

ISTANBUL TECHNICAL UNIVERSITY ★ GRADUATE SCHOOL OF
SCIENCE ENGINEERING AND TECHNOLOGY

**SYSTEM DESIGN FOR ORGANIC CARBON AND NUTRIENT REMOVAL
FROM SEWAGE BASED ON ENERGY RECOVERY**

Ph.D. THESIS

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Department of Environmental Engineering

Environmental Biotechnology Programme

DECEMBER 2012

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To my mother and my daughters Cemre & Ezgi,

FOREWORD

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ABBREVIATIONS

ATV (A 131)	: Alman standardı aktif çamur tasarım kılavuzu
A/O	: Anaerobic/Aerobic reaktor
AWSA	: Ankara Water and Sewerage Administration
BOD	: Biochemical Oxygen Demand
BNR	: Biological Nutrient Removal
CHP	: Combined heat and power
COD	: Chemical Oxygen Demand
CSTR	: Completely Stirred Reaktor
DAF	: Dissolved Air Flotation
EA	: Extended Aeration
EANR	: Extended Aeration with Nutrient Removal
EBPR	: Enhanced Biological Phosphorus Removal
EEC	: European Economic Community
ECM	: Energy Conservation Measure
EPA	: Environmental Protection Agency
EU	: European Union
HRT	: Hydraulic Retention Time
IWSA	: Istanbul Water and Sewerage Administration
MEUP	: Ministry of Environment and Urban Planning
MLE	: Modified Lutzack Ettinger
MLSS	: Mixed Liquor Suspended Solid
MSS	: Mass of suspended solids
NR	: Nitrified recycle
PAH	: Polycyclic aromatic hydrocarbon
PCB	: Polychlorinated biphenyl
PE	: Population Equivalent
RAS	: Return Activated Sludge
SBR	: Sequencing batch reactor
SCADA	: Supervisory control and data acquisition
SF	: Security factor
SRT	: Sludge Retention Time
SS	: Suspended Solids
SVR	: Sludge volume ratio
TCOD	: Total Chemical Oxygen Demand
TDS	: Total dissolved solids
TKN	: Total Kjeldahl Nitrogen
TN	: Total Nitrogen
TP	: Total Phosphorus
TSI	: Turkey Statistical Institute
UASB	: Upflow anerobic sludge blanket
UCT	: University of Cape Town
USEPA	: United States Environmental Protection Agency
UV	: Ultraviolet

UWTR	: Urban Wastewater Treatment Regulation
VFA	: Volatile Fatty Acids
VIP	: Virginia initiative plant
VSS	: Volatile Suspended Solids
WEF	: Water Environment Federation
WPCR	: Water Pollution Control Regulation
WWTPTAB	: Wastewater Treatment Plant Technical Aspects Bulletin
WWTP	: Wastewater Treatment Plants

LIST OF SYMBOLS

$\hat{\mu}_a$	: Maximum growth rate for autotrophs
$\hat{\mu}_h$	: Maximum growth rate for heterotrophs
$\dot{I}_{XB}$	: The amount of nitrogen taken from biomass
$\dot{I}_{XE}$	: The amount of nitrogen taken from inert biomass
C_S	: Biodegradable COD concentration
K_{NH}	: Half saturation constant for amonia
N_D	: Removed nitrite nitrogen
N_{DP}	: Denitrification potential
N_{OX}	: The amount of oxidized nitrogen
N_X	: The amount of nitrogen retained by biomass
OR_H	: Total oxygen demand for heterotrophic microorganisms
OR_{HS}	: Oxygen demand of easy biodegradable substrat
OU_h	: Maximum hourly oxygen consumption (total)
P_X	: The amount of biomass
P_{XA}	: The amount of biomass produced in the autotroph system
P_{XAE}	: The amount of biomass produced in the inert autotroph system
P_{XE}	: The amount of biomass produced in the inert heterotrophic system
P_{XH}	: The amount of biomass produced in the heterotrophic system
P_{XI}	: The amount of inert particulate matter accumulated in the system
S_I	: Soluble inert COD
S_{NH}	: Concentration of amonia
S_{NO}	: Concentration of oxidized nitrogen
S_{OR}	: Oxygen concentration in recycle stream
S_S	: Readily biodegradable COD
X_I	: Particulate inert COD
X_T	: Total biomass concentration in the reactor
Y_{NA}	: Net autotrophic yield coefficient
Y_{NH}	: Net heterotrophic yield coefficient
b_A	: Endogenous decay rate for autotrophs
b_H	: Endogenous decay rate for heterotrophs
f_{ES}	: Dissolved inert fraction of biomass
f_{EX}	: Particulate inert fraction of biomass
f_e	: Inert fraction of endogenous biomass
θ_{XA}	: Aerobic sludge age
θ_x	: Sludge age
A_{ST}	: Surface area of secondary settling tanks
η	: Correction factor
Q	: Flow rate
q_A	: Spesific amonia oxygen rate
q_A	: Surface overflow rate of secondary settling tanks
q_{SV}	: Sludge volume surface loading rate of secondary settling tanks
R_I	: Internal return slundge ratio
RS	: Return sludge ratio

S_E	: Effluent substrat concentration
SF	: Safety factor for nitrification
S_{NH_e}	: Ammonia nitrogen standard
SNO_E	: Nitrate-nitrogen standard
SP_d	: Daily waste activated sludge production (solids)
SS_{BS}	: Suspended solids concentration in the bottom sludge of secondary
SS_{RS}	: Suspended solids concentration of the return (activated) sludge.
TN_E	: Total nitrogen standard
$t_{SS, dim}$	: Sludge age upon which dimensioning is based
t_{Th}	: Thickening time of the sludge in the secondary settling tank
V_D	: Volume of the biological reactor used for denitrification
R	: Recycle ratio
c	: Volume correction factor

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SYSTEM DESIGN FOR ORGANIC CARBON AND NUTRIENT REMOVAL FROM SEWAGE BASED ON ENERGY RECOVERY

SUMMARY

As in the world, also in our country, which is a developing country, increase in population and economical data show an increasing demand for energy. Great part of energy demand is met by fossil fuel and hydroelectric power plants. As a result of reduction of natural sources and increase of environmental pollution, use of renewable energy sources, developing clean and sustainable energy policies, gain importance. Energy production from biomass, which has an important place in renewable energy sources, is one of the sources gradually becomes important. When energy production from biomass is considered, one of the things that should be evaluated is to obtain energy (methane gas) from removal of biomass in anaerobic digester during wastewater treatment. When the energy amount consumed by wastewater treatment plant is considered, such a situation means that some of the energy requirement of the plant is provided by the plant itself. Studies show that 2% of the energy consumed in general in our country is consumed by wastewater treatment plants. Therefore, the subjects of the use of energy potential of wastewater treatment plants and the provision of the energy requirement of the plant by the plant itself, are gaining importance.

In this study energy potential of municipal wastewater is calculated and an approach on how much of the energy demand, that is calculated depending on population, can be regained by anaerobic sludge digester is developed. This approach mainly consist of evaluating energy requirement and energy regain data obtained from two main topics of municipal wastewater treatment: design and operation.

For this purpose, first, for wastewater flow-rate and pollutant load, which are two important parameters in the design of wastewater treatment plants, input data of 30 wastewater treatment plants are evaluated statistically, for flow, COD, BOD, AKM, TKN and TP range data is defined.

Under the first main title, the evaluation of design in basis of energy, A²O process, which is one of the process used in nutrient removal for population in range of 50,000-1,000,000, and extended aeration process which is widely used in Turkey, also as N and P limits are added to discharge parameters, a third process extended aeration process with nutrient removal, are taken as basis.

These 3 design processes are calculated in ATV (A 131) program which is a German standard, active sludge design guide, and the results are compared. Also results obtained by conceptual calculation methods for A²O process are compared with values calculated using ATV. Using data obtained from conceptual designed municipal wastewater treatment plants, for 50,000 and 200,000 population equivalent, energy consumption of A²O and extended aeration processes are compared. Furthermore energy consumption depending on population of wastewater

treatment plants are calculated and evaluated by using sludge data of wastewater treatment plants with conceptual design.

The distribution of energy requirement according to units, of the A^2O process and extended aeration activated sludge process, designed in this study, is calculated and the energy requirement ratio of physical treatment units, biological treatment units and sludge treatment units are defined in percentage.

In the second part of the design, data of 15 settlements with population in the range of 45,000-1,500,000 and projected by Municipality of Environment and Urbanism (Adıyaman, Aksaray, Akşehir, Bartın, Ceyhan, Çarşamba, Diyarbakır, Erdemli, Erzurum, Lüleburgaz, Merzifon, Polatlı, Seydişehir, Siverek, Soma,) are analysed.

Within these settlements, data of energy consumption and energy regain of cities of Seydişehir, Akşehir, Aksaray, Adıyaman and Diyarbakır, where nutrient removal (A^2O and extended aeration with nutrient removal) is used, Soma, Polatlı, Ceyhan, Lüleburgaz and Erzurum, where C+N removal is used, Bartın, Merzifon, Çarşamba and Siverek, where only carbon removal is used and have aerobic and anaerobic sludge digester, are compared.

The wastewater treatment process for each city was chosen according to the limits of the receiving water, where the treated water is discharged. Among these cities, only in one city, direct discharge to the receiving water is planned. The other cities are planned, taking into account the limits of the receiving water, with binary preference of three different processes.

Under the first title of evaluation of operation data in energy base, the 2009 data of municipal wastewater treatment plants of Balıkesir, Bursa, Fethiye, Siirt, Sivas and Konya which are in operation, are analysed. Under the second title, energy consumption of wastewater treatment plant in operation in İstanbul, are compared with previous studies about municipal wastewater treatment plants and data from other countries.

In this study, the results of conventional activated sludge with nitrogen and extended aeration activated sludge with nitrogen removal were compared for their annual kwh energy per population requirement and as the energy requirement of extended aeration is higher, conventional sludge system is found to be more convenient. In both processes, it is observed that as population increase, energy consumption decreases.

Except the municipal AAT with gas utilisation, in all chosen processes, as population increases energy consumption in kwh/PE.a decreases. When these processes are compared, EA process requires more energy compared to N, N and P removal, A^2O and activated sludge processes.

In the areas where only carbon removal is efficient, for secondary treatment sludge stabilization, aerobic and anaerobic sludge stabilizations are compared. Plants that make only carbon removal and anaerobic sludge stabilisation are compared with plants that make only carbon removal and aerobic sludge stabilization are compared (Figure 8.6). As anaerobic sludge stabilisation consumes less energy, it is found to be more convenient. Also in both sludge processes, as population increases energy consumption (kwh/PE.a) decreases.

Under the light of all these considerations, energy consumption and regain ratio of municipal treatment plants are evaluated, approaches of energy consume reduction are assessed. Data on potential of treatment plant to meet the operating cost by the

amount of energy derived from wastewater treatment are obtained. As in many other countries, use of biomass energy would provide many profit in sustainable energy politics in our country.

ORGANİK KARBON VE NÜTRİENT GİDEREN KENTSEL ATIKSU ARITMA TESİSLERİNDE ENERJİ BAZLI DİZAYN SİSTEMİ

ÖZET

Dünyada olduğu gibi, gelişmekte olan ülkeler arasında yer alan ülkemizde de nüfus artışı ve ekonomik göstergeler artmakta olan enerji ihtiyacını ortaya koymaktadır. Enerji ihtiyacının büyük bir kısmı daha çok fosil yakıtlardan ve hidroelektrik santraller tarafından karşılanmaktadır. Doğal kaynakların azalması ve çevre kirliliğinin artmasıyla yenilenebilir enerji kaynaklarının kullanımı, temiz ve sürdürülebilir enerji politikalarının geliştirilmesi önem kazanmaktadır. Yenilenebilir enerji kaynakları arasında önemli bir yere sahip olan biyokütleden enerji üretimi günümüzde giderek artan şekilde değer kazanan kaynaklardan birisidir. Biyokütleden enerji elde edilmesi düşünüldüğünde değerlendirilmesi gereken bir husus da atıksuların arıtılması sırasında oluşan biyokütlenin anaerobik çürütücüde giderilmesi sırasında enerji (metan gazı) edilmesidir. Böyle bir durum, atıksu arıtma tesislerinin tükettiği enerji miktarıda göz önüne alındığında tesis için gereken enerjinin bir kısmının tesis tarafından temin edilmesi anlamına gelmektedir. Yapılan çalışmalardan ülke genelinde tüketilen enerjinin yaklaşık % 2'sinin kentsel arıtma tesislerinde harcandığı ortaya çıkmaktadır. Bu sebeple kentsel arıtma tesislerindeki atıksuyun enerji potansiyelinin kullanımı ve kentsel atıksu arıtma tesisinin kendi enerji ihtiyacını karşılaması konusu önemini artırmaktadır.

Bu çalışmada Kentsel atıksuların enerji potansiyeli hesaplanarak, nüfusa bağlı olarak hesaplanan enerji ihtiyacının ne kadarlık kısmının anaerobik çamur çürütücü ile geri kazanılabileceği üzerinde bir yaklaşım geliştirilmiştir. Bu yaklaşım esas olarak kentsel atıksu arıtma tesislerinin iki ana başlık olan tasarım ve işletme verilerinden çıkan enerji ihtiyacı ve enerji geri kazanımı kıyaslama verilerinin değerlendirilmesinden oluşmaktadır.

Bu amaçla öncelikle atıksu arıtma tesislerinin tasarımında önemli iki parametre olan atıksu debisi ve kirlilik yükleri için 30 kentsel atıksu arıtma tesisleri giriş verileri istatistiki olarak değerlendirilerek, debi, COD, BOD, AKM, TKN ve TP için aralık verileri belirlenmiştir.

Birinci ana başlık olan tasarımın enerji bazında değerlendirilmesinde, nüfusu 50,000-1,000,000 aralığındaki tesisler için nütrient gideriminde kullanılan proseslerden biri olan A^2O prosesi ile Türkiye'de yaygın kullanımı olan Uzun havalandırmalı aktif çamur prosesi ve deşarj parametrelerine N ve P limitlerinin eklenmesiyle, uzun havalandırmalı aktif çamur prosesine ilave olarak nütrient giderimi prosesleri esas alınmıştır.

Bu üç tasarım prosesi Alman standardı, aktif çamur tasarımı kılavuzu olan ATV (A 131) programında hesaplanmış sonuçlar birbiriyle kıyaslanmıştır. Ayrıca A^2O prosesi için geleneksel hesaplama yöntemi ile elde edilen sonuçlar ATV'den hesaplanan değerlerle karşılaştırılmıştır. Klasik tasarım yöntemiyle yapılan kentsel atıksu arıtma tesisi verilerinden 50,000 ve 200,000 eşdeğer nüfus için A^2O ve uzun havalandırmalı

aktif çamur prosesleri enerji tüketimleri kıyaslanmıştır. Ayrıca kentsel atıksu arıtma tesislerinin nüfusa bağlı enerji potansiyeli, klasik tasarım yöntemiyle yapılan kentsel atıksu arıtma tesisi çamur verileri kullanılarak hesaplanmış ve yorumlanmıştır.

Bu çalışmada tasarımı yapılmış A^2O prosesi ile uzun havalandırmalı aktif çamur prosesi enerji gereksiniminin ünitelere göre dağılımı hesaplanmış ve fiziksel arıtma üniteleri, biyolojik arıtma üniteleri ve çamur arıtımı ünitelerinin enerji gereksinim oranları yüzde olarak belirlenmiştir.

Tasarımın ikinci aşamasında, Çevre ve Şehircilik Bakanlığı tarafından projelendirilen, nüfusu 45.000-1.500.000 aralığında olan, 15 şehir için kentsel atıksu arıtma tesisi tasarımı yapılmıştır. Her bir şehir için tasarlanan atıksu arıtma tesisi prosesi, arıtılan suyun deşarj edeceği alıcı ortam limitlerine göre seçilmiştir. Bu şehirlerden sadece bir tanesinde direk alıcı ortama deşarj planlanmıştır. Diğer şehirler alıcı ortam deşarj limitleri dikkate alınarak 3 farklı prosesin ikili tercihi olarak tasarlanmıştır.

Bu yerleşimlerden nütrient giderimi (A^2O ve nütrient gideren uzun havalandırmalı aktif çamur) yapılan Seydişehir, Akşehir, Aksaray, Adıyaman, ve Diyarbakır şehirleri, C+N giderimi yapılan Soma, Polatlı, Ceyhan, Lüleburgaz ve Erzurum, sadece carbon giderimi yapıp aerobik ve anaerobik çamur çürütücüsü olan Bartın, Merzifon, Çarşamba ve Siverek yerleşimlerinin enerji tüketim ve geri kazanım verileri kıyaslanmıştır.

Bu çalışmada nütrient gideren klasik aktif çamur ile nütrient gideren uzun havalandırmalı aktif çamur tasarım sonuçları yıllık nüfus başına gerekli enerji ihtiyaçları açısından karşılaştırılmış ve uzun havalandırmalı aktif çamur prosesi enerji ihtiyacı daha fazla olduğundan klasik aktif çamur sistemi daha uygun bulunmuştur. Her iki proseste de nüfus arttıkça enerji tüketiminin azaldığı görülmektedir.

Gaz çevrim üniteleri bulunan kentsel AAT dışında, seçilen bütün proseslerde nüfus arttıkça kwh/PE.a cinsinden enerji tüketimi azalmaktadır. Bu prosesler karşılaştırıldığında, uzun havalandırmalı aktif çamur prosesi gerek N, gerekse Nve P gideriminde, A^2O ve klasik aktif çamur prosesine göre daha fazla enerji gerektirmektedir.

Sadece karbon gideriminin yeterli olduğu alanlarda, ikincil arıtma çamur stabilizasyonu için aerobik ve anaerobik sludge stabilizasyonu karşılaştırılmıştır. Sadece karbon giderimi yapıp anaerobik ve aerobik çamur stabilizasyonu yapan tesisler karşılaştırılmış ve anaerobik çamur stabilizasyonun daha az enerji tükettiğinden daha uygun bulunmuştur. Her iki çamur prosesinde de nüfus arttıkça birim enerji tüketimi (kwh/PE.a) azalmaktadır.

İşletme verilerinin enerji bazında değerlendirilmesinin birinci başlığında işletilmekte olan Balıkesir, Bursa, Fethiye, Siirt, Sivas ve Konya kentsel atıksu arıtma tesislerinin 2009 yılı verileri analiz edilmiştir. Bu verilerden kentsel atıksu arıtma tesislerinin enerji tüketimleri, eşdeğer nüfus başına yıllık kwh enerji tüketimi ve m^3 atıksu başına enerji tüketimi kwh olarak hesaplanmıştır. İkinci başlığında, İstanbul'da işletilmekte olan atıksu arıtma tesislerinin enerji tüketimleri, daha önce kentsel arıtma tesisleri ile ilgili yapılmış çalışmalarla ve diğer ülkelerin verileri ile karşılaştırılmıştır.

Tüm bu değerlendirmeler ışığında, nüfusa bağlı olarak kentsel arıtmalardaki, enerji tüketim ve geri kazanım oranları değerlendirilmiş, enerji tüketiminin azaltılması yaklaşımları değerlendirilmiştir. Atıksu arıtmadan elde edilecek enerji miktarının,

arıtma tesisinin işletme maliyetlerini karşılama potansiyeline ilişkin veriler elde edilmiştir. Diğer bir çok ülkede olduğu gibi ülkemizde de biyokütle enerjisinin değerlendirilmesi sürdürülebilir enerji politikalarında önemli getiriler sağlayacaktır.

1. INTRODUCTION

1.1 Purpose of the thesis

The increasing energy requirement in the world and in our country as the result of the rapid increase in population, uncontrolled use of sources and energy demand, is also apparent in wastewater treatment plants as in all areas. The studies in the literature show that energy consumption in the municipal wastewater treatment plants make up 15-30% of the cost in the big plants and 30-40% of the cost in the small plants. When the energy requirement of the wastewater treatment plant is met by the energy obtained from anaerobic sludge units, it can help to prevent the waste of sources. Due to the high proportion of energy consumption in the operating costs, studies about the energy recovery and reuse in municipal wastewater treatment plants are gaining importance.

In this study entitled Energy-based Approach in Municipal Wastewater Treatment Plants, the aim was to evaluate the per capita energy requirement of activated sludge systems which are widely used in carbon and nutrient removal in municipal wastewater treatment plants and an approach on the population dependent energy potential in municipal wastewaters, the energy requirement for treatment and the amount of energy which can be recovered by anaerobic sludge digesters, is developed.

There are practice related problems in urban treatment plants in Turkey due to the absence of databank including operation information of present plants and inadequate analyses of local conditions.

Evaluation of characteristics related to population and district is becoming unlikely since the design parameters are being used according to literature due to the measurements with single sampling and difficulties in performing long, season-related measurements which finally led to an operation and investment problems caused by design errors.

Operation records and measurement results of present treatment plants should be considered for determining the characterization and quantity of wastewater in planning of urban treatment plants. Data should also be collected in databank formed by Ministry of Environment and Urbanization and employees in charge of planning infrastructure plants should be provided access to this databank.

The main goal of this study is to evaluate urban wastewater treatment plants as an energy source to determine minimum population value that meets the energy requirement of treatment plants. The energy potential that provides additional energy depending on population is also researched. Optimum values of converting urban treatment plants of sites populated over 50,000 person into energy production facilities are evaluated.

According to the data taken from TSI, flow quantity of municipal wastewater collected with sewerage system is approximately 3.26 billion m³. 1.51 m³ of this volume is treated in biological and advanced wastewater treatment plants. There is a growing interest in taking advantage of the energy potential in wastewater treated with correct design according to the regulations and in meeting the energy requirement of wastewater treatment plant itself.

1.2 Scope of the thesis

In the context of energy-based approach in municipal wastewater treatment plant, an approach, from the changes in the influent loads of the municipal wastewater treatment plants, to the energy content of the wastewater and its capacity to supply its own energy, is developed.

Within this context, the general profile of existing municipal wastewater treatment plants in our country are examined and the statistical data related to municipal wastewater treatment conditions, infrastructure and the receiving water that these wastewaters are discharged, are evaluated.

Municipal wastewater in Turkey is examined in all municipalities, and information on the percentage of the residential wastewater collected by the sewage systems, on the percentage of the wastewater that is treated in treatment plants (physical, biological and advanced treatment) and the type of receiving waters to which the wastewater is discharged are provided with statistical data.

In the third chapter, two important parameters for the design of wastewater treatment plants, namely the wastewater flow rate and pollution loads, are evaluated in terms of municipal wastewater treatment plants; studies conducted on influent wastewater (15 different municipal wastewater treatment plants that are designed), the values used in the designing of the treatment plants in operation and their operation conditions (15 different municipal treatment plants in operation) as well as values used in other countries, are presented. Pollution loads on population basis given in Table 3.33 are used for determining the influent characteristics of municipal wastewater treatment plant.

In the 4th chapter, literature search on physical and biological treatment units, that are used extensively in municipal wastewater treatment and sludge treatment processes are conducted and activated sludge systems with nutrient removal are emphasized in biological treatment.

In the 5th chapter, entitled Energy Requirement of Wastewater Treatment Plants, the amount of energy consumption in the municipal treatment plants with different populations obtained from the literature, is compared with the amount of energy obtained from anaerobic sludge digesters and results are provided.

In the 6th chapter, the COD, BOD, SS, Total N and Total P values of municipal wastewater treatment plants operated for Turkey in general are used to evaluate different design approaches. Treatment plant is designed and the results are evaluated considering the influent loads for different population values (50000, 100000, 200000, 400000, 1000000). Using those inputs in ATV-131 and conceptual design, outputs of aerobic part in biological treatment are obtained.

In the design processes, the reactor volume, the amount of oxygen consumption and the amount of sludge, which are obtained from ATV program or calculated in conceptual design of A^2O process, are compared.

In the 7th chapter, the energy requirement of municipal wastewater treatment plants are evaluated under four titles, based on data of the treatment plants operated in İstanbul and in different cities in Turkey, data of 14 municipal wastewater treatment plants designed by Ministry of Environment and Urbanization and data of the wastewater treatment plants designed in the 6th chapter of this study.

From the unit wastewater loads calculated by the evaluated design parameters, energy potentials depending on population are calculated. Data have been obtained concerning the meeting potential of energy produced in wastewater treatment in covering the operation expenditures in treatment plant.

Distribution of energy demand and energy production of chosen model treatment plant are given separately based on the treatment units. With these data, population based energy potentials are studied using unit wastewater loads. Data, on the potential of the amount of energy obtained from wastewater treatment to meet the cost of operation of the plants are determined.

Facility operation will gain importance when wastewater is accepted as an energy source and the operation expenditures are defrayed from its own energy output. Mainly effluent analyses are used to measure the performance of treatment plants. Biogas quantity produced per day will allow a more concrete control of system performance when wastewater is considered as an energy source.

2. THE INFRASTRUCTURE SYSTEM OF WASTEWATER IN TURKEY AND LEGAL REGULATIONS

2.1 Introduction

In this chapter, the conditions of the existing municipal wastewater infrastructure is examined. Municipal wastewater is examined on the basis of municipality and information on the percentage of the residential wastewater collected by the sewage systems, on the percentage of the wastewater that is treated in treatment plants (physical, biological and advanced treatment) and the type of receiving waters to which the wastewater is discharged, are provided with statistical data.

2.2 The Infrastructure System Of Wastewater In Turkey

According to the results of Municipal Wastewater Statistics Survey, which was applied to all municipalities, 2,421 municipalities out of 3,225 municipalities were served by sewerage systems. It is determined that, in 2008, municipal population that is served by sewerage systems has a share of 73% in Turkey's population and a share of 88% in total municipal population. Population ratios based on sewerage and treatment services provided between 2001 and 2008 are given in the Table 2.1.

In 2008, out of 3.26 billion m³ of wastewater collected by sewerage systems, 44.7 % was discharged into sea, 43.1% into rivers, 3.5% into dams, 2.1% into lakes and artificial lakes, 1.5% on to land, and 5.1% to other receiving bodies. Data regarding the receiving bodies that wastewater discharges were made between 2001 and 2008 are given in Table 2.2.

Table 2.1 : Number and population of municipalities served by sewerage systems and wastewater treatment plants (TSI,2008).

Year	Number of municipalities	Total municipal population	Number of municipalities served by sewerage system	Municipal population served by sewerage system	Rate of population served by sewerage system in total municipal population (%)	Number of municipalities served by WWTP	Municipal population served by WWTP	Municipal population served by WWTP (%)
2001	3 227	53 407 613	2 003	43 034 156	81	238	18 455 498	35
2002	3 227	53 421 379	2 115	44 342 222	83	248	18 955 305	35
2004	3 225	53 935 050	2 226	46 149 479	86	319	24 369 119	45
2006	3 225	58 581 515	2 321	50 856 943	87	362	29 643 258	51
2008	3 225	58 581 515	2 421	51 673 078	88	442	32 518 318	56

Table 2.2 : Amount of wastewater discharged from municipal sewerage by receiving bodies (TSI,2008).

Year	Number of municipalities questionned ⁽¹⁾	Total amount of wastewater discharged (1000 m ³ /year)	Sea (1000 m ³ /year)	Lake and Artificial Lake (1000 m ³ /year)	River (1000 m ³ /year)	Land (1000 m ³ /year)	Dam (1000 m ³ /year)	Other ⁽²⁾ (1000 m ³ /year)	Wastewater discharged per capita in municipalities (l/capita-day)
2001	3 215	2 301 152	836 493	37 971	1 223 002	41 353	88 942	73 390	147
2002	3 215	2 497 657	885 981	38 403	1 356 297	37 013	96 434	83 528	154
2004	3 213	2 922 783	1 178 001	43 006	1 380 516	40 007	99 551	181 702	174
2006	3 225	3 366 894	1 522 695	46 415	1 410 614	120 525	121 532	145 113	181
2008	3 225	3 261 455	1 458 461	67 193	1 404 164	50 374	115 405	165 857	173

⁽¹⁾ Number of metropolitan municipalities are included.

⁽²⁾ Refers to wastewater discharges to septic tanks, carstic formations, etc.

There were 236 municipal wastewater treatment plants serving 442 municipalities in 2008. 29 of wastewater treatment plants were physical, 158 were biological, 32 were advanced, and 17 were natural. Out of 3.26 billion m³ of wastewater discharged via sewerage, 2.25 billion m³ was treated in wastewater treatment plants. The rate of biological treatment was 38.3%, while the rate of physical treatment was 32.7%, advanced treatment was 28.8% and natural treatment was 0.3%. Rate of population served by wastewater treatment plants was 46% in Turkey's population, and it was 56% in total municipal population. The ratios according to the treatment options between 2001 and 2008 are given in Table 2.3.

Table 2.3 : Status of wastewater treatment plants (TSI,2008).

	(1000 m ³ /year)			
Year	2002	2004	2006	2008
Total number of treatment plants	145	172	184	236
Total capacity	2,358,507	3,410,352	3,648,198	4,143,140
Total amount of wastewater treated	1,312,379	1,901,040	2 140 494	2,251,581
Number of physical treatment plants	28	35	26	29
Physical treatment capacity	771,081	1,384,634	1,329,470	1,537,719
Amount of wastewater treated physically	344,509	598,769	714,404	735,710
Number of biological treatment plants	114	133	135	158
Biological treatment capacity	1,320,124	1,750,532	1,510,835	1,594,640
Amount of wastewater treated biologically	745,852	1,071,217	926,581	861,428
Number of advanced treatment plants	3	4	23	32
Advanced treatment capacity	267,302	275,186	807,893	1,000,814
Amount of wastewater treated by advanced method	222,018	231,054	499,509	648,536
Number of natural treatment systems	-	-	-	17
Natural treatment system capacity	-	-	-	9,967
Amount of wastewater treated by natural treatment systems	-	-	-	5,906

Number of Wastewater Treatment Plants (WWTP) according to population groups are given in Table 2.4 based on the data taken from Turkey Statistical Institute (TSI,2008).

Table 2.4 : Number of municipal WWTPs based on population (MEUP 2009, TSI2008).

Population Groups	Number of Settlement	Sewage Connection Ratio	Number of WWTPs (secondary +advanced)
≥100,000	152	81.3%	76
50,000-99,999	96	29.9%	18
10,000-49,999	317	21.5%	46
2,000-9,999	1455	8.2%	62

2.3 Legal regulations

Turkey aims to join the European Union in near future and towards this goal, acquis must be put into practice after rearranged on Turkey basis. Number of large and small scaled wastewater treatment plants should be designed in scope of meeting the clause conformity of European Union Urban Wastewater Treatment Directive. In the directive mentioned, all of the water bodies in the country are evaluated as sensitive areas. Advanced wastewater treatment plants are necessary for all sites with population equivalent of 10,000 or above 10,000 related to the harmony with this directive. In addition, sites with range of population equivalent between 2,000-10,000 require the design of suitable treatment plants.

Considering the legislation in our country, Environmental Law (26.04.2006 revision, Official Journal of 09.08.1983) constitute the basis of legal regulations about environment. There are three legislation and two bulletin that determine urban wastewater discharge and treatment in scope of Europe Urban Wastewater Treatment Directive (91/271/EEC), Water Pollution Control Regulation (Official Journal of 4.09.1998, revised in 24.04.2011, no 27914), Urban Wastewater Treatment Regulation (Official Journal of 08 January 2006, no 26047), Sensitive and Less Sensitive Water Areas Bulletin (Official Journal of 27.06.2009, no 27271), Land Application of Domestic and Municipal Treatment Sludges Regulation (Official Journal of 3.08.2010, no 27661) and Wastewater Treatment Plants Technical Aspects Bulletin (WWTPTAB) (Official Journal of 20.03.2010, no 27527).

In Water Pollution Control Regulation, domestic wastewaters are classified according to pollution loads and required values of wastewaters coming from domestic wastewater sources directly and/or after treated in urban wastewater treatment plants are given. These values will be applied until 31.12.2014. Provisions in Urban Wastewater Treatment Regulation will be applied for the sites populated over 2,000 person since 31.12.2014 instead of Water Pollution Control Regulation. Discharge limits in both regulations are given comparatively in Table 2.5

Table 2.5 : The comparison between discharge limits in the water pollutant control regulation and the urban wastewater treatment regulation.

REGULATION PARAMETER	Water Pollution Control Regulation(1998)								Urban Wastewater Treatment Regulation (2006)
	P= 84-2000		P= 2000-10000		P=10000-100000		P > 100000		
	5-120 kg/day 2 hour 24 hour		120-600 kg/day 2 hour 24 hour		600-6000 kg/day 2 hour 24 hour		>6000 kg/day 2 hour 24 hour		
BOD ₅ (mg/l)	50	45	50	45	50	45	40	35	25
COD(mg/l)	180	120	160	110	140	100	120	90	125
SS(mg/l)	70	45	60	30	45	30	40	25	35 ⁽¹⁾ 60 ⁽²⁾
TP(mg/l)									2 ⁽³⁾ 1 ⁽⁴⁾
TN(mg/l)									15 ⁽³⁾ 10 ⁽⁴⁾

(1) (>10,000 P.E.) (3) (10,000-100,000 P.E.)

(2) (2,000-10,000P.E.) (4) (>100,000 P.E.)

The nitrogen and phosphate derived from domestic and industrial discharges cause eutrophication problems in receiving water bodies which generally limits the potential use of receiving water due to algal activity unless the input of nutrients is reduced and/or controlled based on legislation. To protect water bodies from eutrophication, in Europe, the EEC Directive 91/271 (CEC, 1991) enforces discharge standards with respect to total nitrogen and phosphate within sensitive areas. Discharge limits for municipal wastewaters are set based on protection of receiving bodies from eutrophication within the harmonization process of EU legislation. In our country, for this purpose, Urban Wastewater Treatment Regulation and Sensitive and Less Sensitive Water Areas Bulletin are added to our legislation. With the regulations added, it can be seen that the discharge limits are reduced and N and P limits are added to the discharge parameters of municipal wastewater.

Population is an indicative factor in discharge limitations in regulations. As it can be seen on Table 2.6, strict implementations are applied by reducing N and P limits for the populations over 100,000 person while for especially sensitive sites populated over 10,000 person is desired advanced treatment. Treatment requirements in Urban Wastewater Treatment Regulation are given in Table 2.6 depending on population, location of district and discharge point. In addition, in clause 4 article 7 of Sensitive and Less Sensitive Water Areas Bulletin is stated “Wastewater treatment plants planned to design in a wastewater collection areas with population equivalent of 10,000 people are designed in the harmony with the declaration possibility of water body as a sensitive area”.

As it can be seen on Table 5 in Urban Wastewater Treatment Plant Regulation (UWTR) and clause 5.6 and 12, treatment option is defined separately according to the characteristic of receiving body (fresh water, gulf, coastal seas, sensitive areas, etc.) by taking population equivalent into consideration. Primary treatment option is decided to be used in cases environment in less sensitive water areas are not affected negatively (clause 5.b). Municipal wastewater discharges to less sensitive coastal seas and estuaries are stated possibly to be more flexible than secondary treatment under the condition of not being less than primary treatment (clause 12.a).

Table 2.6 : Population Equivalent Basis Treatment Options (UWTR,2006).

POPULATION	DISCHARGE POINT	TREATMENT OPTION
General Provisions		
<2,000 P.E.	Fresh water, discharge to estuary	Suitable treatment ^(clause 6.c)
<10,000 P.E.	Discharge to coastal sea	Suitable treatment ^(clause 6.c)
2,000-10,000 P.E.	Fresh water, discharge to estuary	Secondary treatment ^(clause6.d)
>10,000 P.E.		Secondary treatment ^(clause6.d)
Sensitive and Less Sensitive Areas		
> 10,000 P.E.	Sensitive Areas	Advanced Treatment (clause8.a.3)
> 10,000 P.E.	Less sensitive areas	Min.primary treatment (clause12.a)
> 10,000 P.E.	Except sensitive and less sensitive areas	Secondary treatment (clause 8.a, 8.a.2)

Construction and operation of treatment plants, including collection system are scheduled. In addition, advanced treatment technologies for the sites populated over 10,000 person providing nitrogen and phosphorus removal and time period until the end of 2022 are provided for completion of other regulation requirements.

Discharge permit of urban wastewater systems to a receiving body is dependent on the provisions about deepwater discharge in the Water Pollution Control Regulation (UWTR. clause 5g).

According to Sensitive and Less Sensitive Water Areas Bulletin and Urban Wastewater Treatment Regulation, determination and monitoring of sensitive and less sensitive areas and procedures and principles of discharges to be made to these areas are designated.

According to Land Application of Domestic and Municipal Treatment Sludges Regulation, limitations related to the soil application of the treatment sludges coming from municipal wastewater treatment plants under suitable conditions are set in a controlled manner that it will not harm soil, plant, animal and human.

Another legislation related to urban wastewater treatment plants is Energy Efficiency Law published in official journal numbered 26510 on 02.05.2007. The aim of this law is to provide efficient use of energy, to prevent unnecessary expense of energy, to reduce the load of energy costs on economy, to increase energy sources and efficient usage of energy in order to protect environment.

It is necessary to design urban wastewater treatment plants in the light of Energy Efficiency Law and operation of treatment plants are required to follow the Energy Efficiency Legislation.

2.4 Conclusions

It is understood from the statistical data explained in this chapter, that for municipal wastewater in Turkey, 804 municipality out of 3225 do not have a sewer system but 81% of the residential areas with populations over 100,000 have a sewer system.

Again according to the data of TSI, there are 236 municipal wastewater treatment plants serving to 46% of the population in Turkey. According to the data taken from TSI, flow quantity of municipal wastewater collected with sewarege system is

approximately 3.26 billion m³. 1.51 m³ of this volume is treated in biological and advanced wastewater treatment plants. There is a growing interest in taking advantage of the energy potential in wastewater treated with correct design according to the regulations and in meeting the energy requirement of wastewater treatment plant itself.

3. THE CHARACTERIZATION AND QUANTITY OF MUNICIPAL WASTEWATER

3.1 Introduction

In this chapter, the aim is the evaluation of wastewater flowrate and pollution loads of wastewater in municipal wastewater treatment plants, which are two important parameters in wastewater treatment plant design. Studies conducted on influent wastewater in Turkey, the values used in the designing of the treatment plants in operation and their operation conditions as well as values used in other countries are presented. Moreover, applications of Rural Affairs and Bank of Provinces and are evaluated.

In this context, the change of influent characteristics of different municipal treatment plants based on population are evaluated and the influent characteristics of the same plant in different years are compared.

3.2 Quantity of municipal wastewater

Wastewater formation varies worldwide between 80 l/p.d and 275 l/p.d. Domestic wastewater flowrates per year designated for various countries are given in Table 3.1.

Table 3.1 : Unit wastewater flowrates for countries (Henze,1995).

Country	Wastewater flowrate (l/p.d)
Germany	150
Belgium	80
Denmark	150
France	95
Holland	135
England	165
Spain	245
Sweden	230
Switzerland	275
Italy	230
Norway	150
Turkey	173*

*Unit wastewater value for 2008.(TSI)

According to data taken from TSI, wastewater quantity discharged in 2008 in 81 cities per person are given in Figure 3.1 depending on population. Average wastewater quantity is determined in 2006 and 2008 as 181 l/p.d and 173 l/p.d, respectively. Starting from 147 l/p.d in 2001, this quantity had an upward trend until 2006, following with a decrease in 2008. The decrease in wastewater quantity can be explained by the decrease in leak flowrate coming from groundwater due to the renovations in collection system.

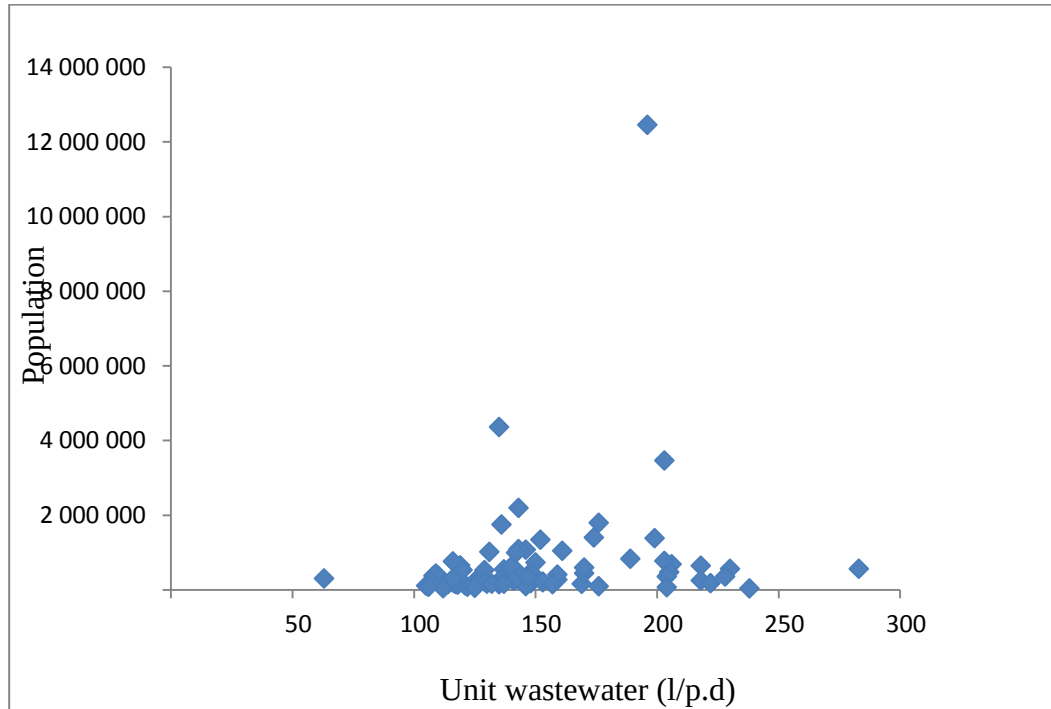


Figure 3.1 : Population based unit wastewater quantities in 2008 (TSI).

In studies performed in Turkey, a water distribution pattern is divided into five groups. These consumers are (1) governmental institutions, (2) commerce and industry sectors, (3) residential buildings, (4) construction sectors and (5) parks, gardens, public fountain and etc. 65-72 % of distributed water was used at residential buildings. Institutional water consumption is very high for especially low populated cities ($P < 100,000$) (Öztürk, et al.,2007).

The relation between water consumption and wastewater formation has many definitions defined by various institutions. Bank of Province “Technical specifications for preperation of drinking water projects” (1985) and “General Directorate of Rural Services” determine the future population based unit water consumption in rural areas.

The values given in Table 3.2 include road, house garden, park, marketplace, motor vehicles, school, hospital, slaughterhouse, otel, etc., as well as the water losses in water supply network (Orhon et al., 2000).

Table 3.2 : Change in water consumption on population basis.

Future Population	Rural Services (l/p.d)	Bank of Province Project Specification (l/p.d)
<3,000	60	60
3,001-5,000	60-80	70
5,001-10,000	70-80	80
10,001-30,000	-	100
30,001-50,000	-	120
50,001-100,000	-	170
100,001-200,000	-	200
200,001-300,000	-	225

Domestic, industrial and infiltration rate are taken into account for calculation of wastewater flow rate. According to Bank of Province flow is calculated the following formula;

$$Q_{\text{municipal}} = Q_{\text{dom}} + Q_{\text{comm}} + Q_{\text{infiltr}} + Q_{\text{other}} \quad (3.1)$$

$$Q_{\text{design}}(\text{m}^3/\text{h}) = Q_{\text{municipal}}/n_1 + Q_{\text{indust}}/n_2 + Q_{\text{infiltr}}/24 \quad (3.2)$$

$$Q_{24}(\text{m}^3/\text{h}) = (Q_{\text{municipal}} + Q_{\text{indust}} + Q_{\text{infiltr}})/24 \quad (3.3)$$

$$Q_{\text{min}}(\text{m}^3/\text{h}) = Q_{\text{municipal}}/n_3 + Q_{\text{infiltr}}/24 \quad (3.4)$$

$$Q_{\text{max}}(\text{m}^3/\text{h}) = Q_{\text{municipal}}/n_4 + Q_{\text{indust}}/n_2 + Q_{\text{infiltr}}/24 + Q_{\text{manhole}} \quad (3.5)$$

$$(Q_{\text{manhole}} = Q_{\text{municipal}} * 0.1/24) \quad (3.6)$$

The coefficients used in the equations are summarised in Table 3.3.

Table 3.3 : Coefficients used in flow rate calculations for wastewater treatment plant design (Bank of Province).

Range		Selection	Explanation
n1	Population, N	Value	Reflects the oscillations in wastewater generation within a day. The value is the function of population.
	N < 1000	10	
	1001 < N < 10,000	12	
	10,001 < N < 100,000	14	
	100,001 < N < 1,000,000	16	
	N > 1,000,000	18-20	
n2	5 – 24	Conditional (8 - 24)	Reflects the daily shift hours of industrial activity. For example, for 3 shifts per day, n2 is taken as 24.
n3	37 – 40	37	Reflects the approximate minimum flow rate after midnight.
n4	12 – 14	12	

In wastewater flowrate predictions for İstanbul made by İstanbul Water and Sewerage Administrator (IWSA), a wide interval is given for unit domestic wastewater flowrate based on counties and years. Annual unit wastewater formations determined in other studies for İstanbul are given in Table 3.4.

Table 3.4 : Unit wastewater values for İstanbul (Orhon,2000).

Year	Camp Tekser	Watson Motor Temel
	(1975)	(1986)
	(l/p.d)	(l/p.d)
1990	130-220	160
2000		200
2010		220
2020	215-290	250

Leak and industrial wastewater flowrates are considered as well as domestic wastewater flowrate in evaluation of wastewater quantity reaching to urban wastewater treatment plants.

Water consumption is taken into account in cases that design flowrate of urban wastewater treatment plants can not be measured by monitoring programs. Suggested value interval for domestic water consumption is 20-65 l/p.d and world average is given as 35-90 l/p.d (Metcalf & Eddy,2003).

Certain ratio of water consumption values is assumed to turn into wastewater (70 - 80%). In consequence of this assumption, wastewater quantity differs within a day as well as it can change day by day. Decision within scientific facts are made by considering location of district, life standard, climate conditions, industrial wastewater discharges, condition of infrastructure system (loss and leakage) as well as the water consumption values (Orhon, et al., 2000).

Calculation of wastewater quantity based on water use brings forward the loss and leak rates in water supply network. Reduction in losses in water supply networks in especially big cities with the purpose of beneficial use of limited sources is becoming more of an issue.

As it is shown in Figure 3.2, approximately 50% of the treated drinking water in city centers with population more than 500,000 is lost and unaccounted in Turkey. On the average, 30% of treated water is lost in distribution systems and leakages from storage tanks. Unaccounted usage of water is about 20%, while only 50% of distributed water is used for residential, industrial and other consumptions. It was reported that total water loss and illegal usages is less than 30% in limited residential places. On the other hand, the level of average water loss is about 20% in European Union (EU) Countries and some countries have water loss levels lower than 10% (Ozturk, et al., 2007).

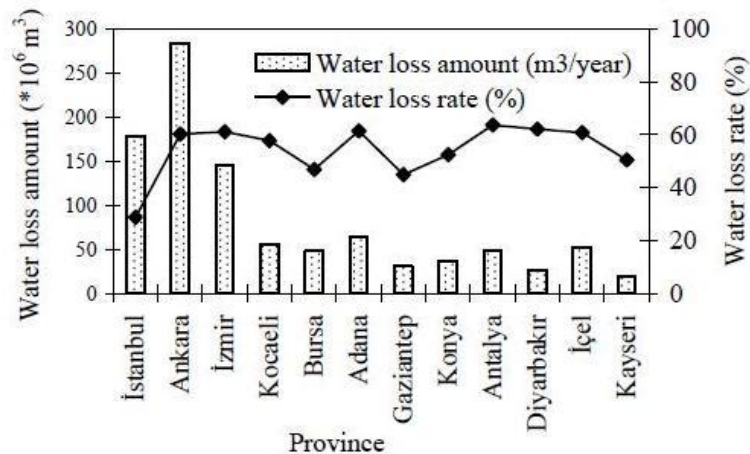


Figure 3.2 : Amount and rates of water loss in metropolitan city centers (Ozturk, 2007).

Based on the data of year 2003 of Turkish Statistical Institute (TSI), across Turkey, the average water loss per year is as high as 51% of the water volume produced (TSI,

2003) As can be seen in these figures, the highest amount of water loss was obtained in Ankara, which was 283,102,040 and the highest percentage of water loss was obtained in Kars, which was 80%. The amount of total water loss for all provincial centers was 1,508,901,825 m³/year and the average water loss was 51% in Turkey, in year 2003. Variation of water loss in terms of annual amounts and percentages with population classes of city centers are given in Figure 3.3. On the other hand the summary in Figure 3.3, shows that the highest amount of water loss was observed for provinces with a population that varied between 100,000 and 500,000 (Ozturk et al., 2007).

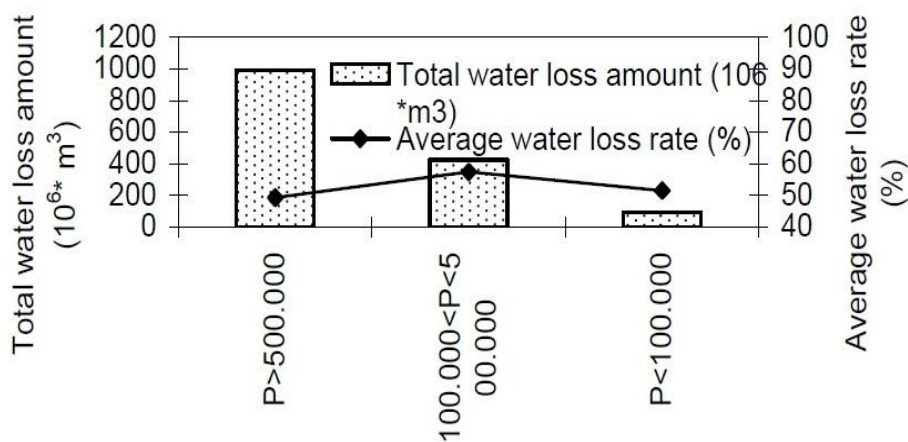


Figure 3.3 : Total amount and average rates of water loss in city centers (Ozturk,2007).

Wastewater generations depending on population in Turkey are given in Wastewater Treatment Plants Technical Aspects Bulletin (Official Journal of 20.03.2010, no 27527). Values related to population interval are given in Table 3.5.

Table 3.5 : Wastewater formation on population basis (WWTPTAB,2010).

Population range	Wastewater formation
	l/p.d
2,000- 1,0000	80
10,000-50,000	90
50,000-100,000	100

Flowrates given in Table 3.5 does not include the leak flowrates to the sewerage system. Quantity of leak flowrate from groundwater reaching to wastewater collection system varies depending on groundwater level and the condition of

collection system. Unit leak flowrate shows changes depending on the height of groundwater level in district, soil structure, leakage ratio of water supply network, the age of sewerage system, whether the site is located seaside or not and etc. (Wastewater Treatment Plants Technical Aspects Bulletin, Official Journal of 20.03.2010, no 27527).

In a study performed in 2004, measurement results gained from wastewater treatment plants operated in 5 different sites with different capacities are evaluated and the values for the design of urban treatment plants are suggested in the Table 3.6 (Erdogan, 2004).

Table 3.6 : Water consumption on population basis and wastewater formation.

Population	Water consumption/p.d	Amount of wastewater l/p.d
5,000 - 10,000	110	80
10,001 - 35,000	120	90
35,001 - 150,000	130	100
150,001 - 400,000	160	125
400,001 - 1,000,000	175	140
1,000,001 -2,000,000	200	160

Influent data for 6 urban wastewater treatment plants in 2009 are evaluated and the results are given in Table 3.7. Annual average wastewater values are calculated as 152 l/p.d and 183 l/p.d respectively from daily wastewater flowrates measured in the influent of present urban treatment plants in Balıkesir and Sivas. As it can be seen on Table 3.6, population value in Balıkesir for 2010 is predicted as 326,000 and population in Sivas for 2018 is predicted as 345,000. There is a 31 l/p.d difference in wastewater quantities reaching to the urban treatment plants, although the population values for Balıkesir and Sivas are similar. Measurement and evaluation should be made for the determination of differences by considering the factors, such as wastewater collecting infrastructure of provinces, groundwater level and district characteristics.

Table 3.7 : Influent wastewater quantities of WWTP in different cities (2009).

Provinces	Population	Amount of wastewaterl/p.d
Siirt ¹	127,000	95
Sivas ¹	345,000	183
Fethiye ¹	92,941	362
Balıkesir ¹	326,000	152
Antalya Hurma ¹	250,000	150
Malatya ¹	720,000	274
Konya ¹	1,000,000	150
Bursa ¹	1,200,000	200
Kayseri	800,000	137
Yenikapı ²	3,000,000	235
Baltalimanı ²	3,000,000	150
K.Çekmece ²	1,400,000	102
B.Çekmece ²	620,000	93
Kadıköy ²	2,230,000	212
Küçüksu ²	1,400,000	117

¹ Sistem Yapı, ² IWSA

Wastewater characterization and flowrate of 15 sites are given on Table 3.8 to form the basis of the master plan, feasibility and design studies, carried out by Ministry of Environment and Urbanization. During the flowrate calculation of plants that have been designed within this scope, provincial bank method has been used.

Table 3.8 : Influent flowrates of treatment plants in different sites.

Sites	(MEUP, 2010) Population	Flow m ³ /d	Amount of ww. (l/p.d)
Adıyaman	245,490	32,817	134
Aksaray	201,782	26,413	131
Akşehir	75,412	15,102	200
Bartın	61,463	12,439	202
Ceyhan	136,220	34,899	256
Çarşamba	81,347	12,936	159
Diyarbakır	1,430,000	196,676	138
Erdemli	58,559	-	94
Erzurum	444,934	62,452	140
Lüleburgaz	138,886	19,451	140
Merzifon	719,991	8,941	124
Polatlı	125,888	19,863	158
Seydişehir	46,792	5,952	127
Siverek	128,082	17,030	133
Soma	111,000	13,764	124

In determination of wastewater quantity, amount of water consumed, literature data, information on population, location of district, life standard, climate condition,

industrial wastewater discharges and most importantly the condition of infrastructure system must be paid attention as well as the measurement. In evaluation of design data for different cities, it can be seen that analysis result of single samples and results of short term measurements are not taken into consideration. In the light of data taken from treatment plants in operation and treatment plants in design process, it can be understood that local conditions and the condition of infrastructure are indicative parameters.

In addition, practices on reducing daily water usage must be disseminated and widely used. Reuse of gray water is an example of these practices. Reuse of gray water will reduce the amount of energy required especially when clean water has to be transferred from long distance. One third of the water used daily in a city can be saved by using treated gray water for flushing (Bieker et al., 2010). Water use per capita in Austria is reduced from 130 L to 100 L using this approach (Enderle et al., 2011).

3.3 Characterization of municipal wastewater

3.3.1 Components of domestic wastewater and classification

Wastewater characterization is made on the collective evaluation basis of pollutants as significant parameters. Components that should be primarily considered in formation of these parameters in domestic wastewaters and their environmental impacts are given on Table 3.9. According to the Table 3.9, study of conventional characterization should be performed on the significant parameters basis, either by taking every single pollutant component separately or collectively (Orhon et al., 2000).

Table 3.9 : Wastewater Components (Henze, et al., 1995).

Component	Parameter	Environmental Impacts
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Microorganisms	Pathogenic bacteria, viruses, gapeworm, etc.	
Biologically degradable organic matters		Oxygen consumption in lake and rivers
Other organic matters	detergents, N, P, NH ₃ , phenol etc.	Toxic effect, bioaccumulation
Nutrients	Nitrogen, phosphorus, ammonia	Eutrophication
Metals	Hg, Pb, Cd, Cr, Cu, Ni	Toxic effect
Other inorganics	Acids, bases	Corrosion, toxic effect
Thermal effects	Warm water	Effect on living conditions
Taste and odor	Hydrogen sulphide	Aesthetic problems, toxic effect
Radioactivity		Toxic effect, accumulation

Domestic wastewaters are classified in literature as strong, medium, weak and diluted to be able to broadly characterize domestic wastewaters and to reveal the differences. Average composition of domestic wastewaters within this classification in terms of organic matter, nutrient is given on Table 3.10, Table 3.11 and Table 3.12.

Table 3.10 : Characterization of wastewater in terms of organics (Henze et al., 1995).

