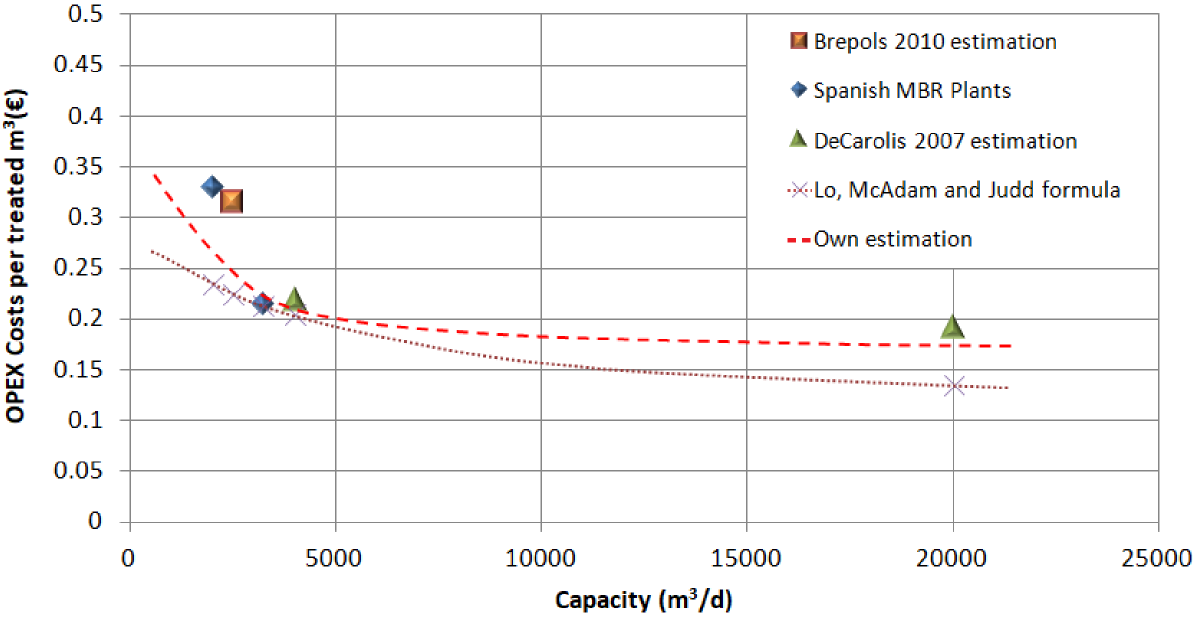

Figure 6.18 MBR OPEX costs trend curve with respect to previously presented data (own elaboration).

An own OPEX costs per treated m^3 trend curve estimation is presented in figure 6.17 with respect to previously analyzed data. This curve has been made taking into account: the accuracy presented by Lo, McAdam and Judd formula with respect to La Bisbal and DeCarolis 4.000 m^3/d capacity estimation, prices under the over-estimated Brepols case and Calasparra plant (which was working at approximately 90% of its capacity), and finally a correction with respect to Lo, McAdam and Judd formula when talking about big-sized plants taking into account DeCarolis estimation for a 20.000 m^3/d capacity.

6.7.3. MBR Versus Conventional Secondary and Advanced Treatments

CAPEX costs of constructing 26 WWTP between 1989 and 2008 in Erfteverband region of Germany are presented in figure 6.20 (Brepols, C., 2010). Those plants are decomposed in three MBR plants, nine conventional activated sludge process plants (CTP) and thirteen conventional activated sludge process plants with additional advanced treatments (CTP with tertiary treatment) to achieve better effluent qualities, usually complemented with a post-denitrification reactor, granular filtration and disinfection.

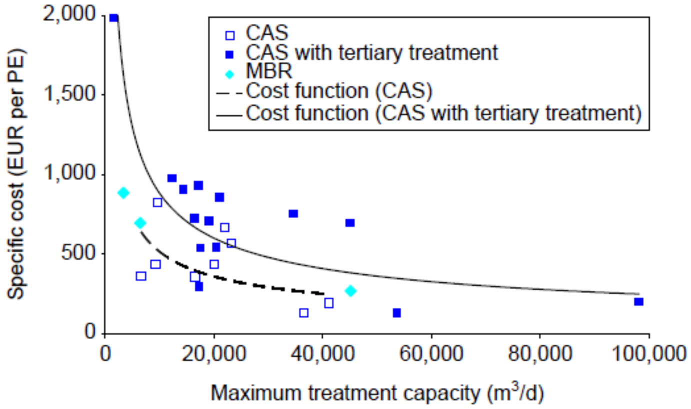

Figure 6.19 CAPEX costs of 26 constructed WWTP in the Erfteverband region between 1989 and 2008 (Brepols, 2010).