Parameter (mg/L)	Wastewater type			
	Strong	Medium	Weak	Diluted
BOD				
Ultimate	530	380	230	150
7 days	400	290	170	115
5 days	350	250	150	100
• Dissolved	140	100	60	40
• Dis., easily degrdb.	70	50	30	20
Settled	250	175	110	70
COD				
Total	740	530	320	210
Settled	530	370	230	150
Dissolved	300	210	130	80
TOC	250	180	110	70
Carbohydrate	40	25	15	10
Protein	25	18	11	7
Fatty acid	65	45	25	18
Fats	25	18	11	7
Detergents	15	10	6	4
Oil and Grease	100	70	40	30
Phenol	0.2	0.15	0.10	0.05
Detergent(anionic)	15	10	6	4

Table 3.11 : Characterization of wastewater in terms of nutrient (Henze et al., 1995).

Parameter (mg/L)	Wastewater type			
	Strong	Medium	Weak	Diluted
TN	80	50	30	20
-ammonia nitrogen	50	30	18	12
-nitrite nitrogen	0.1	0.1	0.1	0.1
- nitratenitrogen	0.5	0.5	0.5	0.5
-organic nitrogen	30	20	12	8
TP	23	16	10	6
-orthophosphate	14	10	6	4
-poliphosphate	5	3	2	1
-organic P	4	3	2	1

Table 3.12 : Domestic wastewater characterization on metals basis (Henze et al., 1995).

Parameter (mg/L)	Wastewatertype			
	Strong	Medium	Weak	Diluted
Aluminium (Al)	1,000	650	400	250
Arsenic (As)	5	3	2	1
Cadmium (Cd)	4	2	2	1
Chromium (Cr)	40	25	15	10
Cobalt (Co)	2	1	1	0.5
Copper (Cu)	100	70	40	30
Iron (Fe)	1,500	1,000	600	400
Lead (Pb)	80	65	30	25
Mangan (Mn)	150	100	60	40
Mercury (Hg)	3	2	1	1
Nickel (Ni)	40	25	15	10
Silver (Ag)	10	7	4	3
Zinc (Zn)	300	200	130	80

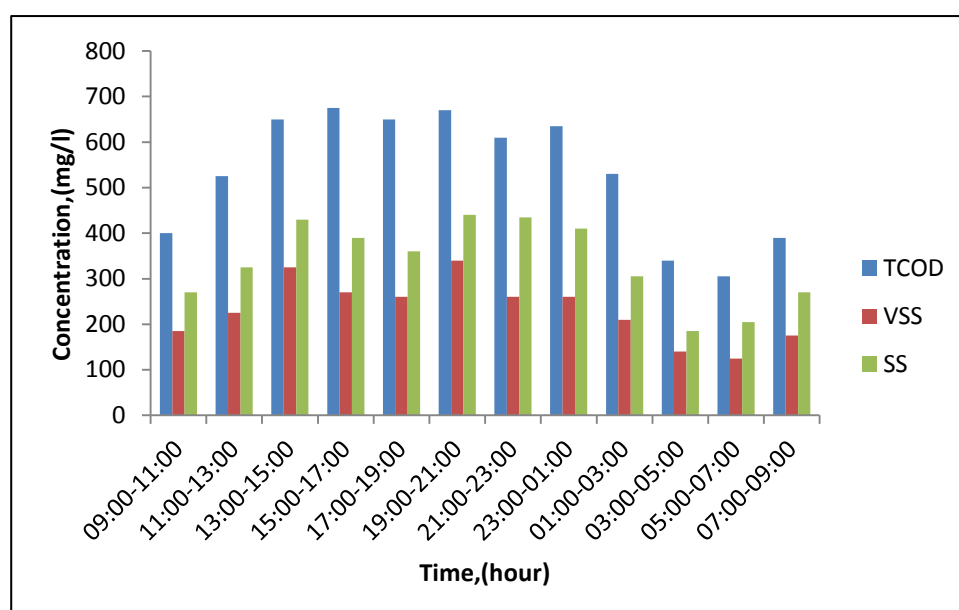
3.3.2 Characterization of municipal wastewater in Turkey

Wastewater characterization could present a difference in the same country depending on various factors, as well as it can differ from country to country. Moreover, wastewater characterization can differ in the course of the day. Influent parameter changes can be seen in Figure 3.4 provided from work in IWSA Pasakoy wastewater facilities.

Wastewater characterization of Paşaköy Advanced WWTP were determined by taking composite sample after grit chamber. Study was performed on 30 samples to characterize dry air flowrate. Results of statistical evaluation for these 30 samples are given in the Table 3.13 and Table A.1 (Sozen et al., 2008),(İnsel et al.,2011).

Table 3.13 : Domestic wastewater characterization in Paşaköy (Sözen,2008).

	TCOD mg/l	Filtered COD mg/l	TKN mg/l	NH ₄ -N mg/l	TP mg/l	SS mg/l	VSS mg/l
Range	300-675	120-275	43-78	29-58	8 - 14	185-440	125-340
Mean	535	200	61	43	11	335	231

**Figure 3.4 :** Diurnal change of influent concentration in Paşaköy.

Various institutions have defined unit loads for wastewater flowrate and some pollutants in conventional parameters. In Wastewater Treatment Plant Process General Specification of Bank of Province, it is foreseen that data given in Table 3.14 should be used in the activated sludge, trickling filter and stabilization pond (Orhan, et al.,2000)

Table 3.14 : Bank of Province Processes General Agreement pollutant loads.

Parameter	Load (g/p.d)
BOD5	50—60
TSS	70—90
N	10—12
P	3—4

Suggested values for typical interval of domestic wastewater pollutant parameters in Turkey are given in Table 3.15 (Metcalf & Eddy, 2003).

Table 3.15 : Suggested intervals for wastewater parameters in Turkey (Metcalf & Eddy, 2003).

BOD ₅ , g/p.d	TSS, g/p.d	TKN, g/p.d	NH ₃ -N, g/p.d	Total P, g/p.d
27 – 50	41 – 68	8 – 14	9 – 11	0.4 – 2

Population basis pollution loads in Turkey are given in Wastewater Treatment Plants Technical Aspects Bulletin (Official Journal of 20.03. 2010, no 27527).Parameters depending on population interval are given in Table 3.16.

Table 3.16 : Changes in pollution loads depending on population (WWTPAB,2010).

Population	COD g/p.d	BOD g/p.d	SS g/p.d	TN g/p.d	TP g/p.d
2000- 10000	55	40	35	5	0.9
10001-50000	75	45	45	6	1.0
50001-100000	90	50	50	7	1.1

*Infiltration flowrate is considered in expression of pollution loads as a concentration equivalent.

Projections of unit pollution loads determination studies for Istanbul are given in Table 3.17. Similar approach is taken by AWSA and unit loads on BOD₅ basis in 1995, 2010 and 2025 are determined as 53g/p.d, 56g/p.d and 60g/p.d, respectively. Most of the studies performed until today are completed based on assumptions, not science-oriented researches. Importance of detailed characterization, especially in design, clarified when the differences between the values in the Table are considered (Orhon et al., 2000).

Table 3.17 : Unit pollutant loads suggested for Istanbul (Orhon, 2000).

Parameter		Camp Tekser (1975) gr/p.d	Watson Motor Temel(1986) gr/p.d	Meriç(1990) gr/p.d
BOD ₅	1990	50	60	60
	2020	75		
SS	1990	75	70	70
	2020	105		
N	1990	8	10	10
	2020	9		
P	1990	2	2	2
	2020	2.5		

Measurements in the world and scientific studies performed in Turkey are evaluated with the necessity of determining wastewater characterization for every single pollutant source and the results of these surveys are evaluated. These studies include touristic areas with significant difference in summer and winter populations, various districts of İstanbul that are intensively populated and different cities with different populations.

In a study performed to represent the sensitive touristic regions, Fethiye, Marmaris and Bodrum are designated as a pilot areas. Wastewaters from these cities are analysed during 5 months between May and September to characterize the summer and winter seasons (Orhon, et al., 1996). It can be seen on Table 3.18 that Fethiye has weak wastewater characteristic while Bodrum has the strongest wastewater (Orhon et al., 2000).

Table 3.18 : Domestic wastewater characterization (Orhon,2000).

Parameter	Marmaris		Bodrum	
	Mean(mg/l)	Range(mg/l)	Mean(mg/l)	Range (mg/l)
COD _{total}	370	215-480	473	335-530
COD _{settled}	236	145-265	319	210-370
COD _{filtered}	96	60-125	128	85-140
TKN	37	31-42	45	32-57
NH ₃ -N	26	21-34	33	26-41
Organic N	11.2	7-15	12.3	0.5-21
TP	7.7	5.6-9	8.8	7-11
SS	194	170-265	227	140-310
VSS	177	160-230	199	120-290
Alkalinity	462	380-520	486	410-540
TDS	2,107	900-2,980	2,470	2,190-2,850
Oil and Grease	127	80-200	162	115-230
Detergent	3.7	2.8-4.9	4.5	3.2-5.4

Influent analyses for municipal wastewater treatment plant in Fethiye performed in 2000 and 2009 are compared in Table 3.19. In Table 3.19, it is seen that in the year 2009 average value of COD and SS concentration increased, whereas average value of N and P parameters decreased. However, it is observed that the measurement in 2009 spread in a broader range.

Table 3.19 : Comparison between influent values of Fethiye Urban Treatment Plant.

Parameter	Fethiye (Orhon, 2000)		Fethiye (Sistemyapı,2009)	
	Mean mg/L	Range mg/L	Mean mg/L	Range mg/L
COD _{total}	227	190 - 245	318	134 – 727
COD _{settled}	158	110 - 245		
COD _{filtered}	52	30 – 145		
TKN	26.80	20 – 37	21	13 – 41
NH ₃ – N	17.9	11.5 - 21	11	4 – 21
Organic N	8.9	2 – 16		
TP	5.4	3.3 – 9	3	1 – 6
SS	150	100 - 270	206	72 – 654
VSS	136	90 – 235		
Alkalinity	412	310-470		
TDS	615	580-680		
Oil and Grease	102	60-210		
Chloride	110	100-120		
Detergent	2.3	1.3-3.2		
pH	7.3	6.9-7.5		

Studies performed for domestic wastewater characterization in Istanbul include samples taken from Kadıköy, Küçükçekmece, Baltalimanı, Sefaköy, Yenikapı, Ömerli, Ataköy and Tuzla. Studies in Kadıköy contain results of two different studies performed in different periods (Orhon et al.,2000).

Influent wastewater values for Baltalimanı and Ataköy stations for different time periods are given in Table 3.20 and Table 3.21. Average of measurement results taken from analyses performed in Ataköy, Yenikapı, Baltalimanı, B.Çekmece and K.Çekmece by IWSA in 2008-2009 are given in Table 3.21. Evaluating the results in different time periods in these sites, it can be seen that while treatment plant pollutant loads in influent in Baltalimanı and Ataköy.

While NH₄-N parameter in Ataköy is approximately 23 during the year 1993, in the years 2008-2009 it has reached to value of 35 mg/l. Also COD parameter in Ataköy increased from approximately 270 mg/l to a value of 386 mg/l.

Table 3.20 : Comparison between influent values of Ataköy UWTP.

Parameter	(Orhon,1997)		(Okutman,2009) ^a			This study ISKI
	Mean	Range	Mean	%70	Range	Mean
COD (mg/l)	270	160-350	406	445	295- 535	386
TKN (mg/l)	37	22-63	41	43	36 – 47	
NH ₄ -N (mg/l)	23	12-40	27	28.8	19 – 34	35
T-P (mg/l)	7	3-12	8.3	9.1	6.0-11.6	
SS (mg/l)			190 ^b	210 ^b	122 – 247 ^b	183
VSS (mg/l)			178 ^b	198 ^b	118 – 227 ^b	
TSS (mg/l)			2874	2920	2600 - 3300	
pH	7,1	6,8-7,5	7,6	7,7	7,2 – 7,9	

^a yaz mevsimi, kompozit numune, ^b 1.2 µm

Table 3.21 : Comparison between influent values of BaltalimanıUWTP.

Parameter	(Orhon,1997)		(Okutman, 2009) a			This study ISKI
	Mean	Range	Mean	%70	Range	Mean
COD(mg/l)	340	265-645	353	368	314 - 408	474
BOD5	150	73-200	35	38	30 - 40	
TKN (mg/l)	35	23.9-57	20.4	23	11 - 26.0	
NH ₄ -N (mg/l)	19.9	10-26.3	7.1	8.4	4.4 - 10.2	40
T-P (mg/l)	6.8	5.8-63	184	195	151 - 262	
SS (mg/l)	140	85-318	148 b	160	126 – 165b	209
VSS (mg/l)	125	120-135	4486 b	4510	4200-4630b	
TSS(mg/l)			7.4	7.5	7.1 - 7.7	
pH			353	368	314 - 408	

Table 3.22 : Average influent concentrations of WWTP in Istanbul(IWSA,2009).

2008-2009	BOD (mg/l)	COD(mg/l)	SS (mg/l)	NH ₃ -N (mg/l)
Ataköy	229	386	183	35
Yenikapı	235	432	209	33.5
Baltalimanı	276	474	209	40
B.Çekmece	260	531	319	32
K.Çekmece	205	424	244	38
Tuzla	-	-	464	79

The domestic wastewater characterization studies for Istanbul are given in Table A.2 in appendix. The rates of total dissolved COD for Atakoy, Tuzla, Kadıkoy II were to be 42%, 32% and 35% respectively. Concentration values representing 70% and 80%

probability values of pollutant parameters must be known to form the basis of designing the wastewater treatment plant. Concentrations of pollutant parameters representing 70% and 80% probability values derived from statistical evaluation are given for 5 different cities in Table 3.22.

From these results, a general overview on influent characterization for urban treatment plant is illustrated as Table 3.23 (Erdoğan, 2004).

Table 3.23 : Probability values of conventional parameters (Erdoğan, 2004).

Tesis	COD (mg/l)		BOD ₅ (mg/l)		SS (mg/l)		TKN (mg/l)		TP (mg/l)	
	Probability		Probability		Probability		Probability		Probability	
	70%	80%	70%	80%	70%	80%	70%	80%	70%	80%
İzmir	483	544	220	240	267	305	45	51	8.1	8.3
Ankara	336	362	185	190	169	184	46	53	9.4	9.8
Isparta	470	510	280	306	165	193	-	-	-	-
Antalya	451	524	310	338	310	350	33	37	6.5	8.1
Tarsus	514	606	248	260	212	241	48	53	24	25

For the wastewater treatment plants in Nevşehir and Tokat, the pollution loads provided in Table 3.23 are used. Industrial wastewater pollution loads are added to these loads (MEF, 2005). Concentrations of pollutant parameters representing 70% and 85% probability values derived from statistical evaluation are given for different urban treatment plants in 8 different cities in Table 3.24. Significant variation in COD parameter is observed due to the industrial wastewater flowrate and concentration, especially in Balıkesir.

Table 3.24 : Influent wastewater characterization of certain urban treatment plants in Turkey.

Sites	COD (mg/L)		BOD ₅ (mg/L)		SS (mg/L)		TKN (mg/L)		TP (mg/L)	
	Probability		Probability		Probability		Probability		Probability	
	70%	85%	70%	85%	70%	85%	70%	85%	70%	85%
Durusu ¹	444	556			143	214	64	72	12	13
Paşaköy ²	732	830			311	368	86	93	16	18
Balıkesir	1,012	1,116	351	379	450	483				
Sivas	426	522	246	283	285	360	42	47	6	6
Siirt	681	789	286	358	413	501	73	82	8	10
Konya	768	934	369	425	400	460	79	83	13	13
Bursa	512	561	234	260	212	234	56	60	9	10
Fethiye	381	480			230	298	22	28	4	4

¹(Ozturk, 2008)

²(Sozen et al., 2008)

Wastewater treatment plant data given for different sites in Table 3.24 are provided by compiling measurements performed in 2008-2009 period. Measurement results of Balıkesir, Bursa, Fethiye, Siirt, Sivas and Konya are obtained from the authorities in charge of operation (Table A.3, Table A.4, Table A.5, Table A.6 and in Table A.7) and the shortest term measurement result contains 3 month period for Konya (Figure 3.25), while these results take a year for other facilities. Annual average of influent data for urban treatment plants are given in Table 3.26 (Sistemyapı, 2009).

Table 3.25 : Design and influent wastewater characterization of Konya wastewater treatment plant (Sistemyapı, 2009).

	SS (mg/l)	COD (mg/l)	BOD ₅ (mg/l)	TN (mg/l)	TP (mg/l)
Design	295	775	320	50	15
August	392.39	857.89	401.61	49.73	12.51
September	303.29	701.13	341.82	57.33	12.7
October	346.13	667.6	386	78.69	10.5
Average	347	742	376	62	12

Table 3.26 : Wastewater analyses of treatment plants in different cities.

	Flow m ³ /day	Population	COD mg/l	BOD mg/l	TSS mg/l	TN mg/l	TP mg/l
Fethiye	33,690	92,941	317	167	205	21.36	3.02
Balıkesir	49,582	326,000	1009	345	408		
Bursa	185,798	1,200,000	471	211	194	51.09	8.24
Sivas	63,050	345,000	410	223	243	42.11	5.91
Siirt	12,146	127,000	618	271	379	67.69	8.05
Konya	150,000	1,000,000	742	376	347	62.00	12
Antalya	37,500	250,000	400	700	500	60.00	12
Malatya	197,478	720,000	485	242	283	44.50	7.3
Kayseri ¹	110,000	800,000	325	130	274	92.5	5.48
Adana ¹			674	199	274	58.4	13.7

¹ (Alaton,2007)

The results of the influent characterization study for between 28.09.2007 and 28.07.2008 in Durusu wastewater treatment plant are presented in Table 3.27.

Table 3.27 : Influent wastewater characterization for Durusu Treatment Plant (Ozturk, 2008).

	TCOD mg/L	SS mg/L	TKN mg/L	Ammonia mg/L	TP mg/L	Nitrate mg/L	pH	Alkalinity mgCaCO ₃ /L
Mean	363	133	54	43	9.5	2.41	7.6	288
Standard Deviation	193	96	18	15	3.9	1.29	0.5	116
Minimum	117	22	11	5	2.8	0.40	6.3	83
Maximum	1,045	437	91	84	20.8	6.70	8,8	546

Measurement results of TCOD and SCOD parameters are designated as 363±193 and 154±108 mg/L, respectively. Average measurement results for TKN, NH₄-N, TP, SS and VSS parameters are determined as 54±18, 43±15, 9,5±3,8, 133±96 and 106±79 mg/L, respectively (Ozturk et al., 2008).

Design criterias of 15 sites within the scope of master plan, feasibility and design studies performed by Ministry of Environment and Urbanization are given in Table 3.26. Measurement results taken from 15 different urban treatment plant influent, sewerage system of the plants and the design parameters used in those plants are determined in evaluation of urban treatment design parameters.

Wastewater analyses are carried out twice in a year, being in dry and rainy period and the results are considered in the evaluation of pollution loads. However, since the samples are only taken as daily and 24 hours, analysis results does not represent the general characterization of wastewater. Therefore, analysis results are not considered in some cities because of the serious impact of groundwaters and rain waters on sewerage system, while in others, due to the single sampling, mostly local conditions are taken into account, with a slight consideration of analysis results.

Different conclusions have been reached under the impact of local conditions when analysis results performed for different sites, design parameters used, analysis results taken from present urban treatment plants and the results of studies carried out in different countries are evaluated. As it can be noticed when looked at the parameters that there is an important difference between the analysis results taken from sewerage system and influent characterization of present urban treatment plants and the analysis results are not taken into consideration in most of the sites.

COD and BOD₅ concentrations are relatively higher than other parameters due to the high industrial wastewater flowrate in certain cities. In the AKM and TKN values, it

is observed that the data values of the existing treatment plant is close to the design values of the new treatment plants. By taking into account that the analyse values are low and existing infrastructure is inadequate, the results of measurements made in different points in the sewer system are not used. In order to stay safe in the design, the design values given in Table 3.28 are used (Cowi,MEUP, 2010).

Table 3.28 : Wastewater analyses of treatment plants in different cities and design parameters.

	COD(mg/l)				BOD ₅ (mg/l)			TSS (mg/l)			TKN (mg/l)			TP (mg/l)		
	PE	Range	Mean	Design	Range	Mean	Design	Range	Mean	Design	Range	Mean	Design	Range	Mean	Design
Adıyaman	245,490	160-480	325	671	75-250	166	338	7-170	89	338	2.24-38.64	24	63	0.537-20.39	10	13
Aksaray	304,371	342-840	607	774	105-400	285	384	100-160	138	377	30.8-57.31	44	57	5.14-12.71	9	15
Akşehir	75,412	540-820	700	495	220-400	332	248	216-457	324	278	57.87-86.8	69	53	4.32-15.51	12	11
Bartın	61,463	232-460	303	396	110-250	160	198	37-77	59	223	36.22-49.73	41	45	4.42-9.20	7	9
Ceyhan	136,220	400-480	440	314	230-260	245	157	20-54	37	176	16.8-32.48	25	35	13.71-20.93	17	7
Çarşamba	81,347	420-620	525	601	250-330	284	301	118-190	139	318	45.92-78.96	63	62	6.4-19.57	13	13
Diyarbakır	1,430,000	344-620	482	601	165-325	245	300	40-75	58	331	49.73-54.5	52	52	7.39-18.4	13	13.1
Erdemli	58,559	500-880	690		270-420	345	315	210-361	286	479	43.3-70	57	62	11.18-17.50	14	17.5
Erzurum	444,934	280-460	370	631	150-250	190	321	18-50	33	343	0.12-38.45	22	54	0.221-19.48	8	13
Lüleburgaz	138,886	456-780	618	653	225-370	298	323	121-150	136	354	42-46.67	44	70	14.79-17.37	16	13.8
Merzifon	719,991	480-520	500	580	240-250	245	290	130-182	156	330	20.16-69.44	45	66	5.41-16.14	11	12
Polatlı	125,888	408-600	504	537	195-275	235	270	100-580	340	291	59.36-60.48	60	57	7.49-10.91	9	12
Seydişehir	46,792	240-320	287	555	110-175	148	277	49-135	88	316	32.48-33.60	33	59	5.04-20.93	11	12
Siverek	128,082	208-420	314	603	100-200	150	302	54-104	79	339	4.60-16.05	10	68	4.86-13.30	9	13.5
Soma	111,000	400-500	450	669	230-250	240	334	149-178	164	369	26.32-41.81	34	74	2.30-9.10	6	15

Sample results taken from sewerage system(analysis) and determined design criterias(design) for 15 urban wastewater treatment treatment plant (Table 3.28)and the concentration values depending on population in present treatment plants Table 3.22 and 3.26 are shown on the Figure 3.5, Figure 3.6, Figure 3.7,Figure 3.8, Figure 3.9. Low concentrations in analysis results are derived from the suboptimal conditions in sewerage systems.

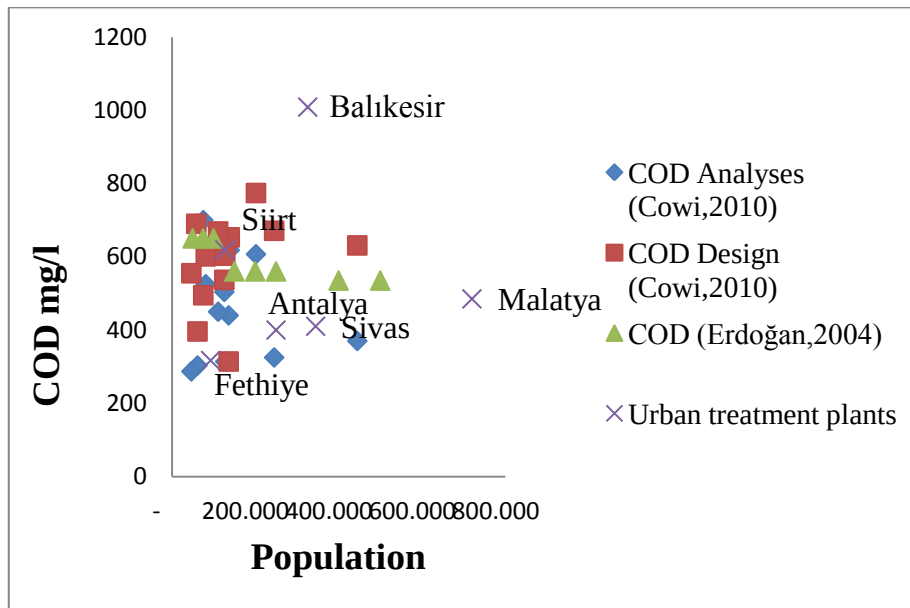


Figure 3.5 : COD changes in different population.

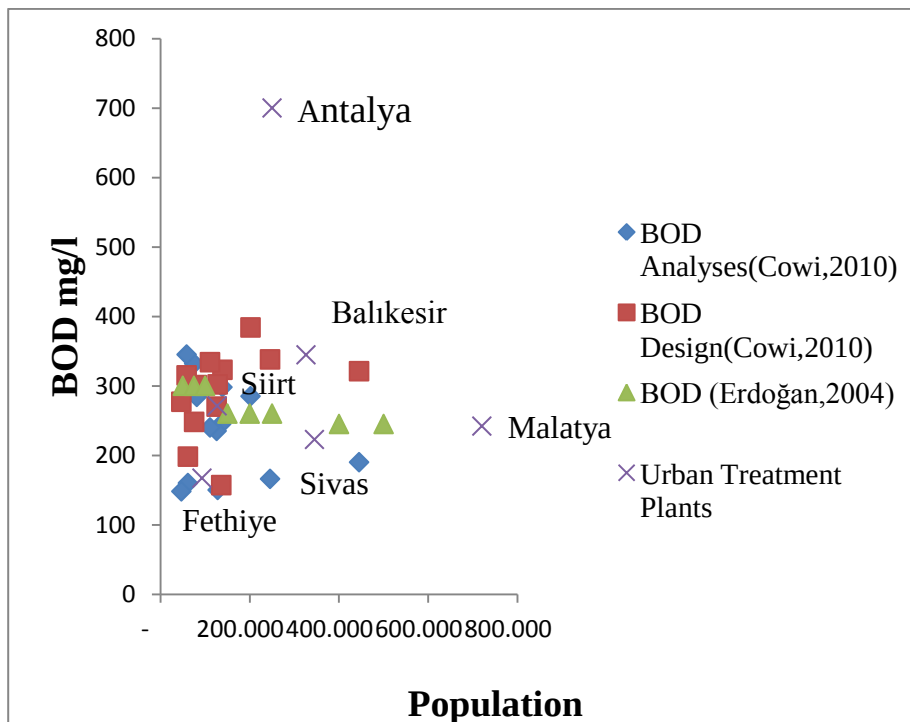


Figure 3.6 : BOD changes in different population.

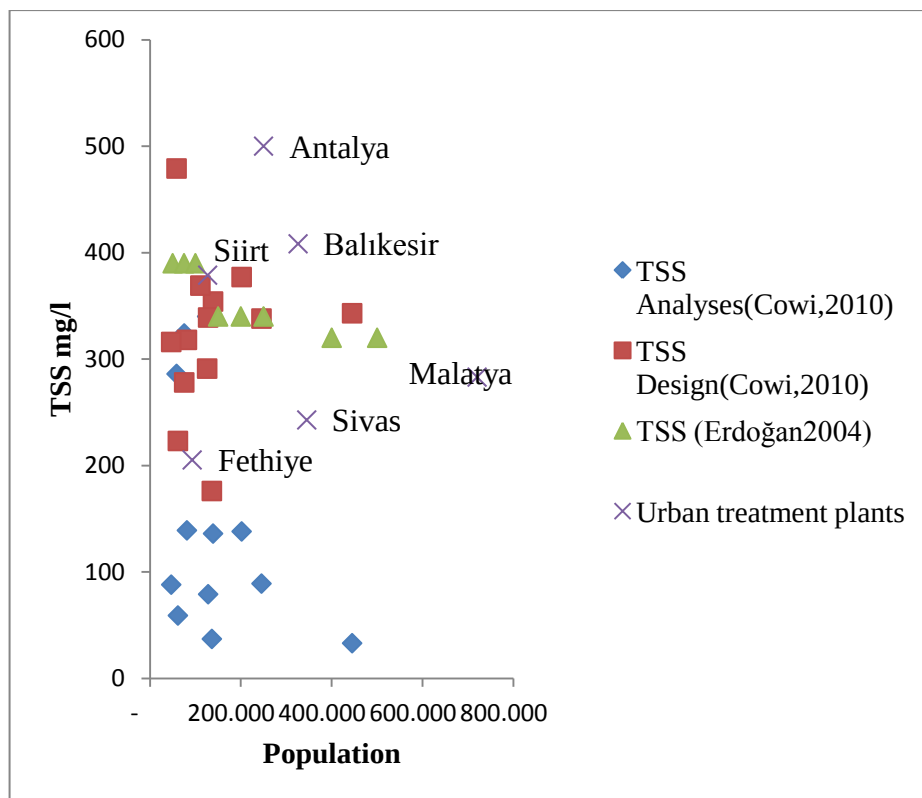


Figure 3.7 : TSS changes in different population.

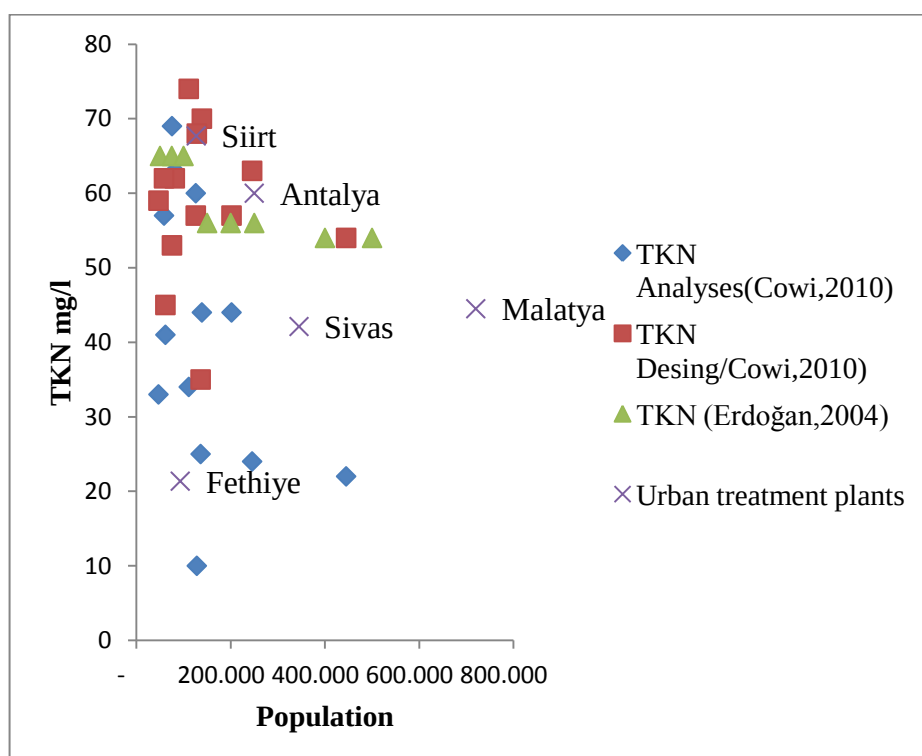


Figure 3.8 : TKN changes in different population.

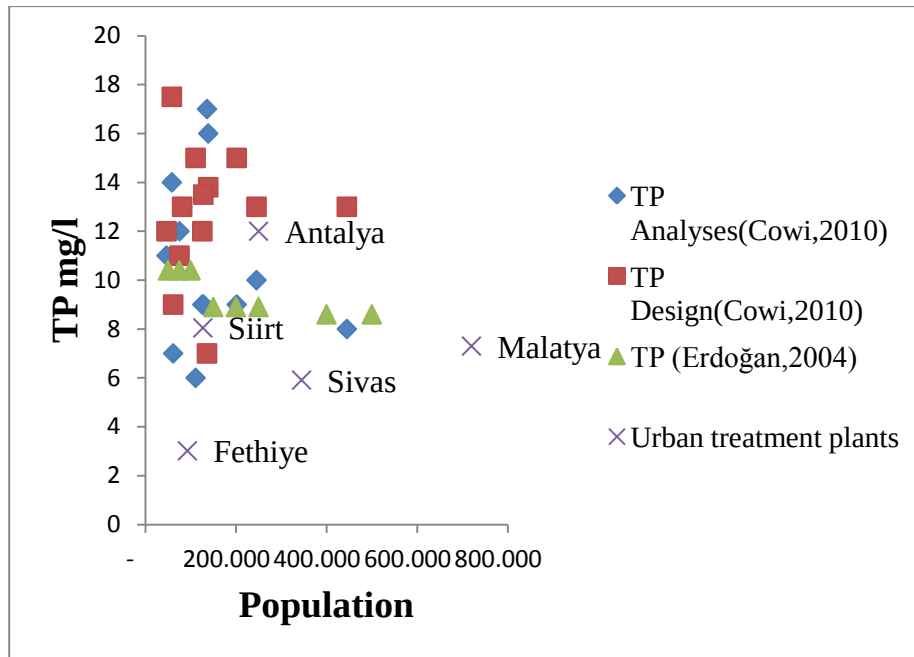


Figure 3.9 : T-P changes in different population.

3.3.3 Characterization of municipal wastewater in different countries

The domestic wastewater characterization of different countries are given in Table 3.29 and Table 3.30. Values on these tables do not represent the countrywide; they are given to provide a general overview. There are differences among countries depending on life standard, general consumption habits and etc.

Table 3.29 : Conventional wastewater characterization made in terms of concentration concerning different countries (Orhon, 2000).

	TCOD (mg/l)	COD _(filt) (mg/l)	BOD ₅ (mg/l)	TKN (mg/l)	NH ₃ -N (mg/l)	TP (mg/l)	VSS (mg/l)	SS (mg/l)	Alkalinity (mg/l)
Brasil	627	157	357	54	30	9.9	297	376	198
Greece	-	-	259	-	39.3	5.9	-	-	-
France	500	350	200	55	-	-	75	150	-
Israel	635	205	201	43	40	-	-	-	-
Mexico	416	270	149	57.4	44	9.3	67	155	709
S.Africa	705	-	300	37.5	-	-	-	-	-
Denmark	-	-	172	44	-	-	-	-	-
Germany	-	-	200	50	-	-	-	-	-

A comparison on municipal wastewater characteristic between Istanbul and European Countries is given in Table 3.31. Influent characteristics of urban wastewater treatment plants in different cities in Turkey measured in 2009 compared to the values in other countries are given in Table 3.32.

Wastewater characterization can change for different times in the same residential areas, as well as it can reach the same result for different countries. In Sivas and Bursa domestic wastewater treatment plant COD input values are measured respectively 471 and 410 for annual average. The values are registered as 477 and 455 for Belgium and Denmark. These values are determined as 618 and 613 for Siirt and England, respectively. The comparison of each of the influent values of wastewater treatment system on a plant basis would be more useful for the evaluation of the result.

Table 3.30 : Measures of conventional parameters and statistical assessment concerning different countries (Pons,2001)

Country	COD mg/l				BOD ₅ mg/l			SS mg/l			N mg/l			P mg/l		
	Plant	A	B	C	A	B	C	A	B	C	A	B	C	A	B	C
Avustria	97	526	498	218	285	256	120	-	-	-	44	40	18	7.1	7	3
Belgium	152	477	382	314	187	152	128	236	174	222	-	-	-	-	-	-
Denmark	836	455	404	440	163	135	119	-	-	-	36	30	31	7.8	6.2	7.6
France	149	634	592	315	268	245	137	302	267	170	52	51	23	9.3	9	3.8
Holland	384	450	426	151	171	159	66	237	194	252	42	40	16	6.7	6.1	6.1
Slovenya	100	581	418	613	267	217	245	426	341	292	37	39	19	5.5	4.8	3.8
Cyprus	6	522	-	-	547	-	-	436	-	-	98	-	-	16	-	-
Finland	7	559	-	161	266	-	78	378	-	144	43.8	-	10.4	7.5	-	1.3
Germany	6	548	-	-	319	-	-	208	-	-	59	-	-	8	-	-
Fas	60	928	900	357	353	350	140	397	400	174	-	-	-	-	-	-
Spain	3	762	-	-	434	-	-	290	-	-	71	-	-	11	-	-
Sweden	17	239	-	-	133	-	-	98	-	-	28	-	-	4	-	-
Switzerland	6	477	-	123	171	-	72	243	-	87	33	-	8.1	6.1	-	17
England	12	613	-	-	212	-	-	150	-	-	40	-	-	-	-	-

A: Mean, B: Media, C: Standard deviation

Table 3.31 : Comparison of wastewater characterization with other countries.

Parameter	Domestic wastewater in Istanbul (Orhon,1997)				Domestic wastewater in Europe (Pons et al., 2002)						
	Baltalimani		Yenikapi		France	Austria	Netherlands	Sweden	Norway	Finland	Germany
	Mean	Range	Mean	Range	Mean	Mean	Mean	Mean	Mean	Mean	Mean
COD (mg/L)	340	265 - 645	680	280 - 1480	634	526	450	477	233	559	548
TKN (mg/L)	35	23.9 - 57	66	27 - 92	52	44	42	33.1	22	43.8	59
NH ₃ -N (mg/L)	19.9	10 - 26.3	37.74	24 - 48.8							
TP (mg/L)	6.8	5 - 8.63	7	3.6 - 13	9.3	7.1	6.7	6.14	3	7.47	8
TSS (mg/L)	140	85 - 318	480	110 - 820	302	—	237	243	143	378	208
VSS (mg/L)	125	120 - 135	65	65 - 69	—	—	—	—	—	—	—
TDS	435	335 - 537	—	—	—	—	—	—	—	—	—
Ph	7.4	7.2 - 7.5	7.24	7.1 - 7.3	—	—	—	—	—	—	—

Table 3.32 : Comparison of municipal wastewater characteristics between cities in Turkey and other countries.

	This study (Sistemyapı,2009)				Other Countries (Pons et al., 2001)						
	Bursa	Fethiye	Sivas	Siirt	Belgium	Denmark	Slovenia	Cyprus	Spain	England	Moresque
COD	471	317	410	618	477	455	581	522	762	613	928
BOD ₅	211	167	223	271	187	163	267	547	434	212	353
SS	194	205	243	379	236	-	426	436	290	150	397
TN	51	21	42	68	-	36	37	98	71	40	-
TP	8,2	3	6	8		7.8	5.5	16	11	-	-

3.4 Conclusions

Wastewater flowrate and pollutant loads are two important parameters in system design when wastewater characteristics are taken into consideration. Long-term monitoring programs are necessary due to variation in these parameters depending on conditions.

For the calculation of the amount of wastewater, several factors together with measurement including the amount of water usage(billed and unbilled), literature data, population data, location of the region, standards of life, climatic conditions, industrial waste water discharges and most importantly the infrastructure system must be taken into account.

In our country water usage per person is mainly in the range of 100-150 l/pd, but it can reach 200-250 l/pd in some cities (TSI 2008). Practices on reducing daily water usage must be disseminated and widely used. Reuse of gray water which is an example of these practices can save approximately one third of the amount of water used daily. Water use per capita in Austria is reduced from 130 L to 100 L using this approach (Enderle et al., 2011).

During the evaluation of the design data from different cities, it is observed that the results of individual samples and results of short-term measurements, are not taken into consideration. The studies conducted until today indicate that the influent wastewater characteristics have a very broad range.

Analyses conducted in the same settlement in different years indicate that different pollutant loads are reached. In the same settlement, the pollution loads are changed by year, increasing or decreasing, based on living conditions and changes in the infrastructure. Hence, it is important to take the changes in the loads into consideration in the design and the treatment plant that would be built should meet the requirements of changes in the influent loads.

As the result of the existing infrastructure in our country, in the determination of the influent values of wastewater treatment plants, the state of the infrastructure should be evaluated prior to measurements.

In the assesment of the pollutant load of the wastewater treatmet plants that are designed in different cities, it is observed that no long-term measurements but only

short-term measurements or measurements on grab samples are made. When the short-term measurements are combined with the inadequacy of the infrastructure system, mostly the literature values are used for the influent concentrations during the design of treatment plants. The data from municipal wastewater treatment plants (influent values of 30 municipal wastewater treatment plant) evaluated in this study indicate a wide range for influent flowrate and pollutant loads. The statistical analysis of these data (Figure 3.10, Figure 3.11, Figure 3.12, Figure 3.13, Figure 3.14) shows that the influent flowrate is between 100 and 150 l/PE.d, COD is between 500 and 600 mg/L, SS is between 250 and 350 mg/L, TKN is between 55 and 65 mg/L and TP is between 12 and 15 mg/L. Furthermore, according to data of TSI, the average wastewater generated in our country is between 100 and 150 L/PE day.

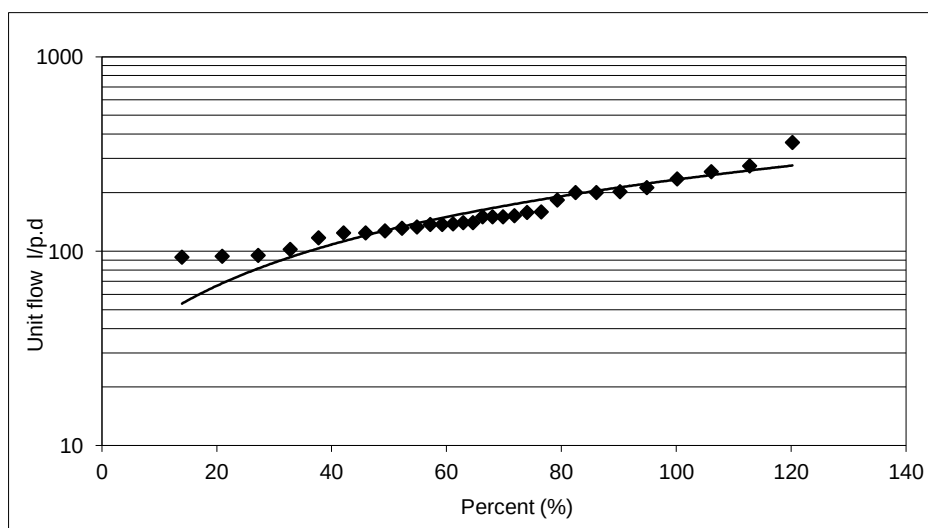


Figure 3.10 : Probability rate of influent flowrates of treatment plants.

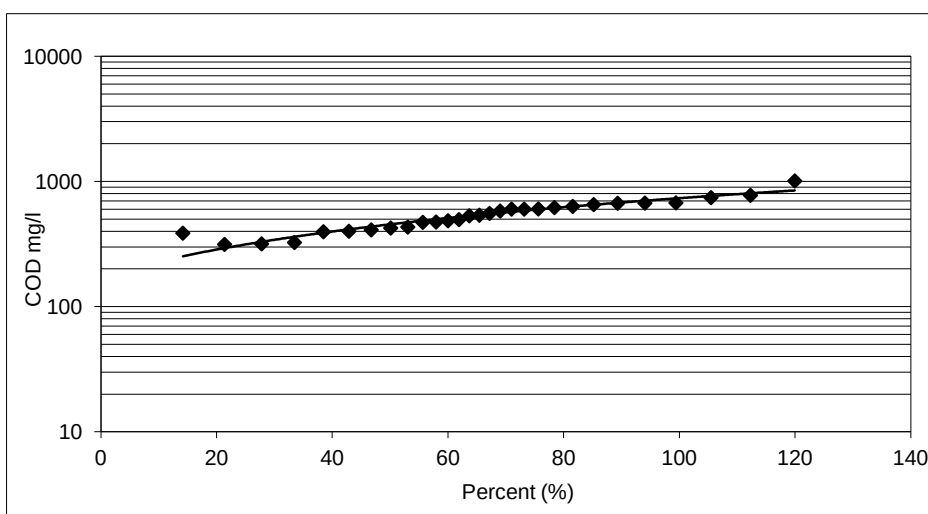


Figure 3.11 : Probability rate of influent COD concentration in treatment plants.

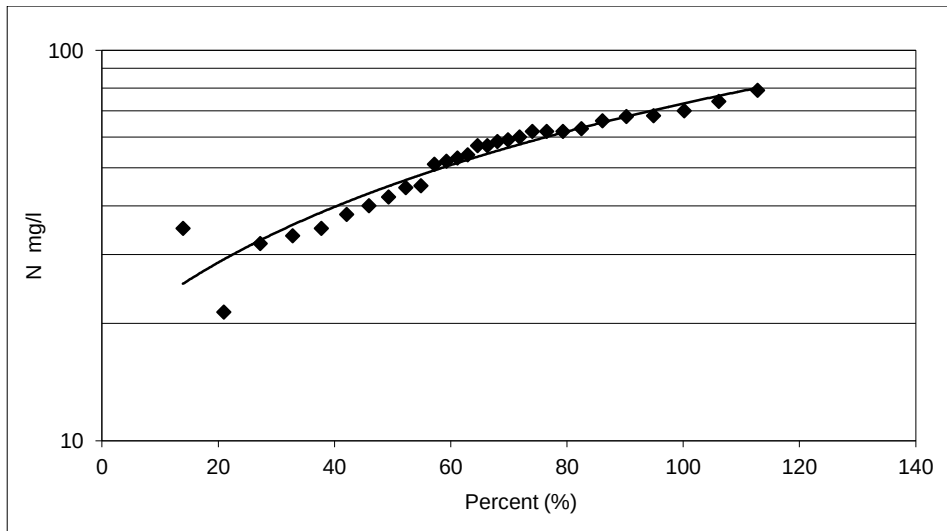


Figure 3.12 : Probability rate of influent N concentration in treatment plants.

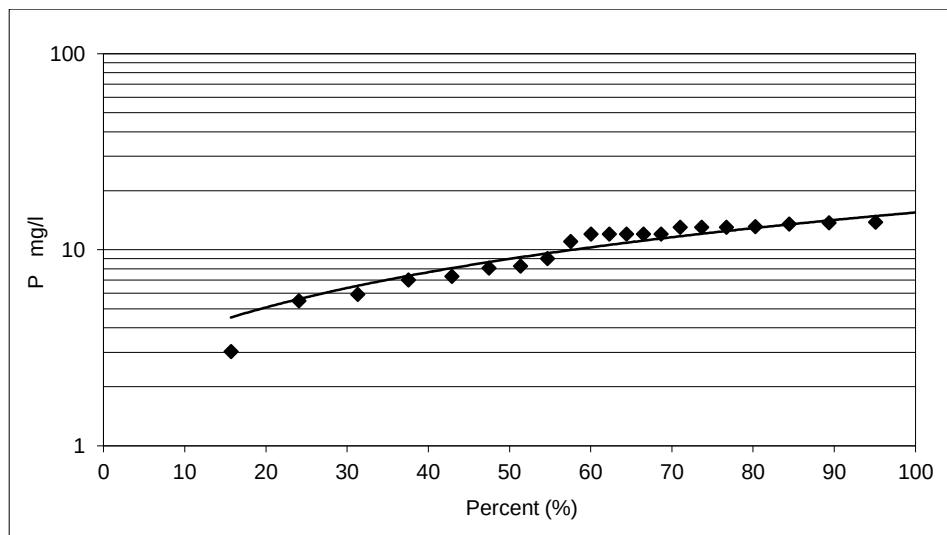


Figure 3.13: Probability rate of influent P concentration in treatment plants.

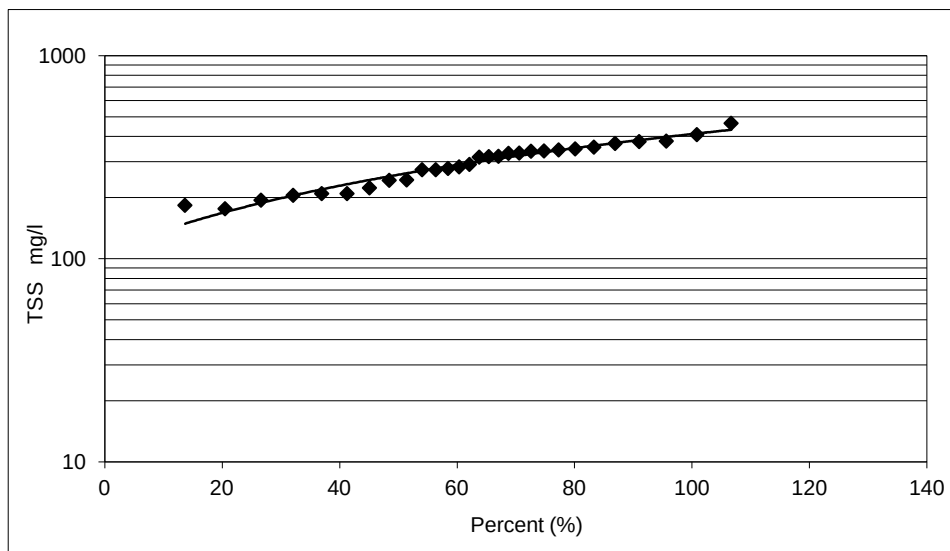


Figure 3.14: Probability rate of influent TSS concentration in treatment plants.

Therefore, in the determination of flowrate and influent pollutant loads of municipal treatment plants, first, the infrastructure should be adequate, then long-term measurements must be made, and at last, these values should be used for the design instead of literature values.

The standard values for wastewater treatment influent loads (Table 3.33 and Table 3.34) by Erdogan, 2005, are used as the influent loads in the design approach of this study.

Table 3.33 : Unit pollutant loads suggested for design on population basis (Erdogan, 2004).

Population (person)	COD		BOD		SS		TKN		TP	
	g/p.d	mg/l	g/p.d	mg/l	g/p.d	mg/l	g/p.d	mg/l	g/p.d	mg/l
5.000-7.500	55.2	690	25.2	315	33.2	415	5.6	70	0.88	11.0
7.501-25.000	60.3	670	27.45	305	36	400	6.03	67	0.96	10.7
25.001-100.000	65	650	30	300	39	390	6.5	65	1.04	10.4
100.001-250.000	70	560	32.5	260	42.5	340	7	56	1.11	8.9
250.001-750.000	74.9	535	34.3	245	44.8	320	7.56	54	1.2	8.6
750.001-2.000.000	80	500	36.8	230	48	300	8	50	1.28	8.0

Table 3.34 : Wastewater loads at Plant Inlet.

Parameter	P.E.	50,000	100,000	200,000	400,000	1,000,000
COD	kg/d	3,250	6,500	14,000	29,960	80,000
BOD	kg/d	1,500	3,000	6,500	13,720	36,800
SS	kg/d	1,950	3,900	8,500	17,920	48,000
Total-N	kg/d	325	650	1,400	3,024	8,000
Total-P	kg/d	52	104	222	480	1,280

In order to use the optimum of the limited sources, all necessary measures should be taken and investment decisions should be made to reduce the water loss in distribution systems that is measured as %51 in Turkey (Ozturk, 2003).

During the harmonization of EU regulations, the discharge standards in environmental regulations have changed and the limitations for N and P in sensitive areas prone to eutrophication are determined based on the population of the settlement with more strict limits, being applied for areas with a population $N > 100,000$ compared to areas with $N = 10,000 - 100,000$.

The Wastewater Treatment Plants Technical Aspects Bulletin includes the population-dependent wastewater generation and population loads, but the pollution loads for populations up to 100,000 are only to be used when measurements is not possible to conduct.

The Ministry of Environment and Urbanization should publish annual reports for the treatment plants in operation and these values should be available when new municipal wastewater treatment plants are designed.

4. LITERATURE REVIEW: BIOLOGICAL NUTRIENT REMOVAL SYSTEMS FOR MUNICIPAL WASTEWATER

4.1 Introduction

Nowadays, municipal treatment plants with only carbon removal are being replaced by activated sludge plants with nutrient removal. At the same time, due to the discharge limits, there is a need to convert the plants with only carbon removal to nutrient removal systems, by developing them. Predenitrification systems are widely used in activated sludge systems with nutrient removal.

In this chapter literature search on physical and biological treatment units and sludge treatment processes are conducted for activated sludge systems with nutrient removal.

4.2 Process design of physical treatment units

4.2.1 Screening

Generally, screening is the first physical treatment unit at wastewater treatment plants. The most important aim of screens is to remove large objects such as rags, paper, plastics, wood scraps and etc. and thus pretreatment or primary treatment in a wastewater treatment plant could be accomplished. In the case of these objects are not removed by primary treatment; many problems can occur such as plant operation and maintenance problems leading to damage of the pumping and sludge removal equipments; weirs, valves, nozzles, channels, pipelines and etc. Screening equipments are normally classified as coarse screens or fine screens. Screens can be cleaned both manually and mechanically but screens that are cleaned manually are limited to bar racks and generally used at small plants.

Mechanically cleaned screens are most widely used for all other applications because the cleaning of those type of screens is not continuously and this cleaning process of screens can be adjusted by timing devices or water level devices (WEF,2009).

Coarse screens are normally the first treatment unit it is used at protective devices in wastewater treatment plants such as; bar screens, coarse woven wire screens, comminutors. The capacity of mechanically cleaned bar racks versus the size of the opening of bar racks was represented in Figure 4.1 (Qasim, 1999). As the screen opening reduces, the quantity of collected screenings increases. Screening process has some common properties even though it does not exist a clear definition of screenable material. For example; the rag content can be estimated as 60 to 70 percent of the total screenings volume for 25 and 100 mm screens, respectively. Generally, coarse screening materials are highly volatile, as 80 to 90 percent or more, and the dry solids content percentage is 15 to 25 and the density 640 to 960 kg/m³ (Metcalf & Eddy, 2003).

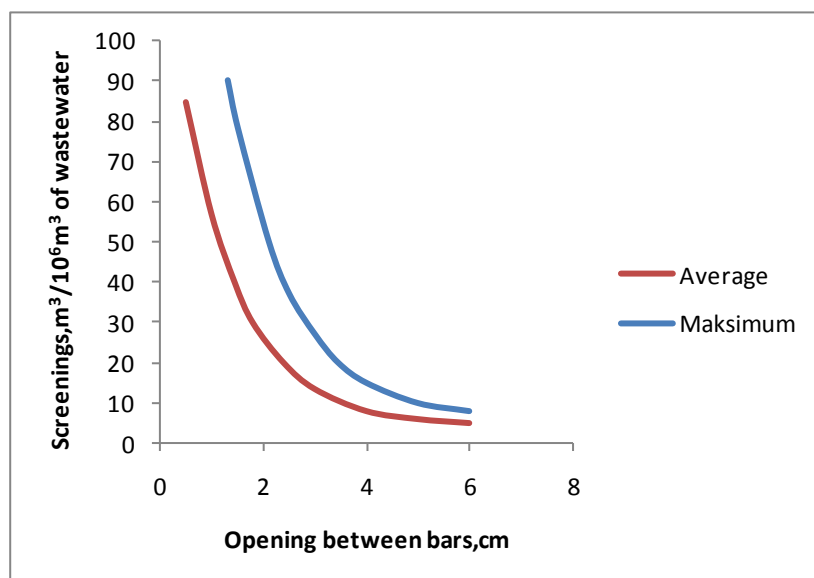


Figure 4.1: Quantities of screenins collected from mechanically cleaned bar racks (Qasim, 1999).

Fine screenings consist of materials that are kept on screens with openings less than 15 mm. Screen openings of 0.09 to 0.25 remove 5 to 10 percent of influent SS, while those of 0.03 to 0.06 can remove 10 to 15 percent. Volatile solids content of fine screenings varies from 65 to 95 percent. Bulk densities of fine screenings are slightly lower and moisture contents are somewhat greater compared to coarse screenings. Because of the putrescible matter, including pathogenic fecal material, within screenings, they must be properly handled and disposed, as well as contain substantial grease and scum that fine screenings contain (Metcalf & Eddy, 2003).

The openings of fine screens range between 0.035 - 6 mm are now available. Various types of microscreens have been developed in recent years that are used for upgrading effluent quality from secondary treatment plants(Qasim, 1999).

Bar racks are the most commonly used devices at medium and large sized wastewater treatment facilities. The design velocity, bar spacing, bar size, angel of inclination, and allowable head loses through the racks are summarized in Table 4.1.

Wash water consumption is important during ringing of screens and by installing solenoid valves to block the wash water flow up, energy can be saved. Thus, screening process is more effectively accomplished leading to reduce the energy required in other parts of the wastewater treatment plant(WEF,2009).