Brepols analyzed the investment costs of the 26 WWTP during his study in 2010, estimating a cost function both for CTP and CTP with tertiary treatment plants. Although it is seen an important price variation case by case in figure 6.18, in average, CAPEX costs of MBR plants per PE were over the CTP plants costs and under the costs of CTP with tertiary treatments plants.

Brepols continued his study analyzing OPEX costs of a MBR plant, now compared with a CAS with tertiary treatments plant. The study used all the analyzed data in terms of CAPEX and OPEX costs to estimate a hypothetical comparison case between a MBR plant and a CTP with tertiary treatments plant of same size (10.000 PE ~ 2.500 m^3/d). Both installations use biological nutrient removal, producing a comparable effluent quality. MBR plant is designed with separate filtration tanks of hollow-fiber membranes. CTP plant adds granular filtration and UV disinfection. It is assumed a SRT of 25 days in both plants, with a MLSS concentration of 4 g/L at CTP and 12 g/L at MBR. Lifetime of civil works and electric equipment is fixed in 30 and 15 years, respectively, in both installations. Membrane lifetime is fixed in 8 years in the MBR plant (Brepols, C., 2010).

Table 6.11 CAPEX and OPEX costs comparison between a MBR and a CTP with tertiary treatment plants (Brepols, C., 2010).

Item	CAS with tertiary treatment (€)	MBR (€)
Civil works	4.960.097	2.248.507
Mechanical equipment	2.152.614	2.500.029
Electrical equipment	711.789	795.464
Membrane filter modules	-	558.000
TOTAL CAPEX Costs	**7.824.500**	**6.102.000**
Energy demand	45.990	82.782
Sludge and waste disposal	25.664	25.664
Personnel	70.000	70.000
Maintenance	96.411	96.420
Others	18.665	12.860
TOTAL OPEX Costs	**256.730**	**287.726**

Despite both MBR plant CAPEX and OPEX costs were considered overrated during chapters 6.7.1 and 6.7.2, respectively, it is assumed the CAS with tertiary treatment estimation is also overrated both in terms of CAPEX and OPEX costs and therefore are comparable each other. In Brepols estimations there is an important reduction of the MBR plant CAPEX costs with respect to a CAS with tertiary treatments plant. Moreover, if is taken into account the extra price that must be destined to buying the land for building the plants, even cheaper the MBR plant would be with respect to a CAS with tertiary treatments plant, as Brepols estimated both paved plant surfaces in 900 m^2 and 2400 m^2, respectively.

On the other hand, the MBR plant OPEX costs are estimated over the CAS with tertiary treatments plant, mainly because of a higher energy demand. However, calculating a 30 years lifetime of total costs of both installations, the MBR plant presents costs of 14.733.780 € under the 15.526.400 € presented by the CAS with tertiary treatments plant.

The idea of that a MBR plant is more expensive than a CAS plant is confirmed by another study made by Bertanza in 2017. After estimating in detail two biz-sized hypothetical cases based on data from the Brescia-Verzian Italian plant, where a MBR and a CAS lines are operating in parallel for a total capacity of 250.000 PE (~ 62.500 m^3/d), the economic assessment showed that the CAS system is the cheapest solution. The MBR plant has a total cost increase (CAPEX + OPEX) of 53% under the most favorable conditions and of 38% under the worst scenario (Bertanza, 2017). Comparing the energy demand in both cases (0,34 kWh/m^3 for a CAS plant and 0,48 kWh/m^3 for a MBR plant (Bertanza, 2017)), the study suggested that a CAS with tertiary treatments plant would have an energy demand of approximately 0,6 kWh/m^3, over the energy demand estimated for the MBR plant. This data suggests that Brepols study could have importantly overrated the energy demand of its MBR plant, which was practically duplicating the energy demand of its CAS with tertiary treatment plant.

Brepols MBR plant energy demand overestimation is confirmed by looking at the results presented by another study. Iglesias analyzed and compared in 2017 the OPEX costs of fourteen MBR plants with respect to sixteen CAS with tertiary treatments plants, in which coagulation-flocculation, granular filtration and disinfection are added after the conventional activated sludge process.