Table 4.1 : Design factors for manually cleaned and mechanically cleaned bar racks (Qasim, 1999).

Design Factor	Manually cleaned	Mechanically cleaned
Velocity through rack (m/sn)	0,3 - 0,6	0,6 - 1,0
Bar size		
Width	4 – 8	8 – 10
Depth	25 – 50	50 – 75
Clear spacing between bars (mm)	25 – 75	10 – 50
Slope from horizontal (degrees)	45 – 60	75 – 85
Allowable head loss, clogged screen (mm)	150	150
Maximum head loss, clogged screen (mm)	800	800

The most common disposal method of screenings are landfilling, disposal by on the plant (only for small plants), incineration alone or with sludge and grit (only for large scale plants), disposal with municipal solid wastes and finally discharge to grinders or mascerators where they are ground or returned to wastewater. These disposal methods are subject to Environmental Regulations on the disposal site(Metcalf&Eddy, 2003).

4.2.2 Grit and grease removal

Generally, all treatment plants include grit removal unit in order to remove sand, dust, cinder, seeds, large organic particles such as food waste and other heavy materials in wastewater heavier than organic matters that are nonbiodegradable.

Location of grit removal unit is preferred to be before the raw wastewater pumps. Sometimes, the depth of incoming sewers is not suitable therefore the location of grit removal unit is not desirable. Due to this reason, grit removal units are most commonly located after the raw wastewater pumps or along with the primary sedimentation tanks.

Incase grit materials are not removed from wastewater, operators come up against many problems in a wastewater treatment plant. In order to protect mechanical equipments, pumps, to prevent clogging of pipelines, deposits in channels and cementing on digesters, primary clarifiers and also to prevent inert material accumulation in aeration unit and digesters, a grit removal unit is necessary (Qasim, 1999). In the absence of a grit removal unit, inert materials will accumulate in digesters leading to reduce effective volume of digester and so as the gas produced which is a renewable source. Grit removal process also consumes energy in some extent therefore efficient grit removal process will reduce the energy required in other parts of the wastewater treatment plants (WEF, 2009).

Selection of grit removal units depends on the quantity and quality of grit, head loss, space requirement and the effect of the grit on other units (Qasim, 1999).

Grit removal facilities can be divided into three general categories:

1. Horizontal – flow grit chambers
2. Aerated grit chambers
3. Vortex type grit chambers

4.2.2.1 Horizontal – flow grit chamber:

Horizontal – flow grit chambers are divided into two categories according to their shapes; rectangular or square type of grit chambers. The flow passing through the grit chamber horizontally is controlled by the dimensions of the basin, influent distribution gates and by special weirs.

Horizontal flow grit chamber is the oldest type of grit chamber and the design velocity is closed to 0.3 m/s. The design velocity is used to maintain sufficient time for grit particles to settle down to the bottom of the grit chamber. The aim of the design velocity is to carry organic particles through the chamber and to resuspend any settled organic particles but to permit the heavier grit to settle out.

Representative design data for horizontal flow grit chambers were given in Table 4.2 (Metcalf&Eddy, 2003).

Table 4.2 : Typical design information for horizontal flow grit chambers (Metcalf&Eddy, 2003).

Item	Value	
	Range	Typical
Detention time, s	45 – 90	60
Horizontal velocity, m/s	0,24 - 0,39	0,3
Settling velocity for removal of:		
0,21 mm material, m/min ^a	0,98 - 1,28	1,16
0,15 mm material, m/min ^a	0,61 - 0,91	0,76
Headloss in a controlled section as percent of depth in a channel, %	30-40	36 ^b

^a If the specific gravity of the grit is significantly less than 2.65, lower velocities should be used.

^b For Parshall flume control

Square type of horizontal - flow grit chambers have been used for over 50 years. Solid particles are swept out by a rotating equipment to collect those particles at the side of the square grit chambers. Settled grits are collected by an inclined rake mechanism or they are pumped by grit pumps through a cyclone which separates organic material from concentrated grit (Metcalf&Eddy, 2003). Little energy is used in velocity type grit basins and after the construction of a basin, it does not exist much additional energy to save (WEF, 2009).

4.2.2.2 Aerated grit chambers

An aerated grit chamber, similar to standard grit chambers, is generally used to remove grits selectively. A current is provided in an aerated grit chamber by means of a diffused compressed air. The rate of the compressed air is adjustable to provide low velocity. Thus, grit particles can settle down in the basin. However, organic particles which are lighter are collected out of the basin. Aerated grit chambers are designed to remove particles over a 0.21 mm diameter; when the air flow is reduced, the spiral flow velocity also reduces and it enables us removal of the smaller particles (Qasim, 1999).

Aerated grit chambers have many advantages and they are generally used at medium and large scale treatment plants. The air flow provides freshened wastewater; air removes additional organic content and odor, thus minimum level of head loss is kept. Also, grease can be removed by means of a skimming equipment in the basin

and grits, having low putrescible organic content, can be discarded through the air control.

Design parameters of an aerated grit chamber are type of grit, detention time of grit particles, air supply, dead spaces, basin geometry, baffle configuration and inlet, outlet structures. Design informations for aerated grit chambers were given in Table 4.3.

Table 4.3 : Typical design information for aerated grit chambers (Metcalf&Eddy, 2003).

Item	Value	
	Range	Typical
Detention time at peak flowrate, min	2-5	3
Dimensions:		
Depth, m	2,1-4,9	
Length, m	7,6-19,8	
Width, m	2,4-7,0	
Width-depth ratio	1:1-5:1	1.5:1
Length-width ratio	3:1 -5:1	4:1
Air supply, m ³ /min • m of length	0,2-0,5	
Grit quantities, m ³ /103 m ³	0.004-0,2	0,015

The optimal operation is implemented by making a balance between the air produced and the air required. Insufficient air supply, unfortunately leads to bed odor due to settling of putrescible matters at the bottom of the tank and storage and disposal of grit materials become difficult (WEF,2009).

4.2.2.3 Vortex type grit chambers

Vortex type grit chambers use gravitational and centrifugal forces at the same time in order to separate grit material. Grit chambers, by means of gravitational and centrifugal forces, are mostly used for storm water, combined sewer flow, municipal and industrial wastewaters (Qasim, 1999).

Grits can be collected mechanically with the help of some equipments in combination with scrapers, screws, buckets, plows and etc. Settled grit can be directed to a central point by bottom slopes or created artificial air flow. Types of grit vortex grit chambers can be listed as tubular conveyors, bucket-type collectors and elevators, screw conveyors, grit pumps, and clamshell buckets. Air pumps are useful for small scale aerated chambers and in some cases a pipe can be used for tubular conveyors (Qasim, 1999).

Grit chambers also use energy to transport the grit and the most important source of energy comes from grit pumps although small motors may be used on center and conveyor. There is no need to operate pumps continuously; it is adequate to operate them on cycles for the control of energy intensity (WEF,2009).

Parameters given below should be taken into account before the design of a grit removal facility.

1. Wastewater characteristics and type of grit to be removed
2. Design average, peak, and low initial flows
3. Information on existing facility if plant is being expanded and site plan
4. Type of grit removal facility to be provided (horizontal – flow grit chamber, aerated grit chamber, vortex grit chamber, combined grit and organics removal system including degritting)
5. Influent pipe data and static head, force main and hydraulic grade line if grit removal is preceded by a pumping station
6. Head loss constraints for grit removal facility
7. Treatment plant design criteria prepared by the concerned regulatory agency
8. Equipment manufacturers and equipment selection guides (Qasim, 1999).

4.2.3 Primary sedimentation

Removal of the settleable organic solids is the basic principle of primary sedimentation tanks. It was noted that 70% of organic solids in wastewater are removed in primary sedimentation tank and the removal efficiency of a primary clarifier is generally 50-70 % for TSS and 30-40% for BOD₅ (Qasim, 1999). Volatile solids comprises 55% - 65% of settled sludge and 3-5% of sludge is solids. It was noted that primary sludge flowrate in primary sedimentation is only 0,3-0,5% of influent wastewater flowrate. Yet, SS concentration in primary sludge was given as 250 times the SS in wastewater (Öztürk,2007).Mechanical scrapers have been used to collect settled solids and then settled solids are pumped to a sludge processing area. Oil, grease and other floating materials are generally collected from the surface of the primary sedimentaiton tank and then the effluent can be discharged over the weirs. Coalescence of flocculant particles is the major mechanism to specify the settling behavior of solid particles in a primary sedimentation tank; better particle coalescence leads to faster settling of particles.

Some removal rates were expected in the primary clarifiers. These removal rates are calculated with the retention time in the primary clarifiers. Resulted removal efficiencies in a primary sedimentation tank were given in Table 4.4.

Table 4.4 : Recommend unit pollutants loads for domestic wastewater (Orhon, 2000).

	Raw wastewater	Settled wastewater	Removal Rate
	gr/capita.d	gr/capita.d	%
COD	90	65	28
BOD	40	30	25
TSS	60	20	66
Total-N	9	8.25	8.3
Total-P	1.2	1.1	8.3

Flocculation mechanism efficiency and better removal of settling flocs depend on chosen chemicals during primary sedimentation. There are also some dissolved materials such as soluble organics, phosphorus, and heavy metals, difficult to precipitate. Thus, chemicals can be added to facilitate the precipitation of those materials (Qasim, 1999).

The design overflow rates for various types of clarification facilities were given in Table 4.5. Primary sedimentation facilities have been generally designed for an overflow rate of $40 \text{ m}^3/\text{m}^2.\text{d}$ at average design flow.

Table 4.5 : Design Overflow Rates for Plain Primary Sedimentation Basins (Qasim,1999).

Condition	Range ($\text{m}^3/\text{m}^2.\text{d}$)	Typical ($\text{m}^3/\text{m}^2.\text{d}$)
Primary clarification prior to secondary treatment		
Average flow	30-50	40
Peak flow	70-130	100
Primary clarification with waste-activated sludge return		
Average flow	25-35	30
Peak flow	45-80	60

Main design points for primary sedimentation facility are given below:

1. Average and peak design flows, including the returned flows from other treatment units
2. All sidestreams from thickener, digester, and dewatering facilities to be considered if flows are returned ahead of primary sedimentation
3. Discharge parameters considered in treatment plant design
4. Available space and topographic map of the plant site
5. Shape of the tank (rectangular, square, or circular)
6. Influent pipe data, to include diameter, flow characteristics, and approximate water surface elevation or hydraulic grade line
7. Head loss constraints for sedimentation facility

The pumping of primary sludge was given as the largest energy-consuming component in a conventional primary treatment process. Used energy arises from primary sludge pumps, primary sludge grinders and primary sludge valves. Finally, it was noted that these energy uses are process driven and established by design constraints such as flow and head requirements on the pumps, therefore they do not readily afford opportunities for energy savings (WEF,2009).

It is possible to increase production of primary sludge in pre-sedimentation tank by polymer or metal salt dosage. Primary sludge digestion characteristic is suitable in comparison with secondary sludge. As a result of this, a good digestion and more biogas production have been provided. However, additional costs are in question during the disposal of sludge which is rich in metal salts. We should pay attention to the effects of a more efficient primary treatment on nitrogen removal. In consequence of high efficiency, generally unwanted BOD/N ratios have been reached. Therefore, problems may occur in the denitrification stage (NL Agency, 2010).

4.2.4 Disinfection

After wastewater treatment, final effluent already contains microorganisms; thus disinfection process is used to inactivate or to destroy most of these microorganisms and pathogens as well. Wastewater contains many types of pathogens and human enteric organisms associated with waterborne diseases, such as typhoid, cholera, paratyphoid, and bacillary dysentery caused by bacteria, amebic dysentery caused by

protozoa and common viral diseases including poliomyelitis and infectious hepatitis. It was noted that disinfection refers to selective destruction of disease-causing organisms in the water supply or wastewater effluent and today waterborne diseases can be controlled by chlorinating both water supply and wastewater effluent. Unfortunately, chlorination process generates chlorinated hydrocarbons which are carcinogens (Qasim, 1999).

Disinfection methods can be classified into three main categories as physical, chemical and radiational. Physical disinfection methods are heat (pasteurization) and ultraviolet radiation. It was reported that due to large volumes of liquid and high energy cost in heating, pasteurization is not applicable to water and wastewater. Ultraviolet disinfection process has been an excellent disinfectant because it has no residual product. The efficiency of ultraviolet radiation depends on the depth of penetration, time of contact, and turbidity or suspended solids that may reduce the effective depth of penetration. Sunlight has also been an effective alternative used for disinfection (Qasim, 1999).

Oxidizing agents have also been used as chemical disinfectant in many different applications for water and wastewater; such as halogens (chlorine, bromine, and iodine), ozone, hydrogen peroxides, potassium permanganate, alcohols; phenol and phenolic compounds; heavy metals; soaps and synthetic detergents; and alkalies and acids. It was noted that ozone is a highly effective disinfectant due to it has no residual product, it should be generated at the site and it has been very effective for destroying odors and color-causing compounds in water (Qasim, 1999).

The most common method of disinfection is represented as the addition of chlorine gas solutions or hypochlorite solutions. Electrical energy is consumed by disinfection with chlorine gas and bulk hypochlorite due to:

- a. Evaporator heaters (chlorine gas),
- b. Pumping of dilution water, and
- c. Metering and pumping of chlorine/hypochlorite solution (WEF, 2009).

Having high penetrating power, gamma rays such as cobalt 60 are emitted from a radioactive source in order to disinfect or sterilize water, wastewater and sludge. This method has been denoted as a reliable method and it has no residual product in the effluent; also causes an alteration of the makeup of sludge to benefit dewaterability

and stability. In contrast, it was pointed out that safety has been threatened by the radiation emitted from the system.

EPA Task Force Report has described the use of ultraviolet (UV) radiation for disinfection of wastewater process as potentially advantageous when comparing with conventional disinfection by chlorination and ozonation. Besides, UV radiation process is noted as a physical process, however other processes are chemical processes and they have chemical residual products.

The low pressure mercury lamps was specified as the primary source of UV energy. The low pressure mercury lamp is nearly globally accepted as the most efficient of UV radiation for disinfection systems and it is also accepted as an effective source. The radiation is generated by hitting an electric arc through mercury vapor, leading to the emissions of UV light (Qasim, 1999).

Main design points for UV disinfection facility are given below:

1. Effluent TSS concentration (must be typically less than 20 mg/L)
2. Peak wet weather, peak dry weather, average design, and minimum initial flows
3. Discharge standards for treatment plant design
4. Determined contact time
5. UV absorbance or transmittance of the effluent to maintain the desired level of germicidal effect; color caused by industrial wastes can be serious.
6. Head loss constraints through the unit; the automatic level control gate requires a minimum of 0.4 m of head loss.
7. Existing site plan with contours and location of the disinfection system
8. Influent and effluent seasonal coliform count for specific flows (annual average, maximum 7-day average, maximum 30-day average, peak dry weather, and peak wet weather) (Qasim, 1999).

4.3 Process design of biological treatment units

Remaining wastewater contaminants following primary treatment, consist of colloidal matter which is highly organic a small amount of dissolved organic matter, nutrients, and dissolved inorganic solids. Colloidal and dissolved organic matters are subject to biological secondary treatments. It was noted that there were numerous

methods used to establish biological secondary treatments. Most biological secondary treatments can be classified as either suspended growth or fixed film systems. An active culture of microorganisms are in suspension in a reactor and air or dissolved oxygen is given to this culture to maintain biological activity. Oxygen demand required for a biological reactor can be supplied by various ways such as diffused, dispersed, or mechanical aeration systems which are most commonly used as aeration devices (WEF,2009).

Numerous configurations used at WWTPs for suspended growth systems were determined. Today's activated sludge systems have been operating for carbonaceous removal and nitrification, biological phosphorus removal, or complete biological nutrient removal. Carbon and nitrogen removal systems use aeration unit throughout the aeration basin(s). It was denoted that these facilities may be constructed as single pass tank or multiple pass tank. The selection of the proper dissolved oxygen within the aeration basins is important for the energy efficiency of the system. The energy efficiency may be increased by choosing high dissolved concentrations between 0.5 to 2.0 mg/l (WEF,2009).

It was pointed out that the assessment of the characterization of the untreated wastewater, the type of the wastewater treatment facility and the required nutrient control level are important points in terms of selection of a nutrient control strategy. Biological phosphorus removal systems and biological nutrient removal (BNR) systems have both aerated and unaerated zones to accomplish the biological treatment.

Biological nutrient removal systems have been designed in 3 stages; anaerobic, anoxic and aerobic zones. Phosphorus removal process occurs in anaerobic tanks and the growth of the phosphorus storing microorganisms (acinetobacter) is also accomplished in this tank. Acinetobacter has an important characteristic that it can store more amount of phosphorus in itself when comparing to the amount necessary for their growth. Bacteria, being exposed to anaerobic stressed conditions, releases phosphorus and then uptakes much more in their structure under aerobic conditions.

Internal recycle flowrate constitutes a considerable amount of the forward flow which is 300 to 400 %, and requires a substantial quantity of water to pump. Pumps must have high capacity in terms of volume and they have to be low head pumps. To maximize energy savings, it was noted that the pumps must be on VFDs in order to

return the correct quantity of flow to maintain the proper operating conditions (WEF,2009).

Nitrate content of aerobic zone named internal turn and the anaerobic effluent are pumped to the anoxic zone for the purpose of denitrification process. The process mechanism was explained; heterotrophic bacteria provide a COD / BOD oxidation by using nitrate instead of oxygen and thus nitrate is reduced to nitrogen gas. Settled domestic sludge has C/N ratio lower than activated sludge.

Therefore, it can be said that removals of only 20-30% can be attributed to the microbial synthesis of nitrogen containing cellular compounds.

The rest of the COD/BOD has been oxidized by heterotrophic bacteria in the next stage in aerobic tank. Surface aeration or diffused aeration equipments supply required oxygen. The role of autotrophic bacteria is to oxidize ammonia into nitrite and nitrate. Growth of bacteria constitutes excess sludge which will be removed as secondary sludge in secondary clarifiers.

4.3.1 The importance of nutrient in wastewater

Carbon removal process often provides many discharge permits in Turkey when discharging effluents to receiving bodies whereas for some sensitive areas nitrogen and phosphorus removal is essential and the limiting factor because there is a risk in terms of eutrophication. Therefore, it was noted that the control of the influent nitrogen and phosphorus is crucial, especially the primary plant nutrients, when discharged to water bodies, are hydraulically susceptible to eutrophication.

It was pointed out that eutrophication has been one of the main reasons in terms of water pollution, could be defined as algal bloom. Algal bloom is caused by the excessive reproduction of algae by excessive nutrient discharge to lakes and estuaries. Nitrogen and phosphorus content of treated wastewater discharges are the principal nutrients for eutrophication having negative impacts on fauna and flora in water bodies. Eutrophication phenomena was a major threat especially for poorly replenished water bodies, such as natural lakes, artificial impoundment, bays and sluggish streams.

It was defined that some blue green bacteria have the ability to fix atmospheric nitrogen gas for their primary production. For this purpose, eutrophication control

strategies generally made up of the control of phosphorus in the effluent discharges to water bodies.

An appropriate utilization of the activated sludge process applied for biological nutrient removal will provide and most probably guarantee a cost effective and sustainable nutrient discharge reduction into receiving waters. Optimal design and control of activated sludge systems will ensure a sustainable nutrient (N, P) removal in an efficient way (İnsel,2005).

As we all know, activated sludge systems have complex behaviour therefore activated sludge models have been widely used tools to understand and to predict the complex biological structure and mechanism of activated sludge systems. Yet, under certain conditions, some external factors such as environmental conditions, operating conditions, interactions between complicated and unknown biological reactions, could make difficult the understanding of the system. For a better understanding of biological processes, it was noted that researches and developments in parallel to advancing technologies in biotechnology, have been provided. Therefore, discoveries in biotechnology for physical and biological processes encourage the increasing trend in the complexity of activated sludge modelling (İnsel,2005).

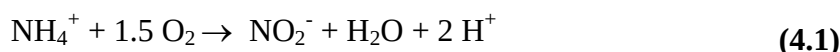
4.3.2 Biological nitrogen removal systems

Nitrogen in municipal sewage consists of two forms which are ammonia and organic nitrogen. Typical composition of domestic wastewater with respect to nitrogenous compounds was summarized in Table 4.6. The table below showed that the total nitrogen content of domestic wastewater varies from 20 to 50 mg/l, consisting primarily of ammonia and organic nitrogen. Typical removal efficiencies obtained with traditional primary and secondary treatment without nitrification/denitrification were also represented in Table 4.6 (Orhon& Artan,1994).

Table 4.6 : Nitrogen Content of Sewage and Its Removal by Conventional Treatment (Orhon&Artan,1994).

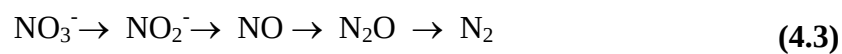
Concentration	Raw Waste (mg/l)	Effluent Primary Settling		Effluent from Secondary Biological Treatment	
		mg/l	% Removal	mg/l	% Removal
Organic N	10 - 25	7 - 20	10 - 40	3 - 6	50 - 80
Dissolved	4 - 15	4 - 15	0	1 - 3	50 - 80
suspended	4 - 15	2 - 9	40 - 70	1 - 5	50 - 80
NH ₃ -N	10 - 30	10 - 30	0	10 - 30	0
NO ₂ -N	0 - 0.1	0 - 0.1	0	0 - 0.1	0
NO ₃ -N	0 - 0.5	0 - 0.5	0	0 - 0.5	0
TN	20 - 50	20 - 40	5 - 25	15 - 40	25 - 55

Almost all nitrogen forms will be readily deaminated in the pH ranges encountered in an activate sludge process. Therefore, for the effluent of conventionally operated activated sludge plants, it can be noted that ammonia is the main dissolved constituent containing nitrogen. Nitrate is the nitrogen form converted to ammonia biologically during nitrification, takes part of the subsequent denitrification process as a source material during the overall nitrogen removal system. It was also specified that a better understanding of the stoichiometry and the kinetics of nitrification and denitrification processes will provide an optimized activated sludge process (Metcalf & Eddy, 2003).



By providing the energy required from nitrogen oxidation, autotrophic microorganisms called as nitrosomonas and nitrobacteria convert ammonia into nitrite and nitrite into nitrate respectively. Nitrification process has been carried out by these bacteria. It was also noted that those type of bacteria use inorganic carbon (CO₂) in the synthesis instead of organic carbon. Being more sensitive to environmental conditions than heterotrophic bacteria, autotrophic bacteria could be affected by certain environmental parameters such as dissolved oxygen concentration, pH and alkalinity, temperature and inhibitors. Therefore, supply of necessary conditions in the design of treatment plant becomes important for these consecutively working nitrification bacteria.

Most common process for nitrogen removal in wastewaters is biological denitrification. Process is based on conversion of nitrate into nitrite and then to nitrogen gas. Thereby, dissolved form of nitrogen which is harmful to the environment is converted into harmless gas form. In this reaction, oxygen is replaced by nitrite and nitrate in microbial respiration. Therefore, denitrification takes place in anoxic zone the absence of oxygen. Unlike nitrification, there are various type of facultative denitrification bacteria in denitrification. If nitrate and oxygen are present at the same time, these facultative bacteria will directly use oxygen as an electron acceptor by inhibiting the denitrification process (Orhon at al., 2000).



Return nitrate nitrogen to anoxic zone is completely consumed if denitrification potential is enough. However, it should be taken into account that oxygen is also included in the internal return to anoxic zone. In this case, some portion of denitrification potential is consumed by the dissolved oxygen returned to anoxic zone (Orhon at al., 2000).

One of widely accepted carbon and nitrogen removal system in activated sludge process was given in Figure 4.2. The activated sludge was separated from each other as two primary section having the aerobic and anoxic zones. These zones can be adjusted by suitable adjustments in the aeration facility such as plug flow reactor or split up; by devising the reactor in series (Insel,2005).

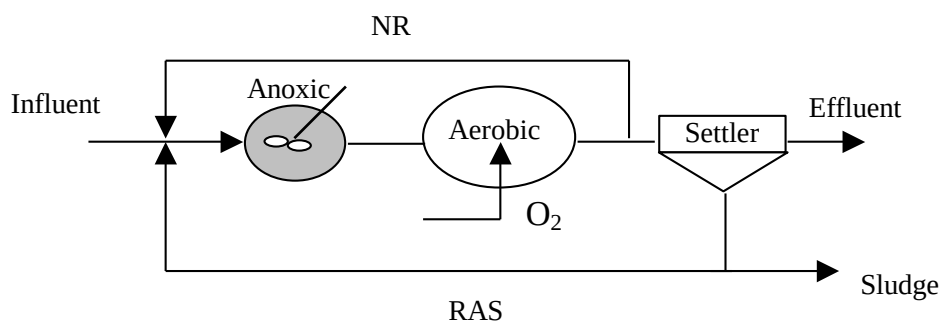


Figure 4.2:Modified Ludzack Eltinger (Insel, 2005).

It was noted that the MLE process was a very adaptable process to existing activated sludge facilities and could easily meet a common effluent standart of <10 mg/l for total nitrogen. For domestic wastewater, influent BOD and NO₃-N concentrations are important; BOD and NO₃-N concentration from 4 to 7 mg/l is generally sufficient and also a BOD/TKN ratio of 4:1 in the influent wastewater is usually

sufficient for effective nitrate reduction by preanoxic processes (Metcalf & Eddy, 2003).

Denitrification process is based on same principles with carbon removal process, with a slight difference of using different electron acceptors. In carbon removal, enough electron acceptor should be provided to the system in order to convert the electron donor organic matter into end products.

The aim of denitrification process is to remove the electron acceptor nitrate and four conditions below should be available in the system for biological denitrification: presence of nitrate (or nitrite), absence of dissolved oxygen, facultative biomass aggregate, presence of electron donor (Orhon et al., 2000).

The role of aerobic zones of BNR systems has been to convert ammonia to nitrate and nitrite by oxidation process. The oxidation of ammonia requires energy and this energy is recovered by the oxidation of nitrogen which has been used as an electron acceptor in the anoxic zones, thus the overall oxygen requirement is reduced. It was determined that the reduction of 1 g nitrate nitrogen is equivalent to the reduction of 2.86 g oxygen (WEF,2009).

Biological conversion of ammonia to nitrate (nitrification) requires oxygen to be supplied at 4.57 times the ammonia nitrogen loading, which is a significant energyconsuming process. Additionally, because nitrification only occurs with low food to microorganism ratios, more energy is required to satisfy endogenous respiration demands. (WEF,2009) Oxygen demand of the system will show a $2.86/4.57 \times 100 = 63\%$ reduction due to the 2,86 g O₂/g N gain in denitrification compared to 4,57 g O₂/g N requirement for nitrification. 15 – 20% saving in total oxygen demand can be provided in systems containing biological denitrification due to the ratio of oxygen demand for nitrification, being 25 – 35% of total oxygen demand (Orhon et al.,2000).

Dissolved oxygen, pH, alkalinity, temperature, inhibitors and the type of organic matter accomplishing the electron donor task in denitrification process are environmental parameters affecting nitrification and denitrification processes rate.

Dissolved oxygen is an important substrate for nitrification and its effect to nitrification process is a widely researched subject by researchers. It is assumed that as dissolved oxygen concentration decreases, this parameter becomes a rate limiting

substrate. Nitrification process occurs in presence of 0.5-2.5 mg/l dissolved oxygen in environment. The value is defined to necessity of 2 mg/l dissolved oxygen value in practice. Dissolved oxygen concentration above 0.2 mg/L has inhibitive effect on denitrification, while this value range between 0.3-1.0 mg/l in activated sludge systems (Orhon et al., 2000).

There are two important reasons for the sensitivity of nitrification rate to pH changes. First reason is that both $[H^+]$ and $[OH^-]$ ions have significant inhibition effect on nitrification bacteria and the other reason is that nitrification process consumes the present alkalinity and cause pH decrease. This situation is observed in cases that pH is above 8,5 and below 7 Optimum nitrification rate is set between 7 and 8,5 pH values. Denitrification bacteria are facultative and prefer aerobic respiration in existence of oxygen and as a result, necessary quantity of electron donor for denitrification decreases.

Denitrification bacteria are more resistant to changes in pH compared to autotrophic bacteria. Denitrification rate is affected negatively in pH values below 6 and above 8 (USEPA, 1975). High pH values can be detected in removal of high amount of nitrate due to the alkalinity production in denitrification process (Orhon et al., 2000).

Alkalinity increase occurs in denitrification process, while there is a consumption of alkalinity in nitrification. Alkalinity of 3.57 g $CaCO_3$ is produced in reduction of 1 g nitrate. However, alkalinity of 7.14 g $CaCO_3$ is consumed in oxidation of 1 g ammonia during nitrification process. Consequently, it can be understood that approximately half of the alkalinity consumed in nitrification can be recovered during denitrification. Due to the capability of alkalinity recovery, denitrification is an important process for especially wastewaters with low alkalinity. As in other biochemical reactions, nitrification and denitrification process is also affected by the changes in temperature. Temperature especially affects the maximum growth rate of autotrophic bacteria. Denitrification rate decreases for the temperature values below 5°C and remains stable above 20°C, optimum at 40°C and reaches its maximum value at 50°C (Orhon et al., 2000).

Site specific conditions, existing process and equipment, and treatment needs affect the selection of a specific process for biological nitrogen removal. Typical design and operation parameters for a nitrogen removal process were given Table 4.7. and

commonly used processes' advantages and limitations, their treatment capability in terms of effluent total nitrogen concentrations were also summarized in Table B.1.

Table 4.7 : Typical design parameters for commonly used nitrogen removal processes (Metcalf & Eddy, 2003).

Process	SRT,d	MLSS, mg/L	Total	Anoxic zone	Aerobic zone	RAS % of influent	Internal recycle% of influent
MLE	7-20	3000-4000	5-15			50-100	100-200
SBR	10-30	3000-5000	20-30	Variable	Variable		
Bardenpho (4-stage)	10-20	3000-4000	8-20	1-3 ^a 2-4 ^c	4-12 ^b 0,5-1 ^d	50-100	200-400
Oxidation ditch	20-30	2000-4000	18-30	Variable	Variable	50-100	
Bio-denitro TM	20-40	3000-4000	20-30	Variable	Variable	50-100	
Orbal TM	10-30	2000-4000	10-20	6- 10	3- 6 ^a 2-3 ^b	50-100	Optional

a:1st stage, b:2nd stage, c:3rd stage, d:4th stage

Ammonia air stripping from effluents with alkali chemicals having pH elevated above 11 was noted as a nitrogen removal process. This process is relatively inefficient and temperature sensitive requiring considerable amounts of chemicals (usually lime and neutralizing acid), has been expensive, and adds offending ammonia to the air. This is why, ammonia air stripping is not commonly used in municipal WWTPs. Biological denitrification was listed as an alternative process following a nitrification process (WEF,2009).

For nitrogen removal, side-stream treatment processes were cost-effective, especially for recycle streams rich in nitrogen and generated from dewatering of anaerobically digested sludges namely centrate (WEF,2009).

4.3.3 Biological phosphorus removal systems

Three forms of phosphorus exist in wastewater; orthophosphate, polyphosphate and organically bound phosphorus. Total inorganic phosphorus consists of simple orthophosphates and polyphosphates or condensed phosphates. For the liberation of organic phosphorus as orthophosphate, a total oxidative destruction of the organic matter is required and acid hydrolysis converts polyphosphates to orthophosphates which are easily assimilated by microorganisms. It was specified that other forms of phosphorus become available to microorganisms only after hydrolysis to orthophosphates (Orhon and Artan, 1994).

The amount of phosphorus in wastewater has been significantly affected by the contribution of industrial discharges, nonpoint sources and the differences in

phosphate content of detergents. Therefore, the sources and quantities of phosphorus in domestic sewage are said to be quite variable and detergents could contribute half the total phosphorus discharge from households.

Typical phosphorus levels in fresh domestic sewage were given in Table 4.8. and it shows that nearly 70% of the total phosphorus in sewage is inorganic. Filtrable orthophosphates range is said to be between 15-35% of the total phosphorus in domestic sewage. It was noted that the ratio of ortho/total phosphate depends on the source of the sewage, and besides on the degree of degradation of phosphorus containing organics (Orhon and Artan, 1994).

Table 4.8 : Phosphorus in Domestic Wastewaters (Orhon and Artan, 1994).

	Concentration as P (mg/l)	Phosphorus Generation (kg/ca-year)
Total	6-20	0.8-1.8
Organic	2-5	0.3-0.6
Inorganic	4-15	0.5-1.2

Two possible techniques used for phosphorus removal were listed as traditional chemical phosphorus removal (CPR) and biological phosphorus removal (BPR). Chemical phosphorus removal can be achieved through precipitation of phosphorus in the soluble form by adding metal salts such as aluminum sulfate (alum) and ferric chloride. Eventhough this technique has been effective, it could be costly due to high chemical costs as well as increased sludge treatment and disposal costs (WEF,2009).

On the other hand, biological phosphorus has been a cost effective process for the removal of phosphorus from wastewater using a non- aerated zone that selects for the growth of specific organisms called polyphosphate accumulating organisms (PAOs). Polyposphate accumulating organisms are said to be capable of storing excessive amounts of phosphorus in their body cells (5 to 7 % of body mass) under certain environmental conditions. Non-aerated zone comes into existence when there is an absence of oxygen and PAO_s take up the readily biodegradable organic carbon in the wastewater and release phosphorus in these conditions. PAO_s take up phosphorus and store it in their body cells in the aerated zone, which then will be wasted in the sludge stream. BPR processes were better suited for phosphorus and nitrogen recovery than CPR processes in terms of nutrient recovery(WEF,2009),(Yağcı,2005).

It was reported that influent COD/P ratio is so important that should be in sufficient amount in the anaerobic zone for phosphorus release. However, high COD/P ratio generally means that the phosphorus storage capacity of sludge can not be used completely. In these conditions, the phosphorus content of sludge will be low. Phosphorus content and low effluent phosphorus concentrations depend on the completely use of maximum phosphorus storage capacity of sludge. In the case of substrate type permits dominant Poly-P organisms, it is sufficient that the maximum ratio of COD/P is 20. Therefore, maximum phosphorus storage capacity of sludge may be used when minimum substrate concentration and COD are at limiting side (Taşlı,1996). In good conditions, they can incorporate phosphorus in the ratio of 3.5 % instead of phosphorus which is % 2 of their dry weight (Geilvoet at al.,2010).

It was noticed that A/O type systems, widely known as Phoredox systems, could bring about enhanced biological phosphorus removal. It can be said that Phoredox has been derived from phosphorus and redox potential words to indicate the importance of low reduction conditions in anaerobic zone. In the case of there is no requirement for nitrification process, Phoredox process consisting of 2 stages as anaerobic (A) reactor prior to the aerobic reactor (O) could be operated similar to predenitrification type plants (see Figure 4.3) (Taşlı, 1996).

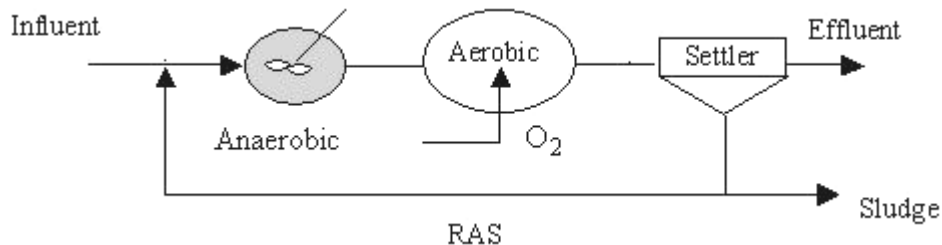


Figure 4.3: A/O Process.

The first reactor is anaerobic since no nitrate is generated in the aerobic reactor without any internal recycle stream. EBPR which is the enhanced biological phosphorus removal, is promoted through uptake of volatile fatty acids in anaerobic reactor. It was noted that the system was not operated for denitrification since there were no anoxic zones in the system layout (İnsel,2005).

Environmental parameters; such as dissolved oxygen, pH, temperature and inhibitors affect Biological phosphorus removal process. Biological phosphorus removal may be adversely effected in biological combined nitrogen and phosphorus removal

systems when the DO concentration in the aerobic zone remains 1.5-3.0 mg/l. It was defined that if the DO is too low, that phosphorus removal may be reduced, nitrification will be limited and a poor settling sludge may be developed. In the case of too high DO, denitrification performance could be limited because of high DO concentration in the recycle stream to the first anoxic zone. As a conclusion, it can be said that higher nitrate nitrogen concentration could affect the phosphorus release performance of the anaerobic zone (EPA, 1987).

Nagashima et al. found that total phosphorus removal in the Modified Bardenpho process was improved from 42 to 92 percent as the pH was increased from 5 to 8. It was concluded that the efficiency of biological phosphorus removal could decline significantly below a pH of 6.5. According to studies done, 90 percent of biological phosphorus removal was possible over a temperature range from 18 °C down to 6 °C. However, it was noted that studies showed a decreased nitrogen removal efficiency below 10 °C (EPA, 1987).

4.3.4 Enhanced biological phosphorus removal (EBPR) with nitrogen removal processes

The design of simultaneous nitrogen and phosphorus removal systems from wastewaters requires a long sludge age. The EBPR part should be integrated with the system in order to maximize enhanced phosphorus removal notwithstanding the relatively low excess sludge production. The design of EBPR consists of minimizing the amount of nitrate and dissolved oxygen introduced to the anaerobic volume, and securing optimal fermentation conditions for the influent readily biodegradable COD in order to maximize available S_A . It was noted that the maximum allowable non-aerated volume fraction should include the required anaerobic and anoxic phases. 35-40% of the total volume may be referred to the anoxic volume and 10-15% to the anaerobic volume. Depending on the required effluent nitrate and phosphate concentrations, adjustments can be made (Orhon et al., 2009).

For activated sludge processes, the most accepted carbon and nitrogen removal systems, the EBPR systems with recent developments were given and explained below (Insel, 2005).

4.3.4.1 Three stage phoredox (A^2O) process

It was explained that the difference of A^2O and the AO process was the presence of an additional anoxic reactor between the anaerobic and aerobic reactor. This system was shown in Figure 4.4. This layout shown in Figure 4.4 is the modification of the AO system designed for denitrification together with EBPR. The nitrate concentration coming to the anaerobic compartment is reduced by the anoxic zone. The hydraulic retention time for the anoxic reactor may be selected approximately 1 hour (İnsel,2005). In the anoxic zone, there exist a deficiency in terms of DO, but chemically bound oxygen in the form of nitrate or nitrite is introduced by recycling nitrified mixed liquor from the aerobic section (Metcalf and Eddy, 2003).

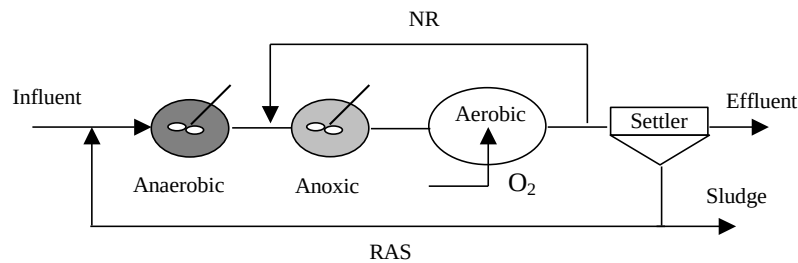


Figure 4.4: Plant layout for A^2O (İnsel, 2005).

4.3.4.2 Modified Bardenpho (4-5 Stage) type plants

The combination of two processes which are preanoxic and postanoxic systems constitute the Bardenpho (4-stage) process used for nitrogen removal. An internal recycle is commonly used to direct the nitrified mixed liquor to the first anoxic compartment. The retention time of the post-anoxic stage is specified as about the same or larger than that of preanoxic zone. It was reported that the nitrate was reduced from 5-7 mgN/l to less than 3 mgN/l in the secondary anoxic and aerobic zone. A plant layout for a 4-stage Bardenpho was given in Figure 4.5 (İnsel,2005).

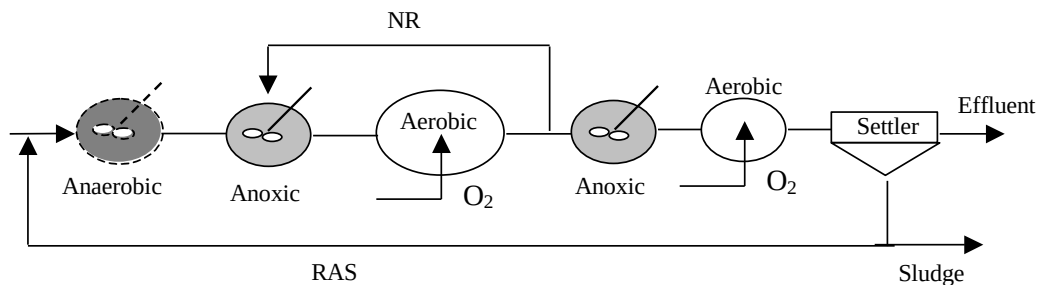


Figure 4.5: Layout for Bardenpho system (İnsel, 2005).

A modification can also be established; a 5-stage Bardenpho process via introducing an anaerobic zone to promote acetate uptake for enhanced biological phosphorus removal. Aside from the A²O layout depicted in Figure 4.5, it was reported that the second anoxic stage removes the additional nitrate using the endogenous carbon source. Nitrogen gas stripping is provided by the final aerobic stage against a failure of activated sludge settling (İnsel,2005).

4.3.4.3 Modified University of Cape Town, UCT

It was noted that the UCT process is developed by the research group of the University of Cape Town (South Africa) for EBPR from weak wastewaters. The aim of this process is to minimize the amount of nitrate in the anaerobic reactor, which is critical for EBPR. For this purpose, nitrate from the settler is recycled to the anoxic reactor ($2 \cdot Q_{\text{influent}}$) and also an additional denitrified recycle, DNR is introduced from the anoxic to the anaerobic compartment. This process was shown in Figure 4.6. The retention time for anaerobic reactor should be longer than the Phoredox systems due to the mixed liquor with lower biomass concentration and the retention time is generally in the range of 1-2 hours for the anaerobic reactor. It was reported that in the Modified UCT process, the anoxic reactor is separated into 2 anoxic compartments. It was explained that the first and second compartments receive the return activated sludge RAS and nitrified recycle, respectively (İnsel,2005).

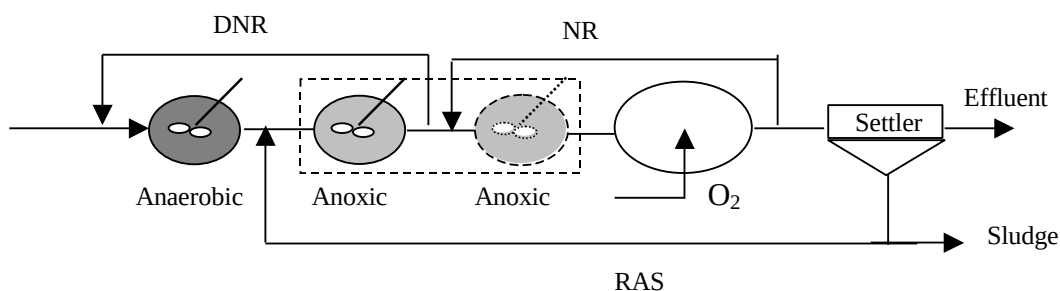


Figure 4.6: Modified University of Cape Town, UCT (İnsel, 2005).

4.3.4.4 Virginia initiative plant, VIP

It was mentioned that the VIP process is developed by Daigger *et al.* (1988). This process was shown in Figure 4.7. The properties of VIP from A²O and UCT plants is that the all zones have been staged consisting at least 2 CSTRs in series. The nitrified recycle (NR) and return activated sludge (RAS) are connected to the first compartment of the anoxic reactor.

It was reported that the plug flow regime of the anoxic reactor provides better denitrification. It was concluded that the nitrate load recycled to the anaerobic reactor was reduced and the VIP process could be operated in high rate and extended aeration mode, moreover the sludge retention time, SRT and hydraulic retention time, HRT for anaerobic and anoxic compartments were generally in the range of 1.5-3 days and 1.0-1.5 hours, respectively (İnsel,2005).

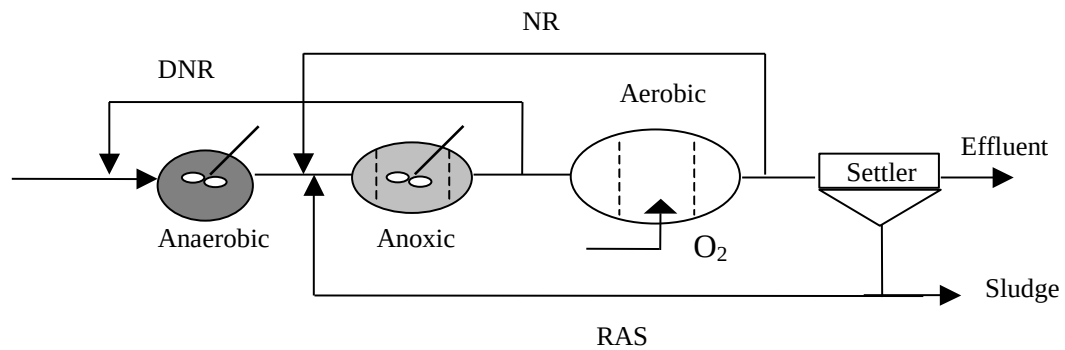


Figure 4.7: VIP process (İnsel, 2005).

4.3.4.5 PhoStrip process

PhoStrip process can be incorporated easily into existing activated sludge plants. Process is flexible; phosphorus removal performance is not controlled by BOD/P ratio. Significantly less chemical usage than mainstream chemical precipitation process. Can achieve reliable effluent orthophosphate concentrations less than 1mg/l (Metcalf and Eddy, 2003).

The design and operating parameters which have been used commonly in nitrogen and biological phosphorus removal processes were indicated in Table 4.9. According to the table, sludge retention time (SRT) comprises a wide range for nitrogen and phosphorus removal processes. It was reported that the mixed liquor suspended solid (MLSS) concentration in a biological reactor should be maintained generally around 2-4 mg/l for nutrient removal processes. Providing greater flexibility, higher hydraulic retention times (HRT) for oxidation ditch type plants (Biodenitro, Orbal) and SBRs were to be preferred for process design. In the case of phosphorus removal, to maintain desired MLSS in a biological reactor, lower HRTs are sufficient for A/O and A²O processes. It was noted that for nitrogen and phosphorus removal, SBRs are more flexible in terms of HRT selection.

For A/O and A²O processes, return activated sludge range is between 25% - 100% because the nitrate load through the anarobic zone should be reduced for better EBPR efficiency. Generally, a minimum RAS value of 50% (sludge thickening factor) has been preferred in order not to sludge blanket build up in the final clarifier. It was specified that the Bardenpho, UCT and VIP processes require high internal recycle rates due to the multi-staged reactor composition. During the operation, high internal recycle rates are necessary from aerobic to anoxic, anoxic to anaerobic reactors.

For oxidation ditches, it is possible to maintain higher internal recycles in a closed loop bioreactor with the aid of mixers. Finally, there is no need to maintain an internal recycle in oxidation ditches (İnsel, 2005).

Table 4.9 : Design and operating parameters for biological phosphorus removal processes (Metcalf and Eddy, 2003).

Process	SRT	MLSS	HRT(h)			RAS	Internal Recycle
	day	g/L	Anaerobic Zone	Anoxic Zone	Aerobic Zone	% of Influent	% of Influent
A/O	2-5	3-4	0.5-1.5	-	1-3	25-100	
A ² O	5-25	3-4	0.5-1.5	0.5-1	4-8	25-100	100-400
UCT	10-25	3-4	1-2	2-4	4-12	80-100	200-400 (anoxic) 100-300 (aerobic)
VIP	5-10	2-4	1-2	1-2	4-6	80-100	100-200 (anoxic) 100-300 (aerobic)
Bardenpho (5-stage)	10-20	3-4	0.5-1.5	1-3 ^a 2-4 ^b	4-12 ^a 0.5-1 ^b	50-100	200-400
PhoStrip	5-20	1-3	8-12		4-10	50-100	10-20
SBR	20-40	3-4	1.5-3	1-3	2-4		

a:1st stage, b:2nd stage

Advantages and limitations of phosphorus removal process are given in Table B.2

4.3.5 Extended aeration activated sludge system

The difference of extended aeration process from the conventional plug flow process is that the operation has been realized in the endogeneous respiration phase of the growth curve leading to long aeration time and low organic loading. This plant is said to be generally applied for prefabricated package plants for small societies (Metcalf & Eddy, 1991).

In an extended aeration process, the reactor is usually equipped with aeration and mixing devices. Wastewater passing from screening unit, will enter to the reactor and then will be combined with the return activated sludge. It was reported that the tank configuration, aeration and mixing devices provide unidirectional reactor flow, thus the energy used for aeration is sufficient to provide adequate mixing with a relatively long hydraulic retention time. Commonly used design parameters for the treatment of domestic sewage by activated processes were given in Table 4.10.

Table 4.10 : Typical design parameters for the treatment of domestic sewage by activated sludge (Orhon and Artan, 1994).

Parameter	Conventional Activated Sludge	Extended Aeration
Sludge age, θ_x , days		
Warm	3-6	10-20
Temperate	5-10	20-30
Cold	10-15	>30
F/M ratio (kgBOD ₅ /kg VSS.day)		
Warm	0.4-0.6	0.2-0.25
Temperate	0.3-0.5	0.1-0.2
Cold	0.2-0.4	<0.1
Hydraulic detention time, θ_h , hrs	3-8	12-36
MLSS, mg/l	2000-3500	3000-5000
MLVSS/MLSS ratio	0.7-0.85	0.5-0.7
f_B , g BOD ₅ /g MLVSS	0.55-0.70	0.40-0.65
Biomass generated, kg VSS/kg BOD ₅ applied	0.45-0.55	0.3-0.35
Sludge production, kg SS/kg BOD ₅ applied	0.9 (0.7-1.2)	0.65 (0.5-0.7)
with primary settling	1.15 (1.0-1.3)	0.85 (0.7-1.0)
without primary settling		
Oxygen requirement, kg O ₂ /kg BOD ₅ applied	0.6 - 1.2	1.2 - 1.4

4.3.6 Secondary clarifier

It was reported that one of the most important unit determining the capacity of a treatment plant is secondary clarifier. The activated sludge system consists of two processes; the aeration tank and the secondary clarifier. They are said to be inseparable and closely linked to the other in terms of their performance.

The purpose of secondary clarifier is to remove solids from the mixed liquor in the activated sludge system. The critical point in an activated sludge treatment system is solid removal and in order to produce stable and well-clarified effluent with low BOD and suspended solids, solid separation is said to be the last step. It was noted that the basic design factors consist of settling characteristics of sludge, tank types and dimensions, detention period, flow distribution, influent structure, weir loading rates and placement, scum removal and surface and solids-loading rates. The flow should be equalized between parallel tanks of the same dimensions. The flow should be distributed proportionally to surface area when the tanks capacities are not equal.

It was mentioned that a secondary clarifier should serve two functions. These functions are the separation of the mixed-liquor suspended solids from the treated wastewater and the thickening of the return sludge. Clarifier depth will influence these functions. It was said that the selection of a depth must be taken into consideration in order to provide the needed volume for these functions indicated above.

The most popular activated sludge tank type was noted as circular and rectangular. Circular tanks have been built with diameters ranging from 3 to 60 m. But the most common application was 10 to 40 m. It was reported that the most important point is not to pass over the tank radius as five times the sidewater depth. Rectangular tanks must be proportioned in order to provide proper distribution of influent flow so that horizontal velocities are not extreme. It was said that the maximum length of rectangular tanks must not exceed 10 to 15 times the depth. Although lengths up to 90 m have been used commonly in large plants. For a well suspended solids separation and a well concentration of return sludge; the liquid depth is said to be an important factor. There is a risk for the large amount of solids because they can escape with the effluent when the design criteria are overcome. Published values for surface and solids loading rates must be used. For this purpose, it is important that the overflow rates should be build on peak flow conditions.

Typical surface and solids loadings rates and liquid water depths used for biological system's design were indicated in Table 4.11. Besides, overflow rate values at average and peak flows are specified as 16-28 and 40-64 m³/m².d. And solids loading at average and at peak flows are determined as 5-8 and 9 kg/m².h (Metcalf & Eddy,1991).

Table 4.11 : Typical design information for secondary clarifiers for the activated-sludge process^a (Metcalf & Eddy,2003).

Type of treatment	Over flow rate		Solids loading		Depth, m
	m ³ /m ² .d		kg/m ² .h		
	Average	Peak	Average	Peak	
Settling following air-activated sludge (excluding extended aeration)	16-28	40-64	4-6	8	3.5-6
Selectors,biological nutrient removal	16-28	40-64	5-8	9	3.5-6
Settling following oxygen-activated sludge	16-28	40-64	5-7	9	3.5-6
Setting following extended aeration	8-16	24-32	1.0-7	7	3.5-6
Setting for phosphorus removal; effluent concentration, mg/L					3.5-5
TotalP = 2	24-32				
TotalP = 1 ^a	16-24				
TotalP = 0.2-0.5 ^b	12-20				

^a Occasional chemical addition required.

^bContinuous chemical addition required for effluent polishing.
Peak is a 2-h sustained peak.

There is a potential for a very little amount of scum to accumulate in the secondary clarifiers even though the secondary clarifier has been well operated. In recent years, stripping of the final tank has become common in removing the scum when the primary clarifier is not used.

A special attention should be taken into consideration for process control in order to maintain high levels of treatment performance with the activated sludge process under a wide range of operating conditions. For process control; maintaining

dissolved oxygen levels in the aeration tanks, regulating the amount of return activated sludge (RAS), and controlling the waste activated sludge (WAS) are specified to be important and also the parameter most commonly used for controlling the activated sludge process is determined as SRT. The mixed-liquor suspended solids (MLSS) concentration may also be used as a control parameter.

It was reported that the return activated sludge is important to maintain the MLSS concentration and to control the sludge blanket level in the secondary clarifier. Moreover, the waste activated sludge flow from the recycle line is frequently selected to maintain the desired SRT. Oxygen uptake rates (OURs) have also been measured as a means of monitoring and controlling the activated sludge process. It was said that routine microscopic observations are important to monitor the microbial characteristics and to detect changes that might negatively impact the sludge settling and the process performance (Metcalf & Eddy, 2003).

In order to gather and to remove the settled solids from the secondary clarifiers; the secondary clarifiers used in activated sludge and fixed film processes contain collector mechanisms. It was clarified that these collector mechanisms use small horsepower motors, typically in the range of 0.5 to 1.5 kW. Higher horsepower motors in the range of 3 to 5kW may slightly be used in large, rectangular secondary clarifiers using cable and drag type collection mechanisms (WEF, 2009).

4.4 Process design of sludge removal processes

It was explained that the management of sludge received from municipal wastewaters is an important part of the wastewater management plan. The development of related regulations addressed to maximize the use and utilization of waste material and energy content and to minimize the impact of waste on the environment are required for the correct management of sludge and other biodegradable wastes.

Organic and inorganic solids are widely consisted of substances having offensive odor potential and waste sludge is composed of those organic and inorganic solids. It was noted that the primary sludge consists of solids present in the raw wastewater, on the other hand secondary sludge consists of chemical or biological solids produced during the treatment processes.

For different treatment processes, typical physical characteristics of sludge were given in Table 4.12 (Qasim, 1999).

Table 4.12 : Physical characteristics of Sludge from Municipal WWTP (Qasim,1999).

Source of sludge	Quality (g/m ³)	Solids (%)	Specific Gravity of Solids	Specific Gravity of Sludge	Description
Primary	105-165	4-8	1,4	1,02	Gray and slimy, extremely offensive odor. VSS is 60-70%. Readily digested in aerobic or anaerobic digesters.
Activated sludge (waste sludge)	70-100	0,8-2 ² 0,2-0,6 ³	1,25	1,005	Biological solids, flocculating, brownish appearance; when fresh has earthy smell. Turns dark and septic rapidly and has disagreeable odor of purification. VSS is 70-80 %. Readily digested in aerobic and anaerobic digester.
Trickling filter (waste sludge)	50-90	2-4	1,45	1,025	Brownish and flocculating. Earthy smell when fresh; undergoes decomposition and turns offensive. VSS 60-75%. Readily digested.
Chemical sludge ¹					
Iron salts	200-250	0,5-3	0,5-3,0	1,04	Grayish brown, slimy and gelatinous. Undergoes decomposition but at a slower rate.
Low lime	240-300	2-8	2-8	1,04	
High lime	500-1000	4-15	4-15	1,05	

¹ chemical addition in primary sedimentation basin for P removal

² from clarifier

³ from aeration basin

VSS of primary sludge, activated sludge and tricking filter are given above as 60-70%, 70-80 % and 60-75% respectively.