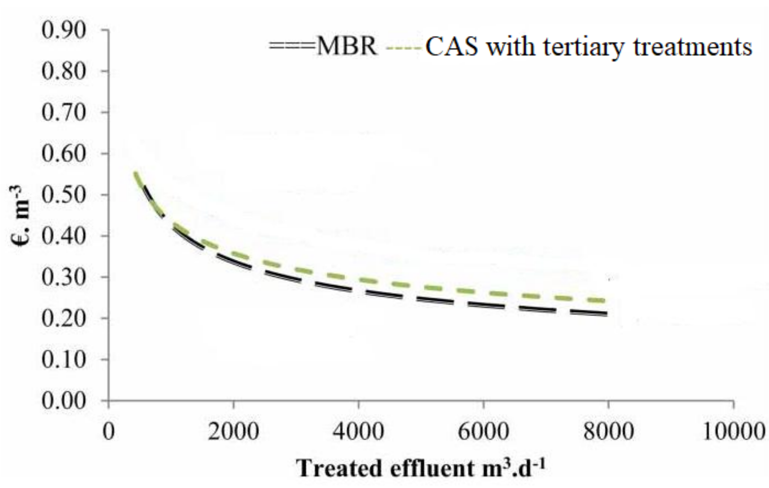

Figure 6.20 OPEX costs functions proposed by Iglesias in 2017 for MBR and CAS with tertiary treatments plants, respectively (adapted from Iglesias, R., 2017).

In general terms, both cost functions estimated by Iglesias indicate lower OPEX costs for a MBR plant than for a CAS with tertiary treatments plant of same capacity. It has to be taken into account that Iglesias CAS with added coagulation-flocculation, granular filtration and disinfection plants, while Brepols compared CAS with only added granular filtration and disinfection plants. Therefore, Brepols CAS with tertiary treatments plant would have higher OPEX costs considering coagulation-flocculation as well. However, after analyzing all the data provided by the three studies in terms of OPEX costs, it is believed that the OPEX costs of a MBR plant are similar to or even lower than a CAS with tertiary treatments plant OPEX costs.

In conclusion, on the one hand is more cost-effective a CAS than a MBR plant. Therefore, if a new WWTP is being projected in order to achieve an effluent quality to discharge the treated wastewater into a non-sensitive environment, constructing a CAS plant is recommended over a MBR plant in economical terms. On the other hand, a MBR plant is more cost-effective than a CAS with tertiary treatments plant, especially in terms of CAPEX costs, which usually represent an important limitation for the municipality owner of the future WWTP. Therefore, choosing a MBR plant over a CAS with tertiary treatments plant to achieve an effluent quality to reuse the wastewater is recommended in economical terms. MBR systems are also recommended for upgrading cases from CAS plants because of their compactness, adaptability and also cheapest CAPEX and OPEX costs with respect to a conventional line of advanced treatments. So, if nowadays a CAS plant is decided to be built in order to achieve discharge qualities taking profit of its lower costs, if in the future the plant is wanted to be upgraded to achieve effluent qualities to reuse the treated wastewater, initially constructing a MBR plant would had resulted in lower costs than constructing a CAS plant and then adapting it to a MBR plant or even adding a line of conventional tertiary treatments, the most expensive of the presented solutions.

6.8. MBR Design

A model has been developed on Excel to design a compact MBR system based on all the information presented and discussed during this thesis. A conventional submerged MBR configuration has been selected, separating biological and filtration zones in order to take advantage of the benefits of providing optimal fine and thick bubble aeration in each zone, respectively. Filtration is performed outside-in by vertical hollow-fiber modules, intermittently aerated in cycles of 40 seconds (10:30 aeration). Relaxation is done during 1 minute in intervals of 10 minutes. The MBR system is preceded by a fine screen size of 0,5 mm and an equalization storage tank.

The model uses some inputs (marked in red) which have to be defined by the user, mostly related with the influent wastewater characteristics, capacity and temperature of the MBR system or some data from the biological or filtration process, respectively. Once all the required information is given, the model calculates the biological growth that will take place inside the biological zone and how it will be affected by the temperature. From this information the characteristics of the biological zone are obtained. Meanwhile, the model calculates the characteristics of the filtration zone, giving as an output a minimum size and a designed air requirement which depend on the characteristics of the membranes that will be used. Zenon cassettes ZW 500 D have been proposed, if the user wants to use different membranes it is possible to change their characteristics. Some of the most important outputs are remarked in green.

The explanation about the model performance is plenty explained in the annex of this thesis, where all the calculations done by the model are presented step by step, including all the data that is used to arrive at the obtained results. A practical case is attached to see the results obtained by the mathematical model calculating a hypothetical MBR plant of 5.000 m^3/d of capacity operating at 25ºC treating a given influent wastewater.

Chapter 7

Conclusions

It has been recognized the importance of reusing treated wastewater to solve the stress of the freshwater resources as well with a sustainable water cycle generation in the long term. Advanced treatment design needed for that purpose is probably the most challenging aspect in a WWTP. It is fundamental to know the incoming wastewater characteristics and the desired treated wastewater quality in order to choose a solution. Limitations in CAPEX and OPEX costs must be taken into account, as well with the available physical space for the WWTP construction or expansion, the compatibility with existing facilities if an upgrading is being carried out, the equipment availability, the personnel or the energy and resource requirements.

MBR systems are a mature alternative to conventional wastewater treatments that can substitute an entire line of secondary and tertiary treatment systems after the preliminary and primary treatments in a WWTP, achieving excellent effluent qualities that allow reusing the treated wastewater in most of urban, agricultural, industrial, recreational or environmental uses. Although MBR effluents achieve concentrations of fecal coliforms really close to the absence, it may be a limiting factor to reuse the treated wastewater in some uses that require the absence of pathogens. Therefore, depending on the system operation it is recommended a disinfection treatment to act as a second pathogen barrier. In terms of CEC, MBR systems are seen as an interesting treatment to achieve good removal performances. The addition of granular activated carbon (GAC) to the biological process is a promising solution to generally provide high CEC removals in MBR. Adding ozonation as a second pathogen barrier is seen as an interesting measure to ensure both the absence of pathogens and high removals of most of known CEC nowadays.

MBR systems work with higher biomass concentrations compared with a conventional activated sludge process, what allows designing compact facilities with an important space demand reduction compared with other alternatives. Therefore, MBR are specially seen as an attractive option for upgrading conventional plants that achieve discharge qualities into plants where reusing the effluent would be possible, because of their low space demand and adaptability to a conventional activated sludge process. It is also seen as an interesting solution for new constructions where space is a limiting factor. Moreover, working with higher biomass concentrations and solid retention times results in a much lower wasting sludge generation, an important operational cost in conventional activated sludge processes.

Submerged MBR are nowadays recommended over external MBR because of their lower OPEX costs. Therefore, the operational criteria, performance, costs and design have been discussed in terms of a submerged MBR. Fundamental operational criteria have been reviewed in order to achieve an optimal system performance. Fouling is the main operational problem in MBR and also one of the main reasons of this technology to have high OPEX costs, affecting on pretreatment needs, cleaning requirements, operating conditions and performance of the system. Many recommendations have been provided in order to apply proper operational tools in terms of fouling prevention. It is important to find an optimal combination between fouling maintenance, control and prevention strategies to achieve high net flux rates in the long term by minimizing the energy consumption and promoting the system productivity without damaging the membranes. Some experimental cases have shown that backwashing could damage the membranes or promote effluent contaminations in the long term. So, relaxation is recommended over backwashing as a physical fouling prevention measure combined with periodical chemical cleanings. Feedback control systems are seen as an interesting tool to achieve an adjustable control of fouling maintenance, control and prevention strategies during the system lifetime.

In terms of costs, on the one hand is more cost-effective a CAS than a MBR plant. Therefore, if a new WWTP is being projected in order to achieve an effluent quality to discharge the treated wastewater into a non-sensitive environment, constructing a CAS plant would be recommended over a MBR plant in economical terms. On the other hand, a MBR plant is more cost-effective than a CAS with tertiary treatments plant, especially in terms of CAPEX costs, which usually represent an important limitation for the municipality owner of the future WWTP. Therefore, choosing a MBR plant over a CAS with tertiary treatments plant to achieve an effluent quality to reuse the wastewater is recommended in economical terms. MBR systems are also recommended for upgrading cases from CAS plants because of their compactness, adaptability and also cheapest CAPEX and OPEX costs with respect to a conventional line of advanced treatments. So, if nowadays a CAS plant is decided to be built in order to achieve discharge qualities taking profit of its lower costs, if in the future the plant is wanted to be upgraded to achieve effluent qualities to reuse the treated wastewater, initially constructing a MBR plant would had resulted in lower costs than constructing a CAS plant and then adapting it to a MBR plant or even adding a line of conventional tertiary treatments, the most expensive of the presented solutions.