Typical solids concentrations and capture values for various solids processing methods are given in Table 4.13.

Table 4.13 : Typical solids concentrations and capture values for various solids processing methods (Metcalf & Eddy, 2003).

Operation	Solids concentration, %		Solids capture, %	
	Range	Typical	Range	Typical
Gravity thickeners:				
primary sludge only	4-10	6	85-92	90
primary and waste-activated	2-6	4	80-90	85
Flotation thickeners:				
with chemicals	4-6	5	90-98	95
without chemicals	3-5	4	80-95	90
Centrifuge thickeners:				
with chemicals	4-8	5	90-98	95
without chemicals	3-6	4	80-90	85
Belt-filter press:				
with chemicals	15-30	22	85-98	93
Filter press:				
with chemicals	20-50	36	90-98	95
Centrifuge dewatering				
with chemicals	10-35	22	85-98	92
without chemicals	10-30	18	55-90	80

In order to ensure energy reduction, it is important to improve the sludge characteristics by sludge disintegration. In most of the full-scale sludge disintegration systems, only 20-70% of the flowrate is treated. It is not possible to choose whether to treat the whole flowrate or partial flowrate in advance; it depends on the technology and equipment applications. Another widely used application is the treatment of the return sludge. Sludge disintegration reduces the amount of sludge, increases dewatering and increases biogas production. Sludge digestion may occur faster and better in the digesters (Lunning et al., 2007).

As a result of this short retention time or this short retention time with better decomposition, organic materials can be completed. Based on literature data, there is 25-35% increase in the degradation of organic matter. Dewaterability is increased after disintegration due to the particule size distribution and small amount of organic matter. The dry weight of sludge cake may increase by approximately 10%.

Disintegration prevents bulking and foaming of the sludge by eliminating the filamentous organisms in the sludge. Therefore, the effective volume in the digester increases. As a result of the increase in the degradation of biomass, more biogas is produced. There is an increase of 25-35% in the production of biogas (Lunning et al., 2007).

4.4.1 Sludge thickening and dewatering

Handling and disposal of sludge is complex and problematic because it contains large volumes of water. It was clarified that the common sludge management processes are thickening, stabilization, dewatering, and disposal. . A quite few unit operations and processes are utilized at different stages of sludge processing and disposal. Many of these unit operations and unit processes are given Table 4.14 (Qasim, 1999). Comparison of alternative methods for dewatering various types of sludge and biosolids are given Table B.3.

Table 4.14 : lternative Unit Operations and Processes for Sludge Processing and Disposal (Qasim, 1999)

Operation	Process	Operation	Process
Thickening	Gravity	Dewatering	Vacuum filter
	Flotation		Filter pres
	Centrifugation		Horizontal belt filter
Stabilization	Chlorine oxidation	Disposal	Centrifugation
	Lime stabilization		Drying beds
	Heat treatment		Land application
	Aerobic digestion		Composting
	Anaerobic digestion		Land filling
Conditioning	Chemical		Incineration
	Elutriation		Recalcination
	Heat treatment		

By means of sludge dewatering (dry matter content 20-25%), savings can be provided in terms of transport costs. Similarly, the calorific value of sludge has been increased as a result of decreased sludge water content. Specific site conditions have to be evaluated in order to determine economical advantages of dewatering. Also, transport and sludge disposal costs have to be taken into account. Sludge dewatering is feasible for large scale treatment plants in terms of economical benefits. Sludges coming from small scale treatment plants may be transferred to dewatering units of central treatment plants (Geilvoet at al.,2010).

Thickening of sludge is a process used to concentrate solids and reduce the water volume in sludge and when the sludge is thickened, it requires less tank capacity and chemical dosage for stabilization and smaller piping and pumping equipment for transport. Common methods used for sludge thickening in large plants are ranged as gravity, flotation, centrifuge, gravity belt, and rotary drum (Qasim, 1999).

When considering all thickening processes, it can be said that each thickening process has its own advantages and disadvantages. Special consideration is required for the selection of a particular application. Usually, DAF for thickening is suitable in waste activated sludge, chemical sludges, or solids that settle slowly, while gravity thickening is simple, economical to operate, and works well with primary and combined sludges. Besides, centrifugal thickening is effective; however, it requires high power and maintenance costs. It was also noted that gravity belt and rotating drum filters are convenient for waste activated and chemical sludges. For medium-sized secondary treatment plants, the gravity thickening of combined sludges is normally said to be most cost-effective. Depending on design performance and economic criteria, evaluation of various sludge thickening processes and their comparative performance were listed in Table 4.15.

Table 4.15 : Comparative evaluation of different sludge thickening processes (Qasim, 1999).

Evaluation Criteria	Gravity	DAF	Centrifugation	Gravity Belt	Rotating Drum
Space requirement	High	Medium	Low	Medium	Medium
Operation and maintenance	Simple	Medium	High	Medium	Medium
Applicable to	Primary and combined	WAS	WAS	WAS	WAS
Conditioning chemicals	None	High	High	Medium	Medium
Power requirement	Low	High	High	Medium	Medium
Capital cost	Low	High	High	Medium	Medium
Operation cost	Low	High	High	Medium	Medium
Thickened sludge Solids concentration	Medium	Low	High	Medium to high	Medium to high
Building corrosion problem if enclosed	High	Medium	None	Medium	Medium
Odor problem	Serious	Moderate	Low	Moderate	Moderate

Removal of moisture is used in sludge dewatering to transport the sludge cake by truck and to apply the sludge cake as biosolids over farm lands, composted, or disposed of by landfilling or incineration. It was noted that the solid particles in municipal sludge are extremely fine, hydrated, and carry electrostatic charges so that these properties of sludge solids make dewatering quite difficult. Thus, sludge conditioning is needed to destabilize the suspension for the use of proper sludge-dewatering devices effectively (Qasim, 1999).

A number of sludge dewatering units are based on characteristics of the sludge to be dewatered and also they are dependent on available space, moisture content requirements of the sludge cake for the final disposal. Drying beds and drying lagoons which are natural dewatering systems are said to be preferred when land is available and the sludge quantity is small. However, the mechanical dewatering systems including vacuum filter, centrifuge, filter press, and belt filter press are usually selected if there is no land available. Below, a comparative determination of these processes was presented in Table 4.16(Qasim, 1999).

Table 4.16 : Comparative evaluation of various sludge dewatering processes.

Dewatering Method	Land Area Needed	Capital Costs	O&M Costs	Energy Requirement	Chemical Needs	Others
Sludge-drying bed	Large	Low	Low	Low	Low	Odors, stabilized sludge applied, influenced by climatic conditions. removal of cake labor-intensive
Sludge lagoons	Very large	Very low	Low	Low	None	Odors, stabilized sludge applied, potential for groundwater pollution influenced by climatic conditions, mosquito problem, unsightly.
Vacuum filter	Large	High	Medium	High	Medium	Vacuum pumps may be noisy
Centrifuge, solid-bowl	Low	Medium	High	High	High	Minimum odors, easy to install, clean appearance
Centrifuge, basket	Low	Medium	High	High	Medium	Minimum odor, same machine can be used for thickening and dewatering, clean appearance
Filter press (recessed plate filter)	Large	High	High	Medium	High	Batch operation
Belt filter press	Medium	Medium	Medium	Low	Medium	Hydraulically limited in throughput, short media life

Sludge dewatering systems can be ranged from simple devices such as natural evaporation and percolation from sludge lagoons or drying beds to very complex mechanical processes such as sludge conditioning, followed by centrifugation, vacuum filtration, filter presses, and belt filters. Sludge-drying beds or lagoons are frequently selected for smaller plants. In the past, vacuum filters were used, but their use has declined because of the availability of other improved methods. In many applications, centrifugal thickening and dewatering have been applied with varying degrees of success. The belt filter press offering continuous operation, has a low energy requirement, and it has relatively lower capital and operation costs (Qasim, 1999).

4.4.1.1 Gravity thickening

Being one of the most common methods used; gravity thickening has been accomplished in a tank similar in design to a conventional sedimentation tank. Generally, a circular tank is used for gravity thickening, and dilute sludge is usually fed to a center feed well. The sludge feeded to the center, is allowed to settle down and thicken. After, the thickened sludge is withdrawn from the tank bottom having a conical shape. Also, the supernatant is drawn off and returned to either the primary settling tank, the influent of the treatment plant, or a return flow treatment process. It was noted that the thickened sludge is pumped to the digesters or dewatering equipment when required; therefore, a storage space must be provided for the sludge (Metcalf & Eddy, 2003).

Two important advantages of gravity thickening are listed as the sludge concentration and the equalization and the storage for sludge to enhance downstream operation. It was reported that gravity thickeners are commonly used to concentrate solids in sludges from the primary clarifier, trickling filter, and activated sludge; the primary and lime sludges thicken easily. Biological sludge complicates gravity settling due to the slow settling characteristic and the resistance to compaction. Thickening performance may vary from two to five times the concentration of solids in the incoming sludge (Qasim, 1999).

Gravity and mechanical thickening units are used for primary and secondary sludges having a dry solids ratio of 0.8-1.0% to 3.5 – 6.0% (Geilvoet et al., 2010). The sludge volume ratio SVR is generally taken between 0.5 – 2 days. This value is indicated as

the volume of sludge blanket held in the thickener divided by the volume of the thickened sludge removed per day. Usually, a higher SVR provides more thicker sludge. Yet, excessive retention times will lead to gasification and buoying of the solids (Qasim, 1999).

The design criteria for gravity thickeners were listed as below:

1. Minimum surface area is based on hydraulic and solids loading
2. Depth of thickener
3. Slope of floor

Design criteria for gravity thickening were listed in Table 4.17.

Table 4.17 : Design criteria for gravity thickening (Qasim,1999).

Type of Sludge	Influent Solids Concentration (percent)	Thickened Solids Concentration (percent)	Solids Loading (kg/m ² .d)	Solids Capture (percent)	Overflow, TSS (mg/L)
Primary	1,0- 7,0	5,0 - 10,0	90 - 144	85 - 98	300 - 1000
Trickling filter	1,0 - 4,0	2,0 - 6,0	35 - 50	80 - 92	200 - 1000
Waste activated sludge	0,2 - 1,5	2,0 - 4,0	10 - 35	60 - 85	200 -1000
Combined primary and waste activated sludge	0,5 - 2,0	4,0 - 6,0	25 - 80	85 - 92	300 - 800

The design criteria for the sludge thickeners for a side water depth is generally said to be 3-4 m and a detention period of 24 h. The hydraulic loading rates are listed as 16-32 m³/m².d for primary sludge, 4-8 m³/m². d for waste activated sludge, and 6- 12 m³/m².d for combined sludge. It was reported that the thickener overflow rate is important due to its relation to solids loading rate. Also, a high hydraulic loading can cause excessive solid carryover however a low loading may create septic conditions. In order to achieve acceptable hydraulic loading; secondary effluent is mixed with the sludge fed into the thickener. The gravity thickeners are frequently said to be a significant odor source and they are commonly covered and provided with odor control measures (Qasim, 1999).

During the gravitational sludge thickening, suspended solids concentration may increase by means of polymer dosage. Anaerobic retention time and produced biogas increase. Suspended solids concentration by polymer dosage is less than suspended solids concentration in mechanical sludge thickening. On the other hand, investment cost for dosing equipment is less (Geilvoet et al., 2010).

4.4.1.2 Dissolved Air Flotation (DAF)

It was noted that the flotation thickening unit can be used efficiently for waste sludges coming from suspended growth biological treatment processes as activated sludge process or suspended growth nitrification process. Other sludges such as primary sludge, trickling filter humus, aerobically digested sludge, and sludges coming from chemical treatment containing metal salts can be thickened by flotation thickening method (Metcalf & Eddy 2003).

During dissolved air flotation, air is introduced into a solution at an elevated pressure. In case of the solution is depressurized, the dissolved air will be released as finely divided bubbles carrying the sludge to the top, then it will be removed. In locations where freezing or odor is a problem, flotation thickeners should be usually enclosed in a building (Metcalf & Eddy 2003).

It was said that the main advantage of flotation over gravity thickening is that particles settling down slowly under gravity will be removed rapidly and completely in a DAF system. Besides, maximum concentration of solids can reach 4-5 percent.

It was determined that dissolved air flotation process is applied by either pressurizing all or only a small part of the incoming sludge and also by pressurizing the recycled flow from the flotation thickener being greatly preferred due to the eliminating need. Otherwise high-pressure sludge pumps are subject to maintenance problems (Qasim, 1999).

4.4.1.3 Centrifugal dewatering

It was determined that centrifugation is a process in which solids are thickened or dewatered from the sludge under the influence of a centrifugal field usually the force of gravity. It was indicated that there exist three basic types of centrifuges for sludge thickening such as basket centrifuge operated on a batch basis, disc nozzle and solid bowl centrifuges continuously operated.

It was explained that the disc-nozzle centrifuges require a wide and careful prescreening process and also a grit removal from the sludge. However, solid bowl type of centrifuges have wide application in sludge-thickening (Qasim, 1999).

The solid-bowl centrifuge is said to be the basic type of centrifuge used for sludge thickening. The solid-bowl type of centrifuge is composed of a long bowl. This bowl

was said to be fixed horizontally and bevelled at one end. It was expressed that sludge is introduced into the unit continuously, and the solids concentrate on the periphery which contains an internal helical scroll, spinning at a slightly different (Metcalf & Eddy 2003).

High power and high maintenance costs are said to be important requirements for centrifugal thickening of sludge. A skilled operation is necessary, otherwise sludge will be difficult to thicken by other methods (Qasim, 1999).

It was reported that centrifuges are compact, entirely enclosed (which may reduce odors), needing small space, and can handle sludges that might otherwise plug filter cloth. On the other hand; complexity of maintenance, abrasion problems, and concentrate high in suspended solids are listed to be important disadvantages of the centrifuge system (Qasim, 1999).

It should be noted that the sludge cake from the centrifuge contains 20-35 percent solids by a capturing efficiency of 85-90 percent. The favorable polymer dosage for the conditioning of sludge prior to centrifuge is listed to be 0.1-0.7 percent of dry solids in the feed. The important design parameters for solid bowl, decanter-type centrifuges were given in Table 4.18.

Table 4.18 : Design parameters for solid bowl, decanter type centrifuges (Qasim, 1999).

Parameter	Range of Values
Bowl diameter	36-152 cm
Capacity	38-600 l/min
Gravitational force	1400-2300 times gravity
Feed solids, waste activated sludge	0.3-2.0 percent
Thickened solids	5-8 percent
Solids recovery	85-95 percent
Polymer usage	0-3 g/kg dry solids

4.4.1.4 Vacum filter

Most widely used filters for dewatering of both raw and digested sludges are rotary vacuum filters. It was noted that vacuum filters constitutes a cylindrical drum

covered with cloth of natural or synthetic fabric. The drum is said to be partly submerged in a vat of sludge and it rotates slowly. The sludge is usually drawn to the filter medium by an internal vacuum maintained inside the drum, and water is withdrawn from the sludge (Qasim, 1999).

4.4.1.5 Filter press

Under high pressure, the water from the sludge is forced for dewatering purpose and this process is achieved by a filter press. Advantages of the filter press are; high concentrations of cake solids achieved, good filtrate clarity, and high solids capture. Disadvantages of the filter press are; mechanical complexity, high chemical costs, high labor costs and limitations on filter cloth life. There exist various type of filter presses to dewater the sludge. Most commonly used filter press types are the fixed volume and variable volume recessed plate filter presses (Metcalf & Eddy,2003). It was specified that the fixed volume, recessed plate filter press constitutes a series of rectangular plates supported face to face in a vertical position on a frame with a fixed and movable head. In addition to this, it was explained that a filter cloth is hung or fitted over each plate. During the filtration process, the plates are said to be held together with sufficient force for sealing them to withstand the pressure applied. Moreover, hydraulic rams or powered screws are used to hold the plates together (Metcalf & Eddy,2003). The solid particles are retained when the water passes through the filter cloth and a cake is formed on the surface of the cloth. It was indicated that the desired cake solids level may reach up to 40 percent solids in plate and frame presses. The time for the press to be filled with the conditioned sludge was reported to be 10-20 min (Qasim, 1999).

4.4.1.6 Belt filter press

It was mentioned that belt filter presses, by using the principles of chemical conditioning, gravity drainage, and mechanically applied pressure, are continuous feed dewatering devices. In the early 1970s, the belt filter press was introduced in the United States and it has become one of the predominant sludge dewatering devices. Belt filter press is an equipment proved to be effective for almost all types of municipal wastewater sludge and biosolids (Metcalf & Eddy,2003).

A belt filter press has 4 basic operational stages containing single or double moving belts to dewater sludge continuously; polymer conditioning zone, gravity drainage

zone for excess water, low pressure zone, and high-pressure zone. Handling of a drier cake, low energy requirement, and continuous operation are main advantages of a belt filter press to be considered. Short media life and a sensitive filtration rate to incoming sludge are main disadvantages for a belt filter press (Qasim, 1999).

There exist variables affecting the performance of a belt-filter such as sludge characteristics, method and type of chemical conditioning pressures developed, machine configuration (including gravity drainage), belt porosity, belt speed and belt width. There is sensitivity from the point of a belt filter press for variations in sludge characteristics, resulting improper conditioning and reduced dewatering efficiency. In the case of variations in sludge characteristic; sludge blending facilities should be included in the system design (Metcalf & Eddy,2003).

The design, operation and performance data for a belt filter press were presented in Table 4.19 and Table 4.20 respectively.

Table 4.19 : Design and operation data of belt filter press (Qasim, 1999).

Condition	Data
Solids in feed sludge	3-10 percent dry wt.
Solids in cake	20-40 percent dry wt.
Polymer for conditioning	0.2-0.5 percent of dry solids
Total suspended solids in filtrate	100-1000 mg/L
Solids capture	90-95 percent
Sludge-dewatering rate based on belt width	200-700 kg/m belt width-h
Hydraulic throughput based on belt width	2-8 L/m belt width-s

Information below should be primarily considered before the design of a sludge-conditioning and dewatering facility:

1. Determination of sludge characteristics including average quantity of sludge dewatered per day, solids concentration, and whether the sludge is raw or stabilized

2. Selection of an ultimate sludge disposal method and specification of the moisture content of the sludge cake
3. Evaluation of site conditions
4. Determination of the operational characteristics of the equipment including energy requirements, specialized maintenance requirements, performance reliability, and simplicity of operation.
5. Determine type of chemicals and the dosages needed for proper dewatering.
6. Develop design data for the selected equipment for dewatering. This includes operational period, solids-loading rate, or dewatering rate.

Table 4.20 : Typical dewatering performance data for belt filterpresses for various types of sludge and biosolids (Metcalf & Eddy,2003).

Type of sludge	Dry feed solids, %	Loading per meter of belt length		Dry poly mer ^a , g/kg dry solids	Cake solids, %	
		L/s	kg/h		Typical	Range
Raw primary	3-7	1.8-3.2	360-550	1-4	28	26-32
Waste-activated (WAS)	1-4	0.7-2.5	45-180	3-10	15	12-20
Primary + WAS (50:50) ^b	3-6	1.3-3.2	180-320	2-8	23	20-28
Primary + WAS (40:60) ^b	3-6	1.3-3.2	180-320	2-10	20	18-25
Primary + trickling filter	3-6	1.3-3.2	180-320	2-8	25	23-30
Anaerobically digested:						
Primary	3-7	1.3-3.2	360-550	2-5	28	24-30
WAS	3-4	0.7-2.5	45-135	4-10	15	12-20
Primary + WAS	3-6	1.3-3.2	180-320	3-8	22	20-25
Aerobically digested:						
Primary + WAS, unthickened	1-3	0.7-3.2	135-225	2-8	16	12-20
Primary + WAS (50:50) ^c , thickened	4-8	0.7-3.2	135-225	2-8	18	12-25
Oxygen-activated WAS	1-3	0.7-2.5	90-180	4-10	18	15-23

^aPolymer needs based on high-molecular-weight polymer (100 percent strength, dry basis).

^bRatio is based on dry solids or the primary and WAS.

During sludge dewatering process; after thickening unit or in pursuit of digestion, belt filter application instead of centrifuge will affect energy consumption in a positive way. Obtained dry matter content should be taken into account. Energy consumption has been affected positively when sludge dewatering is applied directly

by using a belt press with cascade alignment instead of single – stage centrifuge or instead of a belt press with prolonged dewatering band (Geilvoet et al., 2010).

4.4.1.7 Sludge drying beds

It was reported that the oldest method of sludge dewatering is sludge drying bed and is still used in small to medium sized plants to dewater digested sludge. Besides, it is relatively inexpensive providing dry sludge cake.

Sludge drying beds are used to dewater settled sludge and digested sludge right after the extended aeration activated sludge treatment process without prethickening. It was specified that after drying process, the solids are removed and then disposed in a landfill. Later, it is used as a soil conditioner. The principal advantage of a drying bed is that the cost is low. Attention required is rare and high solids content is available in the dried product. The principal disadvantages of a sludge drying bed are large space requirement, effect of climatic change on drying characteristics, labor intensive sludge removal, insects and potential odors. There exist five types of drying beds such as; conventional sand, paved, artificial media, vacuum assisted and solar (Metcalf & Eddy, 2003).

Usually, conventional sand drying beds are used for small and medium sized communities. In cities with a population over 20,000 habitants, alternative means of sludge dewatering should be taken into consideration. For larger municipalities; the initial cost, the cost of sludge removal, replacing of sand and the large area requirement frequently prevent the use of sand drying beds. Typical data for various types of biosolids were shown in Table 4.21.

Table 4.21 : Typical area requirements for open sludge drying beds for various types of biosolids (Metcalf & Eddy, 2003).

Type of biosolids	Area ^a m ² /person	Sludge loading rate kg dry solids/m ² .yr
Primary digested	0.1	120-150
Primary and trickling-filter humus digested	0.12-0.16	90-120
primary and waste-activated digested	0.16-0.23	60-100
Primary and chemically precipitated digested	0.19-0.23	100-160

^aCorresponding area requirements for covered beds vary from about 70 to 75 percent of those for the open beds.

4.4.1.8 Sludge lagoons

Owing to the periodical removal of the sludge and the refill of the lagoon, sludge lagoons are said to be an economical method similar to drying beds. Sludge must be stabilized to reduce odor problems. It was noted that sludge-drying lagoons consist of shallow earthen basins and the supernatant is decanted from the surface and returned to the plant, while the sludge liquid evaporates. Sludge-drying time depends on the climatic conditions and the depth of sludge application. Usually, 20-40 percent solids is reached in 3-6 months in the sludge cake and solids capture in drying lagoons is 90-100 percent. Sludge cake is removed by mechanical equipment. Solids-loading rates for drying lagoons are suggested to be $37 \text{ kg/m}^3\cdot\text{yr}$ of lagoon capacity (Qasim, 1999).

4.4.1.9 Sludge conditioning

In order to enhance water removal, sludge conditioning consists of chemical and/or physical treatment of the sludge. Moreover, some conditioning processes also disinfect sludge, control odors, alter the solids nature, provide limited solids destruction, and improve solids recovery (Qasim, 1999).

Conditioning of sludge and biosolids by using chemicals to dewater the sludge is said to be economical due to the increased yields and greater flexibility obtained. Depending on the nature of the solids to be treated, chemical conditioning can reduce the 90 to 99 percent incoming moisture content to 65 to 85 percent. Chemical conditioning leads to coagulation of the solids and release of the absorbed water. Chemical conditioning which is attained by inorganic or organic chemicals is said to be related principally to mechanical sludge-dewatering systems such as centrifugation, belt-filter presses, and pressurefilter presses. It was specified that chemicals used in conditioning include ferric chloride, lime, alum, and organic polymers (Metcalf & Eddy 2003).

Elutriation and thermal conditioning are the most important physical sludge conditioning methods. It was mentioned that other methods such as freezing, ultrasonic vibration, solvent extraction, and irradiation are less commonly used methods (Qasim, 1999).

4.4.2 Anaerobic digestion

In the absence of oxygen, anaerobic digestion is a microbial process converting the organic matters into the end products such as carbon dioxide and methane. Anaerobic digestion is a widely used process and a renewable energy source whereby to the biogas production rich in methane and carbon dioxide helping replace fossil fuels. On the other hand, there exist a nutrient rich digestate which can be used as fertiliser after.

Anaerobic digestion is a process used to remove high strength wastes such as sludges containing high levels of suspended solids. In municipal wastewater treatment plants, primary sedimentation tank and the secondary clarifiers are the main sources of sludge. There also can be an additional sludge quantity coming from chemical precipitation, screening and grinder, and filtration devices if the plant constitutes these processes.

The use of anaerobic treatment processes can be determined by explaining and considering its advantages and its disadvantages. The main advantages are listed as energy considerations, lower biomass yield, fewer nutrients required, and higher volumetric loadings. (Metcalf & Eddy, 2003). Advantages of anaerobic processes lead to the development of new aerobic process configurations capable of treating medium, low strength soluble and colloidal wastes as municipal wastewaters. There exist also potential disadvantages for anaerobic processes such as operational considerations, the need for alkalinity addition, and the need for further treatment. The principal advantages and disadvantages of anaerobic treatment were listed in Table 4.22.

Table 4.22 : Advantages and disadvantages of anaerobic processes compared to aerobic processes (Metcalf & Eddy 2003).

Advantages	Disadvantages
Less energy required	Longer start up time to develop necessary biomass inventory
Less biological sludge production	May require alkalinity addition
Fewer nutrients required	May require further treatment with an aerobic treatment process to meet discharge requirements
Methane production, a potential energy source	Biological nitrogen and phosphorus removal is not possible
Smaller reactor volume required	Much more sensitive to the adverse effect of lower temperatures on reaction rates
Elimination of off-gas air pollution	May be more susceptible to upsets due to toxic substances
Rapid response to substrate addition after long periods without feeding	Potential for production of odors and corrosive gases
Less energy required	
Less biological sludge production	
Fewer nutrients required	
Methane production, a potential energy source	
Smaller reactor volume required	
Elimination of off-gas air pollution	
Rapid response to substrate addition after long periods without feeding	

Anaerobic digestion having complex structure and successive, parallel reactions consists of three stages; hydrolysis, acidogenesis, acetogenesis and methanogenesis. These three stages have been explained thoroughly. It was noted that in hydrolysis phase, insoluble organic molecules such as proteins, cellulose, lipids and complex organics are converted into glucose, amino acids, and fatty acids. In acidogenesis phase, solubilized organic matter from hydrolysis are converted into organic acids, such as propionic, butyric, and other acids, by facultative organisms. In acetogenesis phase, alcohols and volatile acids are oxidized into acetic acid, carbon dioxide and hydrogen and finally in methane phase, volatile organic acids are converted into methane and carbon dioxide (Switzenbaum, 1991).

In anaerobic digestion, microorganisms have significant properties and also have significant structure and functions of soluble and insoluble organics. It was reported

that researches have shown the major stages of anaerobic digestion and the metabolic characteristics of bacteria. The main groups for anaerobic bacteria and their functions in anaerobic digestion were specified in Table 4.23.

Table 4.23 : The main groups of anaerobic bacteria.

Type of bacteria	Function On Anaerobic Digestion
Acidogens	Production of volatile fatty acids
Acetogens	Production of Acetate and Hydrogen
Methanogens	Production of CH ₄ by Carbon Dioxide Reduction
	Production of CH ₄ by Acetate Decarboxylation

One of the major environmental parameters is determined as temperature in anaerobic digestion. During anaerobic process, metabolic activities of microorganisms mainly depend on temperature. Microorganisms can work efficiently under specific temperature levels such as; Thermophilic at 50°C - 65°C, Mesophilic at 20°C - 45°C and Psychrophilic below 20°C.

The anaerobic process is said to be mainly controlled by the methane-forming bacteria being very sensitive to pH, substrate composition and temperature. The bacteria type being highly active in mesophilic (27 - 43°C) and in thermophilic (45 - 65°C) conditions have been indicated as methane bacteria. The most commonly used operational temperature condition of anaerobic digesters is 35 - 40°C (Qasim, 1999).

It was noted that anaerobic treatment technologies are world saving processes with high removal efficiency even when there exist high organic loadings, moreover anaerobic treatment processes have worldwide application. Because of the sensitivity of the anaerobic treatment process to environmental conditions; microorganisms responsible for the biodegradation and the effectiveness of the anaerobic treatment can be easily affected in terms of the metabolic activity. It was mentioned that accumulation of volatile fatty acids, due to easy adaptation of acidogenic microorganisms against changed conditions thanks to their faster growth rate, are the most common and limiting problem of anaerobic digestion.

By providing optimum environmental conditions and the specific requirements of the microorganisms at the same time, desired conditions may be reached in anaerobic digestion. Most important environmental factors are said to be temperature, pH, alkalinity, mixing, inhibition, toxicity and nutrient requirements.

When compared to aerobic processes, application of anaerobic digesters has been increasing rapidly. For the selection of proper reactor to provide desired level of treatment efficiency; the concentration and the composition of biomass is said to be a significant parameter. It was indicated that the two important criteria when selecting the proper reactor type are the effectiveness of the reactor to decrease the pollution parameters of wastewater and required effluent characteristics.

Anaerobic processes, according to the growth rate of the microorganisms, may be classified into Suspended Growth Reactors and Attached Growth Reactors. Common names of anaerobic digestion for ‘Suspended Growth Reactors’ are listed to be standart rate, single-stage high rate, single stage two stage, anaerobic contact process and upflow anerobic sludge blanket (UASB).

Common names of anaerobic digesters for ‘Attached Growth Reactors’ are anaerobic filter process and expanded bed reactor types. Completely mixed and contact type anaerobic digesters have worldwide applications, however anaerobic filters, fluidized or expended beds upflow anaerobic sludge blankets (UASB) are the most commonly used reactor types.

It was reported that the type of wastewater and its characteristics have been important in the evaluation and design of anaerobic processes. The solids retention time is said to be a fundamental design and operational parameter for all anaerobic processes. For an effective treatment performance, SRT values greater than 20 days and 30 °C temperature are required for anaerobic processes. Typical organic loading rates for anaerobic suspended growth processes were given in Table 4.24 (Metcalf & Eddy, 2003).

Table 4.24 : Typical organic loading rates for anaerobic suspended growth processes at 30 °C (Metcalf & Eddy, 2003).

Process	Volumetric organic loading kg COD/m ³ .d	Hydraulic retention time,d
Complete mix	1.0-5.0	15-30
Anaerobic contact	1.0-8.0	0.5-5
Anaerobic sequence batch reactor	1.2-2.4	0.25-0.50

Determination of the required volume based on a loading factor is used in sizing of digesters. Typical design criteria for sizing mesophilic high rate anaerobic digesters were indicated in Table 4.25 and 4.26.

Table 4.25 : Typical design criteria for sizing mesophilic high rate complete mix anaerobic sludge digesters (Metcalf & Eddy, 2003).

Parameter	Units	Value
Voluma criteria:		
Primary sludge	m ³ /capita	0.03-0.06
Primary sludge+trickling-filter humus sludge	m ³ /capita	0.07-0.09
Primary sludge + activated sludge	m ³ /capita	0.07-0.11
Solids loading rate	kg VSS/m ³ .d	1.6-4.8
Solids retention time	D	15-20

Table 4.26 : Anaerobic digester design criteria for biogas production.

	(Öztürk,2007)	(Metcalf,2003)	Kullanılan
Yield (grVSS/grCOD)	0.04-0.1	0.024-0.06	0.05
b (1/d)	0.02-0.04	0.02-0.04	0.03
Solids retention time (d)	Low rate30-60	15-20	
	High rate 15-20		15
VSS (%)	Primary sludge60-80		70
	Activated sludge 59-88		85

5. LITERATURE REVIEW: ENERGY CONCEPT OF MUNICIPAL WASTEWATER TREATMENT PLANT

5.1 Introduction

In this chapter, worldwide applications are provided and the amount of energy consumed in municipal treatment plants with different populations are compared with the amount of energy obtained from anaerobic sludge digesters.

The amount of energy used in municipal treatment plants are provided based on both population and treatment units.

5.2 Gas utilisation facilities

In order to evaluate the recovery options of treatment sludge, it is necessary to examine the components of the sludge. Sludge is a mixture of which the major part of the non-toxic organic carbon components (make up 60% of the dry base) are of biological origin and more than 95% is water. Its other components are nitrogen and phosphorus-including compounds, toxic inorganic and organic pollutants, pathogens and other microbiological pollutants, and inorganic compounds. There are many alternatives to produce energy from the organic components in the sludge. These alternatives can basically be divided into nine groups:

- (1) Anaerobic respiration in treatment sludge
- (2) Biofuel production from treatment sludge
- (3) Direct electricity production with microbial fuel cells in treatment sludge
- (4) Energy production through incineration of treatment sludge
- (5) Incineration of treatment sludge in coal power plants
- (6) Pyrolysis and gasification of treatment sludge
- (7) Use as fuel and raw material for the production of construction material
- (8) Supercritical wet oxidation of treatment sludge
- (9) Hydrothermal treatment of treatment sludge (Rulkens,2007)

The content of anaerobic digestion gas is about 65 to 70 percent CH_4 by volume, 25 to 30 percent CO_2 and small amounts of N_2 , H_2 , H_2S , water vapor and other gases. It was indicated that the digester gas has a specific gravity of approximately 0.86 relative to air. The production of digestion gas has been accepted as the best feature to indicate the efficiency and the progress of digestion tank. Just because the digestion gas may be used as fuel, it was noted that the design engineer should be familiar with its production, collection, and use (Metcalf & Eddy, 2003).

Total gas production can be generally estimated from decrease in the percentage of volatile solids. Typical values changes between 0.75 to 1.12 m^3/kg of volatile solids destroyed (Metcalf & Eddy, 2003). Gas production can differentiate over a wide range, depending on the volatile solids content of the sludge feed and the biological activity in the digester. Excessive gas production rates can occur during startup and may cause foaming and escape of foam and gas from around the edges of floating digester covers. For egg-shaped and shallow cylindrical digesters, foaming can be a problem by clogging the gas outlet unless foam control is provided. If stable operating conditions have been achieved and the gas production rates indicated above are being maintained, it will be possible to obtain a well-digested sludge. It is possible to estimate gas production roughly on per capita basis. The gas yield is 15 to 22 $\text{m}^3/10^3\text{persons.d}$ in primary plants treating domestic wastewater. In secondary treatment plants, the gas production yield is increased up to about 28 $\text{m}^3/10^3\text{persons.d}$. (Metcalf & Eddy, 2003) Gas production is 35 $\text{m}^3\text{ gas/person.d}$ and heating value is 6.2 kwh/m^3 (Stillwell et al., 2010).

In cylindrical digesters, gas is said to be collected under the cover of the digester. There exist three principal types of covers which are floating, fixed and membrane type covers. Floating covers fit on the surface of the digester. It allows the volume of the digester to change without releasing air to enter the digester. Gas and air must not be allowed to mix, otherwise an explosive mixture may result.

There is risk for explosions to occur in wastewater treatment plants. In order to prevent possible explosions; gas piping and pressure relief valves must include adequate flame traps. The covers may also be used as gas holders for a limited storage of gas. High rate digesters have capacity to produce about two volumes of gas per volume of digester capacity/d. It was noted that floating covers can be used in single stage or in the second stage of two-stage digesters.

At standard temperature and pressure (20 °C and 1 atm), methane gas is expected to have a lower heating value which is named the heat of combustion. It is less than the heat of vaporization of any water vapor present. Because digester gas, with only 65 percent methane, has the lower heating value of approximately 22,400 kJ/m³. In comparison with digester gas, natural gas being a mixture of methane, propane, and butane has a heating value of 37,300 kJ/m³ (Metcalf & Eddy, 2003).

For large plants, digester gas has a possibility to be used as fuel in boilers and internal combustion engines. Then, they are used to pump wastewater, operate blowers, and generate electricity. It was reported that hot water from heating boilers or from engine jackets and exhaust heat boilers may be used for sludge heating and for building heating, or gas-fired sludge-heating boilers may be used for sludge heating and for building heating. It was also said that energy recovery is more efficient if prime movers are designed to run hot because heat rejected at low temperatures (Metcalf & Eddy, 2003).

Cogeneration is also an efficient way to evaluate the digester gas. Cogeneration is usually defined as a system to generate electricity and to produce another form of energy which is generally steam or hot water.

It was specified that digester gas can be used to power an engine-generator to generate electricity and then the jacket water produced from the internal combustion engine may be used in heating of digester or building. If there exist surplus power, it is possible to sell it to local electric utilities.

The content of digester gas was reported to be hydrogen sulfide, nitrogen, particulates and water vapor. There exist a necessity for this gas to be frequently cleaned in dry or wet scrubbers before it is used in internal-combustion engines. Hydrogen sulfide concentrations is excess of approximately 100 ppm by volume. Therefore, it may require the installation of hydrogen sulfide removal equipment (Metcalf & Eddy, 2003).

The heating requirement of digesters is used to raise the incoming sludge temperature to digestion tank temperature, to compensate heat losses through walls, floor and roof of the digester and to make up the losses occurring in the piping between the source of heat and the tank. It was reported that the heating of sludge in digestion

tanks is provided by pumping the sludge and supernatant through external heat exchangers and back to the tank (Metcalf & Eddy, 2003).

5.3 Energy demand for municipal wastewater treatment plants

According to statistical data obtained from different countries, it can be seen that as the number of nutrient removal wastewater plants increase, also the amount of energy used in the treatment plants increase within the rate of use across the country. From collecting of the municipal wastewater to its discharge by ensuring limits of the receiving environment, energy is used in every stage. As a result of the gradual increase of the population, environmental pollution also increases. Therefore together with obtaining discharge standards, it is very important that also energy consumption is reduced.

Energy consumption of wastewater treatment facilities varies according to local conditions, the amount of service provided, the selected treatment type and sludge removal. Importance of local conditions and analyzing of influent load is mentioned in chapter 3 in detail.

The ratio of energy use in wastewater treatment plant to energy consumption in general is 0.7 % in Germany (Mauer, 2009) 0.1-0.3 % in USA (Stillwell, 2010). Looking at the annual energy consumption per capita in municipal treatment, the value is given in the range of 30-50 kwh/PE in Austria (Queensland government, 2005) and 30-60 kwh/PE in Germany, 15-40 kwh/PE of which is made up of biological step. It is analysed that in the evaluation made according to population in Germany, in the settlements with a population of 1000-5000, this value goes up to 150 kwh/PE (Kolish et al., 2009). Annual electric energy consumption in conventional active sludge system is approximately 0.4 kwh/m³ or 30-40 kwh/PE (Mauer, 2009).

Table 5.1 : Municipal WWTP electric energy consumption data (Geilvoet, 2010).

Processes	Mean	Range	Unit
Total plant	27	20-34	kwh / PE-a
Aeration tank	15	10-20	kwh / PE-a
Dewatering	0,13	0,04-0,27	kwh / kg ds
Transport	0,016	0,0028-0,0333	kwh/m ³ km

According to a study that is supported by the German Federal Environment Agency, based on data obtained in connection with population, in settlements with population up to 1000, the ratio of electricity consumption to total energy consumption is 75kwh/PE, where as the same ratio decreases to 32 kwh/PE in settlements with more than 100,000. Both ratios are illustrated graphically in Figure 5.1 (Kolisch et al.,2009).

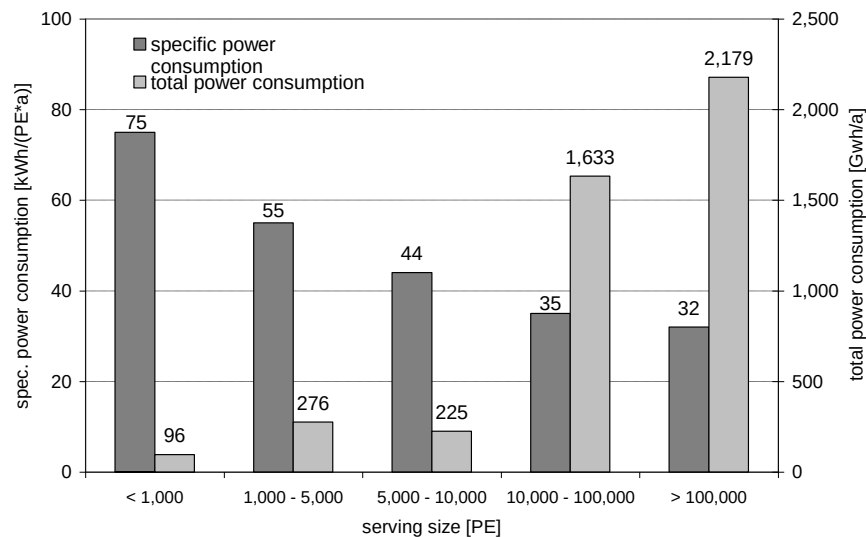


Figure 5.1 : Specific and absolute power consumption of sewage plants in Germany (Kolisch,2009).

This energy requirement expresses largely the electricity required for wastewater treatment plant units. The heat required for digestion is obtained by electricity generation from biogas. On the other hand in case chemicals are used for processes such as dewatering, removal of phosphorus, more energy is required for indirect processes (such as production, transport). In order to see direct and indirect energy consumption of wastewater treatment plants, energy requirement due to chemical use should taken into consideration.

Generally, the large portion of energy is consumed in the secondary treatment process.85% of the treatment plant in the US with flows exceeding 7510 m³/d utilize activated sludge for secondary treatment.The typical distribution of energy consumption at an activated sludge facility is shown in Figure 5.2.Regardless of facility size, this distribution would be remain relatively consistent (Ast,2008).

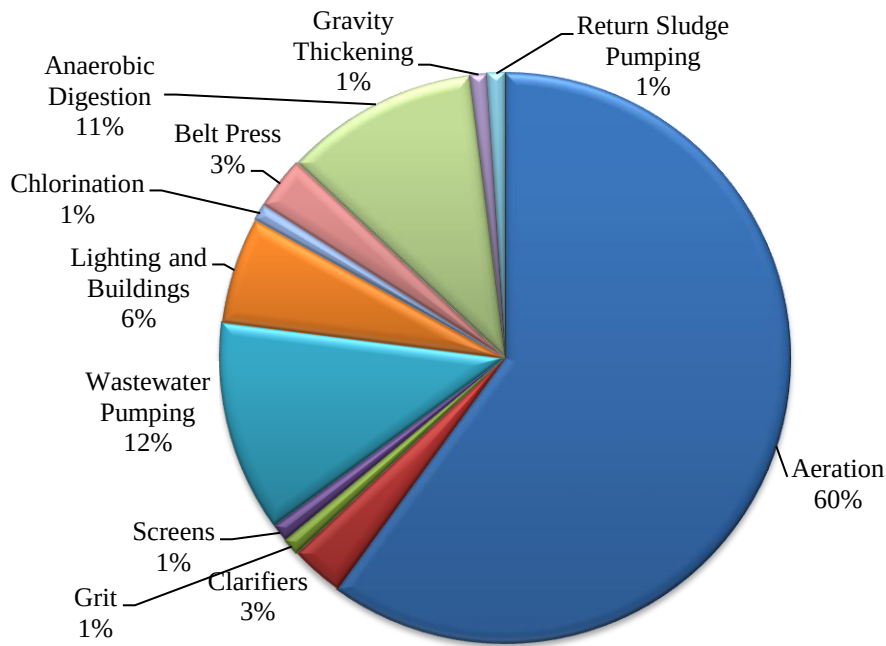


Figure 5.2 : Electricity requirement for typical activated sludge facilities (Ast,2008).

Nutrient removal of municipal wastewater activated sludge plant is not necessarily associated with high energy consumption. Not only theoretical evaluations but also practical examples from wastewater treatment plants in Austria, shows that the demand of electricity for the aeration (of the aeration tank) in activated sludge plants equipped with anaerobic digesters ranges between 9 to 13 kwh/PE.a. The demand of electricity for the other electrical equipments is approximately between 7 to 12 kwh/PE.a. 12 to 14 kwh/PE.a electrical or mechanical energy can be obtained with digester gas. Accordingly nutrient removal plants demand external electrical or mechanical power of at least 5 to 10 kwh/PE.a (Novak,2003).

Aproximately 85 percent (Metcalf & Eddy, 2003) of the wastewater treatment plants in the United States provide secondary and higher levels of treatment. The distribution of units of advanced wastewater treatment plants based on energy consumption according to Federation of America Water Environment that is made by the water environment measuring and monitoring, are given in Table 5.2.

Table 5.2 : Energy requirement for advanced wastewater treatment plants with nitrification (WEF, 2009).

Item	Electricity use, kwh/d					
	160 m ³ /h	800 m ³ /h	1600 m ³ /h	3170 m ³ /h	7900 m ³ /h	15770 m ³ /h
Wastewater pumping	171	716	1,402	2,559	6,030	11,818
Screens	2	2	2	3	6	11
Aerated grit removal	49	87	134	250	600	1,200
Primary clarifies	15	78	155	310	776	1,551
Aeration (diffused air)	532	2,660	5,320	10,640	26,600	53,200
Biological nitrification	346	1,724	3,446	6,818	16,936	33,800
Return sludge pumping	54	256	508	869	1,952	3,757
Secondary clarifies	15	78	155	310	776	1,551
Chemical addition	80	290	552	954	2,187	4,159
Filter feed pumping	143	445	822	1,645	3,440	6,712
Filtration	137	247	385	709	1,679	3,295
Gravity thickening	6	15	25	37	75	138
Dissolved air flotation	na ^a	na	2,022	3,268	7,008	13,237
Aerobic digestion	1200	2400	na	na	na	na
Anaerobic digestion	Na	Na	1,700	3,200	7,800	15600
Belt filter pres	Na	228	457	689	1,385	2,545
Chlorination	1	5	27	53	133	266
Lighting and buildings	200	400	800	1,200	2,000	3,000
Totals	2951	9,631	17,921	33,514	79,383	155,840
Average flowrate, m ³ /s	0.044	0.22	0.44	0.88	2.19	4.38
Unit electricity use, kwh/m ³	0.78	0.51	0.47	0.44	0.42	0.41
Energy recovery (from biogas combustion)	Na	na	3,500	7,000	17,500	35,000
Net consumption	2,951	9,631	14,412	26,514	61,883	120,840
Unit net electricity use, kwh/m ³	0.78	0.51	0.38	0.35	0.33	0.32
UV disinfection	77	384	768	1,536	3,840	7,680
Gravity belt thickener	Na	na	343	517	1,039	1,909

^aNot applicable

5.4 Production of biodiesels from municipal sludge

There are many studies in the literature about the disposal of municipal sludge from wastewater treatment plants by anaerobic digestion and gaining energy from this process. It is estimated that each year USA has 628 to 4940 million kwh recovery per year. In Texas, 40.2 to 460 million kwh saving per year is obtained with anaerobic digestion and this savings can go up to 51.9 to 1,030 million kwh by the incineration of biological solids (Stillwell et al., 2010).

The potential energy in wastewater, including from residuals, liquid stream constituents, and as recoverable heat, exceeds the energy to treat it by 10-fold; yet currently, this renewable potential resource largely goes to waste. Overcoming this mode of operation will require concerted efforts on the part of multiple stakeholders to help utilities become energy efficient and potentially energy self-sufficient (Zahreddine,2010).

Anaerobic processes are not energy consumers like aerobic processes, on the contrary they are energy producers. The energy balance of a 20°C strong wastewater is provided in Table 5.3. According to the conditions given in the table 5.3, aerobic process needs an energy of 528 kwh/day, whereas anaerobic process produces 3472 kwh/day. Approximately 583 kwh/day of the total energy produced by anaerobic process is needed to increase the temperature of the wastewater from 20°C to 30°C. A temperature of 30°C, is the bottom value for the mesophilic temperature range and it is a very desirable value anaerobic treatment. Hereby, the potential net energy production that is obtained as a result of anaerobic treatment is 2889 kwh/day. This value is 5 times more than the energy needed for aerobic treatment (Metcalf& Eddy, 2003).

Wastewater strength plays a critical role in the energy balance comparison of aerobic and anaerobic processes in case the wastewater temperature needs to be increased. Based on the same assumptions provided in Table 5.3 to obtain the energy balance, both aerobic and anaerobic processes require the same amount of energy influent when the biodegradable COD concentration is 1270 mg/L (Metcalf and Eddy, 2003).

Table 5.3 : Comparison of energy balance for aerobic and anaerobic processes for the treatment of a wastewater^a (Metcalf & Eddy, 2003).

Energy	Value, kwh/d	
	Anaerobic	Aerobic
Aeration ^{b,c}		528
Methane produced ^{d,e}	3472	
Increase wastewater temp. to 30 °C	583	
Net energy, kJ/d	2,889	528

^awastewater characteristics: Q:100 m³/d; 10kg/m³;20 °C

^bOxygen required = 0.8 kg/kg COD removed.

^cAeration efficiency = 1.52 O₂/kwh and 3600 kJ=1 kwh

^dMethane production = 0.35 m³/kg COD removed.

^eEnergy content of methane = 35,846 kJ/m³ (at 0°C and 1atm).

The calorific value of methane gas (CH_4) is 10 kwh/ Nm^3 . By means of combined heat and power units (CHP), approximately 32% of this 10kwh/ Nm^3 can be transformed into electricity. In case of direct coupling of the gas engine and blower, it can also be transformed into mechanical power (Nowak, 2003).

Data concerning biogas production of UASB reactor with 5000 m^3 municipal wastewater capacity per day in India are given in Table 5.4 below.

Table 5.4 : Biogas production and quality (Draaijer,1992).

Parameters	Unit	Value
Biogas yield	m^3/kg COD removed m^3/m^3 wastewater	0,05 - 0,10 0,025 - 0,030
H_2S	(vol. %)	0,5
CO_2	(vol. %)	5
CH_4 estimated	(vol. %)	75 – 80
N estimated	(vol. %)	14 – 19

In 2008, 24% of the electric requirement of municipal wastewater treatment plants in Holland is met by anaerobic digestion(NLAgency, 2010). CH_4 contents and calorific values of biogases from various types of wastes are given in Table 5.5.

Atwo stage biological treatment (A/B plant) in order to treat loads varying from 90000 to more than 2000000 PE weekly avarages depending on touristic season , is provided by the municipal WWTP Strass. There are 31 communities that drain their sewage to this plant. 55-65 % of the organic load is eliminated by the high loaded A-stage with intermediate clarification and a separate sludge cycle. The A-stage is operated at half a day sludge retention time SRT, meanwhile in the B-stage the target SRT is about 10 days. In the low loaded B-stage N-elimination is operated by pre-denitrification, in order to reach an annual N-removal efficiency of around 80% at maximum ammonia effluent concentration of 5 mg/L. In order to reach to maximum load flexibility of the system all activated sludge tanks can be aerated. By using on-line ammonia measurements airflow and aeration periods are controlled. The PE specific calorific energy input to the plant of 1760 kJ/PE corresponds to 120 g COD/PE and the thermal energy flux of 14100 kJ/PE corresponds to 200 L/PE at 16.8 °C (Wett et al., 2007).

Table 5.5 : CH₄ contents and calorific values of biogases from various type of wastes (Öztürk, 2007).

Type of biogas	CH ₄ (%)	Calorific value (kwh/m ³)
Gas from sludge digester	60-70	6-7
Gas from anaerobic industrial WWTP	50-85	5-8,5
Gas from anaerobic treatment of farm wastes	55-75	5,5-7,5
Landfill gas	35-55	3,5-5,5

In Greater Amman, Samra station in the wastewater treatment plant designed for a population of 2.2 million, applies developed solutions for wastewater treatment (activated sludge and primary sedimentation) and sludge treatment (digester and compost). With a capacity of an average of 267,000 m³ per day, it discharges a good quality wastewater into the natural environment. More than 90% of the energy required for a treatment plant of this scale is produced by biogas obtained from digestion (Fievez, 2009).

In the USA, 80% of wastewater treatments plants are plants with a capacity of 3785m³/d or smaller. In Figure 5.3, the results of comparison of 54 wastewater treatment plants are given. The average energy requirement and is 35 kwh/m³ and unit energy requirement changes between 6 kwh/m³ and 122 kwh/m³ (Ast, 2008).

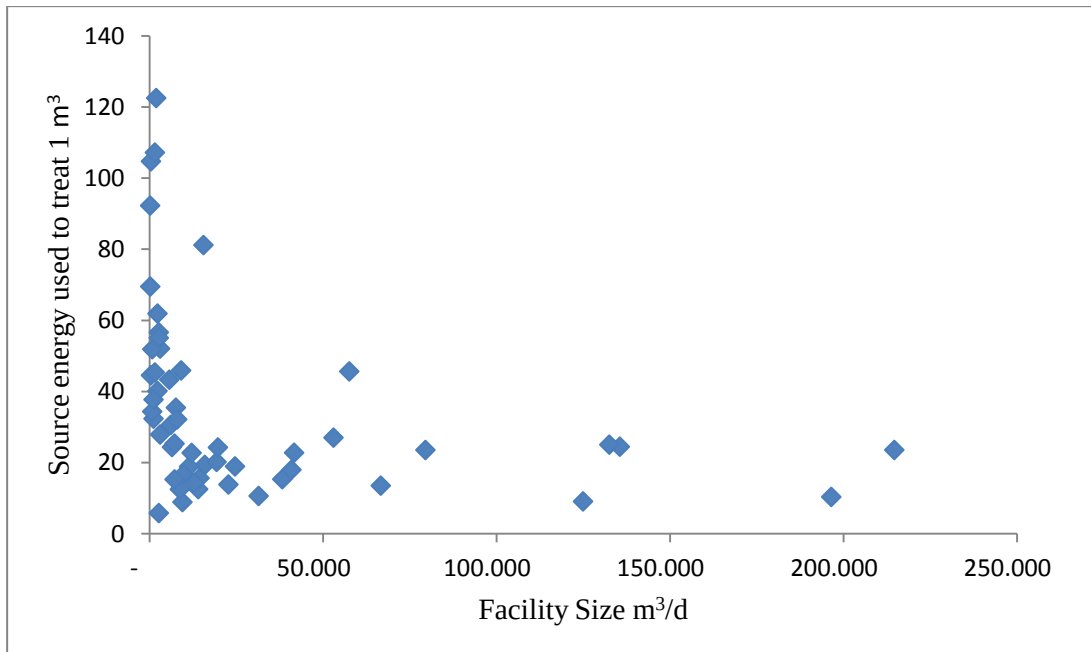


Figure 5.3 : Energy use per flow vs. average daily flow (Ast,2008).

It is commonly known that the potential energy that is available in the raw wastewater influent considerably exceeds the electricity requirement of the treatment process. Energy that is captured in organics entering the plant can be related to the COD load of the influent flow. A capita specific energy input of 1760 kJ per PE in terms of 120 g COD of organic matter can be calculated by using the calorific measurements that were presented by Shizas and Bagley (2004). This specific organic load partly releases the captured energy during aerobic and anaerobic degradation process. Energy from carbo-hydrate degradation is differentiated under three categories – ET thermal energy, ES syntheses energy and EE electricity (Wett et al., 2007)

The results of the measured mass flow and balance study of Ulu Pandan, the second largest wastewater treatment plant in Singapore, are evaluated. In UPWRP the global specific energy consumption was 0.52 kWh/m^3 , electricity generation was 0.12 kWh/m^3 , the energy efficiency as a result is 28.8%. Based on the datas, the largest electricity consumer, with 42.4% of the total energy consumed, is aeration for biological treatment. This result is similar to other wastewater treatment plants. For odour removal and inlet pumping, the energy consumed is calculated as 27.6%. Eventhough this result is higher compared to similar wastewater plants in the world, Ulu Pandan has considerations to protect the surrounding and the inlet pumping locations are high (Yeshi et al.,2012).

The values obtained in the studies conducted for the calculation of the amount of the sludge that would be produced from the 15 municipal wastewater treatment plants projected in the Master plan for İstanbul, are shown in Table 5.6. Table 5.6 consists of the amount of sludge in 2010 that is formed for primary treatment, secondary treatment and tertiary treatment options. Primary treatment is considered as screen and grit removal, secondary treatment as biological treatment and tertiary treatment as nutrient removal in addition to biological treatment (Çakmakçı et al., 2005).