A mathematical model has been developed on Excel to design a compact MBR system based on all the information presented and discussed during this thesis. The model uses some inputs defined by the user to provide all the basic information needed to design a real MBR plant. The effects of the projected temperature have been seen transcendental, importantly affecting the biological growth and therefore the outputs related with the biological zone. Despite its relative simplicity, the model allows the user getting adapted to his particular case thanks to its high flexibility. It has been found that the SRT is not directly related with the biological growth process and therefore it can be decoupled from the HRT. It would be interesting for further developments to investigate and calculate the pH effects of the influent wastewater in the MBR system performance.

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Annex

MBR Design Model

A model has been developed on Excel to design a compact MBR system based on all the information presented and discussed during this thesis. A conventional submerged MBR configuration has been selected, separating biological and filtration zones in order to take advantage of the benefits of providing optimal fine and thick bubble aeration in each zone, respectively. Aeration inside the filtration zone is made intermittently in cycles of 40 seconds (10:30 aeration). The MBR system is preceded by a fine screen size of 0,5 mm and an equalization storage tank.

Filtration is performed outside-in by vertical hollow-fiber modules, selected because of their advantages in price, energy demand, compaction density and movement possibilities with respect to other alternatives. Specifically, Zenon cassette model ZW 500 D has been chosen. Any of these cassettes consists in 48 modules of approximately 2700 PVDF UF hollow fibers of 0,1 µm pore size, with element sizes of 0,711 x 0,643 x 0,076 m. A membrane surface area of 1.650 m^2 is included in each cassette, packed with a density of 448 m^2/m^2 resulting in 3,7 m^2 of footprint and 2,5 m of height. Some operational criteria are coming defined by the supplier recommendations: 25,4 $L/m^2 \cdot h$ of flux, 0,009 $m^3/min/m^2$ of air scour flowrate in the membrane zone and a vacuum pressure of less than 80 kPa. Relaxation is done during 1 minute in intervals of 10 minutes. Maintenance cleanings are done three times per week during 10 minutes where the membranes are backwashed and relaxed in alternating intervals of 30 seconds, two of them applying 250 mg/L of NaCOl and the third applying 2 wt% of citric acid. An intensive chemical cleaning is assumed to be done once per year.

The MBR system is designed to achieve effluent qualities similar or even better than a conventional activated sludge process followed by coagulation-flocculation, granular filtration and disinfection. Treated wastewater is able to be reused in all urban, agricultural, industrial, recreational or environmental uses which not demand absence of fecal coliforms or nutrients removal. Although the MBR system is able to achieve average coliform concentrations less than 10 CFU/100 mL and therefore almost the absence, it is recommended a second pathogen removal barrier after the MBR if the effluent is going to be reused in most restricting uses in terms of fecal coliforms.

1.1. Inputs

Influent wastewater characteristics, MBR system capacity and temperature, membrane zone flux, air scour flowrate and vacuum pressure (given by the supplier) and MLSS and SRT (decoupled from the biological process) are introduced as inputs in the Excel file from the biological zone, marked in red, to design the specific case. Some of the inputs are accompanied by notes giving recommended ranges of values.

1.2. Biological Process

1.2.1. Biological Growth

1.2.1.1. Growth rate of microorganisms [r_g (mg/L·d)]

$$r_g = \frac{dX}{dt} = \mu \cdot X \tag{1.2.1.1}$$

Where:

X: Biomass microorganisms concentration (mass/volume) = MLVSS (mg/L)

μ: Specific cellular growth velocity (time^{-1})

1.2.1.2. Monod expression applied to the specific cellular growth velocity

$$\mu = \mu_m \cdot \frac{S}{K_s + S} \tag{1.2.1.2}$$

Where:

μ_m: Maximum specific cellular growth velocity (time^{-1})

S: Dissolute substrate concentration limiting the cellular growth = Influent BOD (mg/L)

K_s: Half velocity constant. Substrate concentration for the half of the maximum specific cellular growth velocity (mass/volume) = 60 mg/L BOD