Table 5.6 : Calculation of sludge amount for Istanbul WWTP in 2010 (Çakmakcı, 2005).

Plant	1.Stage treatment		2. Stage treatment		3. Stage treatment	
	kg/d	m ³ /d	kg/d	m ³ /d	kg/d	m ³ /d
Riva	130,360	2,556	195,329	10,637	221,020	13,832
Baltalimanı	61,100	1,196	91,615	4,994	103,630	6,488
B.Çekmece	16,851	330	25,383	1,392	28,650	1,798
K.Çekmece	129,247	2,534	193,560	10,533	219,073	13,707
Tuzla	94,432	1,852	141,354	7,688	160,021	10,010
B.Marmara	2,978	58	4455	242	5,045	315
Yenikapı	111,866	2,193	167,647	9,131	189,682	11,872
Terkos	589	8	148	32	666	42
Kilyos	182	2	52	10	202	13
Büyükada	2,393	31	567	129	2,715	169
Heybeliada	2,345	31	574	128	2,648	166
Burgazada	766	10	192	43	860	54
Kınalıada	1,376	18	333	75	1,556	92
Paşaköy	66,392	871	15,047	3,605	75,203	4,701
Tepeören	9,274	182	13,867	753	15,706	982
Total	630,151	11,872	850,123	49,392	1,026,677	64,241

5.5 Energy saving in wastewater treatment plants

Energy consumption on waste water treatment system is depending on its location, strenght of wastewater, level of treatment, type of treatment process selected, and mode of operation, configuration, sludge management,hydraulic conditions, inplant energy recovery design,but then comprasion of performance criteria and optimization of energy continuous improvement must be done. The problems in plants and the cause that increase the energy consumption need to be analysed, these issues are considered for next plant project.

Especially the improvements plans for operation about the department which use most energy like pumps and aeration need to be done on design step.

Reducing the amount of domestic sewage treatment plant energy is important to avoid leakage to the ground water system. Otherwise, the flow rate will increase on both of the sewage and wastewater treatment systems.That will also lead to consume more energy for both of systems. As a result of preventing leakage, that will be provided to use smaller tanks and equipment.

15% to 30% of the operating cost at large treatment facilities and 30% to 40% at small facilities are due to energy consumption in municipal wastewater treatment (Zahreddine,2010).

In United states, 2% of the total amount of the electricity produced, is consumed by water and wastewater utilities. Typically, 30% of total operating cost of the plant is spent on energy use. It is expected that in the near future, next 20 to 30 years, the electricity requirement is going to increase 30 to 40% in addition. It is important to focus on improving the efficiency of electric use and reducing the cost of treatment in the design and operation of wastewater treatment plants, as there are concerns about the adequacy of fuel supplies, cost of energy, and the increasing higher levels of treatment that results in increased energy consumption (Metcalf & Eddy,2003).

It is possible to provide great benefit by effectively monitoring the wastewater treatment plant in term of energy efficiency. In developed countries like United States and Holland, energy consumption of the wastewater treatment plants are monitored. Annual energy cost of the wastewater treatment plant corresponds to 5-10 % of the operation cost of the treatment plant (Kolish et al., 2009).

Aeration system is a set of equipment supplied by different manufacturers, come together to ensure the transfer of oxygen. The system consists of blower, pipes, valves, diffusers, and control equipments. Other aeration equipments are defined as surface aerators, air brushes, the tower of activated sludge systems , etc. Many factor affects ability to work any equipment that meet dissolved oxygen demand in wastewater. These are diffuser type, tank geometry, diffuser depth, turbulence, external air pressure, temperature, aeration equipment, and their distance from the settlements with each other, the daily changes in temperature of wastewater and organic loadings. The use of fine bubble diffuser becomes more efficient than the use of coarse bubble diffuser in treatment plants and consume less energy. Sending to extra air to aeration tanks causes to loss of energy.

One possible ECM to consider is replacing a larger blower with smaller blowers that allow for better matching of the air flow needed to provide the required oxygen demand of the aeration process.

In general, blower systems should be capable of operating at 1/5th of their full capacity. Cantwell, et al. recommended that aeration systems be designed with 4

blowers at 33% capacity each of design air flow, or 2 blowers at 33% each of design air flow plus 2 blowers at 50% each of design air flow. Although this is a good rule of thumb, there are many configurations that can operate efficiently, especially at larger plants (Zahreddine,2010).

To maintain a dissolved oxygen concentration of 2mg/l in the aeration tank has been a general rule. In case the carbonaceous demand and the nitrogenous oxygen demands have been satisfied, maintaining any dissolved oxygen concentration above 2mg/l is excessive and a waste of energy. There are many wastewater treatment plants today, that operate at low dissolved oxygen concentration once they meet their carbonaceous and nitrogenous oxygen demand. There are treatment facilities that maintain a dissolved oxygen concentration in the range of 0.5 to 2.0 mg/L at the end of the aeration basin (WEF,2009).

In the case of membrane aerators, under operating conditions, the oxygenation capacity is, at the best, about 2.2 to 2.5 kg O₂/kwh. Assuming a value of 2.35 kg O₂/kwh and a DO of 1.5 mg/L in the oxic zones, the energy demand for the aeration is 0.50 kwh per kg of oxygen consumed at 15°C (Nowak,2003).

At the present time, dissolved oxygen meters and blower control systems consume much more reliable and appropriate amounts of energy. The amount of stable dissolved oxygen in aeration tank ensure energy savings. Because, the load of air blower remains constant and / or the air blower is set a smaller range to fixed energy usage. Placing fine bubble diffuser to aeration tanks will reduce the capacity of the air blower to meet the demand of the process air. Working in low capacity air blower will provide the energy savings (WEF, 2009).

Fortunately, many wastewater treatment plants, by taking energy efficiency measures and by modifying their treatment process reduce energy costs by 30% or more. Total energy usage of wastewater treatment plants can be reduced by 3-6% just by optimized aeration and improved pumping system (Stillwell,2010).

As researches show, it is possible to save as much as 20% of all energy consumption in a wastewater treatment plant (Mauer et al., 2010).

The most important thermal energy needed for domestic wastewater treatment plants are due to heat the sludge digestion tanks. Thermal energy is used for heating the digesters fed with sludge and minimize to the reactor in the heat losses.

The heat obtained from Cogeneration is sufficient to heat digestion process (Geilvoet et al.,2010). In colder areas, the reactor cooling rate due to the loss of heat is between 0.25°C and 0.75°C in a day. If heat insulation can be made sufficiently heat losses can be reduced by up to 30% of the total heat requirement. In this way, the thermal insulation of a facility to pay off the cost of the investment period is very short. There are no more differences between the temperature gradient of up to 45°C with the advent of high heat transfer efficiency of thermophilic sludge digesters and mesophilic digesters (Öztürk, 2007).

At the present time, dissolved oxygen meters and blower control systems consume much more reliable and appropriate amounts of energy. The heating the raw sludge digesters to digester temperature and the additional heat transfer loss of the digester surface leads to higher specific heat demand for wastewater treatment plants with separate anaerobic sludge stabilization compared to simultaneous stabilization. The heat requires for heating the sludge in anaerobic digester is about 70% of WWTP's total heat requirement. 20% of the heat requirement is due to transmission loss of the digester transfer and about 10% of the total heat requirement is for heating the buildings. The heat requirement for heating the sludge is about 13.5 kwh/PE.a and therefore target value for wastewater treatment plants with digester is 19.7 kwh/PE.a. the targeted values for WWTP including simultaneous aerobic sludge stabilization and digester is approximately 2.0 kwh/PE.a (Kolish et al.,2009).

Contradictory to sewage plants with simultaneous aerobic sludge stabilization, the sewage plants with sludge digestion show specific heat demand which is significantly higher than the target value. Average difference is about 64%. The reason of this in many cases is the inefficient operation of sludge prethickeners yielding to a higher volume load of the digester with a larger heat demand and a gas production that is reduced (Kolish et al.,2009).

Another issue is the improvement of the sludge with sludge disintegration. Sludge disintegration is defined as the deformation of the structure of the sludge by applying external tension. Disintegration can be achieved physically, chemically or biologically. Disintegration process changes many features of the sludge. When this process is applied, the floc structure of the structure deteriorates and microbial cell walls are destroyed. When cell walls are destroyed, intracellular materials that are protected by cell walls are released into the liquid phase and are transformed into

soluble form. Even though disintegration process can be mistaken as sludge digestion, it is very different than digestion which involves only the reduction of organic material and it is a more advanced treatment process (Filibeli vd., 2006).

Throughout desintegration, as the result of forces applied to sludge, a significant and sudden reduction in the particle size of sludge occurs. The reduction in the size of the particle, in general ensures easier hydrolysis of the solids in the sludge as the result of increasing surface area associated with the reduction in the partical volume. As the result of an effective disintegration, the majority of the organic materials in the sludge are released into liquid phase. The solid sludge particals that cannot pass into liquid phase consists mainly of inorganic materials and as a result, disintegrated sludge can reach higher solids content after dewatering (Filibeli vd., 2006).

Anaerobic digestion of the disintegrated sludge, provides high stabilization as the result of severe fragmentation of the organic material and in this way,the amount of waste sludge can be reduced by 30-40 % compared with conventional anaerobic digestion. Another change in the structure of the sludge during disintegration process, is the reduction of the viscosity of the sludge. The reduction in the viscosity of the sludge is very important since it facilitates the mixing and pumping of the sludge (Filibeli vd., 2006).

Between 20% to 70% of the flow is treated in most of the full scale sludge disintegration application. . It cannot be chosen beforehand whether you would use all the flow or a part of the flow, it depends on the chosen technology and equipment. Another application which is used frequently is return sludge treatment. Sludge disintegration provides to reduce the amount of sludge, developes the dewatering, and increases the biogas production (Luning et al., 2007).

During the digestion, sludge can decompose faster and better. Organic material can be completed by better decomposition with these short retention time and same retention time. Degradation of organic material increase about 25% -35% as literature. After disintegration, dewaterability is developed as the results of adapted particle size of sludge and low quantity of organic material. Dry weight of sludge cake can increase as 10%.Disintegration prevents sludge swelling and formation of foam by treating the filamentous material in sludge. In this way effective volume of disintegration is increased. Biogas is produced as a result of increase of biomass decomposition. Biogas production is increased as 25%-35% (Luning et al.,2007).

Another important energy consumption is wastewater pumps which are used in order to raise wastewater from low point to higher point. There are many pumping applications that are wastewater or treated wastewater, sand, grease and floating solids, dilute or concentrated raw sludge or digested sludge, excess sludge or chemical application of sewage sludge as a result of pumping of the upper phase in a wastewater treatment plant. Conveyed sludge reaches generally as far as the bottom content of the solid material.

1. Excess sludge (Total solid percentage is to % 1).
2. Primary sludge (Total solid percentage is to % 4).
3. Thickened or digested sludge (Total solid percentage is to % 8).

While choosing pumps and pipes total operating expenses should be considered rather than just looking at the low initial costs. When the initial investment cost and energy consumption are combined, the pump and the pipe that provides the lowest life cycle should be selected. The energy needed in the plant just to pump the influent wastewater, can reach up to %15 to %70 of the energy that the wastewater treatment plant uses overall. This amount can reach up to %90 of the energy that is used within the whole plant when the total energy that is used in the operation of all of the pumps is taken into consideration. The amount of energy that is used by the pumps in the influent pumping stations is calculated by how many hours the pumps are operated in their most efficient points. It is a fact that energy calculation is a standard calculation, on the otherhand design can be developed in such a way to optimize the pumping power (WEF,2009).

Appropriate sizing of pumps is key to the efficient operation of a wastewater treatment plant. Pumps sized for peak flow conditions that occur infrequently, or worse for projected future peak flow conditions toward the end of the pump's design life, operate the majority of the time at reduced flow that is below the pump's best efficiency point. Pumping system energy efficiency improvements typically result from lowering of pumping capacity to better match system demand, replacing inefficient pumps or modifying pump impellers, replacing standard efficiency motors with high efficiency motors, installing Variable Frequency Drives (VFDs) to provide pump output throttling in variable output pumping systems (Zahreddine et al.,2010).

In order to improve process performance and energy efficiency, three main areas were identified :

(i) pre-concentration of the influent COD by the PSTs in order to supply much COD to the anaerobic digesters for production of biogas and electricity.

(ii) reduction of aeration energy consumption by automatically controlling aeration and aerobic SRT using on-line sensor; and

(iii) increasing of the solid line performance and operation by pre-treatment of sludge, effective operation of anaerobic digester, applications of high efficiency engines and side line deammonification, etc. It is possible to increase the energy efficiency from the current 28.8% to ~50% by adopting these alternatives (Yeshe et al., 2012).

For energy optimization, a control and automation system including supervisory control and data acquisition (SCADA) must be created. SCADA offers control of selected processes from a central examination room. It allows the monitoring of the current status of processes and equipments, alarms and work schedules.

6. DESIGN APPLICATION FOR MUNICIPAL WASTEWATER TREATMENT PLANT

6.1 Introduction

In this chapter, different design approaches are evaluated by using influent COD, BOD, SS, Total N and Total P values given in the 3rd chapter for municipal wastewater treatment plants and used in general in Turkey. In this approach, A²O process which is one of the processes used for nutrient removal for the plants with a population between the range of 50,000-1,000,000 and extended aeration process used widely in Turkey are taken into consideration. Due to the addition of N and P limits to our discharge standards in Turkey (Municipal Wastewater Treatment Regulation, 2006), a third alternative is formed by adding nutrient removal to extended aeration process. For these three design processes, the reactor volume, oxygen consumption and sludge amount are calculated with the German Standard program for activated sludge design ATV (A131) and results are compared. In addition, for A²O process conceptual calculation methods are used and results are compared with the values obtained from ATV.

6.2 The selection of treatment units for municipal wastewater

Discharge standards are formed including also nitrogen and phosphorus parameters besides carbon removal to protect the quality of receiving bodies. Correct designs of these kind of treatment systems are essential for meeting the discharge standards especially in sensitive areas. As the result of the changes of discharge standards, carbon removal systems are converted into systems with nutrient removal, if the population is more than 10.000 in the settlements in sensitive areas. There is a growing demand in converting present systems with carbon removal into systems with nutrient removal.

In this context A²O process and extended aeration process are designed with flow rate below.

1. Three-stage “Phoredox” (A^2O) process for biological N and P removal

Biologic nutrient removal treatment consists of screening, grit and grease removal, primary sedimentation, activated sludge tanks with anaerobic, anoxic and aerobic zones, secondary sedimentation units. A schematic arrangement of the A^2O process is shown in Figure 6.1.

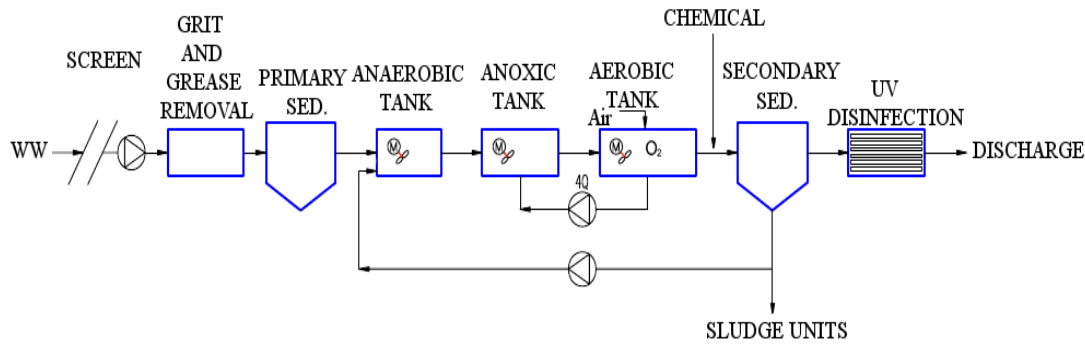


Figure 6.1 : Three-stage “Phoredox” (A^2O) process.

2. Extended aeration activated sludge system

Extended aeration activated sludge for carbon removal consist of screening, grit and grease removal, extended aeration activated sludge tanks and secondary sedimentation units.

3. Extended aeration activated sludge system with nutrient removal

Extended aeration active sludge system with nutrient removal consists of screening, grit and grease removal, anaerobic tank, extended aeration activated sludge tanks with anoxic and aerobic zones, secondary sedimentation units.

6.3 Influent values for municipal treatment plants

As a result of data, WWTP, which are operated, that are detailed in chapter 3 (Table 3.33) inlet wastewater flow and loads in Table 6.1 and Table 6.2 are given for input data to be used in design. Wastewater flow rate determination in Table 6.1 are used in quantities of wastewater volume. That requires detailed information that reflects a realistic relationship among substrat, biomass and electron acceptor for biological nitrogen and phosphorus removal. Based on the calculation methods of Province Bank, a peak factor of 2 is calculated for domestic wastewater.

Table 6.1 : Wastewater quantities and quality at plant inlet.

Total Polulation Equivalent	P.E.	50,000	100,000	200,000	400,000	1,000,000
Water demand	l/P.d	100	100	125	140	160
Dry weather daily flow	m ³ /d	5,000	10,000	25,000	56,000	160,000
Qort, Average Flow	l/s	58	116	289	648	1,852
	m ³ /h	208	417	1,042	2,333	6,667
Qmax, Wet Weather Flow	l/s	116	231	579	1,296	3,704
	m ³ /h	417	833	2,083	4,667	13,333

Pollutant loads and concentrations are given as the equivalent population in the raw wastewater values depending on population (industrial loads are also taken into account) in Table 6.2.

Table 6.2 : Wastewater loads and concentrations at plant inlet.

Parameter	P.E.	50,000	100,000	200,000	400,000	1,000,000
Loads						
COD	kg/d	3,250	6,500	14,000	29,960	80,000
BOD	kg/d	1,500	3,000	6,500	13,720	36,800
SS	kg/d	1,950	3,900	8,500	17,920	48,000
Total-N	kg/d	325	650	1,400	3,024	8,000
Total-P	kg/d	52	104	222	480	1,280
Concentrations						
COD	mg/l	650	650	560	535	500
Dissolved COD	mg/l	260	260	224	214	200
BOD	mg/l	300	300	260	245	230
SS	mg/l	390	390	340	320	300
Total-N	mg/l	65	65	56	54	50
NH ₃ -N	mg/l	42	42	36	35	33
Total-P	mg/l	10	10	9	9	8

The values after primary sedimentation was taken from Table 6.3 as the input values of ATV and a conceptual design. Primary sedimentation removal efficiency as given in Table 4.4 were used as 28% COD, 25% BOD, 66% TSS , 8.3% Total N, 8.3% Total-P values There is no pre-settlement for extended aeration, because of this reason values in the raw wastewater were taken from Table 6.2. Alkalinity value was accepted as 10-15 mmol / l.

Table 6.3 : Loads and concentration at Outlet of Primary Sedimentation.

Parameter	P.E.	50,000	100,000	200,000	400,000	1,000,000
Loads						
COD	kg/d	2,340	4,680	10,080	21,571	57,600
BOD	kg/d	1,125	2,250	4,875	10,290	27,600
SS	kg/d	663	1,326	2,890	6,093	16,320
Total-N	kg/d	298	596	1,284	2,773	7,336
Total-P	kg/d	48	95	204	440	1,174
Concentrations						
COD	mg/l	468	468	403	385	360
Dissolved COD	mg/l	187	187	161	154	144
BOD	mg/l	225	225	195	184	173
SS	mg/l	133	133	116	109	102
Total-N	mg/l	60	60	51	50	46
NH ₄ -N	mg/l	39	39	33	32	30
Total-P	mg/l	10	10	8	8	7

6.4 Design method applications

For this study, same input wastewater values were solved by ATV-A 131 method. The results were compared with the conventional method. When, Nitrogen and phosphorus removal systems for design applications are examined, It is observed that different methods have been carried out.

Some basic conditions must be taken into account such as reliable wastewater characterization in the design of activated sludge systems, substrat parameter is selected on the basis of BOD or COD and wastewater COD fraction. Studies on the COD fractions of wastewater is increasing in recent years.COD is considered as an important parameter that requires detailed information which reflects a realistic relationship among substrat, biomass and electron acceptor for biological nitrogen and phosphorus removal.

Both in ATV program and conceptual design, it is accepted that all of the organic material is removed. In design, calculation based on COD is preferred, in the design parameters used in both methods, the values are chosen as close as possible. Formulas used in both design, are given comparatively in terms of N and P removal, Sludge Production and Oxygen utilization in Table 6.4

Table 6.4 : ATV-131 compared with the conceptual design calculations.

Design parameter	Conceptual design	ATV-131
Sludge age, d	$\frac{1}{\theta_{XA}min} = \hat{\mu}_A(T) - b_A(T)$ $\theta_{XA} = SF \theta_{XA}min$ $\hat{\mu}_A(T) = \hat{\mu}_A(20) * 1.103^{T-20}$ $b_A(T) = b_A(20) * 1.072^{T-20}$ $b_H(T) = b_H(20) * 1.072^{T-20}$	$t_{ss} = SF * 3.4 * 1.103^{15-T}$ $t_{ss,dim} = t_{ss,aerob} \frac{1}{1 - V_D/V}$
	$fe = 2-2.5$	
	$\theta_X = \frac{\theta_{XA}}{1 - V_D/V}$	$t_{ss,aerob,dim} = SF * 3.4 * 1.103^{15-T} \frac{1}{1 - V_D/V}$
N Removal	$N_X = (i_{XB} + i_{XE} f_{EX} b_H c \theta_X) \frac{Y_H (C_{SI} - S_S)}{1 + c b_H \theta_X}$	$S_{NO3,D} = C_{N,0} - C_{orgN,E} - S_{NH4,E} - S_{NO3,E} - X_{orgN,WAS}$
	$N_{OX} = C_{NDI} + S_{NH1} - S_{NH} - N_X$	
	$Q N_{DP} = \frac{OR_{HS}}{2.86} + \frac{\eta V_D (OR_H - OR_{HS})}{cV} \frac{1}{2.86}$	$X_{orgN,WAS} = 0.02 \text{ to } 0.025 \cdot C_{COD,0}$
	$N_D = N_{DP} - R * \frac{S_{OR}}{2.86}$	
	$S_{NO} = S_{NO1} + N_{OX} - N_{DP} + R * \frac{S_{OR}}{2.86}$	$\frac{Q_d \cdot 2.9 \cdot S_{NO3,D}}{1000} = V_D \cdot 0.75 \cdot \frac{OU_{d,C}}{V_{AT}}$
		$OU = C_{COD,0} - S_{COD,inert,E} - X_{COD,WAS}$
	$\frac{OR_{HS}}{Q} = (1 - Y_H) S_{S1}$	$X_{COD,WAS} = X_{COD,inert,0} + (C_{COD,0} - S_{COD,inert,0} - X_{COD,inert,0}) \cdot Y \cdot \frac{1}{1 + b \cdot t_s} + 0.2 \cdot X_{COD,BM} \cdot t_s \cdot b$
Oxygen utilization, OR_{HS} kgO ₂ /d	$\frac{OR_H}{Q} = [(1 - Y_H) + (1 - f_{EX} - f_{ES}) c b_H \theta_X \frac{Y_H}{1 + b_H c \theta_X}] C_{S1}$	

(T) :The temperature factor

Table 6.4 : ATV-131 compared with the conceptual design calculations (continue).

Design parameter	Conceptual design	ATV-131
		$SP_d = SP_{d,C} + SP_{d,p}$
Sludge production ¹ kgSS/d	$Y_{NH} = \frac{Y_H}{1 + cb_H\theta_X}$ $Y_{NA} = \frac{Y_A}{1 + b_A\theta_{XA}}$	$t_{SS} = \frac{M_{SS,AT}}{SP_d} = \frac{V_{AT} \cdot SS_{AT}}{SP_d}$ $= \frac{V_{AT} \cdot SS_{AT}}{Q_{WS,d} \cdot SS_{WS} + Q_d \cdot X_{SS,EST}}$
		$SP_{d,c} = B_{d,BOD} \cdot (0.75 + 0.6 \cdot \frac{X_{SS,IAT}}{C_{BOD,IAT}} - \frac{(1 - 0.2) \cdot 0.17 \cdot 0.75 \cdot t_{SS} \cdot F_T}{1 + 0.17 \cdot t_{SS} \cdot F_T})$
	$P_X(COD) = P_{XH} + P_{XE} + P_{XA} + QX_{I1}$	
	$P_{XH}(COD) = Y_{NH}QC_{S1}$	$SP_{d,p}$
	$P_{XE}(COD) = f_{EX}cb_HY_{NH}QC_{S1}\theta_X$	$= Q_d$
	$P_{XA}(COD) = Y_{NA}QN_{OX}$	$\cdot \frac{(3 \cdot X_{P,BioP} + 6.8 \cdot X_{P,PreC,Fe} + 5.3 \cdot X_{P,PreC,Al})}{1000}$
	$P_{XAE}(COD) = f_{EX}b_AY_{NA}QN_{OX}\theta_{XA}$	$F_{(T)} = 1.027^{(T-15)}$
	$P_{XI}(COD) = QX_{I1}$	
	$P_{XF} = Qf_{XF}X_{AKM}$	
	$P_T(SS) = I_{SSXH}P_{XH} + I_{SSXE}P_{XE} + I_{SSXA}P_{XA}$ $+ I_{SSXAE}P_{XAE} + I_{SSXI}P_{XI}$ $+ P_{XF}$	

(T):The temperature factor

6.4.1 ATV –A 131 Design of activated sludge systems

6.4.1.1 Identification of the input data

ATV design program designs an activated sludge system that includes biological phosphorus removal sizing, in settlements with different populations(ATV-A 131,2002). In activated sludge systems that are solved with ATV-A 131 method, in addition to carbon removal, modifications as nitrification, denitrification, aerobic sludge stabilization and in addition to all methods, phosphorus settlement can be chosen. Applied design procedur, begins with definition of input information and extends to required nitrogen, reactor volume in line of phosphor removal and oxygen requirement. Design stages are given below;

1. A decision is made on which nitrification/denitrification process will be used for the nitrogen removal. COD-based design was chosen for extended aeration and preanoxic zone denitrification process.

2. Determination of the necessary safety factor (SF) taking into account the dimensioning capacity of the plant and, in case, the measured diurnal load fluctuations. SF = 1.8 should be taken in less than BOD 1,200 kg/day designing capacity due to changes in effective input load. SF = 1.45 should be also taken in the calculations as for $\geq 6,000$ kg / day.

3. Identification of the input data, flow characteristics and relevant load case 1 (BOD, COD, TSS, TN, $\text{NH}_3\text{-N}$, TP, dissolved COD concentration), dimensioning, load case 2, proof of nitrification with lowest temperature, load case 3 calculation of oxygen uptake with highest temperature are entered in Table 6.3. The values in Table 6.3 were used for Input COD components.

According to studies for Istanbul, dissolved COD range was between 30-40%. According to studies made in different countries, dissolved COD was 17-43% in Table 6.5 (Orhon et al., 2000). ATV is used in the design value of 40%. Primary sedimentation efficiencies in Table 6.6 are given.

Table 6.5 : Inlet COD Components.

Parameter	Unit	ATV	Conceptual design	Value, % ATV	Value, % (Orhon, 2000)
Total COD	mg/l	$C_{\text{COD,iat}}$	C_{T1}	100	100
Dissolved COD	mg/l	$S_{\text{COD,iat}}$	S_1	40	17-43
Inert	mg/l	$S_{\text{CODi,iat}}$	S_{I1}	5	2-20
Degradable	mg/l	$S_{\text{CODd,iat}}$	S_{S1}	35	7-33
Particular COD	mg/l	$X_{\text{COD,iat}}$	X_1	60	47-83
Inert	mg/l	$X_{\text{CODi,iat}}$	X_{I1}	15	4-26
Degradable	mg/l	$X_{\text{CODd,iat}}$	X_{S1}	45	43-75

Table 6.6 : Recommended primary sedimentation efficiencies for domestic wastewater.

Parameter	Efficiency (Orhon, 2000) %	Efficiency Settlement time (ATV)	
		0.5 to 1.0 h	1.5 to 2.0 h
		%	%
BOD5	25	25	33
COD	28	25	33
SS	66	50	64
TKN	8.3	9	9
P	8.3	11	11

4. For plants with nitrogen removal the mass of the nitrate to be denitrified is to be determined by means of a nitrogen balance. Input values of effluent nitrogen and phosphorus are entered to the system (Total nitrogen standard, T_{NE} , Nitrate nitrogen standard, S_{NO_E} , Ammonia nitrogen, S_{NHE} , Organic matter standard, S_E , Design temperature). The N and P rates used in the calculations of holding biomass have been given in Table 6.7. These values are used unchanged.

Table 6.7 : Incorporation of nitrogen and phosphorus values for COD related (ATV-DVWK-A 131E).

Parameter	Range	Value
Nitrogen embedded in biomass, $X_{N,BM}$	0.02-0.05 $C_{COD,IAT}$	0.025 $C_{COD,IAT}$
Normal uptake of phosphorus in biomass $X_{P,BM}$	0.005 $C_{COD,IAT}$	0.005 $C_{COD,IAT}$
Enhanced uptake of phosphorus $X_{P,BioP}$		
with anaerobic tank	0.005-0.007	0.007 $C_{COD,IAT}$
...with low temperature and $S_{NO3,EST} \geq 15 \text{ mg/l}$	0.0025-0.005	0.005 $C_{COD,IAT}$

5. Taking into account the selected denitrification process, the necessary proportion of the denitrification volume to the biological reactor volume (V_D/V) is to be determined. The sludge age ($t_{SS,dim}$) is to be calculated accordingly.

6. Selection of the Sludge Volume Index (SVI); settleable characteristic of sludge (SS_{AT}) is defined by sludge volume index, and mixed liquid suspended solids concentration (MLSS).). That affects size of biological reactor and secondary settling tank.

7. Selection of the sludge thickening time (t_{Th}) in the secondary settling tank dependent on the biological process selected and determination of the concentration of (dry) suspended solids in the bottom sludge (SS_{BS}) as function of SVI and t_{Th} .

8. Determination of the return sludge suspended solids concentration (SS_{RS}) from the achievable concentration of suspended solids in the bottom sludge and the dilution of the sludge removal stream dependent on the selected sludge removal system. Selection of the return sludge ratio (R_S) and estimation of permissible suspended

solids concentration of the activated sludge in the biological reactor. (SS_{EAT}). Values used in design are given at Table 6.8.

Table 6.8 : ATV program sludge process design data.

Parameter	Unit	Value	Range
Sludge volume index (SVI)	l/kg	100	100-150
Thickening time (t_{Th})	h	2.5	2-2.5
Return sludge ratio (R_s)	-	0.75	max. 0.75
Dry matter suspended solids ratio (SS_{RS}/SS_{BS})	-	0.70	-
Suspended solids (SS_{EAT})	kg/m ³	4,07	-

Dry matter suspended solids at the bottom of tank SS_{BS} 13.57 kg/m³, dry matter suspended solids in the return sludge was found to be SS_{RS} 9.5 kg/m³.

9. Determination of the surface area of the secondary settling tank (A_{ST}) from the permissible surface overflow rate (q_A) or the sludge volume loading rate (q_{SV}). Determination of the depth of the secondary settling tank from partial depths for the functional zones and other specifications.

10. Determination of the waste sludge production (SP_d), if required taking into account waste sludge from phosphorus removal and the possibly dosed external carbon for denitrification.

11. Calculation of the required mass of solids in the biological reactor ($M_{SS,AT}$) for the selected sludge age.

12. Calculation of the volume of the biological reactor. Dimensioning of an anaerobic mixing tank for biological phosphorus removal.

13. Determination of the relevant oxygen consumption for the design of the aeration facility.

14. Checking of the remaining alkalinity and/or the necessity for dosing -lye taking into account consumption and gain in alkalinity from ammonification, nitrification, denitrification and phosphate precipitation as well as the oxygen utilisation and diffuser depth (The latter only to determine the pH in the biological reactor).

6.4.1.2 Output of The ATV program

N and P removal in the activated sludge system has been designed by using the values of table 6.12. for cities for the equivalent population of between 50,000 to 1,000,000. The results have been given at Table 6.9, Table 6.10 and Table 6.11. Extended aeration system design for the results of same population are given in the Table 6.12 and Table 6.13.

Table 6.9 : Output data of the ATV program in different populations for the A²O process.

Population	COD/N	θ_x	S_F	R	N_{DP}	P uptake	
		d			kg/kg COD	Normal	EBPR
50.000	7.80	10.20	1.80	300	0.072	2.34	3.28
100.000	7.80	9.78	1.72	300	0.072	2.34	3.28
200.000	7.90	10.43	1.53	400	0.077	2.02	2.82
400.000	7.70	9.90	1.45	400	0.077	1.93	2.69
1.000.000	7.83	9.80	1.45	350	0.077	1.80	2.52

Table 6.10 : Output data of the ATV program in different populations for the A²O process.

Population	Primary Sludge kg/d	Excess Sludge kg/d	P Bio. kg/d	P Chem. kg/d	Total Sludge kg/d	OU _h kg/h
50.000	1,287	968	49	81	2,385	121
100.000	2,574	1,954	98	161	4,787	242
200.000	5,610	4,157	212	366	10,345	493
400.000	11,827	8,977	453	903	22,160	1029
1.000.000	31,680	23,985	1,210	1,821	58,696	2732

Table 6.11 : Output data of the ATV program in different populations for the A²O process.

Population	V_{ANAE} m^3	V_{ANOX} m^3	V_{AERO} m^3	V_T m^3	V_D/V	N mg/l	P mg/l
50.000	208	1,102	1653	2,963	0.40	14.60	2
100.000	417	2,127	3,191	5,735	0.40	14.60	2
200.000	1,042	6,069	6,069	13,180	0.50	9.80	1
400.000	2,333	12,531	12,531	27,395	0.50	10.50	1
1.000.000	6,667	32,765	32,765	72,197	0.50	9.80	1

ATV output data for carbon removal system by extended aeration activated sludge system are given in Table 6.12.

Table 6.12 : Output data of the ATV program in different populations for extended aeration process.

Population	V m ³	Total Sludge kg/d	OU _h kg/h	N mg/l	P mg/l	R
50.000	9,205	1,579	158	46.80	6.80	300
100.000	18,410	3,157	308	46.80	6.80	300
200.000	54,643	6,830	643	40.00	6.20	433
400.000	96,906	14,536	1,383	38.60	6.30	415
1.000.000	226,583	38,859	3,666	35.50	5.50	300

As calculated $\Theta_x = 20$ day, $S_F = 5.88$

Table 6.13 : Output data of the ATV program in different populations for the extended aeration process with nutrient removal.

Population	V _D /V	SF	COD/N	N _{DP}	N mg/l	P mg/l	R	OU _h kg/h
50.000	0.17	4.95	7.80	0.053	14.60	2	290	137
100.000	0.17	4.95	7.80	0.053	14.60	2	290	258
200.000	0.21	4.71	7.90	0.058	9.50	1	433	545
400.000	0.21	4.71	7.70	0.058	9.60	1	415	1,172
1.000.000	0.19	4.83	7.83	0.056	9.80	1	400	3,118

$\theta_x = 20.3$

Table 6.14 : Output data of the ATV program in different populations for the extended aeration process with nutrient removal (reactor volume).

Population	V _{ANAE} m ³	V _{ANOX} m ³	V _{AER} m ³	V _T m ³
50.000	208	1,507	7,355	9,070
100.000	417	3,013	14,710	18,140
200.000	1,042	10,919	41,077	53,038
400.000	2,333	19,160	72,080	93,573
1.000.000	6,667	45,770	195,127	247,564

Table 6.15 : Output data of the ATV program in different populations for the extended aeration process with nutrient removal (sludge removal).

Population	Excess Sludge kg/d	P Bio. kg/d	P Chem. kg/d	Total Sludge kg/d
50.000	1,576	68	7	1,651
100.000	3,151	137	14	3,301
200.000	6,817	294	217	7,328
400.000	14,508	629	599	15,736
1.000.000	38,784	1,680	1,084	41,547

When, as a result of calculation A^2O and extended aeration are compared with each other, increasing volume of extended aeration systems is between 2.8 and 3.6. The amount of oxygen increases in the order of 1.3 for extended aeration. Over 200,000 populations for extended aeration in ATV program design give warning as “with respect to the given size class simultaneous aerobic sludge stabilization is not recommended”. Taking into account that the extended aeration system is not adequate to meet the discharge standards for N and P parameters, it can only be applied in the low sensitive areas where primary treatment achieves a satisfactory treatment performance.

The design manual of ATV A131 is a “static” design method for activated sludge plants. Diurnal load fluctuations as normally ascertained municipal wastewater treatment plants with respect to the output ammonia and nitrate concentrations are taken into consideration by some safety factors. The oxygen consumption to use the aeration equipment shall be calculated on the base of the diurnal load fluctuation. If the diurnal load fluctuations are higher than normal, it is suggested to control the results by dynamic simulation. In case of noticable seasonal fluctuations, the configuration shall be controlled for the respective loads. The result may be that the activated sludge tank volume has to be ranged for the e.g. lower load at lower temperatures but the oxygen demand may exhibit the seasonal higher load in summer (Kayser, 2000).

6.4.2 Output of the conceptual design of activated sludge system with biological nutrient removal (A^2O)

Activated sludge systems due to design differences are evaluated in terms of sludge age, denitrification potential, reactor volumes, excess biological sludge generated, amount of oxygen utilized. In the design of biological treatment, defining the kinetic

and stoichiometric parameter is one of the most important aspects. The values should be determined experimentally for each wastewater. Typical values generally reported with domestic sewage is given Table 6.16.

Table 6.16 : Typical values of kinetic and stoichiometric constants for domestic sewage (Orhon&Artan,1994).

Coefficient	Unit	Value	
		Range	Typical
$\mu^{\wedge}$	day-1	1-6	3
Ks	mg/l BOD5	25-100	60
	mg/l COD	15-70	40
Yh	g VSS/g BOD5	0.4-0.8	0.6
	g VSS/g COD	0.3-0.6	0.4
Kd	day-1	0.02-0.10	0.06

The data which are stated in Table 6.1 and Table 6.6 are used as inlet data . Kinetic and stoichiometric coefficients used in the design are given in Table 6.17. The parameter of maximum autotrophic growth rate and decay rates were assigned to 0.70 d⁻¹ and 0.17 d⁻¹, respectively for reference temperature of 20 °C (İnsel et.al,2011).

Table 6.17 : Accepted kinetic and stoichiometric constants for municipal wastewater (Orhon,2000).

Parameter	Value	Parameter	Value
Heterotrophic microorganisms		η	0.6
Y_H	0.67 gr COD/gr COD	i_{XB}	0.086 gr N/gr COD
$\mu_H(15^\circ)$	2.14 1/d	i_{XE}	0.06 gr N/gr COD
$\mu_H(20^\circ)$	3.50l/d	f_{EX}	0.2
$\mu_H(25^\circ)$	5.71 1/d	f_{ES}	0.1
$b_H(15^\circ)$	0.17 1/d	Ks	20 mg/l
$b_H(20^\circ)$	0.24l/d	K_{NH}	1 mg/l
$b_H(25^\circ)$	0.34 1/d	i_{SSXH}	0.83
Autotrophic microorganisms		i_{SSXE}	0.83
Y_A	0.24 gr COD/gr N	i_{SSXI}	0.61
$\mu_A(15^\circ)$	0.43 1/d	f_{XF}	0.15
$\mu_A(20^\circ)$	0.70l/d	f_{x1}	0.1
$\mu_A(25^\circ)$	1,14 1/d	Fs	0.83
$b_A(15^\circ)$	0.12 1/d	Fs1	0.07
$b_A(20^\circ)$	0.17l/d		
$b_A(25^\circ)$	0.24 1/d		

The most important parameter in the design and operation is sludge age activated sludge systems. Primarily the aerobic sludge retention time required for growth of the organism is selected. As autotrophs grow slower than heterotrophs, the sludge age is the limiting step for growth of organisms.

Anaerobic sludge age and output ammonia concentration is calculated by the following formulas. The minimum sludge retention time is calculated by multiplying safety factor (f_e) to take into account such as temperature, dissolved oxygen. This factor is selected from the range of 2-2.5.

$$\min\theta_{XA} = \frac{1}{\hat{\mu}_a - b_A} \quad (6.1)$$

$$\theta_{XA} = f_e \min\theta_{XA} \quad (6.2)$$

The value which are used on ATV program depending on the population and the same value stated on Table 6.9 are taken for f_e value.

Output ammonia concentration;

$$S_{NH} = \frac{K_{NH}(1 + b_A)\theta_{XA}}{\theta_{XA}(\hat{\mu}_a - b_A) - 1} \quad (6.3)$$

$S_{NH} > S_{NH_e}$

This is the first step that targeted treatment yield is checked in term of ammonia. If calculated ammonia nitrogen does not provide the desired quality, then selected denitrification rate or internal recirculation rate should be reviewed. At this stage, how much of the nitrogen in the input flow is oxidized in the aerobic volume and capacity of denitrification should be checked.

The second important point in the system is the growth of heterotrophic organism. Heterotrophs can be in metabolic activities both in aerobic and anoxic environments. Therefore anoxic volume ratio (V_D/V), that shows the activities of heterotrophic organism in total volume, should be determined. From the fact that concentrations of microorganisms are the same in aerobic and anoxic volumes, we can say that volume ratio shows the mass ratio. In order to prevent the deterioration of characteristics of biomass, it is recommended that the V_D/V ratio that is selected, is maximum 0.5 (Orhon et al., 2000).

Total system sludge age is calculated as depending on the V_D/V ratio in below.

$$c = 1 - (1 - \eta) \frac{V_D}{V} \quad (6.4)$$

Total sludge age;

$$\theta_X = \frac{\theta_{XA}}{1 - V_D/V} \quad (6.5)$$

There is no easy degradable organic matter in designed for the removal of nitrogen high sludge age systems during the output stream. Slowly degradable organic matter in the system is considered to be completely hydrolyzed and removed.

$$S_T = S_I + S_P = S_{I1} + cf_{ES}b_H Y_{NH} C_{S1} \theta_X \quad (6.6)$$

$$C_{SI} = f_s C_{TI} \quad (6.7)$$

$$S_{I1} = f_{SI} C_{TI} \quad (6.8)$$

The total oxygen and the necessary biological sludge demand are calculated by formulas for providing the N standard in respectively.

The amount of N within biomass;

$$N_X = (\dot{I}_{XB} + \dot{I}_{XE} f_{EX} b_H c \theta_X) \frac{Y_H}{1 + cb_H \theta_X} C_{S1} \quad (6.9)$$

$$N_{OX} = C_{NDI} + S_{NH1} - S_{NH} - N_X \quad (6.10)$$

Denitrification potential;

$$QN_{DP} = \frac{OR_{HS}}{2.86} + \frac{\eta V_D (OR_H - OR_{HS})}{cV \cdot 2.86} \quad (6.11)$$

Total oxygen demand of system is determined by electron acceptors of heterotrophic and autotroph microorganisms in aerobic reactor. A portion (equivalent of denitrification potential) of the total electron acceptor that needed by heterotrophic microorganisms are supplied by nitrate nitrogen in anoxic volume (Orhon et al., 2000)

Two important parameters that especially considered are oxygen consumption and excess sludge for the design of activated sludge systems.

Oxygen requirement;

$$\frac{OR_{HS}}{Q} = (1 - Y_H)S_{S1} \quad (6.12)$$

$$\frac{OR_H}{Q} = [(1 - Y_H) + (1 - f_{EX} - f_{ES})cb_H\theta_X \frac{Y_H}{1 + b_Hc\theta_X}]C_{S1} \quad (6.13)$$

$$OR_T = OR_H + 4,57 QN_{OX} - 2,86 QN_D \quad (6.14)$$

$$N_D = N_{DP} - R * \frac{S_{OR}}{2.86} \quad (6.15)$$

$$S_{NO} = S_{NO1} + N_{OX} - N_{DP} + R * \frac{S_{OR}}{2.86} \quad (6.16)$$

$$S_{NO} = \frac{N_{OX}}{1 + R} \quad (6.17)$$

Total internal recirculation;

$$R = \frac{N_D}{S_{NO}} = \frac{N_{OX} - S_{NO}}{S_{NO}} \quad (6.18)$$

$$E_D = \frac{N_{OX} - S_{NO}}{N_{OX}} = \frac{R}{1 + R} \quad (6.19)$$

In predenitrification systems while oxidized nitrogen concentration is recirculated from aerobic reactor to anoxic volume, it transport oxygen at the same time. The same amount of organic carbon is consumed as the amount of oxygen transferred with recirculation that results in reduction of electron acceptor requirement which is calculated as N_{DP} in V_D volume. Therefore it is recommended that the selected recycle ratio is maximum 5. In table 6.18, the recycle ratio and change of recirculated nitrogen is given.

$$R = R_I + R_S \quad (6.20)$$

Table 6.18 : The relation between denitrification efficiency and recycle ratio (Orhon,2000).

Recycled Nitrate nitrogen %	Minimum recycle ratio
33	0.5
50	1.0
67	2.0
75	3.0
80	4.0
83	5.0

Total amount of sludge is calculated as total of biological sludge generated in the system and input stream. Design values were taken after primary clarifier.

In activated sludge system, depending on the increasing sludge age, a great decline in the amount of total sludge production is observed. This decline is usually taken as an indicator that internal respiration occurs in the system. The concept of the yield has a constant value that reflects the numerical balance between biomass produced in the activated sludge system and the nutrient that is used. In the activated sludge systems amount of biomass determined by yield is never observed. The observed yield reduces as the result of the affect of internal respiration on growth and it is called net yield (Y_N .) This ratio is not a constant rate , it is an important design parameter that changes inversely proportional to sludge age (Okutman et al.,2010).

In activated sludge system, concept of yield and net yield are defined as Y_H and Y_{NH} for heterotrophic microorganisms and Y_A and Y_{NH} for autotrophic microorganism.

$$Y_{NH} = \frac{Y_H}{1 + cb_H\theta_X} \quad (6.21)$$

$$Y_{NA} = \frac{Y_A}{1 + b_A\theta_x} \quad (6.22)$$

The total amount of sludge in the system;

$$P_X(KOI) = P_{XH} + P_{XE} + P_{XA} + QX_{I1} \quad (6.23)$$

$$P_{XH}(COD) = Y_{NH}QC_{S1} \quad (6.24)$$

$$P_{XE}(COD) = f_{EX}cb_HY_{NH}QC_{S1}\theta_X \quad (6.25)$$

$$P_{XA}(COD) = Y_{NA}QN_{OX} \quad (6.26)$$

$$P_{XAE}(KOl) = f_{EX}b_A Y_{NA}QN_{OX}\theta_{XA} \quad (6.27)$$

$$P_{XI}(KOl) = QX_{I1} \quad (6.28)$$

$$X_F = X_{SS} + X_{VSS} \quad (6.29)$$

$$P_{XF} = Q * X_F \quad (6.30)$$

Total biomass concentration in reactor depends on settling characteristics of the sludge and the relations between aeration and sedimentation tanks. Therefore, X_T value should be selected above 2000 gr SS/m³ to prevent foam formation. The X_T interval is suggested to be 3500-4500 gr SS/m³. If there is a primary sedimentation step in treatment, the interval can be taken as 2500 – 3500 gr/m³ (Orhon et al., 2000).

$$V = \frac{P_{XT}\theta_x}{X_T} \quad (6.31)$$

Hydraulic retention time is chosen between 0,5 and 2 hours for phosphorus removal and it's chosen one hour for anaerobic tank volume. Results is given in Table 6.32.

$$V_{ANAER} = Q * \theta_h \quad (6.32)$$

The data used in design is given Table 6.3. The results obtained formulas above are given in Table 6.19 and Table 6.20.

Table 6.19 : Output data for the conceptual design in different populations (A²O).

Population	θ_x, d	θ_{XA}, d	SF	$\frac{V_D}{V}$	N _{DP} mg/l	N mg/l	R
50,000	9	5.8	1.80	0.32	34.7	13.6	3.1
100,000	8.1	5.5	1.72	0.32	34.5	13.5	3.1
200,000	7.9	4.9	1.53	0.38	32.4	9.6	5.0
400,000	9.4	4.7	1.45	0.49	37.7	10.0	4.9
1,000,000	7.8	4.7	1.45	0.40	29.9	8.9	5.5

Table 6.20 : Output data for the conceptual design in different populations A²O (continue).

Population	Primary Sludge kg/d	Excess Sludge kg/d	Total Sludge kg/d	V _{ANAE} m ³	V m ³	Total V m ³	O _{RT} kg/h
50,000	1,287	1,456	2,743	208	3,549	3,757	75
100,000	2,575	2,856	5,430	417	6,600	7,017	149
200,000	5,610	6,163	11,773	1,042	13,850	14,892	307
400,000	11,827	12,583	24,410	2,333	33,131	35,464	669
1,000,000	31,680	34,754	66,434	6,667	77,783	84,450	1,742

6.5 Conclusions

In our country, different methods are used in the design of municipal treatment. When approaches used in the design are evaluated, it is seen that the assumptions of the numerical values of essential data like influent wastewater characterization, kinetic and stoichiometric coefficients, safety factors may differ from each other. These differences are reflected in the net biomass and oxygen demand calculations and therefore cause different results in determining energy demands.

In order to have a reliable approach in the design, safety factors are used. In the design made with the ATV program, safety factors of 1.8 to 1.45 in A²O process depending on the population (PE 50,000-1,000,000), 4.95 to 4.83 (PE 50,000-1,000,000) in extended aeration systems with nutrient removal, and 5.8 in extended aeration systems are used. In the conceptual design of A²O process, safety factors of 2-2.5 are used. However in order to avoid the differences in safety factors, the safety factors of ATV program were used in the conceptual design of A²O process in this study.

A²O process (ATV), extended aeration process(ATV), extended aeration with nutrient removal process(ATV), conceptual design(A²O) outputs, oxygen consumption and comparisons of the production of biological sludge and required tank volumes are given in Table 6.21, Table 6.22 and Table 6.23.

Extended aeration process requires less equipment and less energy due to its sludge units, the oxygen requirement lead to a need for more aeration and hence more energy requirement. As it can be seen from Table 6.21 and Figure 6.2, the oxygen requirement calculated in conceptual design for predenitrification with phosphorus

removal(A^2O), is approximately 55% more compared to the one calculated with ATV. Extended aeration process requires 20% less oxygen when nutrient removal is obtained. The reason for this is the amount of oxygen recovered from denitrification.

Table 6.21 : Comparison of different process designs in terms of oxygen consumption (kg/h).

Process	50,000	100,000	200,000	400,000	1,000,000
A^2O process (ATV)	121	242	493	1,029	2,732
EA process (ATV)	160	319	677	1,457	3,855
EA+Nutrient (ATV)	137	258	545	1,172	3,118
A^2O process (Conceptual design)	75	149	307	669	1,742

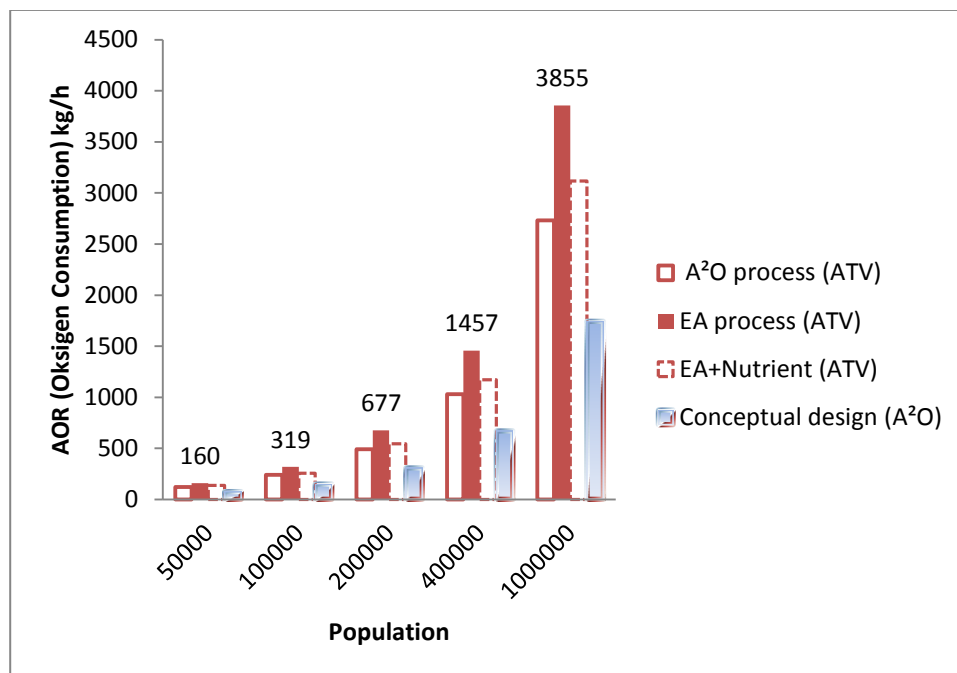


Figure 6.2 : Comparison of different design process based on oxygen consumption

As it can be seen from Table 6.22 and Figure 6.3, extended aeration process produces less sludge compared to the A^2O process regardless of the nutrient removal. When the ATV results and conceptual design results are compared, ATV is found to calculate approximately 10% less sludge. While the sludge age is 9-10 days in ATV, it is calculated as 7-9 in conceptual design.

Table 6.22 : Comparison of different process designs in terms of excess sludge (kg/d).

Process	50,000	100,000	200,000	400,000	1,000,000
A ² O process (ATV)	2,385	4,787	10,345	22,160	58,696
EA process (ATV)	1,579	3,157	6,830	14,536	38,859
EA+Nutrient (ATV)	1,651	3,301	7,328	15,736	41,547
A ² O process (Conceptual design)	2,743	5,430	11,773	24,410	66,434

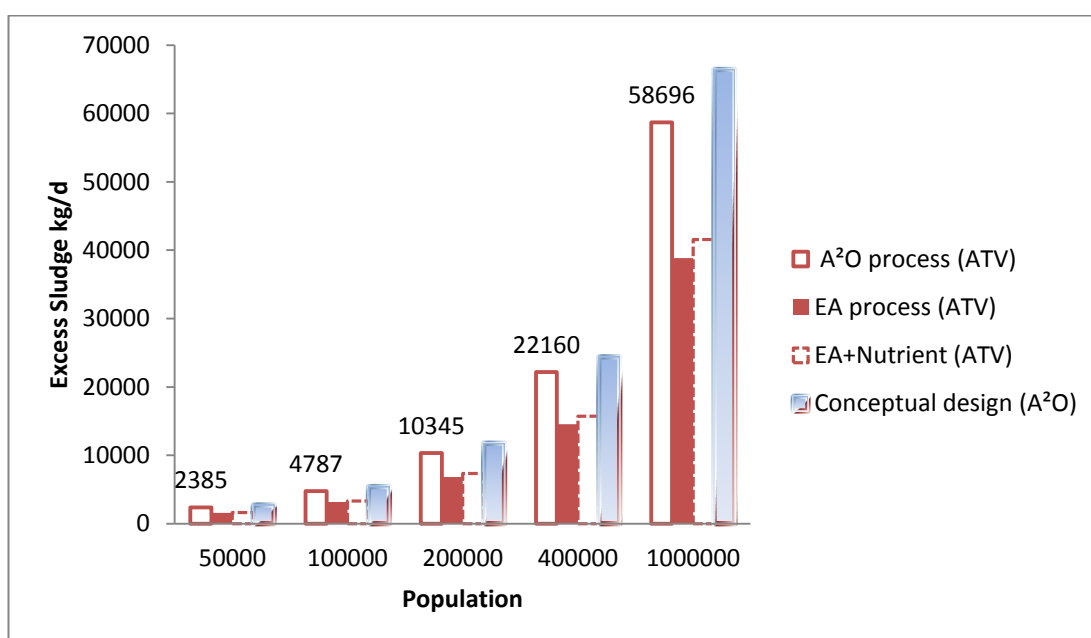


Figure 6.3 : Comparison of different design process based on excess sludge.

As it can be seen from Table 6.23 and Figure 6.4, extended aeration process requires a reactor volume that is approximately 4 times higher due to its high sludge age. The reactor volume for A²O process calculated in ATV is higher than the value obtained in conceptual design.

It can be seen from Table 6.23 and Figure 6.4, as extended aeration process works in high sludge age, reactor volume is 4 times higher. The reactor volume calculated for A²O process in ATV program, has approximately the same values as conceptual design. Extended aeration with nutrient removal process requires approximately 2-4% more reactor volume.