1.2.1.3. Relationship between cellular growth and substrate use

$$\mu = Y \cdot q \qquad (1.2.1.3\ [1])$$

Where:

Y: Maximum Yield constant during a logarithmic growth period. Relationship between the mass of formed cellules and the substrate consumed mass (cellular mass/substrate mass) = 0,6 mg VSS/mg BOD

q: Specific substrate use velocity (substrate mass/cellular mass·time)

Applied to maximum velocities:

$$\mu_m = Y \cdot q_m \qquad (1.2.1.3\ [2])$$

Where:

q_m: Maximum specific substrate use velocity (substrate mass/cellular mass·time) = 5 mg DQO/mg VSS·d

1.2.1.4. Substrate utilization rate [r_{su} (mg/L·d)]

$$r_{su} = -\frac{k \cdot X \cdot S}{K_s + S} \qquad (1.2.1.4)$$

Where:

k: Maximum rate of substrate utilization per unit mass of microorganisms = $\frac{\mu_m}{Y}$

1.2.1.5. Relationship between rate of growth and rate of substrate utilization

$$r_g = -Y \cdot r_{su} \qquad (1.2.1.5)$$

1.2.1.6. Endogenous decay coefficient [r_d (mg/L·d)]

$$r_d = -k_d \cdot X \qquad (1.2.1.6)$$

Where:

k_d: Endogenous decomposition coefficient (time^{-1}) = 0,06 d^{-1}

1.2.1.7. Net rate of growth [r'_g (mg/L·d)]

$$r'_g = \frac{\mu_m \cdot X \cdot S}{K_s + S} - k_d \cdot X \qquad (1.2.1.7)$$

1.2.1.8. Net specific growth rate [μ' (d^{-1})]

$$\mu' = \mu_m \cdot \frac{S}{K_s + S} - k_d \qquad (1.2.1.8)$$

1.2.1.9. Observed Yield [Y_{obs} (-)]

$$Y_{obs} = \frac{Y}{(1 + k_d \cdot \theta_c)} \qquad (1.2.1.9)$$

Where:

θ_c: Mean cell-residence time (time) = 10 d

1.2.1.10. Temperature effect on the reaction rate

$$r_T = r_{20} \cdot \theta^{(T-20)} \qquad (1.2.1.10)$$

Where:

r_T: Reaction rate at a temperature T

r_{20}: Reaction rate at a temperature of 20ºC

θ: Activity-temperature coefficient

T: Temperature (ºC)

Parameters directly affected by the temperature: μ_m $(\theta = 1{,}07)$;k_d $(\theta = 1{,}04)$

1.2.2. Biological Zone

1.2.2.1. Effluent substrate concentration [S_e (mg/L)]

$$S_e = \frac{K_s(1 + k_d \cdot \theta_c)}{\theta_c \cdot (Yk - k_d) - 1} \qquad (1.2.2.1)$$

1.2.2.2. Biological Zone Volume

$$V = \frac{Y \cdot Q \cdot \theta_c \cdot (S - S_e)}{X \cdot (1 + k_d \cdot \theta_c)} \quad (1.2.2.2)$$

Where:

V: Biological zone volume (volume)

Q: Daily capacity (volume/time)

1.2.2.3. Biomass production rate or volatile sludge mass [$P_{X,VSS}$ (kg/ d)]

$$P_{X,VSS} = Y_{obs} \cdot Q \cdot (S - S_e) \quad (1.2.2.3)$$

1.2.2.4. Total sludge mass [$P_{X,TSS}$ (kg/ d)]

$$P_{X,TSS} = P_{X,VSS}/0.85 \quad (1.2.2.4)$$

Where:

0.85: Fraction between volatile and total sludge masses

1.2.2.5. Wasting rate of the reactor

$$\theta_c = \frac{V \cdot X}{Q_w \cdot X + Q \cdot X_e} \quad (1.2.2.5)$$

Where:

Q_w: Wasting rate of the reactor (volume/time)

X_e: Volatile suspended solids concentration in the effluent (mass/volume)