Table 6.23 : Comparison of different process designs in terms of biological reactor volume (m³).

Process	50,000	100,000	200,000	400,000	1,000,000
A ² O process (ATV)	2,963	5,735	13,180	27,395	72,197
EA process (ATV)	9,205	18,410	54,643	96,906	226,583
EA+Nutrient (ATV)	9,070	18,140	53,038	93,573	247,564
A ² O process (Conceptual design)	3,757	7,017	14,892	35,464	84,450

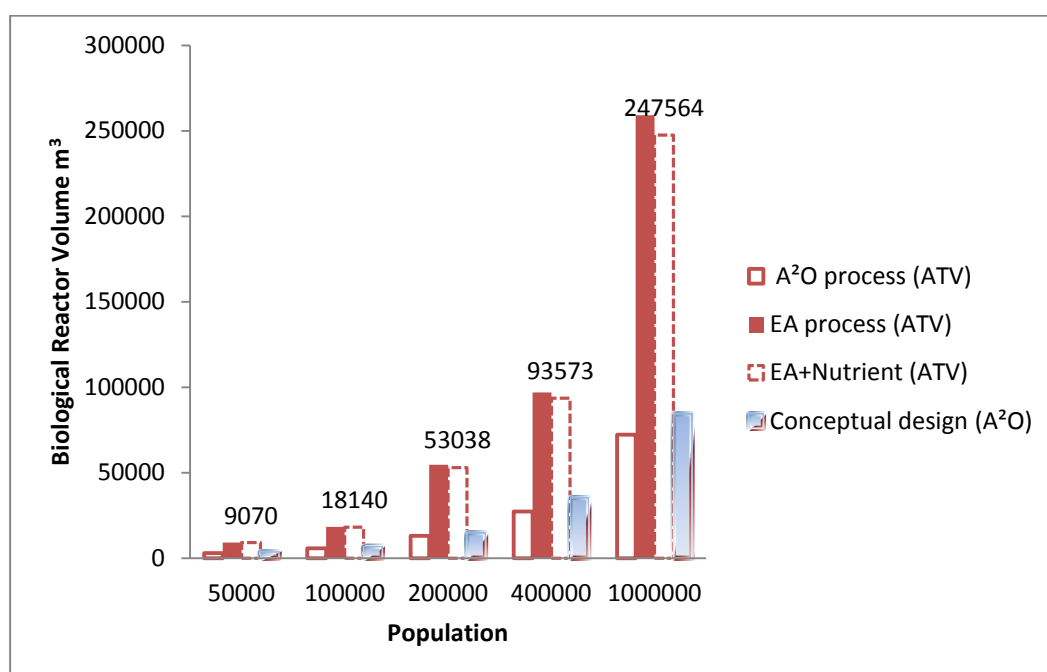


Figure 6.4 : Comparison of different design process based on biological reactor volume.

7. ENERGY DEMAND AND RECOVERY OF MUNICIPAL WASTEWATER TREATMENT PLANTS

7.1 Introduction

The energy requirement of municipal wastewater treatment plants, data of the treatment plants operated in İstanbul and in different cities in Turkey and data of 14 municipal wastewater treatment plants designed by Ministry of Environment and Urbanization and the waste water treatment plants designed in the 5th chapter of this study, are evaluated under 4 titles.

Under first title, 20 biological treatment plants and 4 advanced biological treatment plants operated in Istanbul are compared according to their annual kwh energy consumption per population equivalent (kwh/PE.a) and their energy consumption per m³ wastewater (kwh/m³).

Under the second title, energy consumption of municipal wastewater treatment plants in the cities of Bursa, Fethiye, Siirt, Sivas and Balıkesir, are compared as the annual kwh energy consumption per population equivalent (kwh/PE.a) and kwh energy consumption per m³ wastewater (kwh/m³).

Under the third title, the energy consumption of cities of Seydişehir, Akşehir, Aksaray, Adıyaman, and Diyarbakır, where nutrient removal is made (A²O and extended aeration with nutrient removal); Soma, Polatlı, Ceyhan, Lüleburgaz, and Erzurum where C+N removal is made; Bartın, Merzifon, Çarşamba and Siverek where only carbon removal is made and where they have anaerobic sludge digestion, are compared as annual kwh energy consumption per population equivalent (kwh/PE.a).

Under the fourth title, from the data of the municipal wastewater treatment plants designed in conceptual style in Chapter 5, the energy consumption of A²O and extended aeration processes for 50,000 and 200,000 population equivalent, was calculated as annual kwh energy consumption per population equivalent (kwh/PE.a).

Furthermore, population dependent energy potentials of the municipal wastewater treatment plants are calculated and evaluated by using the sludge data of municipal wastewater treatment plants build in conceptual design.

7.2 Overview of the use of electricity in urban wastewater treatment in Turkey

Benchmarking and continuous improvement studies have been made about the sharing of information can not be done about the energy used by treatment plants operated by our country. Although, energy consumption of wastewater treatment plant depends on its location, strength of wastewater, level of treatment, type of treatment process selected, and mode of operation, hydraulic conditions, in-plant energy recovery design. Comparison of the performance criteria of the same types treatments should be sharing continuous improvement stages.

Minimum energy consumption should be planned at the design stage. One of the issues to be taken into account in the operation of wastewater collection and treatment systems, is to plan adapting to energy consumption of wastewater loads changes during the day. The pollution load changes in a day is given in third chapter (Figure 3.4). While, energy requirements are evaluated in the same type, similar distribution is also obtained to valid for energy requirement. When, wastewater charges are changed throughout the day, aeration requirements will also change. Therefore, aeration units should not run continuously at full capacity, it should run to adapt to load changes.

Operational problems encountered and matters that are increasing energy consumption should be analyzed and taken into consideration for future design. Particularly, pumps and aeration are the highest energy consumption steps. So, improvement plans for operation on these issues are to be established at the design stage. For this reason, during the planning of wastewater treatment plants, the area where plant will be installed should be checked for appropriate hydrolic condition. Approximately, 62% (Demir,2010) of the energy used by ISKI for Istanbul is used in pumping stations. If wastewater should be transported by gravity or less pumping power, energy savings during operation of wastewater treatment plant will be provided on a large scale.

There are 24 wastewater treatment plant in Istanbul. Energy consumed by these plants is about 5% of the total energy used in Istanbul. 13% of the energy consumed

is used for wastewater treatment facilities. In addition, Istanbul has 20 biological and 4 advanced biological wastewater treatment plants. Typical profiles of different types of waste water treatment plants in Istanbul for annual energy use are below. The majority of the energy is consumed by pumps in wastewater pretreatment facilities.

Biological treatment applications includes pretreatment, aeration tank, final sedimentation, and sludge treatment units. Gümüşyaka treatment plant is activated sludge system with sequencing batch reactor (SBR). Energy consumption values of Biological treatment facilities, located in Istanbul is given in Table 7.1 and Figure 7.1.

Table 7.1 : Consumed energy for biological WWTP in Istanbul.

Biological TP	Population (1)	(IWSA annual report, 2011)			Energy cons. kwh/PE.a
		Mean flow (m ³ /d)	Energycons. (kwh-a)	EnergyCons (kwh/m ³)	
Bahçeşehir	26,000	7,833	956,445	0.33	37
Gümüşyaka	6,800	3,104	254,796	0.22	37
Çantaköy	15,000	2,063	327,554	0.44	22
Akalan	1,103	315	48,665	0.42	44
Belgrat	395	58	24,168	1.14	61
Örencik	1,000	175	37,646	0.59	38
Kestanelik	2,150	476	169,619	0.98	79
Örcünlü	460	144	45,652	0.87	99
Çanakça	2,310	389	103,857	0.73	45
İzzettin	1,050	378	65,172	0.47	62
İhsaniye	1,700	204	50,170	0.67	30
Başakköy	430	172	54,890	0.87	128
Oklalı Köyü	1,250	137	59,039	1.18	47
Yazlık	500	137	53,000	1.06	106
Ömerli	2,000	474	118,699	0.69	59
Ağva	25,169	2,565	427,805	0.46	17
Kömürlük	500	173	37,099	0.59	74
Sahilköy	2,000	100	73,945	2.02	37
Yeniköy	800	85	23,526	0.76	29
Öğümce	520	31	32,203	2.85	62

(1) Populations are taken from İSKİ and yerelnet.gov.tr web site.

As it is not possible in İstanbul to match wastewater catchment with population in some areas, it is better to evaluate energy consumption per m³ and kwh/PE.a together.

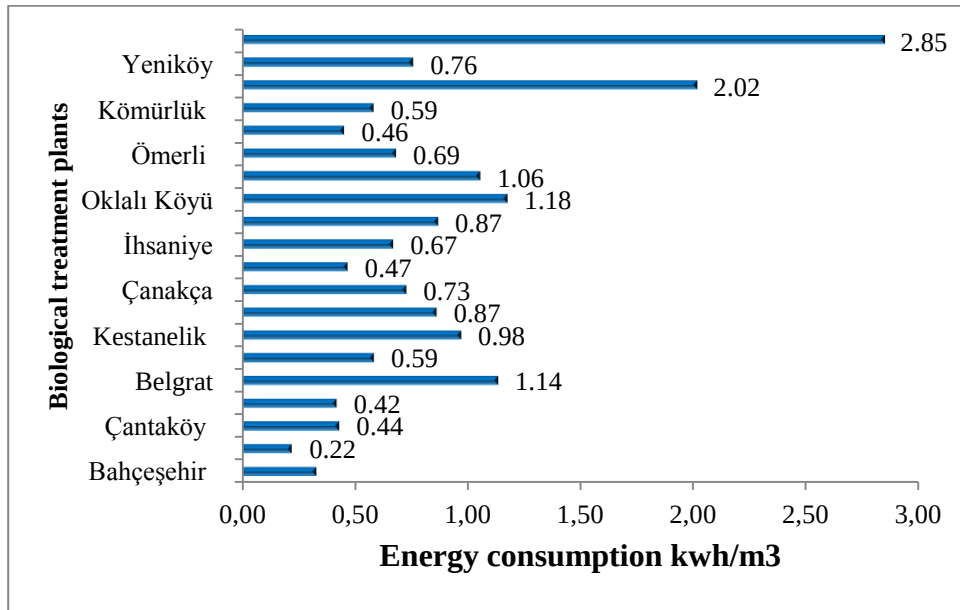


Figure 7.1 : Energy consumption of Istanbul municipal biological WWTP.

Advanced biological treatment plants in Istanbul consist of pretreatment, anaerobic, anoxic and aerobic tanks, secondary sedimentation, sludge treatment units, odor removal, and energy production units. In three of these advanced biological plants (Ataköy, Tuzla, Paşaköy), there are electric and heat producing cogeneration units with different capacities. Energy consumption of these facilities are given in Table 7.2 and Figure 7.2. As shown in Figure 7.2, capacity and pumping stations affects the amount of energy consumption.

Table 7.2 : Consumed energy for advanced biological WWTP in Istanbul.

(IWSA annual report, 2011)					
Plant	Population	Mean flow m ³ /d	Energy cons. kwh-a	Energy Cons. kwh/m ³	Energy cons. kwh/PE.a
Ataköy	2,500,000	320,808	44,324,248	0.38	17.73
Terkos	7,000	1,465	131,943	0.25	18.85
Tuzla	1,000,000	308,608	31,599,283	0.28	31.60
Paşaköy	830,000 ¹	123,405	16,469,452	0.37	19.84

⁽¹⁾Berktaş, 2008

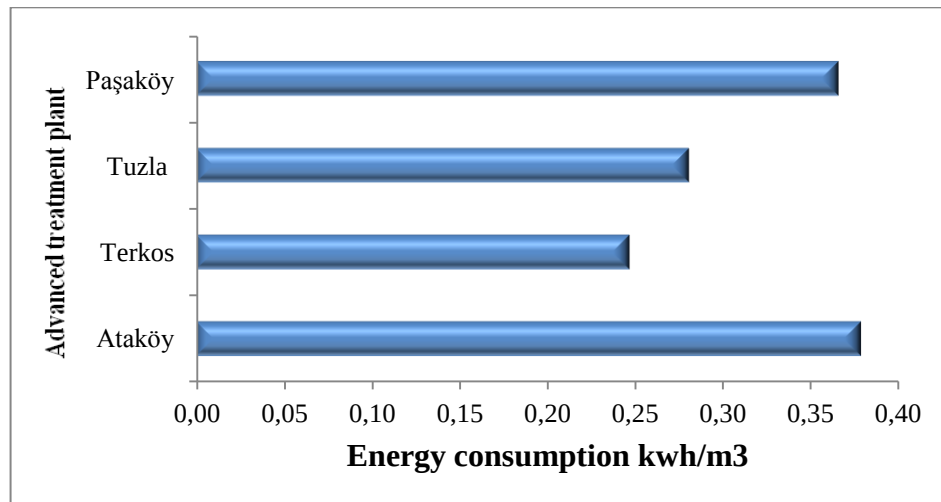


Figure 7.2 : Energy consumption of Istanbul Advanced Biological WWTP.

Treatment units distribution of consumed energy for Advanced Biological Wastewater Treatment Plant Paşaköy is given in below. The most widely used energy units and the energy consumption is respectively measured aeration (38%), inlet pumps (19%), sludge dewatering (15%), sludge drying (13%), secondary settlement (12%). Odor removal (1%) and disinfection system (1%) have the least energy consumption (Demir, 2010).

As described in Table 7.1, Table 7.2, advanced wastewater treatment plants require even greater amounts of electric energy. When the biological treatment and advanced biological treatment plants in İstanbul are compared, even though the treatment type is the same, different results are obtained. Generally, unit costs declined with increasing capacity but the cost varies depending on the pump station. It is necessary to evaluate the energy consumption of the plants in plant basis, and parallel to that, a comparison which allows an analysis of their performance criteria should be made, obtained results should be evaluated.

When we look at the unit energy consumption of other treatment plants, energy consumption of plants with different types of treatment in the cities of Balıkesir, Bursa, Fethiye, Siirt and Sivas are given below.

Balıkesir wastewater treatment plant consists of pretreatment, biological treatment (trickling filter) sludge dewatering systems. Treatment units contain respectively coarse screens, fine screens, grit removal, primary sedimentation, trickling filter pumps, trickling filter, secondary sedimentation, sludge stabilization tank, blower for

sludge, sludge stabilization unit sludge dewatering. Energy consumption values are given as COD, flow rate and the equivalent population in Table 7.3.

Table 7.3 : Balıkesir WWTP Energy consumption values.

Balıkesir	Sistemyapı,2008		Specific Energy Cons.	Unit Energy Cons.
	Flow	Energy Cons.		
	m ³ /day	kwh/year		
January	49,245	3,159,499	0.18	13,16
February	47,140	3,205,399	0.19	13,36
March	54,720	3,114,328	0.16	12,98
April	49,645	2,922,845	0.16	12,18
May	56,770	2,756,986	0.13	11,49
June	53,259	3,190,142	0.16	13,29
July	44,760	3,313,615	0.20	13,81
August	38,175	3,383,848	0.24	14,10
September	49,245	3,432,338	0.19	14,30
October	48,432	3,400,259	0.19	14,17
November	52,230	3,382,555	0.18	14,09
December	51,365	3,306,506	0.18	13,78
Average	49,582	3,214,027	0.18	13,39

Bursa wastewater treatment plant consists of pretreatment, biological treatment and sludge dewatering systems. After the pretreatment unit that consists pumps, screens, grid removal and flow measurement equipments, treatment units go to selector tank and biological treatment begins. During biological treatment process that consists of selector, anaerobic, anoxic and aerobic tank, after last sedimentation it is discharged. Sludge units consist of sludge grit, sludge stabilization tank, belt type condenser, belt press sludge dewatering and sludge stabilization. 2009 energy consumption values of Bursa wastewater treatment plant is given as COD, flowrate and population equivalent in Table 7.4.

Table 7.4 : Bursa WWTP Energy consumption values.

Bursa	Sistemyapı,2009		Specific Energy Cons. forFlow kwh/m ³	Unit Energy Cons. kwh/PE.a
	Flow m ³ /day	Energy Cons. kwh/year		
January	186,472	23,346,683	0.34	19,46
February	206,604	22,299,894	0.30	18,58
March	206,217	22,393,752	0.24	18,66
April	199,320	23,938,902	0.30	19,95
May	184,797	23,897,596	0.32	19,91
June	182,926	24,727,015	0.37	20,61
July	152,259	22,318,905	0.36	18,60
August	169,678	23,952,103	0.39	19,96
September	176,991	23,694,788	0.37	19,75
October	177,191	23,924,850	0.37	19,94
November	188,798	23,374,570	0.32	19,48
December	198,320	24,097,143	0.32	20,08
Average	185,798	23,497,183	0.33	19,58

Fethiye wastewater treatment plant consists of pretreatment, biological treatment and sludge dewatering systems. The values of the energy consumption of Fethiye WWTP are given as flow rate and the equivalent population in Table 7.5.

Table 7.5 : Fethiye WWTP Energy consumption values.

Fethiye	Sistemyapı,2009		Specific Energy Cons kwh/m ³	Unit Energy Cons. kwh/(PE).a
	Flow m ³ /day	Energy Cons. kwh/year		
January	42,172	4,277,070	0.28	30.60
February	45,288	4,333,280	0.26	31.00
March	39,255	3,970,470	0.28	28.40
April	35,830	3,864,985	0.30	27.65
May	33,557	3,969,375	0.32	28.40
June	27,167	3,686,500	0.37	26.37
July	25,750	4,018,285	0.43	28.75
August	25,454	3,557,290	0.38	25.45
September	28,068	4,539,505	0.44	32.47
October	26,829	4,263,565	0.44	30.50
November	30,568	4,441,320	0.40	31.77
December	44,354	4,802,670	0.30	34.36
Average	33,691	4,143,693	0.35	29.64

Siirt advanced biological wastewater treatment plant consists of coarse, fine screen, inlet pumping station, grit removal with blower, biological treatment, sludge dewatering systems. For Siirt daily capacity of wastewater treatment plant is given as 20,178 m³ values of the energy consumption of Siirt WWTP are given as flow rate and the equivalent population in Table 7.6.

Table 7.6 : Siirt WWTP Energy consumption values.

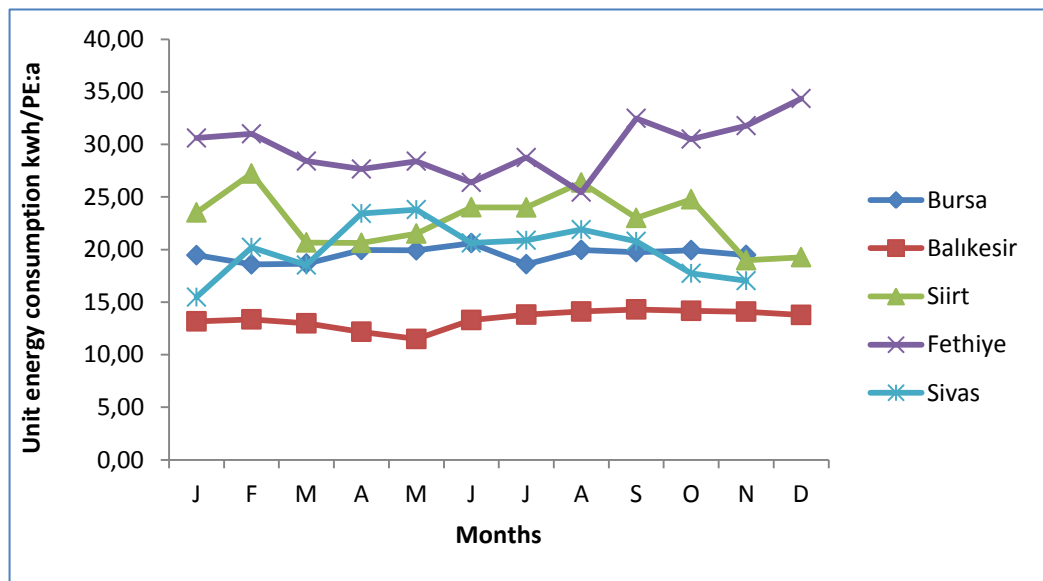
Siirt	Sistemyapı, 2009			
	Flow	Energy Cons.	Specific Energy Cons.	Unit Energy Cons.
	m ³ /day	kwh/year	kwh/m ³	kwh/PE.a
January	1,716	1,941,800	3.10	23.52
February	10,109	2,246,940	0.61	27.22
March	9,318	1,706,375	0.50	20.67
April	13,335	1,702,725	0.35	20.63
May	15,494	1,776,090	0.31	21.52
June	14,811	1,982,680	0.37	24.02
July	14,715	1,980,855	0,37	24.00
August	13,805	2,176,495	0,43	26.37
September	13,473	1,899,095	0.39	23.01
October	13,261	2,045,095	0.42	24.77
November	13,387	1,566,945	0.32	18.98
December	12,330	1,590,670	0.35	19.27
Average	12,146	1,884,647	0.36	22.83

Sivas advanced wastewater treatment plant consists of pretreatment, biological treatment (aerated activated sludge) and sludge dewatering units. The values of the energy consumption of Sivas WWTP are given as flow rate and the equivalent population in Table 7.7.

Table 7.7 : Sivas WWTP Energy consumption values.

Sivas	Sistemyapı,2009		Specific Energy Cons. kwh/m ³	Unit Energy Cons. kwh/(PE).a
	Flow	Energy Cons.		
	m ³ /day	kwh/year		
January	-	-	-	-
February	50,358	3,641,915	0.20	15.47
March	67,395	4,757,740	0.19	20.21
April	65,000	4,359,545	0.18	18.52
May	65,623	5,514,201	0.23	23.42
June	59,385	5,602,808	0.26	23.80
July	46,406	4,853,651	0.29	20.61
August	46,288	4,915,367	0.29	20.88
September	48,036	5,155,019	0.29	21.89
October	47,097	4,890,666	0.28	20.77
November	52,049	4,173,516	0.22	17.73
December	41,814	4,009,809	0.26	17.03
Average	53,586	4,715,840	0.25	20.03

Changes of energy consumption in municipal wastewater plants depending on months are given in Figure7.3.

**Figure 7.3 : Changing of energyconsumptionin WWTP of different cities.**

When we look at the unit energy consumption in the cities of Balıkesir, Bursa, Fethiye, Siirt and Sivas, which are operated, it is observed that unit consumption in Fethiye is higher. When we look at the distribution of monthly energy use, it is observed that the difference in the population in summer and winter, is not reflected in the results. the energy consumptions of municipal treatment plants are given in Table 7.8.

Table 7.8 : WWTP average energy consumption in different cities.

	Flow (Debi)	Population Equivalent	Specific Energy Cons.	Unit Energy Cons.	N/ COD
	m ³ /day	PE	kwh/m ³	kwh/(PE).a	
Balıkesir	49,582	240,000	0.18	13.39	na
Bursa	185,798	1,200,000	0.33	19.58	0.11
Fethiye	33,691	139,789	0.36	29.64	0.07
Siirt	12,146	82,550	0.36	22.83	0.11
Sivas	53,586	235,457	0.25	20.03	0.10

The energy consumption values of cities askwh/m³ and kwh/PE.a are given in Figure 7.4 and Figure 7.5 respectively.

In the raw wastewater, the energy requirement of nutrient removal plants is strongly linked with N:COD ratio, because of oxygen consumption for nitrification, and the need for reduced COD removal by primary sedimentation in case of a high rate of N:COD ratio. Thus, a low ratio of N:COD is an advantage both for nitrogen removal and for low energy demand. A minimum requirement of external electric power of 9 kwh/(pe.a) can be issued from theory for a nutrient removal plant with an N:COD ratio of 0.10 in the influent, on the other hand for treatment plants with lower N:COD ratios of about 0.08 the energy requirement from the grid can be as low as 5 kwh/(pe.a) (Nowak, 2003).

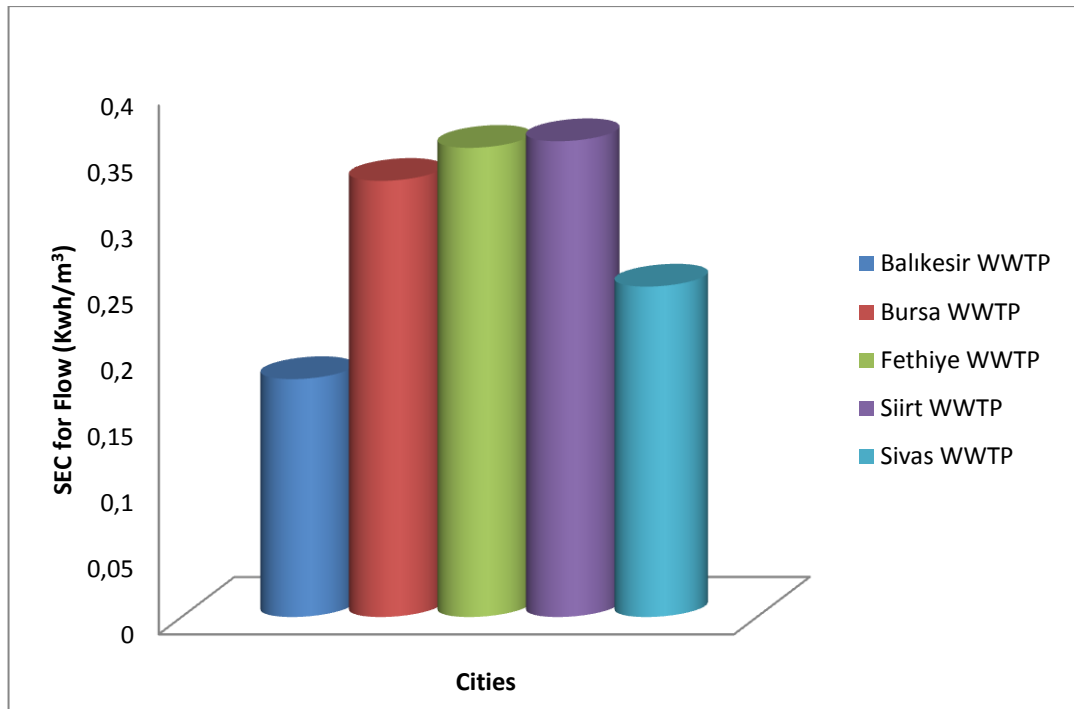


Figure 7.4 : Specific Energy Cons. for flow versus in different cities WWTP.

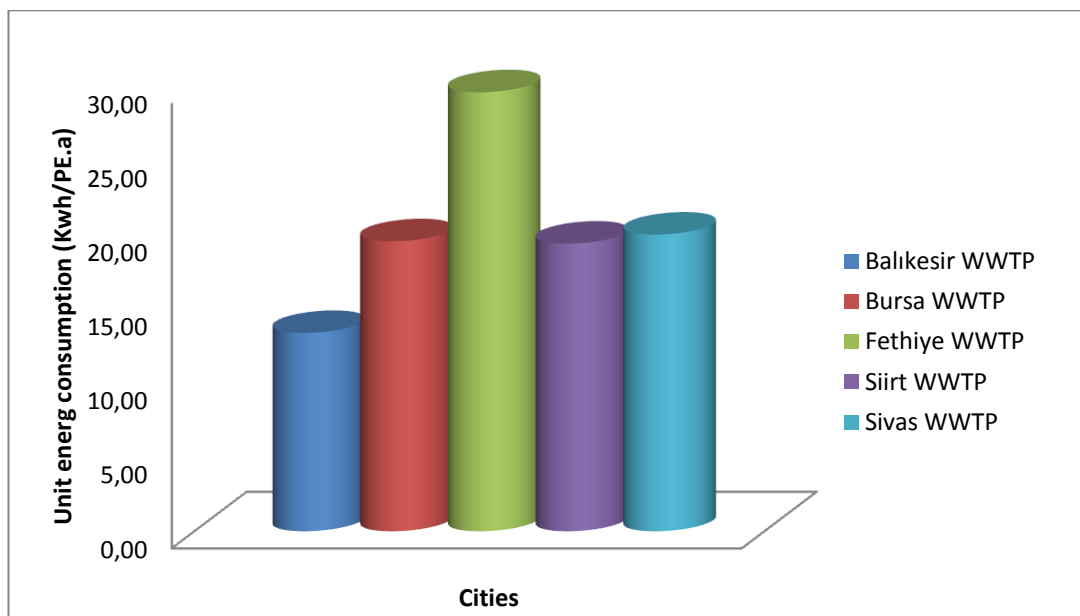


Figure 7.5 : Specific Energy Cons. for population equivalent in different cities WWTP.

As it can be seen in Figure 7.4, the energy consumption in the municipal wastewater treatment plants operated in five different cities, range between 0.18-0.36 kwh/m³ , and are reduced with increasing population.

7.3 Biogas production and energy recovery from municipal wastewater treatment plant design in Turkey

Within this project, municipal wastewater treatment plants for 15 cities were projected. The wastewater treatment process for each city was chosen according to the limits of the receiving water, where the treated water is discharged. Among these cities, only in one city, direct discharge to the receiving water is planned. The other cities are planned, taking into account the limits of the receiving water, with binary preference of 3 different processes.

Among these cities, for cities of Seydişehir, Akşehir, Aksaray, Adıyaman, and Diyarbakır, where nutrient removal is made, A²O and extended aeration with nutrient removal alternatives, for Soma, Polatlı, Ceyhan, Lüleburgaz and Erzurum where C+N removal is made, N removal conventional activated sludge and activated sludge with extended aeration alternatives, for Bartın, Merzifon, Çarşamba and Siverek where only carbon removal is made, aerobic and anaerobic sludge digester alternatives were compared according to their annual kWh energy consumption and annual kWh energy consumption per population equivalent.

The results of two treatment alternatives were determined and compared with respect to unit COD and population for energy demand of the wastewater treatment plants.

The units and flow chart of A²O and extended aeration processes, which are discussed mainly in this study, are given below. The units belong to the other two processes are given in the process explanations of the related cities.

1. Three-stage “Phoredox” (A²O) process for biological N and P removal

Biological nutrient removal consists of screening, grit and grease removal, primary sedimentation, activated sludge tanks with anaerobic, anoxic and aerobic zones, secondary sedimentation, sludge thickening, anaerobic sludge digestion, sludge dewatering, gas utilisation facilities, energy recovery system units. A schematic arrangement of the BNR process is shown in Figure 7.6, Figure 7.7 and Figure 7.8.

2. Extended aeration activated sludge system

Extended aeration activated sludge system consists of screening, grit and grease removal, extended aeration activated sludge tanks, secondary clarifiers, sludge thickening, sludge dewatering unitlerinden oluşmaktadır. A schematic arrangement of the extended aeration treatment plant is shown in Figure 7.9.

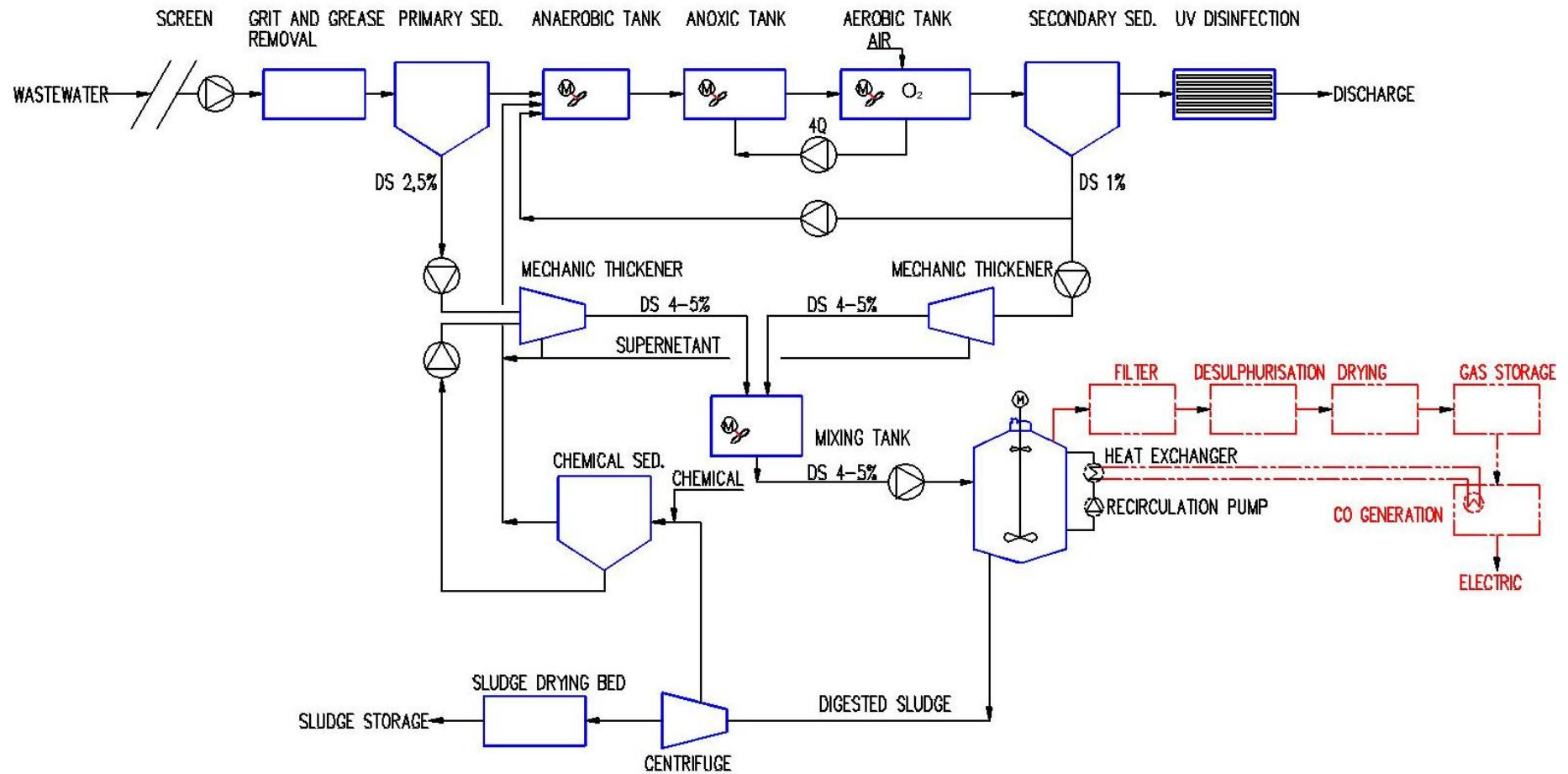


Figure 7.6 : Biological nutrient removal A²O process flow chart.

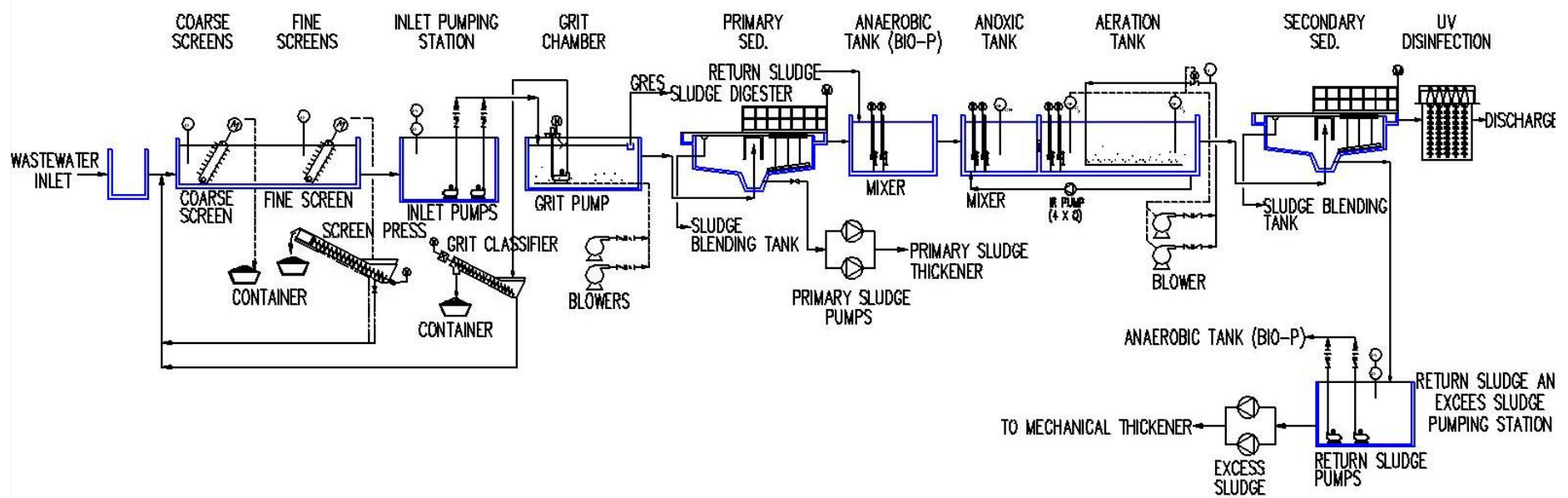


Figure 7.7 : Biological nutrient removal A²O process energy usage equipment (main stream).

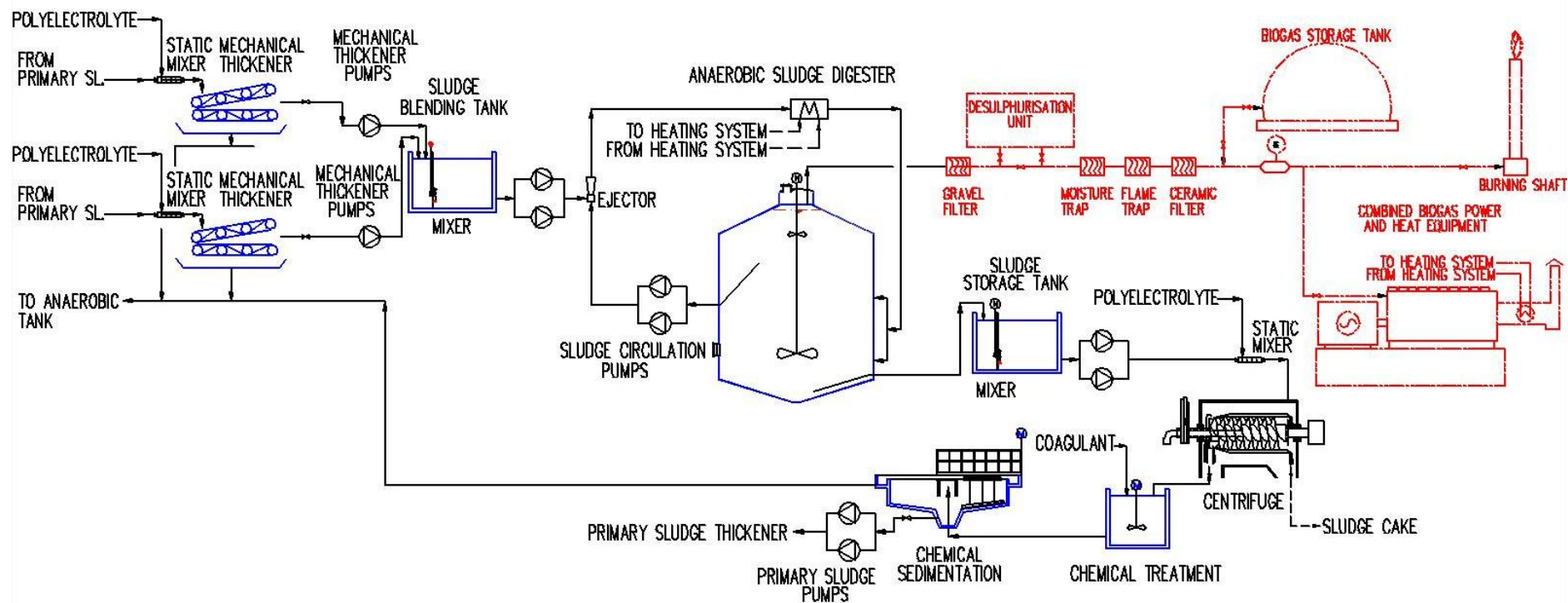


Figure 7.8 : Biological nutrient removal A²O process energy usage equipment (sludge stream).

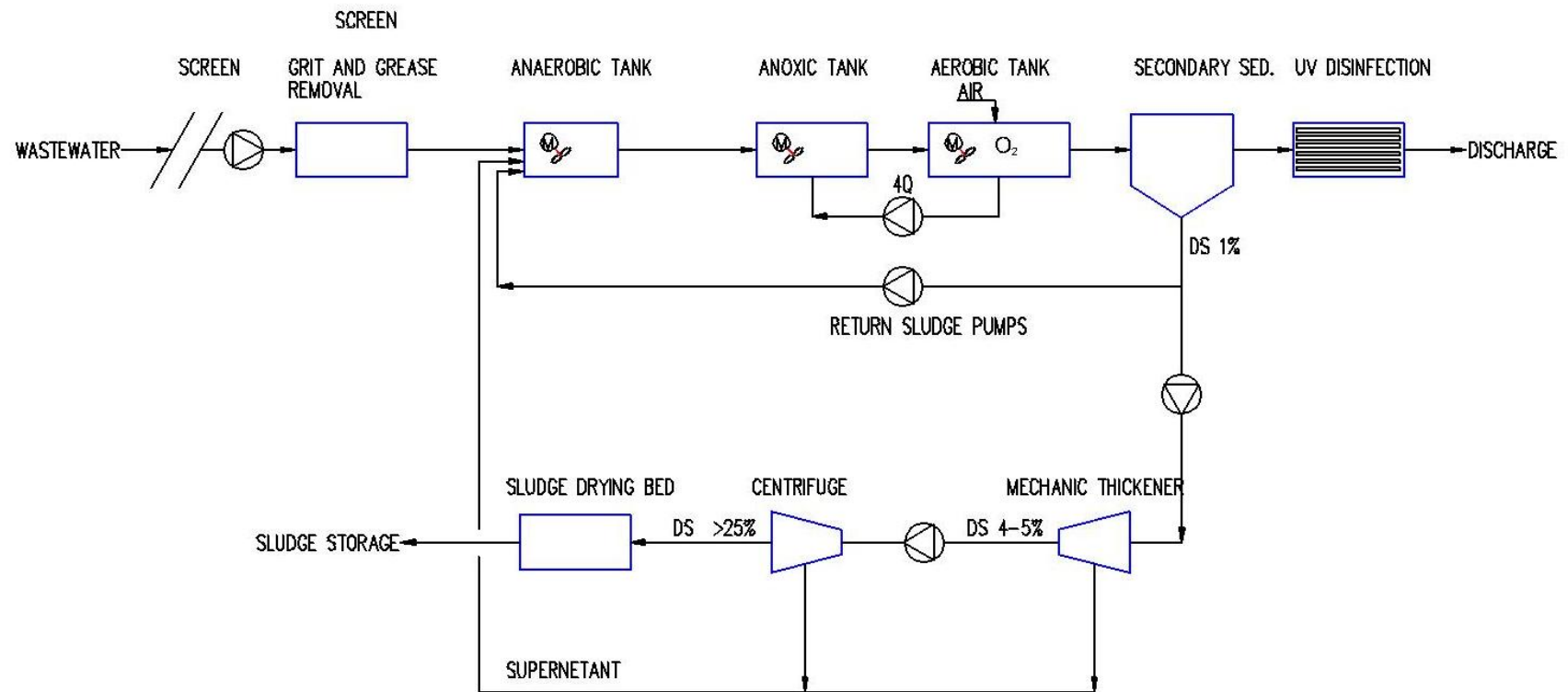


Figure 7.9 : Extended aeration process with nutrient removal flow chart.

Wastewater treatment plants in 14 different cities (Seydisehir, Bartin, Merzifon, Aksehir, Çarşamba, Soma, Polatli, Siverek, Ceyhan, Luleburgaz, Aksaray, Adiyaman, Erzurum, Diyarbakir) that were projected by the Ministry of Environment and Urbanization were evaluated in terms of energy requirement. Treatment units are classified according to the energy distributions.

Energy requirement is evaluated separately for each process. In addition specifying the pumps in a title, rates in the system of the pumps, which are used, is examined. Pumping title consists of wastewater and sludge transfer pumping as well as hot water circuit pumps, polydosage. The energy requirements of all pumps used in the design of domestic wastewater treatment plants were collected under the pumps title. Each of the energy requirements of domestic wastewater treatment plant are given in tables and graphs for the following cities in accordance with Table 7.9. Data are not included in sludge disposal process after dewatering.

Table 7.9 : Equipment commonly used in wastewater treatment facilities requiring electrical energy.

Unit		Functionality
Pumping	M	Inlet pumps
Physical Treatment	M	Coarse screen, conveyor , grit chamber blowers, primary clarifier scraper, UV disinfection,
	S	Fine screen (mechanical clean), conveyor, grit chamber bridge, grit classifier, Grit pumps, scum pumps, grease pumps, primary clarifier scraper, , screen press, secondary clarifier scraper, mixer, grace and grit chamber, blower, scrapers for primary sedimentation tank, primary clarifier sludge pumps,
Biological Treatment	M	Activated tank blower, activated tank mixers,
	S	Aeration fan, monorail, crane, secondary sedimentation internal recirculation pumps, return sludge pumps, excess sludge pumps,
Sludge Thickening And Dewatering	M	Centrifugal decantor , sludge blending tank mixers, chemical building
	S	Belt thickener, gravity thickener scraper, conveyor, poly preperation, phosphorus removal, sludge decantor, sludge dewatering filtrate pumps, gravity thickener sludge pumps, poly dosage pumps, belt thickener pumps, sludge blending tank pumps,
Anaerobic Digester Sludge	M	Anaerobic digester (inclusive, mixer), digester sludge circulation pumps, heating distribution (hot water circuit pumps),
Gas Part	M	Power generator, desulfurization unit,
	S	Biogas storage tank, CHP unit, boiler, heat exchanger, flare, gas holder

M: Main, S: Sub

According to municipal wastewater treatment plant given below, the distribution of energy consumption contain also maintenance, employee buildings for repair process, laboratories, and additional processes that require energy, administration building. All of these buildings consume energy in winter and summer, due to weather condition for heating or cooling. Electric energy is used for lighting of connected walkways, working areas and outdoors. These additional processes are grouped under another title.

Adiyaman biological nutrient removal WWTP (alternative 1) consists of screening, grit and grease removal, primary sedimentation, activated sludge tanks with anaerobic, anoxic and aerobic zones, secondary sedimentation, sludge thickening, anaerobic sludge digestion, sludge dewatering, gas utilisation facilities, energy recovery system units.

Adiyaman extended aeration with nutrient removal WWTP (alternative 2) consist of screening, grit and grease removal, anaerobic tank, extended aeration activated sludge tank with anoxic zone, secondary clarifiers, sludge thickening, sludge dewatering units. Both of alternative design outputs are given in comparison Table 7.10, Figure 7.10 and Figure 7.11.

The amount of energy consumed by all the pumps used in Adiyaman WWTP is calculated as 4579 kwh/d for A²O (alternative 1) and 4110 kwh/d for extended aeration(alternative 2). This constitutes approximately 25% of the total energy demand.

Table 7.10 : Comparison of electricity requirements for Adıyaman wastewater treatment.

Units	Power Cons. (A ² O) kw/d	Power Cons. (EANR) kw/d	Power Percent (A ² O) %	Power Percent (EANR) %	Power Cons. (A ² O) kwh/PE.a	Power Cons. (EANR) kwh/PE.a
Inlet Pumping Station	2,112	2,112	11.65	10.16	3.14	3.14
Coarse Screen	67	67	0.37	0.32	0.10	0.10
Fine Screen	45	45	0.25	0.22	0.07	0.07
Grit and Grease with blower	468	468	2.58	2.25	0.70	0.70
Primary Sedimentation	111		0.61		0.17	
Activated Sludge	11,257	15,429	62.10	74.23	16.74	22.94
Secondary	1568	1558	8.65	7.50	2.33	2.32
Mechanical Thickener		474		2.28		0.70
Gravity Thickener	21		0.12		0.03	
Anaerobic Digester	1,017		5.61		1.51	
Mechanical Thickener	308		1.70		0.46	
Sludge Dewatering Centrifuge	371	627	2.05	3.02	0.55	0.93
Anaerobic Digester (Gas Part)	775		4.28		1.15	
Other	6	4	0.03	0.02	0.01	0.01
Toplam	18,126	20,784	100	100	26.95	30.90

EANR: Extended aeration with nutrient removal

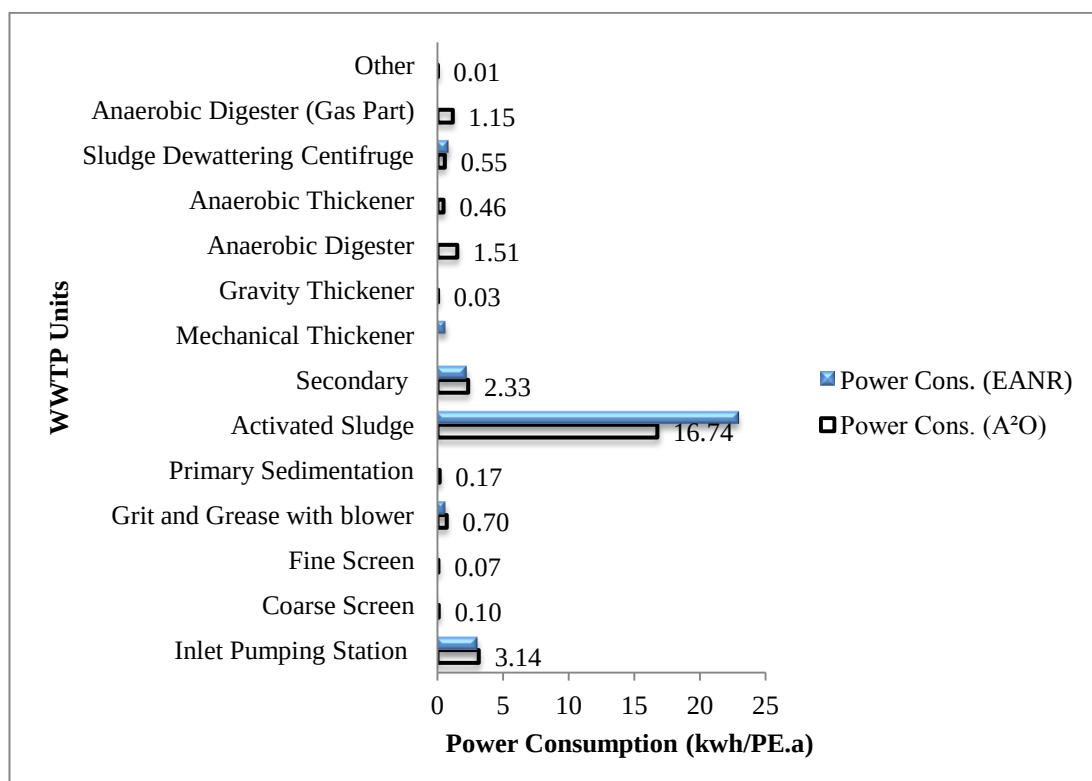


Figure 7.10 : Comparison of energy use profile for Adıyaman WWTP.

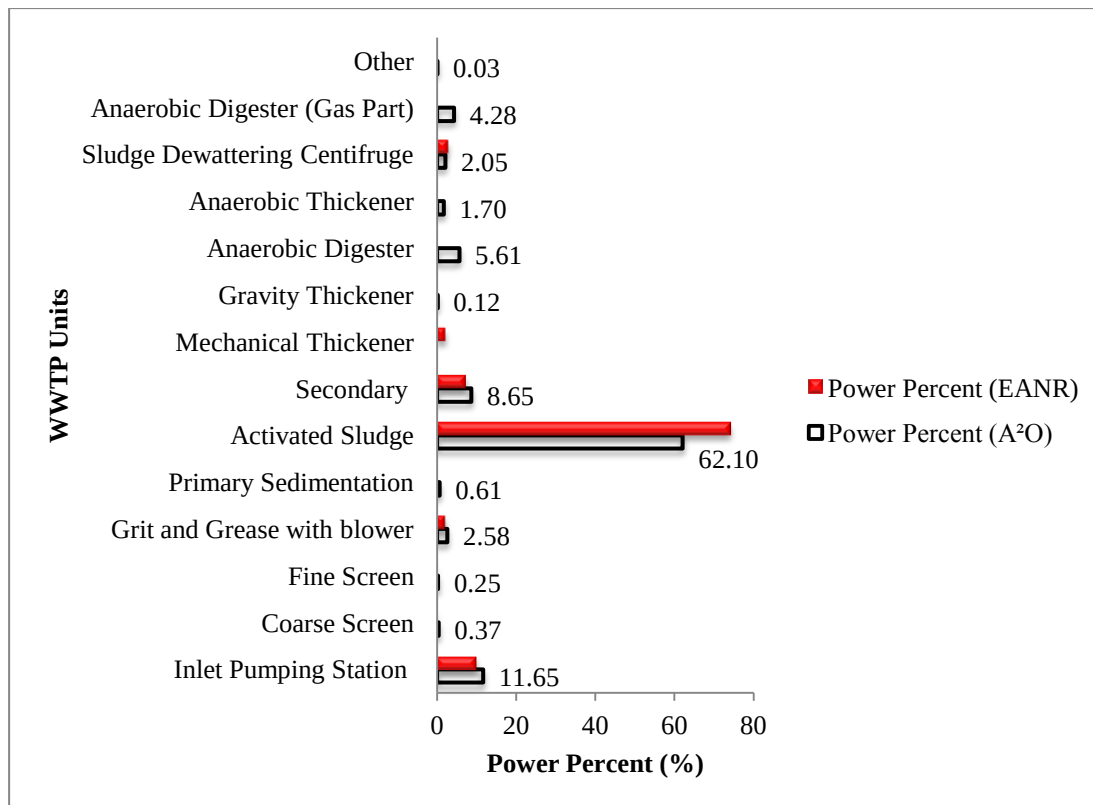


Figure 7.11 : Comparison of unit energy use profile for Adiyaman WWTP.

Aksaray biological nutrient removal WWTP (alternative 1) consists of screening, grit and grease removal, primary sedimentation, activated sludge tanks with anaerobic, anoxic and aerobic zones, secondary sedimentation, sludge thickening, anaerobic sludge digestion, sludge dewatering, gas utilisation facilities, energy recovery units.

Aksaray extended aeration with nutrient removal active sludge system (alternative 2) consists of screening, grit and grease removal, anaerobic mixing tank, extended aeration activated sludge tank with anoxic zone, secondary clarifiers, sludge thickening, sludge dewatering units. Both of alternative design outputs are given in comparison Table 7.11, Figure 7.12 and 7.13.

Table 7.11 : Comparison of electricity requirements for Aksaray wastewater treatment plant.

Units	Power Cons. (A ² O) kw/d	Power Cons. (EANR) kw/d	Power Percent (A ² O) %	Power Percent (EANR) %	Power Cons. (A ² O) kwh/PE.a	Power Cons. (EANR) kwh/PE.a
Inlet Pumping Station	1,728	1,728	11.45	10.57	3.13	3.13
Coarse Screen	39	39	0.26	0.24	0.07	0.07
Fine Screen	34	34	0.23	0.21	0.06	0.06
Grit and grease removal (with blower)	460	460	3.05	2.81	0.83	0.83
Primary Sedimentation	111		0.74		0.20	
Activated Sludge	8,924	11,768	59.15	71.98	16.14	21.29
Secondary Sedimentation	1,298	1,275	4.42	7.80	2.35	2.31
Gravity thickener (primary sludge)	232		1.54		0.42	
Gravity thickener (digester sludge)	229		1.52		0.41	
Mechanical thickener		222	0.00	1.36	0.00	0.40
Anaerobic digester	975		6.46		1.76	
Centrifuge	714	482	4.73	2.95	1.29	0.87
UV disinfection	336	336	2.23	2.06	0.61	0.61
Other	6	6	0.04	0.04	0.01	0.01
Total	15,086	16,350	100	100	27.29	29.58

EANR: Extended aeration with nutrient removal

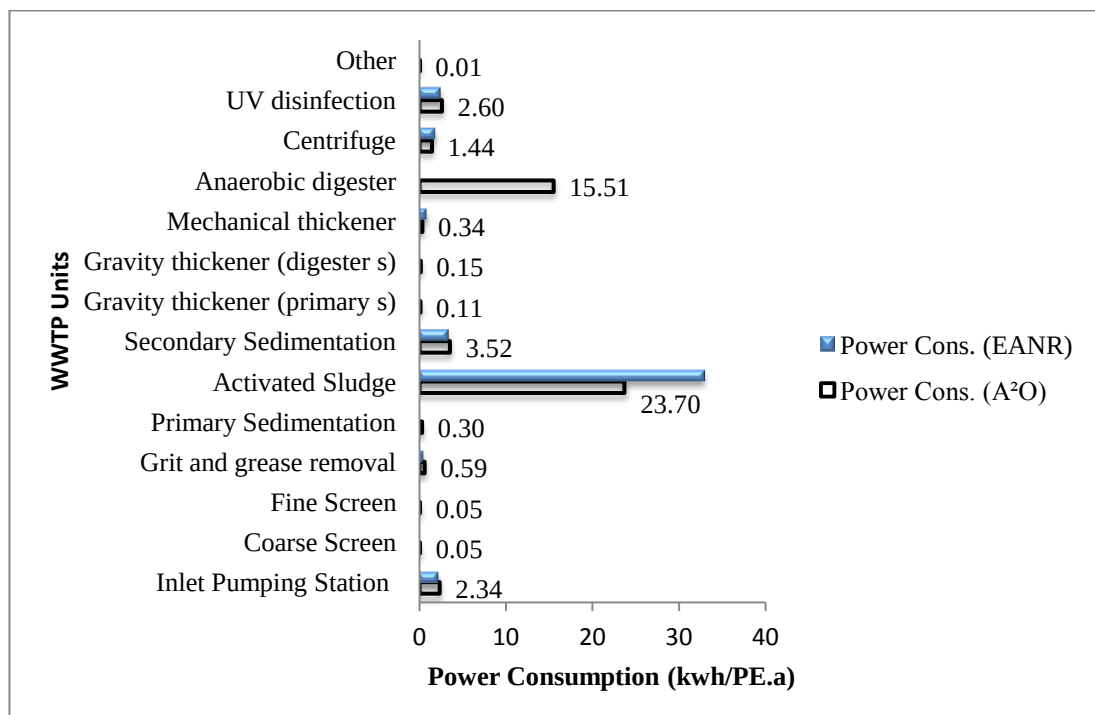


Figure 7.12 : Comparison of energy use profile for Aksaray WWTP.

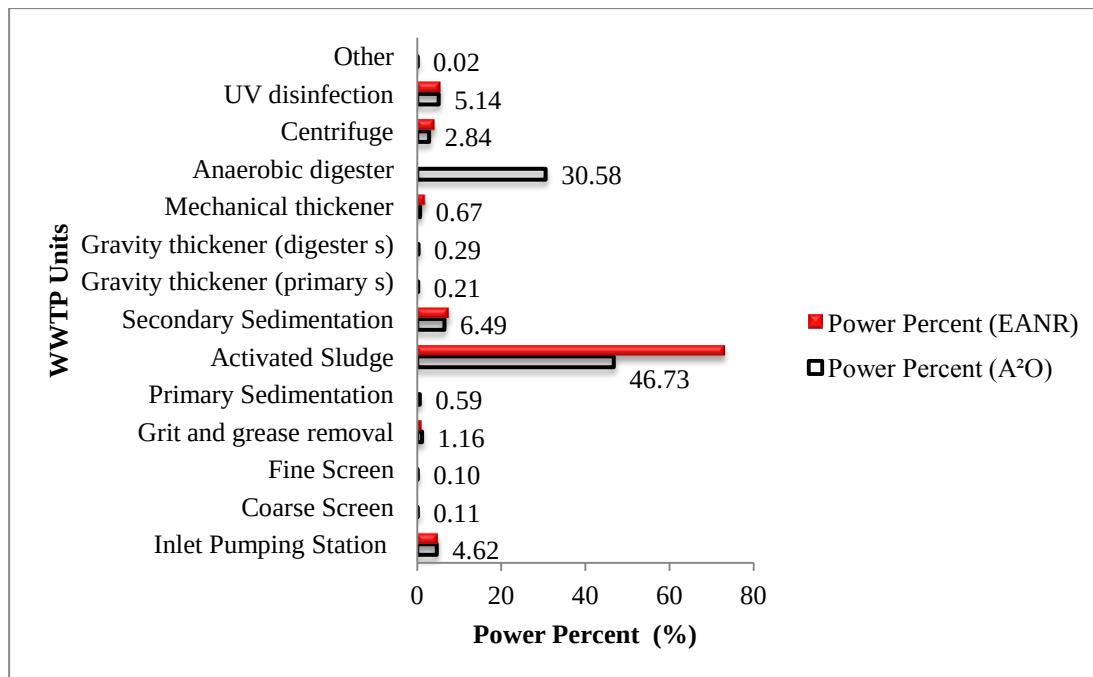


Figure 7.13 : Comparision of unit energy use profile for Aksaray WWTP.

Akşehir biological nutrient removal WWTP (alternative 1) consists of screening, grit and grease removal, primary sedimentation, activated sludge tanks with anaerobic, anoxic and aerobic zones, secondary sedimentation, sludge thickening, anaerobic sludge digestion, sludge dewatering units.

Akşehir extended aeration with nutrient removal active sludge system (alternative 2) consists of screening, grit and grease removal, anaerobic mixing tank, extended aeration activated sludge tank with anoxic zone, secondary clarifiers, sludge thickening, sludge dewatering units. Both of alternative design outputs are given in comparison Table 7.12, Figure 7.14 and 7.15.

Table 7.12 : Comparison of electricity requirements for Akşehir wastewater treatment plant.

Units	Power Cons. (A ² O) kw/d	Power Cons. (EANR) kw/d	Power Percent (A ² O) %	Power Percent (EANR) %	Power Cons. (A ² O) kwh/PE.a	Power Cons. (EANR) kwh/PE.a
Inlet Pumping Station	1,440	1,440	15.95	13.83	6.97	6.97
Coarse Screen	60	60	0.66	0.58	0.29	0.29
Fine Screen	30	30	0.33	0.29	0.15	0.15
Grit&grease rem.with blower	184	184	2.04	1.77	0.89	0.89
Primary sedimentation	48		0.53		0.23	
Activated Sludge Tank	4,657	7,093	51.60	68.12	22.54	34.33
Secondary Clarifier	1,269	1,244	14.06	11.95	6.14	6.02
Gravity thickener	11		0.12		0.05	
Mechanical Thickener		20	0.00	0.19		0.10
Anaerobic digester	999		11.07		4.84	
Belt thickener	27		0.30		0.13	
Centrifuge	248	289	2.75	2.78	1.20	1.40
Other	53	53	0.59	0.51	0.26	0.26
Total	9,026	10,413	100	100	43.69	50.40

EANR: Extended aeration with nutrient removal

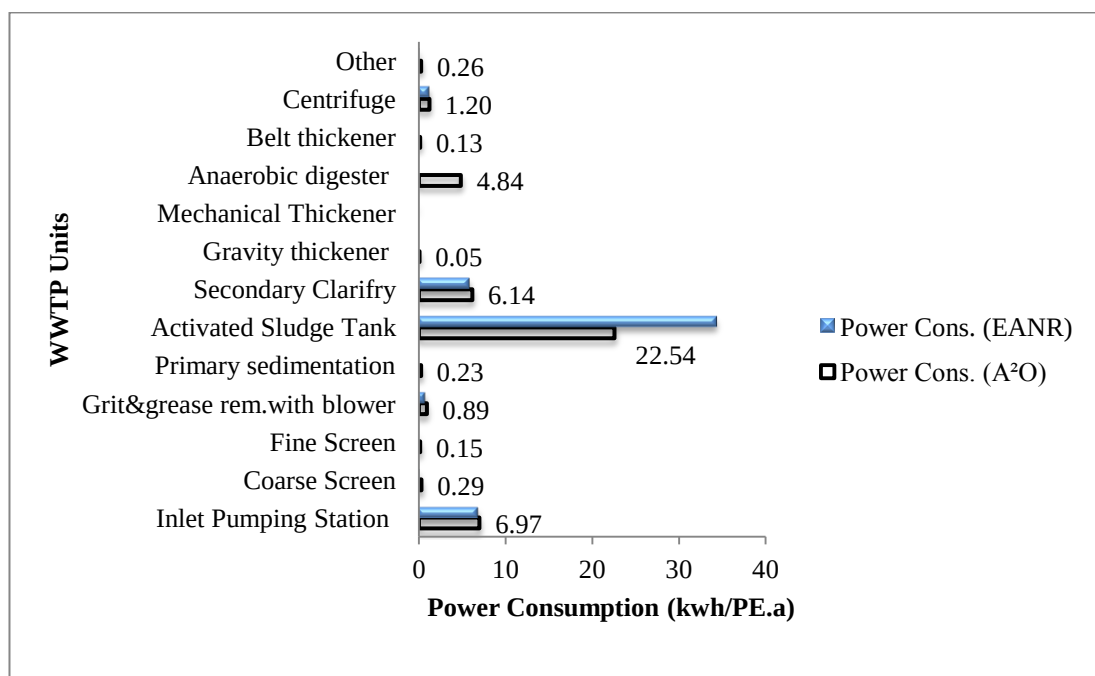


Figure 7.14 : Comparison of energy use profile for Akşehir WWTP.

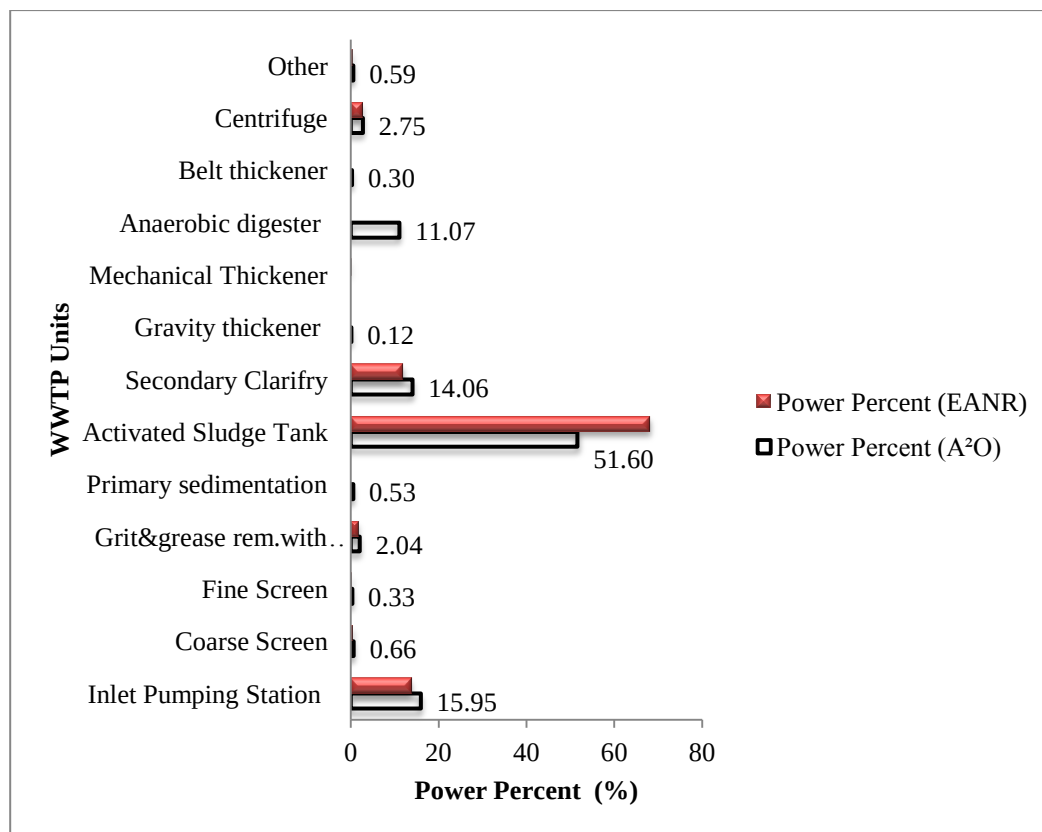


Figure 7.15 : Comparison of unit energy use profile for Akşehir WWTP.

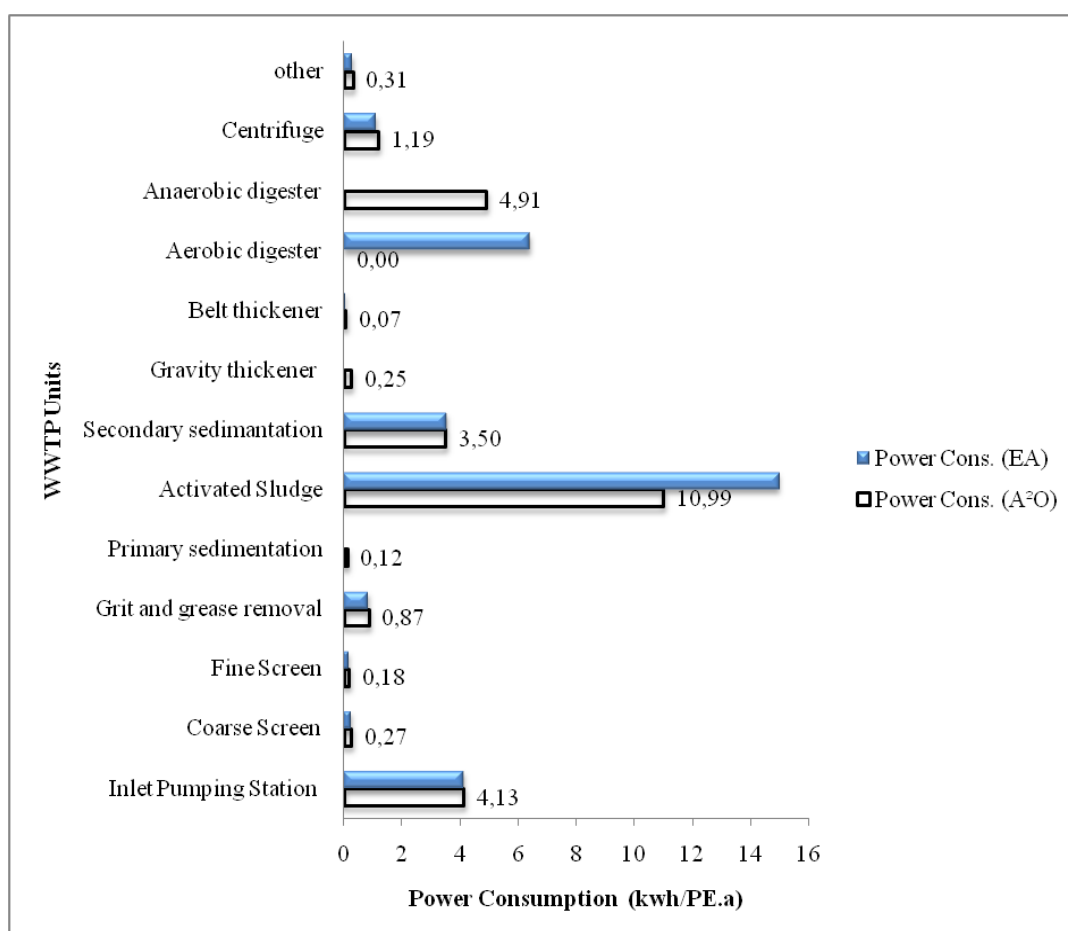
The project of an activated sludge system that makes carbon removal is planned in Bartın. Bartın activated sludge and anaerobic sludge digestion (alternative 1) consist of screening, grit and grease removal, primary sedimentation, activated sludge tanks, secondary sedimentation, sludge thickening, anaerobic sludge digestion, sludge dewatering units.

Bartın activated sludge and aerobic sludge digestion (alternative 2) consist of screening, grit and grease removal, activated sludge tank, secondary clarifiers, sludge thickening, aerobic sludge digester and sludge dewatering units. Both of alternative design outputs are given in comparison Table 7.13, Figure 7.16 and 7.17.

Table 7.13 : Comparison of electricity requirements for Bartın WWTP.

Units	Power	Power	Power	Power	Power	Power
	Cons.	Cons.	Percent	Percent	Cons.	Cons.
	(1)	(2)	(1)	(2)	(1)	(2)
	kw/d	kw/d	%	%	kwh/PE.a	kwh/PE.a
Inlet Pumping Station	696	696	15.43	12.95	4.13	4.13
Coarse Screen	45	45	1.00	0.84	0.27	0.27
Fine Screen	30	30	0.67	0.56	0.18	0.18
Grit rem. with blower	146	146	3.24	2.72	0.87	0.87
Primary sed.	20		0.44		0.12	
Activated Sludge	1,851	2,522	41.03	46.92	10.99	14.98
Secondary sediment.	590	599	13.08	11.14	3.50	3.56
Gravity thickener	42		0.93		0.25	
Belt thickener	12	13	0.27	0.24	0.07	0.08
Aerobic digester		1,081		20.11		6.42
Anaerobic digester	826		18.31		4.91	
Centrifuge	200	190	4.43	3.53	1.19	1.13
Other	53	53	1.17	0.99	0.31	0.31
Total	4,511	5,375	100	100	26.79	31.92

(¹) Alternative 1, (²) Alternative 2

**Figure 7.16 :** Comparison of energy use profile for Bartın WWTP.

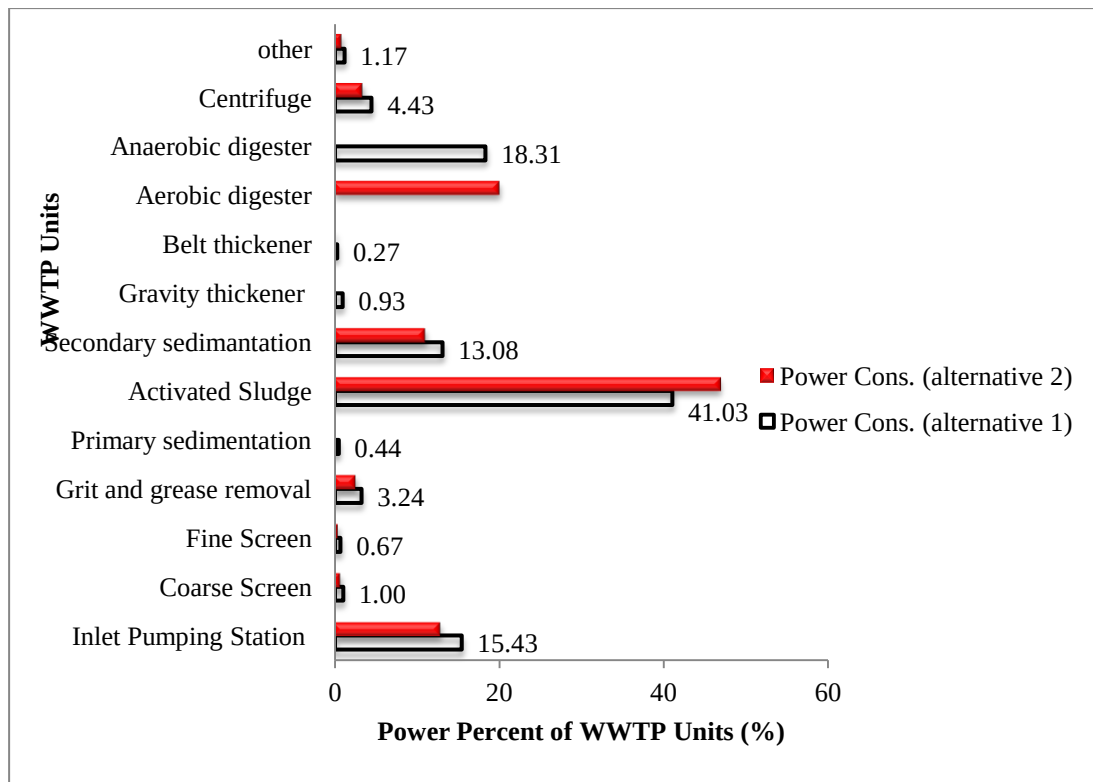


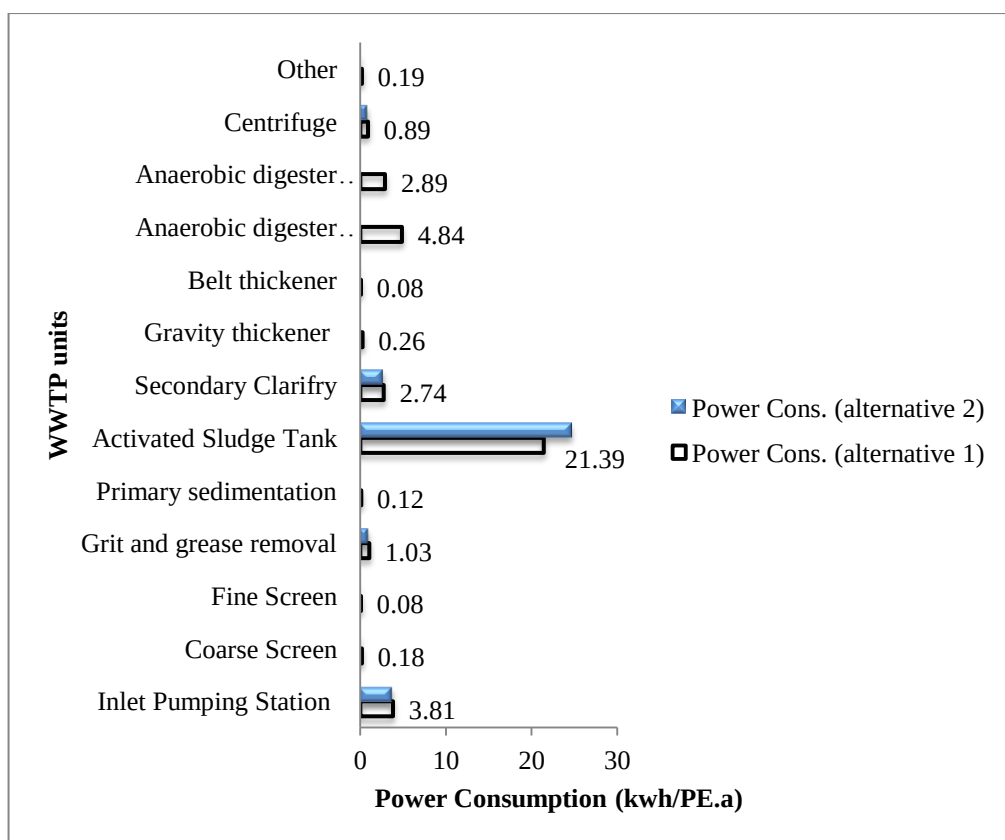
Figure 7.17 : Comparison of unit energy use profile for Bartın WWTP.

Ceyhan wastewater treatment plant is designed as biological treatment that makes carbon and nitrogen removal. Ceyhan biological activated sludge system (alternative 1) consists of screening, grit and grease removal, primary sedimentation, activated sludge tanks with anoxic and aerobic zones, secondary sedimentation, sludge thickening, anaerobic sludge digestion, sludge dewatering, gas utilisation facilities units.

Ceyhan extended aeration activated sludge system (alternative 2) consists of screening, grit and grease removal, extended aeration activated sludge tank with anoxic zone, secondary clarifiers, sludge thickening, sludge dewatering units. Both of alternative design outputs are given in comparison Table 7.14, Figure 7.18 and 7.19.

Table 7.14 : Comparison of electricity requirements for Ceyhan WWTP.

Units	Power	Power	Power	Power	Power	Power
	Cons.	Cons.	Percent	Percent	Cons.	Cons.
	(1)	(2)	(1)	(2)	(1)	(2)
	kw/d	kw/d	%	%	kwh/PE.a	kwh/PE.a
Inlet Pumping Station	1,421	1,421	9.89	11.28	3.81	3.81
Coarse Screen	68	68	0.47	0.54	0.18	0.18
Fine Screen	30	30	0.21	0.24	0.08	0.08
Grit and grease removal	385	385	2.68	3.06	1.03	1.03
Primary sedimentation	44		0.31		0.12	
Activated Sludge Tank	7,982	9,200	55.54	73.02	21.39	24.65
Secondary Clarifier	1,022	1,038	7.11	8.24	2.74	2.78
Gravity thickener	97		0.67		0.26	
Belt thickener	30	31	0.21	0.25	0.08	0.08
Anaerobic digester	1,808		12.58		4.84	
Anaerobic digester (gas part)	1,080		7.51		2.89	
Centrifuge	333	355	2.32	2.82	0.89	0.95
Other	72	72	0.50	0.57	0.19	0.19
Total	14,372	12,600	100	100	38.51	33.76

⁽¹⁾ Alternative 1⁽²⁾ Alternative 2**Figure 7.18 :** Comparison of energy use profile for Ceyhan WWTP.

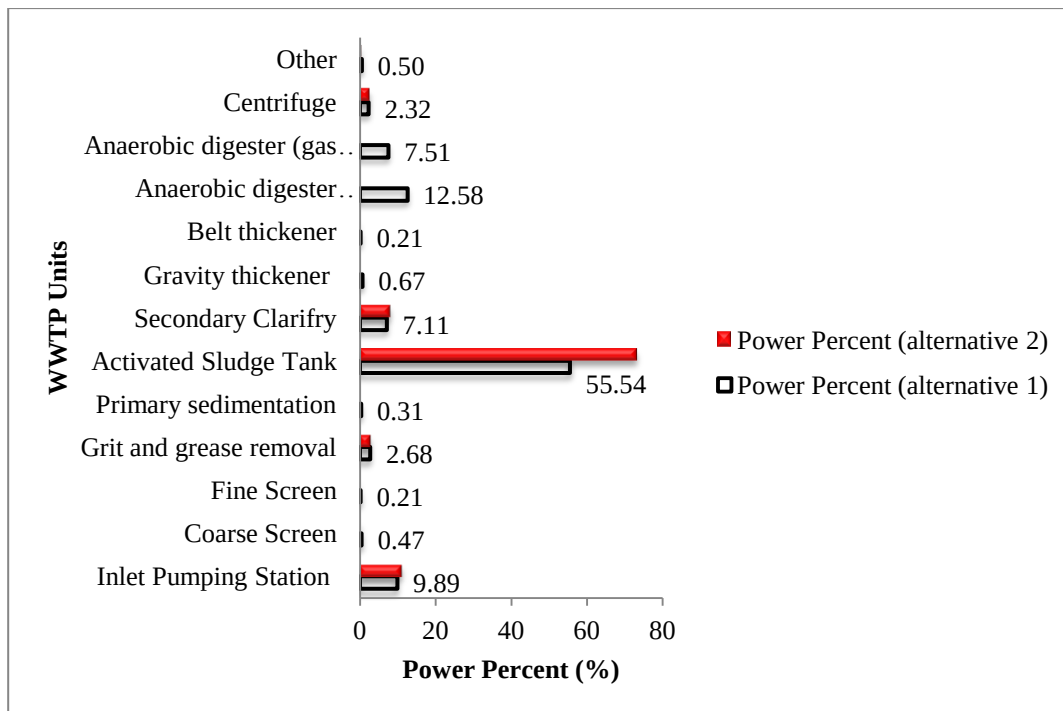


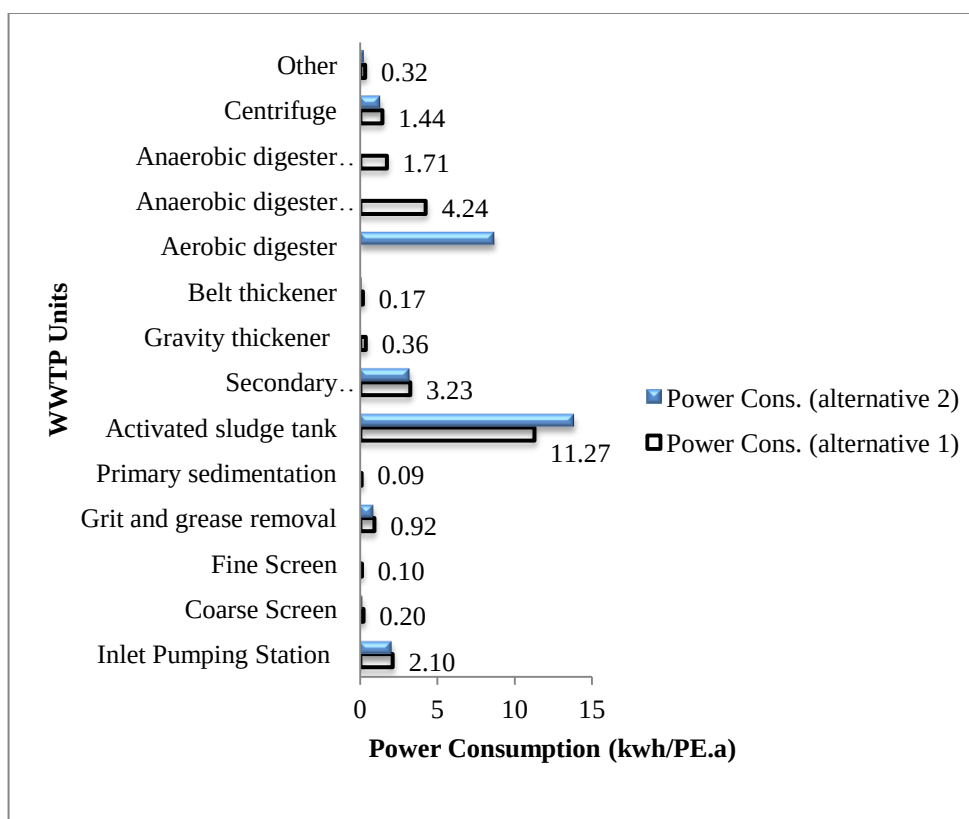
Figure 7.19 : Comparision of unit energy use profile for Ceyhan WWTP.

In Çarşamba an activated sludge system that makes carbon reduction is designed.Çarşamba activated sludge and anaerobic sludge digestion (alternative 1) consist of screening, grit and grease removal, primary sedimentation, activated sludge tanks, secondary sedimentation, sludge thickening, anaerobic sludge digestion, sludge dewatering, gas utilization facilities units.

Çarşamba, activated sludge and aerobic sludge digestion (alternative 2) consist of screening, grit and grease removal, activated sludge tank, secondary clarifiers, sludge thickening, aerobic sludge digester and sludge dewatering units. Both of alternative design outputs are given in comparison Table 7.15, Figure 7.20 and 7.21.

Table 7.15 : Comparison of electricity requirements for Çarşamba WWTP.

Units	Power	Power	Power	Power	Power	Power
	Cons.	Cons.	Percent	Percent	Cons.	Cons.
	(1)	(2)	(1)	(2)	(1)	(2)
	kw/d	kw/d	%	%	kwh/PE.a	kwh/PE.a
Inlet Pumping Station	468	468	8.03	6.78	2.10	2.10
Coarse Screen	45	45	0.77	0.65	0.20	0.20
Fine Screen	23	23	0.39	0.33	0.10	0.10
Grit and greese removal	204	204	3.50	2.95	0.92	0.92
Primary sedimentation	20		0.34		0.09	
Activated sludge tank	2,511	3,087	43.09	44.69	11.27	13.85
Secondary sedimentation	720	727	12.36	10.53	3.23	3.26
Gravity thickener	80		1.37		0.36	
Belt thickener	38	38	0.65	0.55	0.17	0.17
Aerobic digester		1938		28.06		8.70
Anaerobic digester (sludge)	944		16.20		4.24	
Anaerobic digester (gas part)	382		6.56		1.71	
Centrifuge	320	305	5.49	4.42	1.44	1.37
Other	72	72	1.24	1.04	0.32	0.32
Total	5,827	6,907	100	100	26.15	30.99

⁽¹⁾ Alternative 1⁽²⁾ Alternative 2**Figure 7.20 : Comparison of energy use profile for Çarşamba WWTP.**

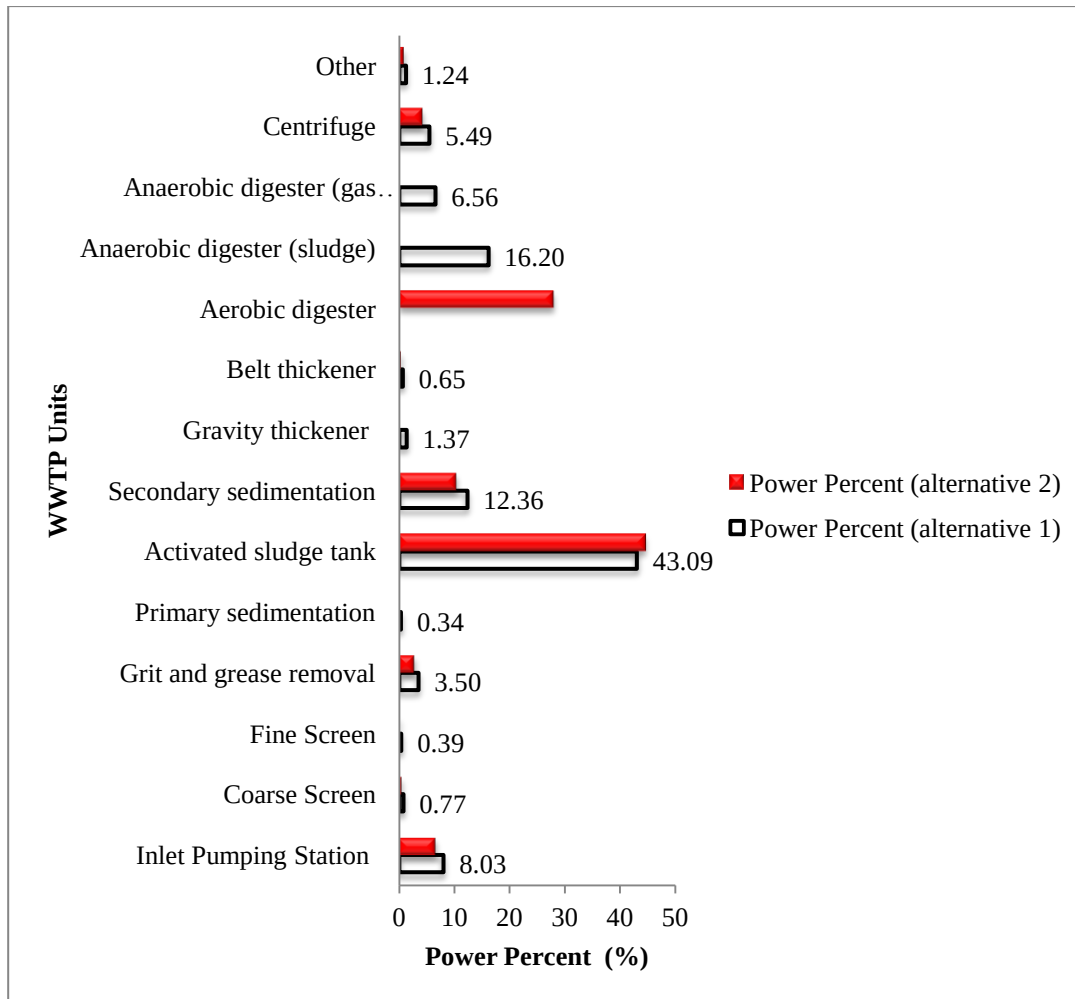


Figure 7.21 : Comparison of unit energy use profile for Çarşamba WWTP.

Diyarbakır biological nutrient removal WWTP (alternative 1) consist of screening, grit and grease removal, primary sedimentation, activated sludge tanks with anaerobic, anoxic and aerobic zones, secondary sedimentation, sludge thickening, anaerobic sludge digestion, sludge dewatering, gas utilisation facilities, energy recovery system units.

Diyarbakır extended aeration with nutrient removal activated sludge (alternative 2) consist of screening, grit and grease removal, anaerobic tank, extended aeration activated sludge tank with anoxic zone, secondary clarifiers, sludge thickening, sludge dewatering units. In addition to that for the pre sedimentation sludge anaerobic digester is considered. Outputs of both alternative designs are given in comparison Table 7.16, Figure 7.22 and 7.23.

The amount of energy consumed by all the pumps (inlet and other) used in Diyarbakır WWTP is calculated as 21,51 kwh/PE.a for A²O and 20,13 kwh/PE.a for

extended aeration with nutrient removal. This constitutes approximately 25% of the total energy demand. Since primary settling tank is present, it is used in the EANR process.

Table 7.16 : Comparison of electricity requirements for Diyarbakır WWTP.

Units	Power Cons. (A ² O) kwh/d	Power Cons. (EANR) kwh/d	Power Percent (A ² O) %	Power Percent (EANR) %	Power Cons. (A ² O) kwh/PE.a	Power Cons. (EANR) kwh/PE.a
Inlet pumping station	9,279	9,279	11.01	11.21	2.37	2.37
Coarse screen	40	40	0.05	0.05	0.01	0.01
Fine screen	369	369	0.44	0.45	0.09	0.09
Grit and Grease Rem.	1,881	1,881	2.23	2.27	0.48	0.48
Primary clarifier	142	142	0.17	0.17	0.04	0.04
Activated sludge tank	51,932	54,075	61.61	65.32	13.26	13.80
Secondary clarifier	7,091	9,417	8.41	11.38	1.81	2.40
Sludge thickening (Gravity thickener)	21	21	0.03	0.03	0.01	0.01
Sludge dewatering (Belt press)	432	558	0.51	0.67	0.11	0.14
Sludge thickening (Mechanical thickener)	1,243	125	1.47	0.15	0.32	0.03
Anaerobic sludge digesters	8,678	3,581	10.30	4.33	2.22	0.91
Sludge dewatering (Centrifuge)	2,718	2,836	3.22	3.43	0.69	0.72
Chemical Building	110	110	0.13	0.13	0.03	0.03
Phosphorus removal system for sludge liquer	290	290	0.34	0.35	0.07	0.07
Other	59	59	0.07	0.07	0.02	0.02
Total	84,286	82,783	100	100	21.51	21.13

EANR: Extended aeration with nutrient removal

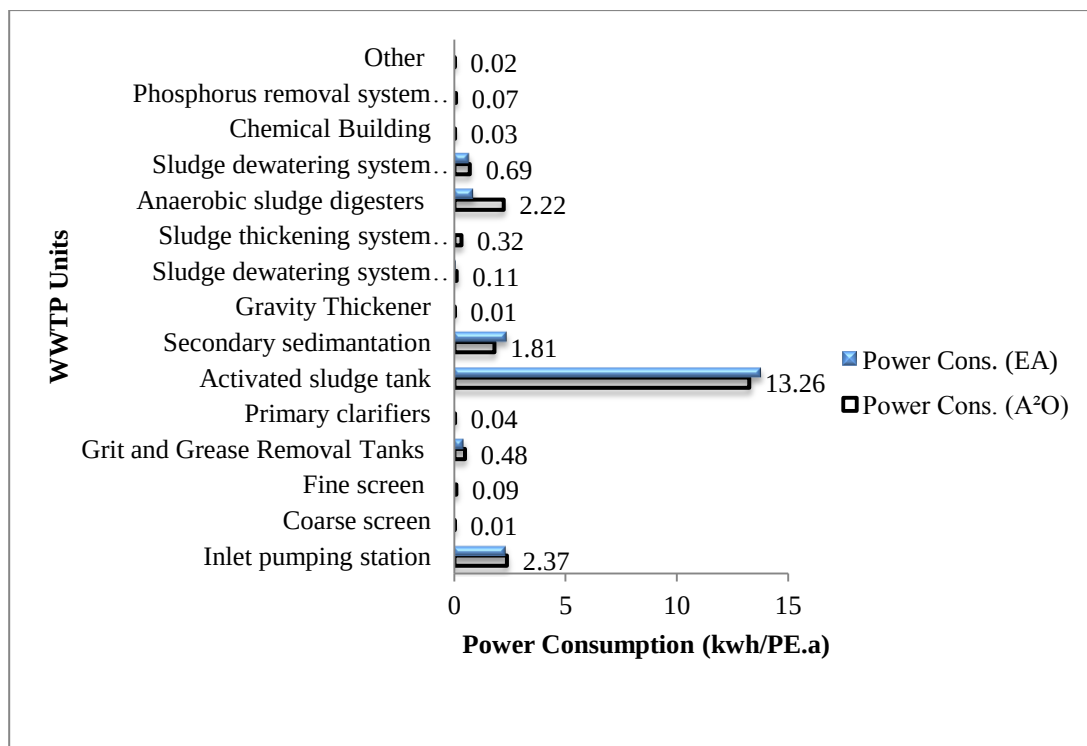


Figure 7.22 : Comparision of unit energy use profile for Diyarbakır WWTP.

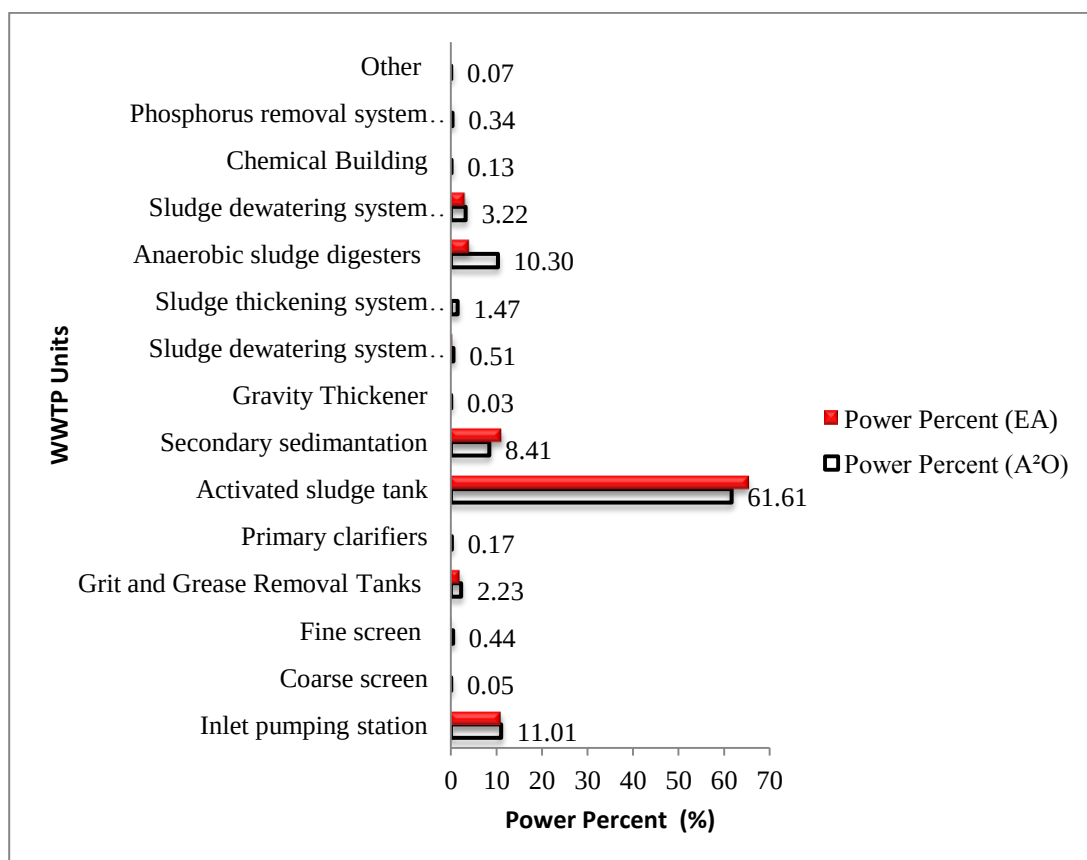


Figure 7.23 : Comparision of unit energy use profile for Diyarbakır WWTP.

As Diyarbakır wastewater treatment project is planned as revision of the existing plant, in the table of energy requirement, the existing anaerobic digester unit is shown for extended aeration process.

Erzurum wastewater treatment plant is designed as biological treatment that makes carbon and nitrogen removal. Erzurum biological activated sludge system (alternative 1) consist of screening, grit and grease removal, primary sedimentation, activated sludge tanks with anoxic and aerobic zones, secondary sedimentation, sludge thickening, anaerobic sludge digestion, sludge dewatering, gas utilisation facilities, energy recovery system units.

Erzurum Extended aeration activated sludge system (alternative 2) consist of screening, grit and grease removal, extended aeration activated sludge tank with anoxic zone, secondary clarifiers, sludge thickening, sludge dewatering units. Both of alternative design outputs are given in comparison Table 7.17, Figure 7.24 and 7.25.

Table 7.17 : Comparison of electricity requirements for Erzurum WWTP.

Units	Power Cons. (1) kw/d	Power Cons. (2) kw/d	Power Percent (1) %	Power Percent (2) %	Power Cons. (1) kwh/PE.a	Power Cons. (2) kwh/PE.a
Inlet Pumping Station	4,290	4,290	11.54	10.92	3.52	3.52
Coarse Screen	104	104	0.28	0.26	0.09	0.09
Fine Screen	37	37	0.10	0.09	0.03	0.03
Grit and grease removal	661	661	1.78	1.68	0.54	0.54
Primary sedimentation	58		0.16		0.05	
Activated Sludge Tank	20,554	30,991	55.30	78.87	16.86	25.42
Secondary Sedimentation	1,971	2,030	5.30	5.17	1.62	1.67
Gravity thickener	329		0.89		0.27	
Belt thickener	74	97	0.20	0.25	0.06	0.08
Anaerobic digester (sludge)	5,424		14.59		4.45	
Anaerobic digester (gas part)	2,726	1,010	7.33	2.57	2.24	0.83
Centrifuge	870		2.34		0.71	
Other	72	72	0.19	0.18	0.06	0.06
Total	37,170	39,292	100	100	30.49	32.23

⁽¹⁾ Alternative 1

⁽²⁾ Alternative 2

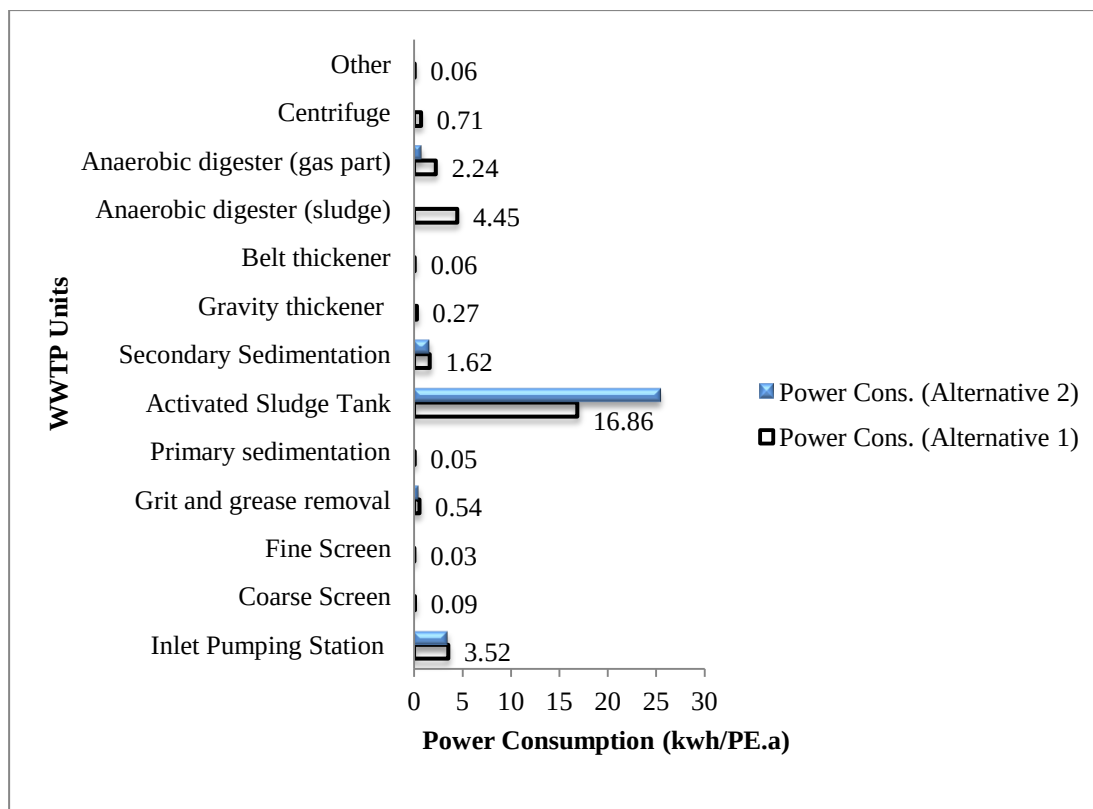


Figure 7.24 : Comparision of energy use profile for Erzurum WWTP.

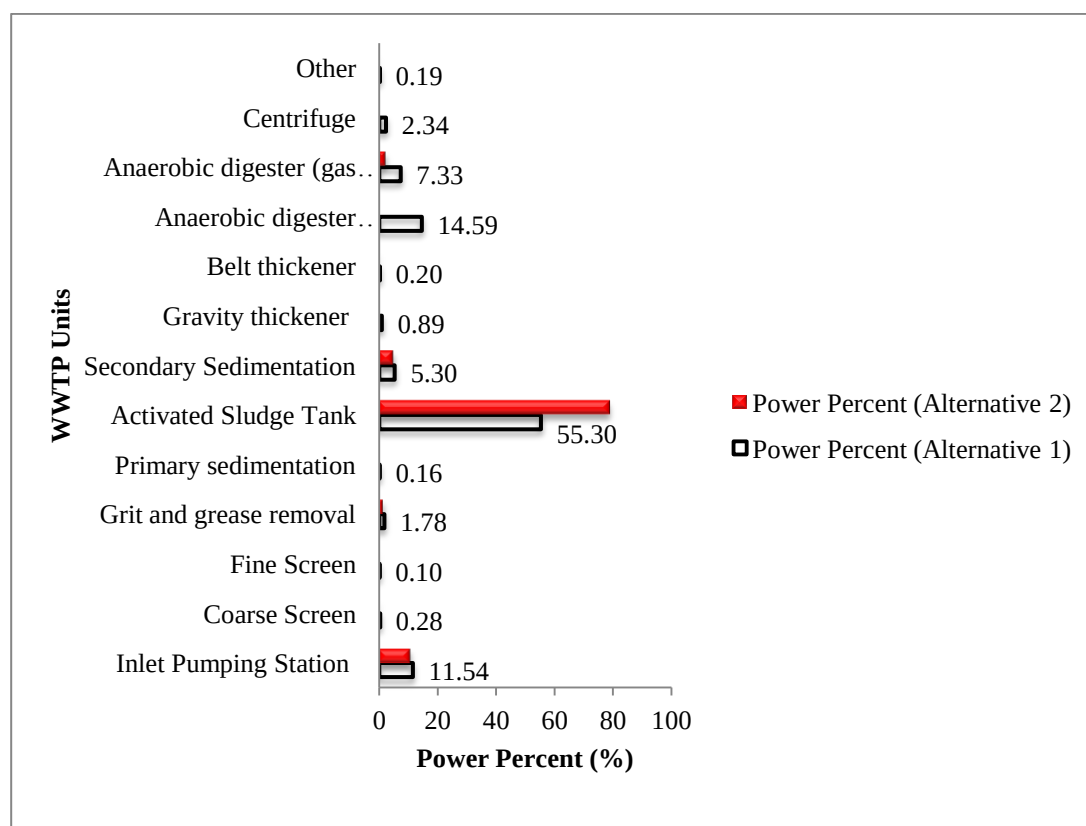


Figure 7.25 : Comparision of unit energy use profile for Erzurum WWTP.

Lüleburgaz wastewater treatment plant is designed as biological treatment that makes carbon and nitrogen removal. Lüleburgaz biological activated sludge system (alternative 1) consist of screening, grit and grease removal, primary sedimentation, activated sludge tanks with anoxic and aerobic zones, secondary sedimentation, sludge thickening, anaerobic sludge digestion, sludge dewatering units.

Lüleburgaz extended aeration activated sludge system (alternative 2) consist of screening, grit and grease removal, extended aeration activated sludge tank with anoxic zone, secondary clarifiers, sludge thickening, sludge dewatering units. Both of alternative design outputs are given in comparison Table 7.18, Figure 7.26 and 7.27.

Table 7.18 : Comparison of electricity requirements for Lüleburgaz WWTP.

Units	Power Cons. (1)	Power Cons. (2)	Power Percent (1)	Power Percent (2)	Power Cons. (1)	Power Cons. (2)
	kw/d	kw/d	%	%	kwh/PE.a	kwh/PE.a
Inlet Pumping Station	1,728	1,728	16.66	14.84	4.54	4.54
Coarse Screen	95	95	0.92	0.82	0.25	0.25
Fine Screen	30	30	0.29	0.26	0.08	0.08
Grit and greese removal with blower	319	319	3.08	2.74	0.84	0.84
Primary sedimentation	33		0.32		0.09	
Activated Sludge Tank	5,860	7,728	56.49	66.36	15.40	20.31
Secondary Clarifier	1,231	1,247	11.87	10.71	3.24	3.28
Gravity thickener (Primary s.)	10		0.10		0.03	
Belt thickener	57	28	0.55	0.24	0.15	0.07
Mechanical thickener		38		0.33		0.10
Anaerobic digester	734		7.08		1.93	
Centrifuge	223	380	2.15	3.26	0.59	1.00
Other	53	53	0.51	0.46	0.14	0.14
Total	10,373	11,646	100	100	27.26	30.61

⁽¹⁾ Alternative 1

⁽²⁾ Alternative 2

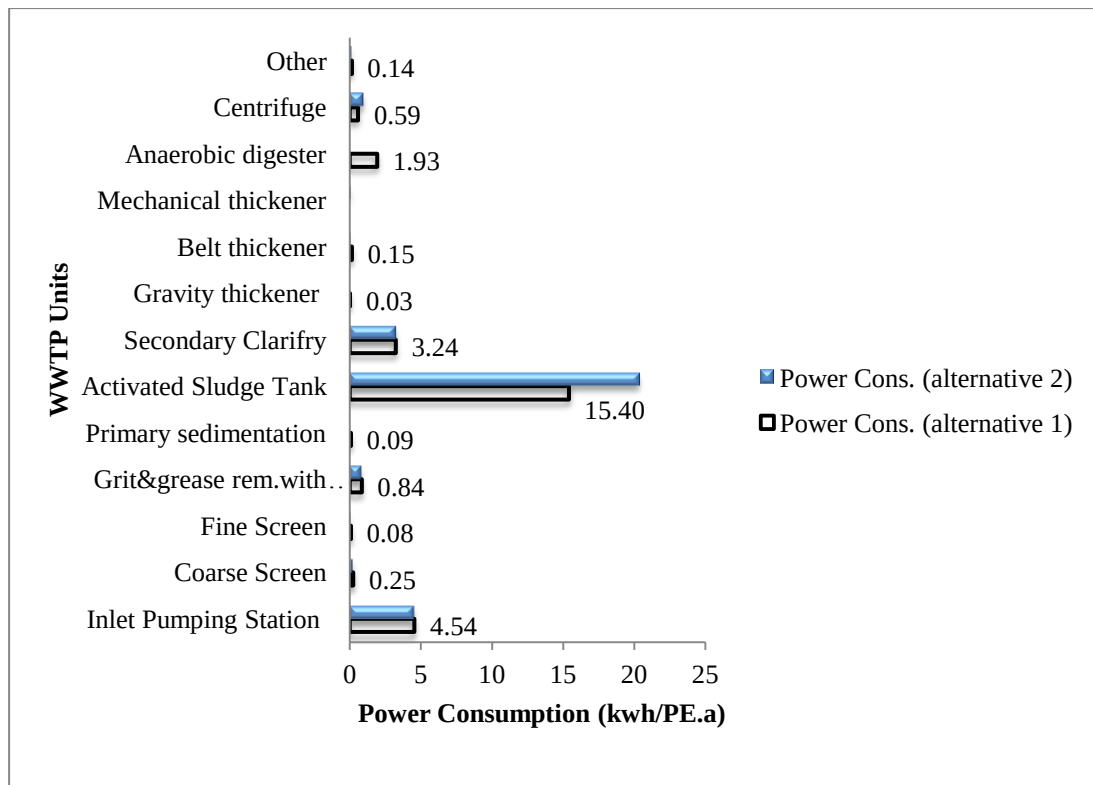


Figure 7.26 : Comparison of energy use profile for Lüleburgaz WWTP.