1.2.2.6. Hydraulic retention time [HRT (h)]

$$HRT = \frac{V}{Q} \quad (1.2.2.6)$$

1.2.2.7. Food-to-microorganism ratio [F/M (mg BOD/mg MLVSS·d)]

$$F/M = \frac{S}{HRT \cdot X} \quad (1.2.2.7)$$

1.2.2.8. Volumetric loading rate [VLR (kg BOD/m3·d)]

$$VLR = \frac{S \cdot Q}{V} \tag{1.2.2.8}$$

1.2.3. Biological Oxygen Demand

1.2.3.1. Oxygen requirement [O_2 (kg /d)]

$$O_2 = Q \cdot \frac{(S - S_e)}{0.68} - 1.42 \cdot P_{X,VSS} \tag{1.2.3.1}$$

Where:

1.42: Cell tissue conversion factor to oxygen mass

0.68: BOD_L conversion factor to BOD_5

1.2.3.2. Theoretical air requirement [Air_t (m^3 /d)]

$$Air_t = \frac{\left(\frac{O_2}{0.232}\right)}{1.201} \tag{1.2.3.2}$$

Where:

0.232: Oxygen amount contained in air by weight

1.201: Air density (kg/m^3)

1.2.3.3. Actual air requirement [Air_a (m^3 /min)]

$$Air_a = \frac{Air_t}{0.08} \tag{1.2.3.3}$$

Where:

0.08: Oxygen transfer efficiency

1.2.3.4. Design air requirement [Air_d (m^3 /min)]

$$Air_d = Air_a \cdot 2 \tag{1.2.3.4}$$

Where:

2: Safety factor used to determine the design volume for sizing blowers

1.2.3.5. Air requirement per unit of volume [Air_q (m^3/m^3)]

$$Air_q = \frac{Air_a}{Q} \qquad (1.2.3.5)$$

1.2.3.6. Air requirement per kg of removed BOD [Air_{BOD} (m^3/kg)]

$$Air_{BOD} = \frac{Air_a}{(S - S_e) \cdot Q} \qquad (1.2.3.6)$$

1.3. Filtration Process

1.3.1. Filtration Zone

1.3.1.1. Required area of membranes [$A_{r,mem}$ (m^2)]

$$A_{r,mem} = \frac{Q}{flux} \qquad (1.3.1.1)$$

Where:

$flux$: Velocity at which the wastewater passes through a spatial unit of membrane. Defined by the supplier recommendation = 25,4 L/m^2·h

1.3.1.2. Number of cassettes [N_{cass} (-)]

$$N_{cass} = \frac{A_{r,mem}}{1650} \qquad (1.3.1.2)$$

Where:

1650: Membrane surface area in m^2 included in each Zenon ZW 500 D cassette

1.3.1.3. Designed area of membranes [$A_{d,mem}$ (m^2)]

$$A_{d,mem} = 1650 \cdot N_{cass} \quad (1.3.1.3)$$

1.3.1.4. Filtration zone minimum area [A_{min} (m^2)]

$$A_{min} = N_{cass} \cdot 3.68 \quad (1.3.1.4)$$

Where:

3.68: Zenon ZW 500 D cassette footprint in m^2

1.3.1.5. Filtration zone minimum volume [V_{min} (m^2)]

$$V_{min} = A_{min} \cdot 2.5 \quad (1.3.1.5)$$

Where:

2.5: Zenon ZW 500 D cassette height in m

1.3.2. Filtration Oxygen Demand

1.3.2.1. Theoretical air requirement [A_t (m^3/min)]

$$A_t = 0.009 \cdot A_{d,mem} \quad (1.3.2.1)$$

Where:

0.009: Air scour flowrate in $m^3/min/m^2$ recommended for the chosen membranes

1.3.2.2. Designed air requirement [A_d (m^3/min)]

$$A_d = A_t \cdot 2 \quad (1.3.2.2)$$

Where:

2: Safety factor used to determine the design volume for sizing blowers

1.4. Outputs

Important calculations obtained related with the biological and filtration zones have been collected and remarked in green inside the Excel file: volume, HRT, designed air requirement, wasting rate of sludge and the effluent BOD from the biological process and the designed area of membranes, minimum area and volume and the designed air requirement from the filtration process, respectively.

1.5. Practical Case

A practical case is attached to see the results obtained by the mathematical model calculating a hypothetical MBR plant of 5.000 m^3/d of capacity operating at 25ºC treating a given influent wastewater.

1.5.1. Influent-Effluent-MBR Characteristics

Influent Characteristics	Concentration	Units
Total Solids (TS)	720	mg/L
Dissolved Solids (DS)	500	mg/L
Suspended Solids (SS)	220	mg/L
Volatile Suspended Solids (VSS)	187	mg/L
Fraction of VSS correponding to COD	1.88	-
Nonbiodegradable VSS (VSSnb)	116.9	mg/L
Biochemical Oxygen Demand (BOD) "S"	220	mg/L
Total Organic Carbon (TOC)	160	mg/L
Chemical Oxygen Demand (COD)	500	mg/L
CODb/BOD	1.6	-
Biodegradable COD (CODb)	352	mg/L
Soluble Biodegradable COD (CODsb)	220	mg/L
Nonbiodegradable COD (CODnb)	148	mg/L
Total Nitrogen	40	mg/L
Total Phosphorus	8	mg/L
Chlorides	50	mg/L
Sulfate	30	mg/L
Alkalinity	50	mg/L
Total Coliforms	10^7	nº/100mL
pH	7	-

MBR System	Value	Units
Capacity	5000	m^3/d
Temperature	25	ºC

Biological Zone		
MLSS	11000	mg/L
VSS/SS	0.85	-
MLVSS	9350	mg/L
HRT	1.65	hours
SRT	20	days
Volume	343	m3
Minimum Dissolved Oxygen Level	2	mg/L
Designed Air requirement	71.26	m3/min
Wasting rate	33.82	m3/d

Membrane Zone		
Flux	25.4	$L/m^2 \cdot h$
Designed Area of membranes	8250	m2
Minimum Area	18.42	m2
Minimum Volume	46.04	m3
Air Scour Flowrate	0.009	$m^3/min/m^2$
Designed Air requirement	148.50	m3/min
Vacuum Pressure	< 80	kPa

Effluent Quality		
Suspended Solids (SS)	1	mg/L
Volatile Suspended Solids (VSS)	0.85	mg/L
Biochemical Oxygen Demand (BOD) "Se"	2.57	mg/L
Turbidity	0.2	NTU
Total Nitrogen	-	mg/L
Total Phosphorus	-	mg/L
Intestinal Nematodes	1	egg/10L
Fecal Coliforms	50	CFU/100mL

1.5.2. Biological Process

Biological Growth [T=20ºC]			Temperature Effects	
Parameter	Value	Units	Value	Units
rg	22039.29	mg/L·d	30911.24	mg/L·d
X	9350	mg/L	9350	mg/L
µ	2.36	d^-1	3.31	d^-1
µm	3.00	d^-1	4.21	d^-1
S	220	mg/L	220	mg/L
Ks	60	mg/L BOD	60	mg/L BOD
Y	0.51	mg VSS/mg BOD	0.51	mg VSS/mg BOD
qm	5.88	mg BOD/mg VSS·d	8.25	mg BOD/mg VSS·d
rsu	-43214.29	mg/L·d	-60610.27	mg/L·d
kd	0.06	d^-1	0.073	d^-1
rd	-561	mg/L·d	-682.54	mg/L·d
rg'	21478.29	mg/L·d	30228.70	mg/L·d
µ'	2.30	d^-1	3.23	d^-1
θc	10	d	10	d
Yobs	0.32	-	0.29	-

Biological Zone		
Parameter	Value	Units
V	343	m3
Px,vss	320.49	kg/d
Px,tss	377.04	kg/d
Qw	33.82	m3/d
HRT	0.069	d
F/M	0.34	mg BOD/mg MLVSS·d
VLR	3.21	kg BOD/m3·d

Biological Oxygen Demand		
Parameter	Value	Units
O2	1144	kg/d
Airt	4104.48	m3/d
Aira	35.63	m3/min
Aird	71.26	m3/min
Airq	10.261	m3/m3
AirBOD	47.19	m3/kg

1.5.3. Filtration Process

Zenon Cassette ZW 500 D		
Parameter	**Value**	**Units**
Membrane Surface Area	1650	m2
Packing density	448	m2/m2
Footprint	3.7	m2
Height	2.5	m

Filtration Zone		
Parameter	**Value**	**Units**
Ar,mem	8202	m2
Nº cassettes	5	-
Ad,mem	8250	m2
Amin	18.42	m2
Vmin	46.04	m3

Filtration Oxygen Demand		
Parameter	**Value**	**Units**
Air Scour Flowrate	0.009	m3/min/m2
Airt	74.25	m3/min
Aird	148.50	m3/min