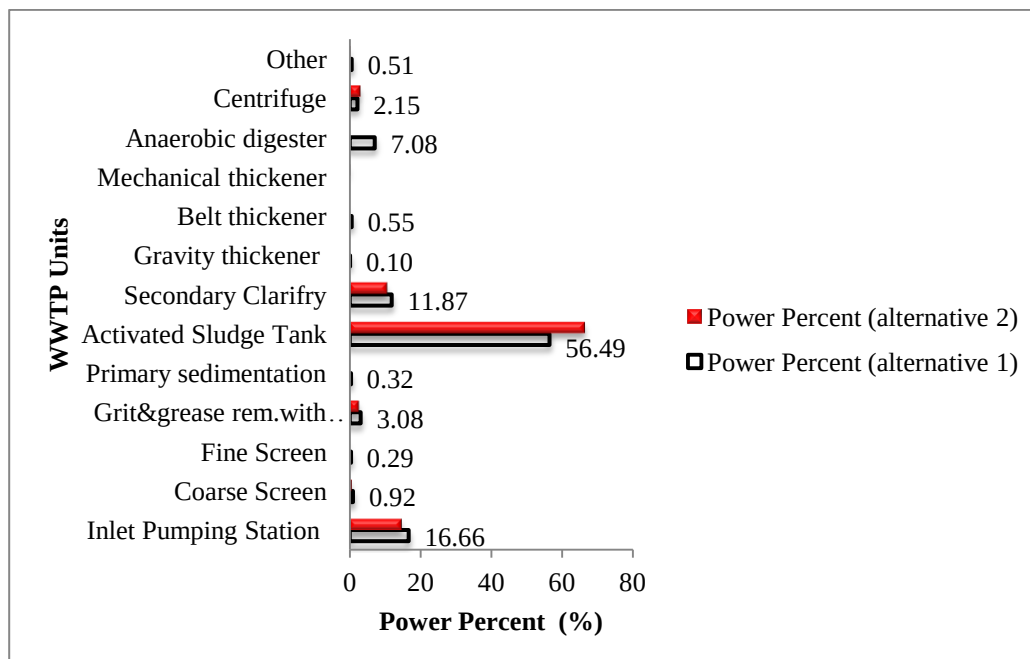


Figure 7.27 : Comparison of unit energy use profile for Lüleburgaz WWTP.

In Merzifon an activated sludge system that makes carbon removal is designed. Merzifon activated sludge and anaerobic sludge digestion (alternative 1) consist of screening, grit and grease removal, primary sedimentation, activated sludge

tanks, secondary sedimentation, sludge thickening, anaerobic sludge digestion, sludge dewatering units.

Merzifon activated sludge and aerobic sludge digestion (alternative 2) consist of screening, grit and grease removal, activated sludge tank (extended aeration), secondary clarifiers, sludge thickening, aerobic sludge digester and sludge dewatering unitlerinden oluşmaktadır. Both of alternative design outputs are given in comparison Table 7.19, Figure 7.28 and Figure 7.29.

Table 7.19 : Comparison of electricity requirements for Merzifon WWTP.

Units	Power Cons.	Power Cons.	Power Percent	Power Percent	Power Cons.	Power Cons.
	(1)	(2)	(1)	(2)	(1)	(2)
	kw/d	kw/d	%	%	kwh/PE.a	kwh/PE.a
Inlet Pumping Station	422	422	10.80	7.85	2.14	2.14
Coarse Screen	61	61	1.56	1.13	0.31	0.31
Fine Screen	30	30	0.77	0.56	0.15	0.15
Grit and grease removal with blower	136	136	3.48	2.53	0.69	0.69
Activated Sludge Tank	1,940	2,580	49.65	48.00	9.84	13.08
Secondary Clarifier	453	489	11.59	9.10	2.30	2.48
Aerobic digester		1,362		25.34		6.91
Anaerobic digester	317		8.11		1.61	
Mechanical Thickener	110	7	2.82	0.13	0.56	0.04
Centrifuge	403	253	10.31	4.71	2.04	1.28
Disinfection (Chlorine)	11	11	0.28	0.20	0.06	0.06
Other	24	24	0.61	0.45	0.12	0.12
Total	3,907	5,375	100	100	19.81	27.25

⁽¹⁾ Alternative 1

⁽²⁾ Alternative 2

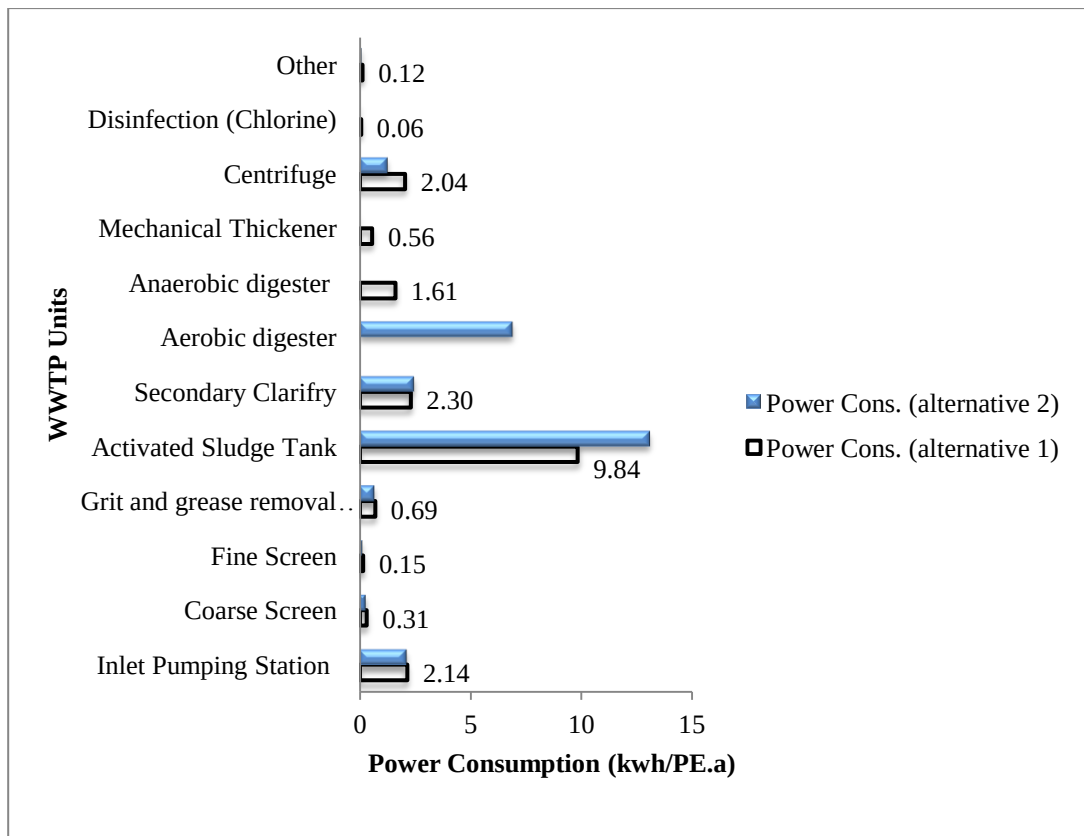


Figure 7.28 : Comparison of energy use profile for Merzifon WWTP.

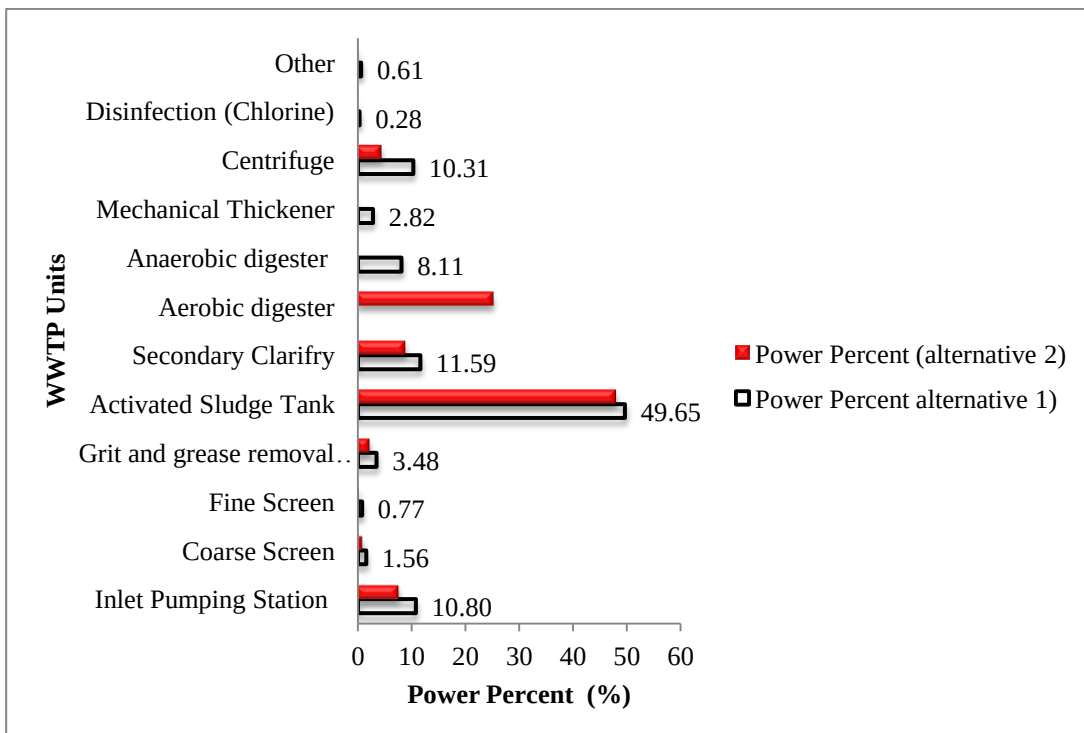


Figure 7.29 : Comparison of unit energy use profile for Merzifon WWTP.

Polatlı wastewater treatment plant is designed as biological treatment that makes carbon and nitrogen removal. Polatlı biological activated sludge system (alternative 1) consist of screening, grit and grease removal, primary sedimentation, activated sludge tanks with anoxic and aerobic zones, secondary sedimentation, sludge thickening, anaerobic sludge digestion, sludge dewatering, gas utilisation facilities units.

Polatlı extended aeration activated sludge system (alternative 2) consist of screening, grit and grease removal, extended aeration activated sludge tank with anoxic zone, secondary clarifiers, sludge thickening, sludge dewatering units. Both of alternative design outputs are given in comparison Table 7.20, Figure 7.30 and 7.31.

Table 7.20 : Comparison of electricity requirements forPolatlı WWTP.

Units	Power	Power	Power	Power	Power	Power
	Cons.	Cons.	Percent	Percent	Cons.	Cons.
	(1)	(2)	(1)	(2)	(1)	(2)
	kw/d	kw/d	%	%	kwh/PE.a	kwh/PE.a
Inlet Pumping Station	672	672	5.72	6.61	1.95	1.95
Coarse Screen	57	57	0.49	0.56	0.17	0.17
Fine Screen	30	30	0.26	0.30	0.09	0.09
Grit&grease removal with blower	242	242	2.06	2.38	0.70	0.70
Primary sedimentation	20		0.17		0.06	
Activated Sludge Tank	5,517	7,260	46.93	71.46	16.00	21.05
Secondary Clarifier	578	604	4.92	5.95	1.68	1.75
Gravity thickener	32		0.27		0.09	
Belt thickener	88	30	0.75	0.30	0.26	0.09
Anaerobic digester (sludge)	2,192		18.65		6.36	
Anaerobic digester (gas part)	1,229		10.45		3.56	
Centrifuge	182	347	1.55	3.42	0.53	1.01
UV disinfection	845	845	7.19	8.32	2.45	2.45
Other	72	72	0.61	0.71	0.21	0.21
Total	11,756	10,159	100	100	34.09	29.46

⁽¹⁾ Alternative 1

⁽²⁾ Alternative 2

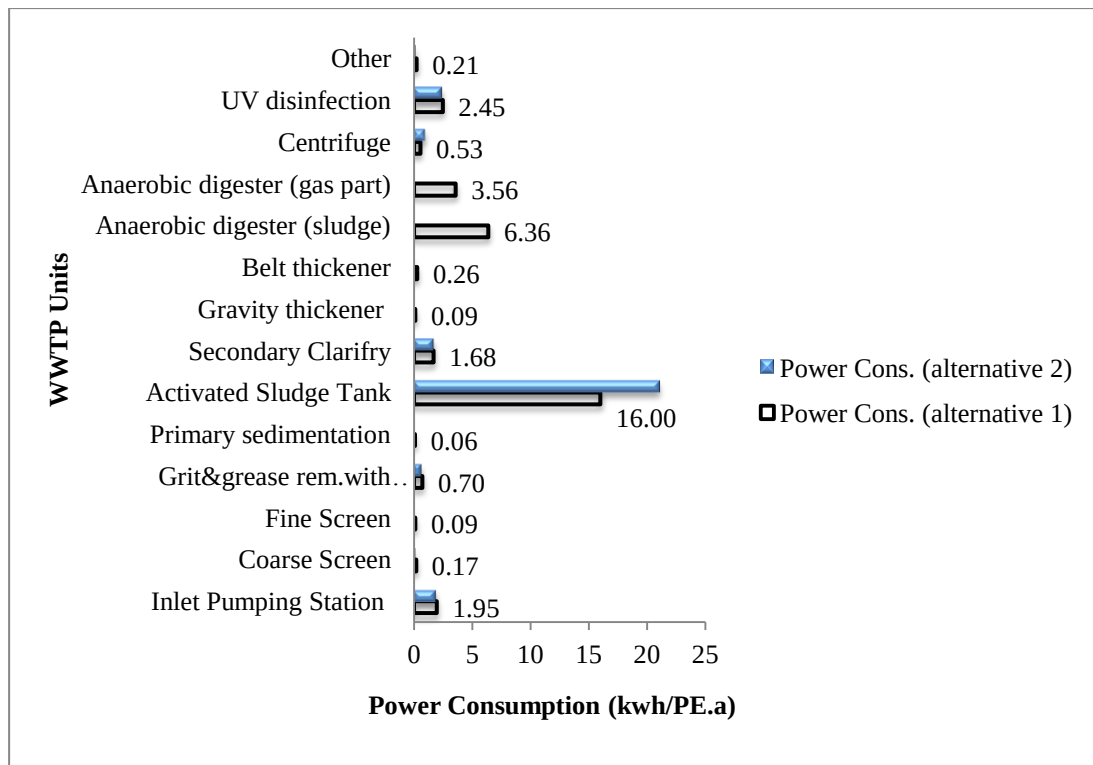


Figure 7.30 : Comparision of energy use profile for Polatlı WWTP.

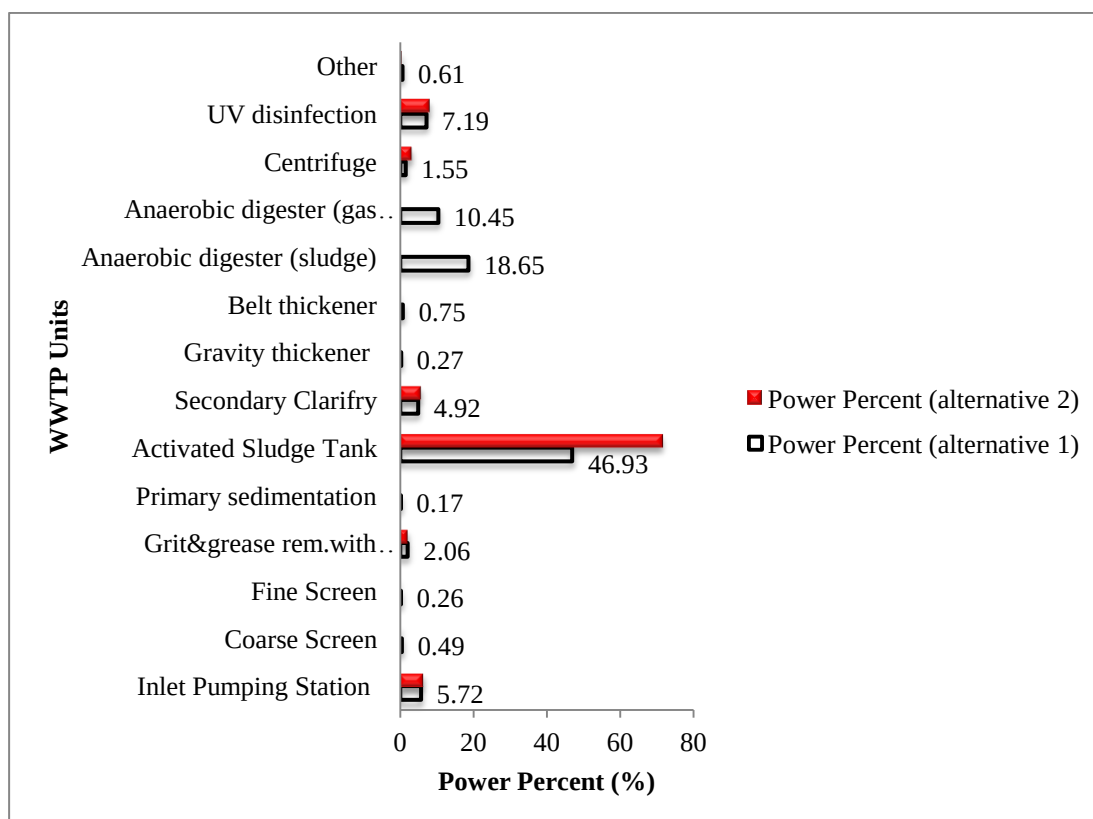


Figure 7.31 : Comparision of unit energy use profile for Polatlı WWTP.

Seydişehir biological nutrient removal WWTP alternative consist of screening, grit and grease removal, primary sedimentation, activated sludge tanks with anaerobic, anoxic and aerobic zones, secondary sedimentation, sludge thickening, anaerobic sludge digestion, sludge dewatering units. Design outputs are given in Table 7.21, Figure 7.32 and Figure 7.33.

Table 7.21 : Comparison of electricity requirements for Seydişehir WWTP.

Units	Power Cons. (A ² O)	Power Percent (A ² O)	Power Cons. (A ² O)
	kwh/d	%	kwh/PE.a
Inlet Pumping Station	518	10.36	4.04
Coarse Screen	33	0.67	0.26
Fine Screen	31	0.62	0.24
Grit&grease removal with blower	105	2.10	0.82
Primary sedimentation	68	1.36	0.53
Activated Sludge Tank	2,767	55.27	21.58
Secondary Clarifier	562	11.23	4.38
Gravity thickener	26	0.52	0.20
Anaerobic digester	296	5.91	2.31
Centrifugal Decanter	389	7.77	3.03
Mechanical Thickener	193	3.86	1.51
Disinfection	11	0.22	0.09
Oher	6	0.12	0.05
Total	5,006	100	39.05

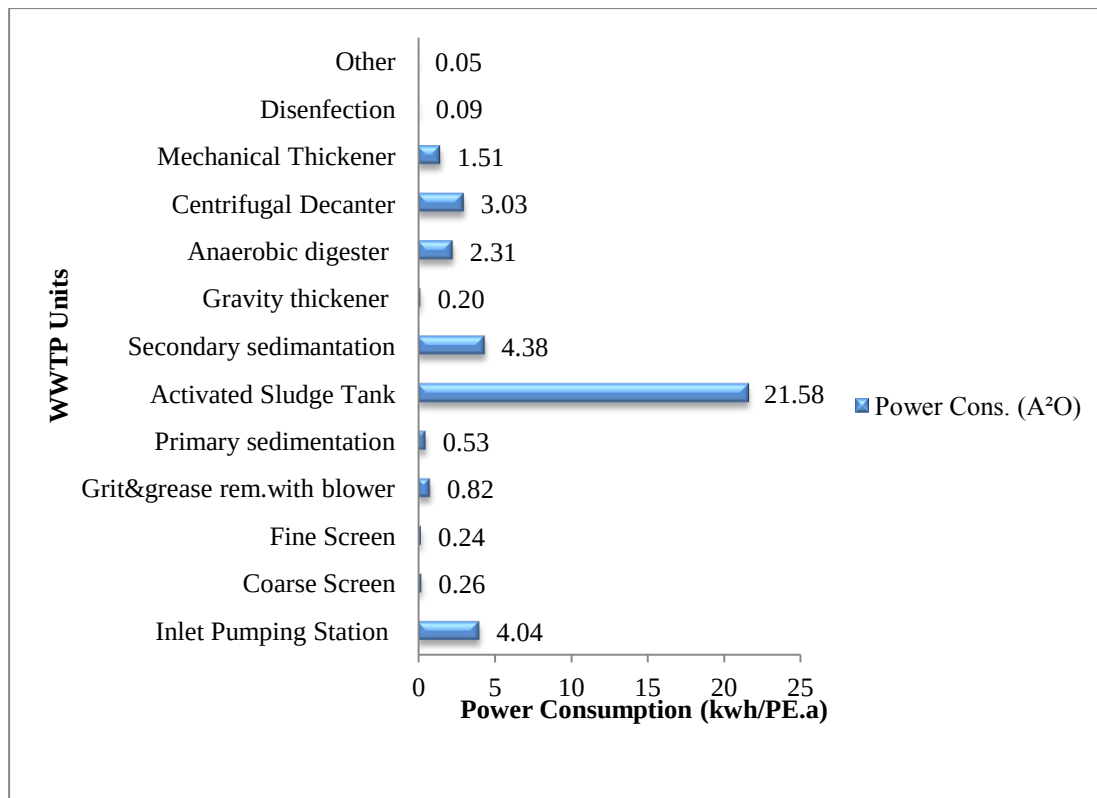


Figure 7.32 : Comparision of energy use profile for Seydişehir WWTP.

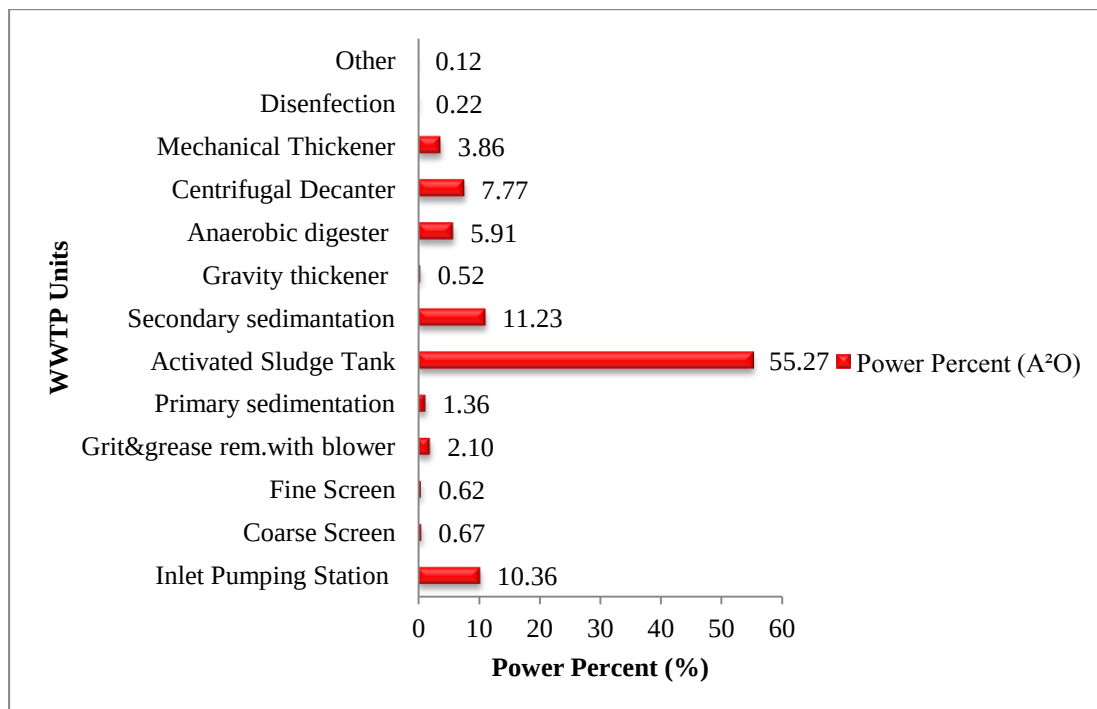


Figure 7.33 : Comparision of unit energy use profile for Seydişehir WWTP.

In Siverek an activated sludge system that makes carbon removal is designed. Siverek activated sludge and anaerobic sludge digestion (alternative 1) consist of screening, grit and grease removal, primary sedimentation, activated sludge tanks, secondary

sedimentation, sludge thickening, anaerobic sludge digestion, sludge dewatering units.

Siverek activated sludge and aerobic sludge digestion (alternative 2) consist of screening, grit and grease removal, activated sludge tank, secondary clarifiers, sludge thickening, aerobic sludge digester and sludge dewatering units. Both of alternative design outputs are given in comparison Table 7.22, Figure 7.34 and 7.35.

Table 7.22 : Comparison of electricity requirements for Siverek WWTP.

Units	Power Cons. (1) kw/d	Power Cons. (2) kw/d	Power Percent (1) %	Power Percent (2) %	Power Cons. (1) kwh/PE.a	Power Cons. (2) kwh/PE.a
Inlet Pumping Station	na	na	na	na	na	na
Coarse Screen	11	11	0.14	0.12	0.03	0.03
Fine Screen	30	30	0.39	0.34	0.09	0.09
Grit&grease rem.with blower	240	240	3.13	2.73	0.68	0.68
Primary sedimentation	28		0.37		0.08	
Activated sludge tank	3,679	3,778	48.05	42.92	10.48	10.77
Secondary sedimentation	1,541	1,573	20.13	17.87	4.39	4.48
Gravity thickener	25		0.33		0.07	
Belt thickener	28	12	0.37	0.14	0.08	0.03
Aerobic sludge digester		2,445		27.78		6.97
Anaerobic digester	1,677		21.90		4.78	
Centrifuge	247	563	3.23	6.40	0.70	1.60
Disinfection	79	79	1.03	0.90	0.23	0.23
Other	72	72	0.94	0.82	0.21	0.21
Total	7,657	8,803	100	100	21.82	25.08

⁽¹⁾ Alternative 1

⁽²⁾ Alternative 2

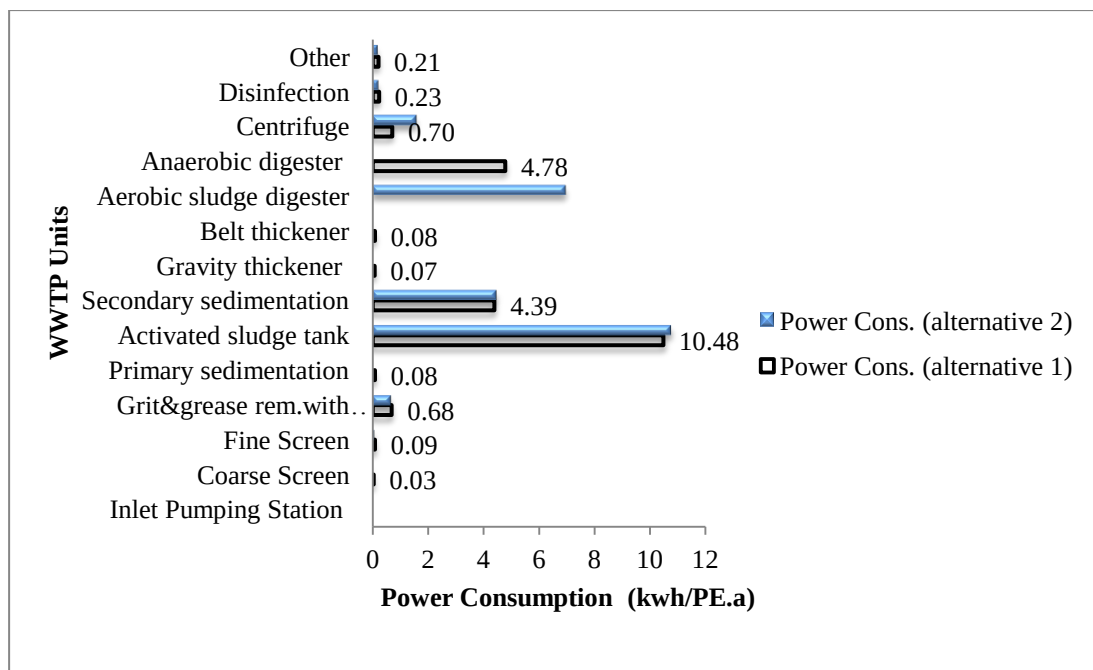


Figure 7.34 : Comparision of energy use profile for Siverek WWTP.

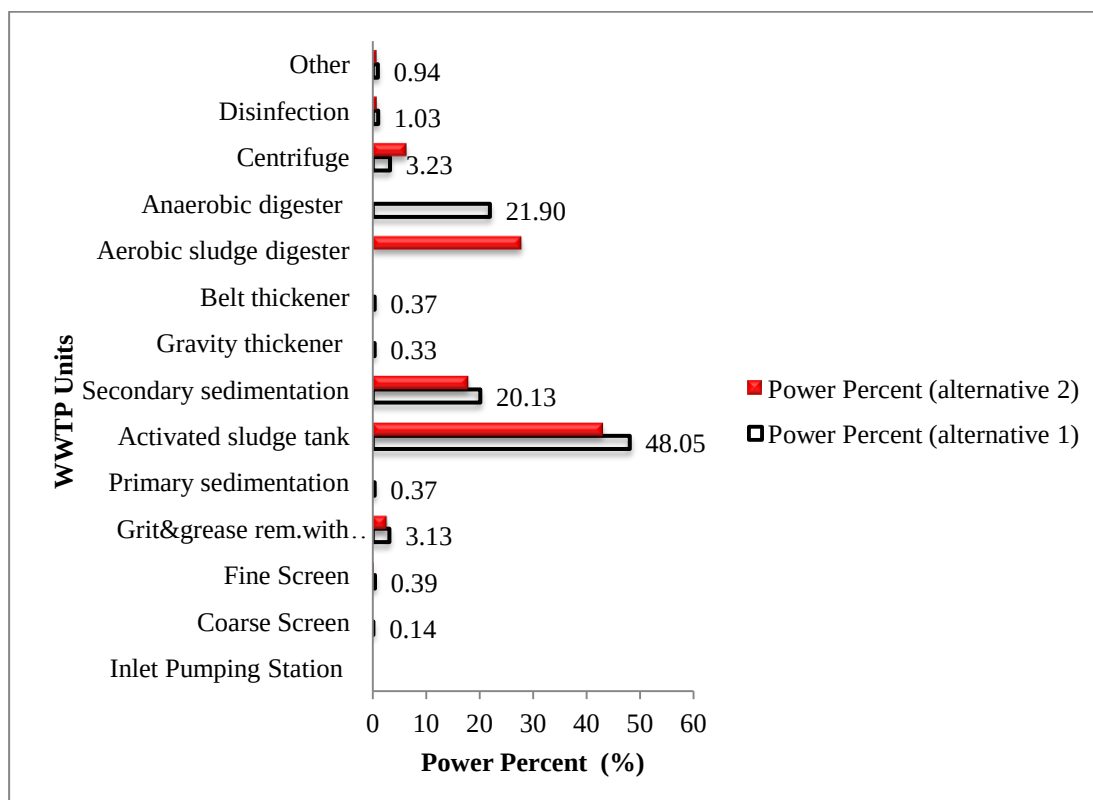


Figure 7.35 : Comparision of unit energy use profile for Siverek WWTP.

Soma wastewater treatment plant is designed as biological treatment that makes carbon and nitrogen removal. Soma biological activated sludge system (alternative 1) consist of screening, grit and grease removal, primary sedimentation, activated sludge tanks with anoxic and aerobic zones, secondary sedimentation, sludge thickening, anaerobic sludge digestion, sludge dewatering units.

Soma extended aeration activated sludge system (alternative 2) consist of screening, grit and grease removal, extended aeration activated sludge tank with anoxic zone, secondary clarifiers, sludge thickening, sludge dewatering units. Both of alternative design outputs are given in comparison Table 7.23, Figure 7.36 and 7.37.

Table 7.23 : Comparison of electricity requirements for Soma WWTP.

Units	Power	Power	Power	Power	Power	Power
	Cons.	Cons.	Percent	Percent	Cons.	Cons.
	(1)	(2)	(1)	(2)	(1)	(2)
	kw/d	kw/d	%	%	kwh/PE.a	kwh/PE.a
Inlet Pumping Station	681	681	7.15	7.92	2.24	2.24
Coarse Screen	45	45	0.47	0.52	0.15	0.15
Fine Screen	23	23	0.24	0.27	0.08	0.08
Grit&grease rem.with blower	147	147	1.54	1.71	0.48	0.48
Primary sedimentation	21		0.22		0.07	
Activated Sludge Tank	5,365	5,848	56.34	67.98	17.64	19.23
Secondary Clarifier	1,507	1,505	15.82	17.49	4.96	4.95
Gravity thickener	26		0.27		0.09	
Belt thickener	29	20	0.30	0.23	0.10	0.07
Anaerobic digester	1,320		13.86		4.34	
Centrifuge	229	204	2.40	2.37	0.75	0.67
Disinfection (Chlorine)	77	77	0.81	0.90	0.25	0.25
Other	53	53	0.56	0.62	0.17	0.17
Total	9,523	8,603	100	100	31.31	28.29

⁽¹⁾ Alternative 1

⁽²⁾ Alternative 2

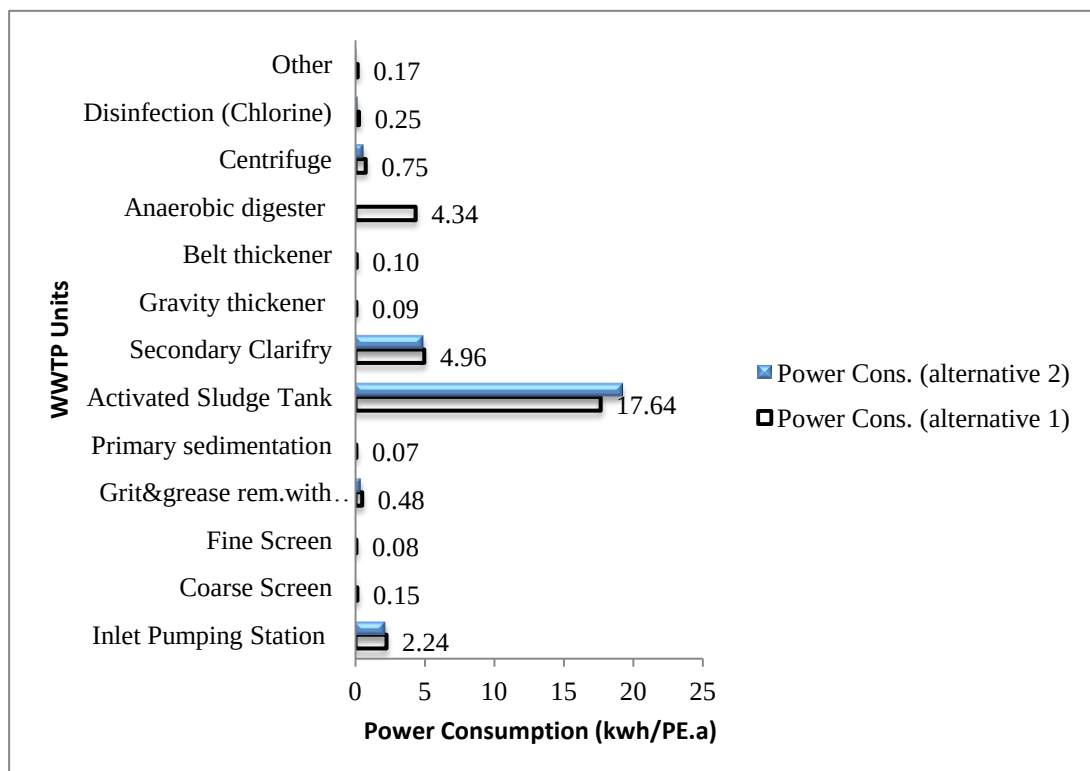


Figure 7.36 : Comparison of energy use profile for Soma WWTP.

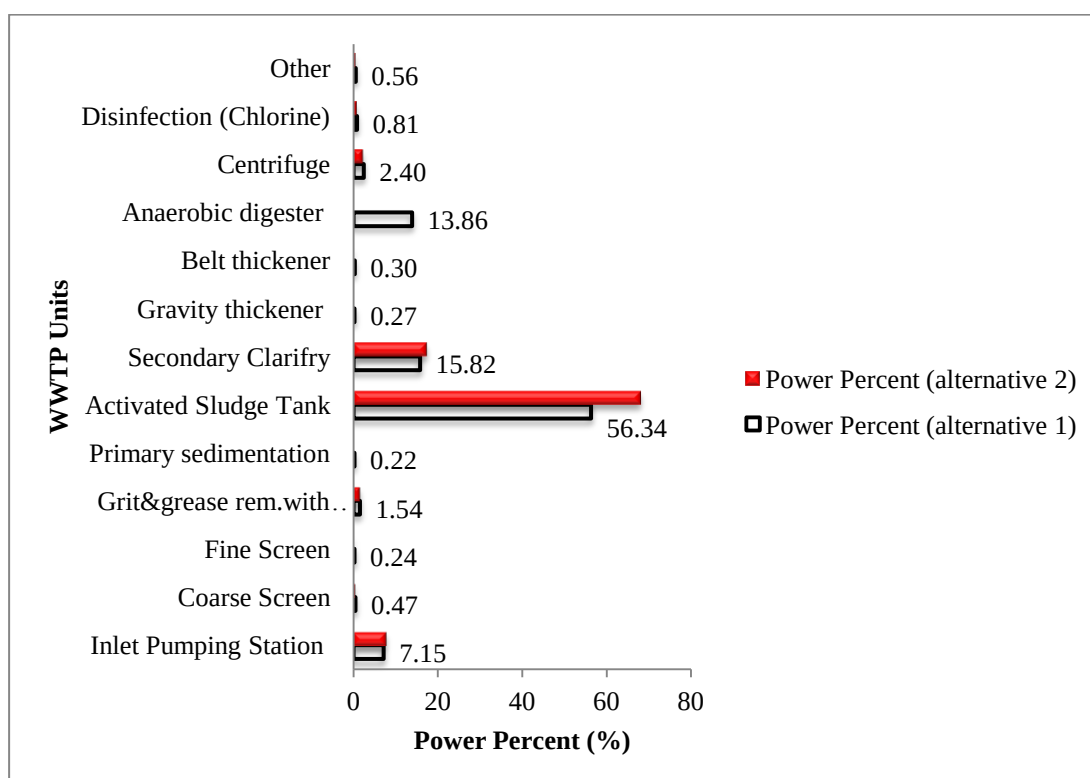


Figure 7.37 : Comparison of unit energy use profile for Soma WWTP.

When energy distribution of cities are compared, it is observed that pumps and aerations units are the main energy consumption units. This energy requirement, refers to electrical energy needed for wastewater treatment plant units. Total energy consumption for both systems were compared on the basis of population. That's given in Table 7.24, Table 7.25 and Table 7.26. The details of the calculation of energy recovery are given in the table C.6 in the attachment.

Table 7.24 : Comparison of electricity requirements for nutrient removal treatment plant.

Cities	Population	A ² O Energy Cons. (kwh/m ³)	A ² O Energy Consumption(kwh/PE.a)	A ² O Energy recovery (kwh/PE.a)	EA with N and P removal (kwh/PE.a)	EA with N and P removal (kwh/m ³)
Seydişehir	46,792	0.84	39.05	na	na	-
Akşehir	75,412	0.60	43.69	na	50.4	1.15
Aksaray	201,782	0.57	27.29	14.55	29.58	1.15
Adıyaman	245,490	0.55	26.95	14.01	30.90	1.08
Diyarbakır	1,430,000	0.43	21.51	12.66	26.15	0.98

When city graphic of the five cities where A²O process is applied (Seydişehir, Akşehir, Aksaray, Adıyaman and Diyarbakır), the necessary energy for the activated sludge tank in all cities, changes between the range of 52-62% of the total energy of the plant. As the population increases, in both processes, except Akşehir, requirement for unit energy decreases. The difference in Akşehir is due to the energy requirement of pumping stations. The energy requirement of pumping stations in other cities are valued at 11-12% while in Akşehir it is 16%. This has increased the requirement of energy unit. In Diyarbakır in addition to the existing treatment plant, in both alternatives for primary sedimentation sludge, anaerobic sludge digester is considered. The energy regain in the all 3 plants are of the order of 55%.

In table 7.25 even though the requirement for energy unit decreases according to the population, in both processes , choices of processes that require more energy like pumping stations, UV disinfections, and addition of gas production processes, unit

energy changes. Within the settlements in table 7.25 only in the wastewater treatment plant in Erzurum, 14.70 kwh/PE.a energy recovery is obtained.

Table 7.25 : Comparison of electricity requirements for C with N removal treatment plant.

		Flow m³/d	C with N removal Energy Cons. (kwh/m³)	C with N removal(kwh/PE.a)	EA with N removal (kwh/PE.a)
Cities	Population				
Soma	111,000	13,764	0.69	31.31	28.29
Polatlı	125,888	19,863	0.59	34.09	29.46
Ceyhan	136,220	34,899	0.41	38.51	33.76
Lüleburgaz	138,886	19,451	0.53	27.26	30.61
Erzurum	444,934	62,452	0.60	30.49	32.23

In Table 7.26, from the two different alternatives, chosen for the carbon removal and sludge treatment, aerobic sludge process requires 15-20% more energy.

Table 7.26 : Comparison of electricity requirements for C removal with different sludge alternatives.

Cities	Population	Flow m ³ /d	C removal with anaerobic sludge digestion (kwh/PE.a)	C removal with aerobic sludge digestion (kwh/PE.a)
Bartın	61,463	12,439	26.79	31.92
Merzifon	71,991	8,941	19.81	27.25
Çarşamba	81,347	12,936	26.15	30.99
Siverek	128,082	17,030	21.82	25.08

In the studies made by Kroiss et al. about the wastewater treatment plants in Austria, two systems with two different sludge alternatives for removing carbon, are compared according to their net energy requirement. The energy results of aerobic and anaerobic sludge stabilization, both of which have two different sludge alternatives, are evaluated and their annual energy requirement is found as 17 kwh/PE.a and 27 kwh/PE.a respectively. In the process where anaerobic sludge stabilization is applied, biogas production and net energy demand is calculated as 2 kwh/PE.a.

In wastewater treatment plants, energy consumption shows differences according to wastewater influent loads, the type of the chosen treatment and sludge treatment.

When the existing and establishing treatment plants are examined, it can be seen that one of the energy consumption points that should be considered, is pumping stations.

The energy requirement in pumping stations is 11.65% in Adıyaman, 15.95% in Akşehir, 15.43% in Bartın and 16.66% in Lüleburgaz, in Siverek goes in the plant with gravity. Depending on the settlement, it can increase in the big cities.

According to the activity report 2010 of İSKİ, 62 % of the energy used by İSKİ, is used in pumping stations. The energy requirement needed just for the municipal sewer wastewater treatment of İstanbul, is approximately 5% of the energy used in İstanbul. There fore when wastewater treatment plant is designed, hydraulic compatibility of the domain where the plant will be founded should be considered.

If we look at the energy potential of the amount of sludge produced from İstanbul municipal wastewater treatment plants, the sludge amount, calculated by accepting that existing treatment plant are advanced treatment, is 1,026,677kg/day (Çakmakçı 2005). If we accept that 50% of the sludge is coming from pre-sedimentation sludge, energy regain is calculated as 783,953 kwh/day. Calculation details are given in the Table C.5 in the attachment. The energy requirement of the pretreatment plant of Paşabahçe, which has a population of 2,000,000, is 644,450 kwh/d.

7.4 Energy self-sufficiency for municipal wastewater treatment plants

In this chapter, energy requirement details for A²O process for 50,000-200,000 population equivalent, that is classically designed according to population, and extended aeration process values, are given according to units, in the Table C1, Table C2, Table C3 and C4. For the energy requirement of pumping stations, 7.1 formula is used and the values found according to population are given below.

Assumming P_w and μ as follows,

$$P_w = \frac{Q H_m}{102 \mu} \quad (7.1)$$

$\mu = 0.70$, $H_m=10$ m, pump power is calculated using the table of standard pump power and 11 kw and 30 kw are found for 50,000 and 200,000, respectively.

In the conceptional design, the energy requirement calculated according to units are given in Table 7.27 and Figure 7.38. It is observed that when population increases, energy requirement per population decreases in Table 7.27 and Figure 7.38.

Table 7.27 : The energy requirements of the units according to the conceptual design and ATV output values.

Process	PE 50,000 A ² O	PE 50,000 EANR	PE 200,000 A ² O	PE 200,000 EANR
Inlet Pumping Station	211	211	1,152	1,152
Coarse screen	30	30	42	42
Fine screen	21	21	34	34
Grit and grease removal	104	104	460	460
Primary Sedimentation	68		111	
Biological Treatment	2,766	3,807	8,924	11,768
Secondary Sedimentation	561	549	1298	1275
Sludge treatment Units	815	363	1979	679
Total (kwh/d)	4,576	5,121	14,001	15,411
Total (kwh/PE.a)	33.41	37.38	25.55	28.12

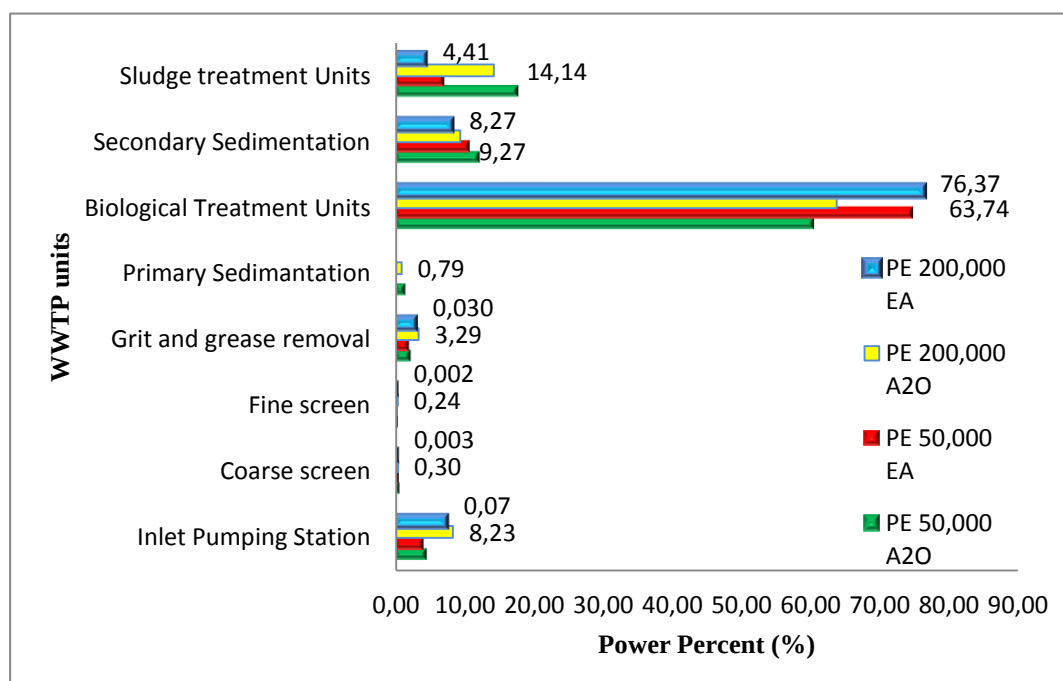


Figure 7.38 : Comparison of energy use profile for A²O and EANR process.

The amount of methane production calculated from data obtained from conceptual design, according to the population are given in Table 7.28, Table 7.29 and Table 7.30.

Table 7.28 : Anaerobic digestion primary and secondary sludge inlet values.

Operation	Unit	50,000	100,000	200,000	400,000	1,000,000
Density	kg/m ³	1,020	1,020	1,020	1,020	1,020
Primary Sludge	kg/d	1,287	2,574	5,610	11,827	31,680
Excess Sludge	kg/d	1,456	2,856	6,163	12,583	34,754
Total sludge production	kg/d	2,743	5,430	11,773	24,410	66,434
Solid content,inlet	%	4	4	4	4	4

Table 7.29 : Calculation table of biogas for primary and secondary sludge.

Operation	Unit	50,000	100,000	200,000	400,000	1,000,000
Primary Sludge	kg/d	1,287	2,574	5,610	11,827	31,680
VSS content of primary sludge	%	0.65	0.65	0.65	0.65	0.65
Primary sludge VSS, inlet digester	kg/d	837	1,673	3,647	7,688	20,592
Degradation	%	50	50	50	50	50
Primary sludge VSS, degraded	kg/d	418	837	1,823	3,844	10,296
Total primary sludge amount out of digester	kg/d	869	1,737	3,787	7,983	21,384
Excess Sludge	kg/d	1,456	2,856	6,163	12,583	34,754
VSS content of excess activated sludge	-	0.85	0.85	0.85	0.85	0.85
Excess sludge VSS, inlet digester	kg/d	1,238	2,428	5,239	10,696	29,541
Degradation	%	50	50	50	50	50
Excess sludge VSS, degraded	kg/d	619	1,214	2,619	5,348	14,770
Total excess sludge	kg/d	837	1,642	3,544	7,235	19,984
Total Sludge at inlet	kg/d	2,743	5,430	11,773	24,410	66,434
Total VSS, inlet digester	kg/d	2,074	4,101	8,885	18,383	50,133
Total sludge VSS, degraded	kg/d	1,037	2,050	4,443	9,192	25,066
Specific gas production (Nm ³ gas/VSS)		0.935	0.935	0.935	0.935	0.935
Gas production (primary sludge)	m ³ /d	391	782	1705	3,594	9,627
Gas production(excess sludge)	m ³ /d	579	1,135	2,449	5,000	13,810
Total Gas Production	m ³ /d	970	1,917	4,154	8,594	23,437

Table 7.30 : Calculation table of electrical and heat energy values from biogas.

Operation	Unit	50,000	100,000	200,000	400,000	1,000,000
Calorific value of digester gas	kJ/m ³	22,400	22,400	22,400	22,400	22,400
Calorific value of digester gas ¹	kwh/d	6,033	11,928	25,846	53,474	145,831
Design of CHP Units						
Gas recovered as electr. Energy	%	35	35	35	35	35
Gas recover.as heat energy	%	50	50	50	50	50
Electrical energy recovered	kwh/d	2,112	4,175	9,046	18,716	51,041
Heat energy recovered	kwh/d	3,017	5,964	12,923	26,737	72,916
Electrical energy recovered kwh/PE.d		15.42	15.24	16.51	17.08	18.63
Heat energy recovered kwh/PE.d		22.02	21.77	23.58	24.40	26.61

¹ kwh:3600 kj

The energy requirement of A²O process for population of 50,000, is calculated as 33.41 kwh/PE.a, and the recovery is found as 15.42 kwh/PE.a. Thus situated, as can be seen in Table 7.31, the net energy requirement is 17.99 kwh/PE.a. For a population of 200,000, this amount decreases to 9.04 kwh/PE.a. When compared with extended aeration process, the difference in energy requirement for a population of 200,000 is 19.08 kwh/PE.a (28.12 kwh/PE.a- 9.04 kwh/PE.a).

Table 7.31 : Energy requirement and recovery values for A²O and EANR process.

Process/Population	Unit	50,000	200,000
A ² O Energy requirement	kwh/PE.a	33.41	25.55
A ² O Energy recovery	kwh/PE.a	15.42	16.51
A ² O Net energy requirement	kwh/PE.a	17.99	9.04
EANR energy requirement	kwh/PE.a	37.38	28.12

When unit energy recovery is compared with the literature, in both population it is observed that values coincide. In the study made by Enderle et al. 2011, the municipality wastewater energy recovery is calculated as 15-18. This value coincides with the values in Table 7.32.

Table 7.32 : Specific energy content (COD, methane, thermal and electrical energy) of different sources (Enderle, 2011).

Parameter	Domestic wastewater	Black water (human excreta)	Sludge from municipal WWTP	“Bio-waste” collected	“Bio-waste” total (estimated)
COD _{total} [kg/(p.a)]	40	ca. 24	ca. 24	14	ca. 30
COD _{converted} [kg/(p.a)]	25-28	18-20	12-15	10-10,5	20-24
Methane[m ³ /(p.a)]	8,8-9,8	6,3-7	4,2-5,3	ca. 3,5	ca. 7.7
Energy _{th} [kwhth/(p.a)]	88-98	63-70	42-53	ca.35	ca. 77
Energy _{el} [kwh _{el} /(p.a)]	31-34	22-24	15-18	ca.12	ca.27

“converted” means “anaerobically convertible to CH₄”

7.5 Conclusions

In this chapter the energy requirement and energy recovery data of the municipal water treatment plants operated in our country and that are in process of design, are compared in kwh/m³ and kwh/PE.d, by taking their process into consideration, and the results are evaluated.

When the existing and establishing treatment plants are examined, it can be seen that one of the energy consumption points that should be considered, is pumping stations. For İstanbul it is stated that 62% of the energy used by İSKİ, is used in pumping stations (activity report of İSKİ). The value that changes according to location is as following in the cities where a plant is designed: 11.65% in Adıyaman, 15.95% in Akşehir, 15.43% in Bartın and 16.66% in Lüleburgaz, in Siverek it enters with gravity. Therefore when wastewater treatment plant is designed, hydraulic compatibility of the domain where the plant will be founded should be considered.

When kwh per m³ energy consumption of all the AAT (biological and advanced WWTP) in İstanbul is evaluated, it can be observed that the values for biological treatment 0.22-2,85 kwh/m³, for advanced treatment 0.25-0.38. When these values are compared with other biological municipality wastewater treatment plants, it is observed that these values coincide with values calculated as 0.18 kwh/m³ for

Balıkesir, 0.33kwh/m³ for Bursa, 0.35 kwh/m³ for Fethiye , 0.25 kwh/m³ for Sivas and 0.36 kwh/m³ for Siirt.

When A²O process is compared with the process of extended aeration with N and P removal in the municipal water treatment plants that are designed, A²O process, even without energy recovery, spends less energy than extended aeration process. When amount of energy regain is taken into consideration, extended aeration requires 2.4-2.8 times more energy (Table 7.25, Figure 7.39).

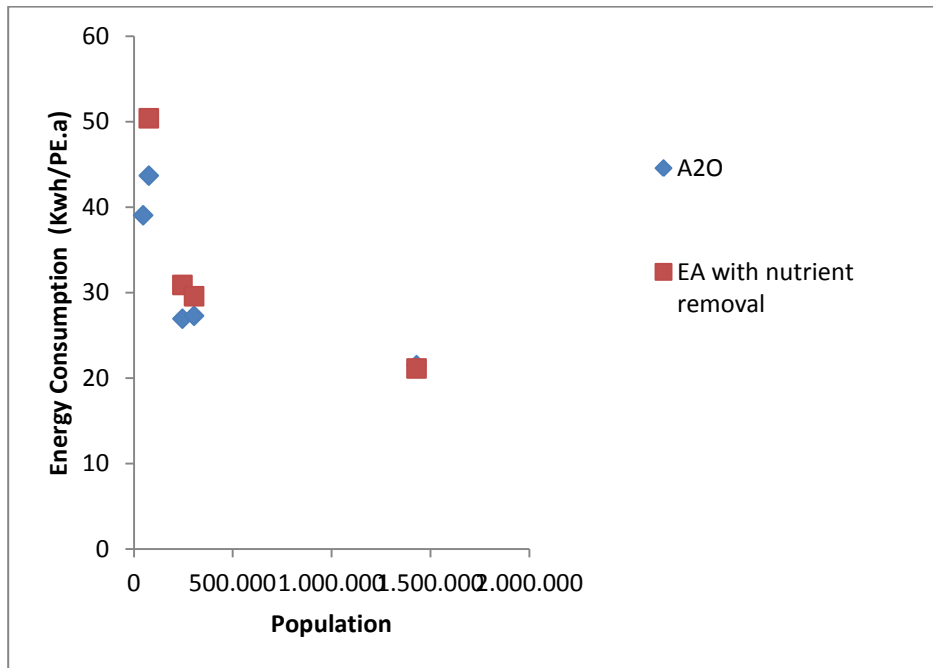


Figure 7.39 : Comparison of electricity requirements for nutrient removal WWTP.

In design projects, when conceptional N removal and extended aeration processes are compared in kwh/PE.a in the basis of energy consumption, it is observed that in both process when population increases, energy consumption decreases. On the otherhand as there are gas recovery units in the water treatment plants in Polatlı and Ceyhan, due to the energy requirements of these units, there is no change depending on the population (Table 7.26, Figure 7.40).

From the results obtained from advanced treatment plants that apply nitrification in United States, for 76,080m³/d wastewater flow , energy use is defined as 0.44 kwh/m³, and net energy use is defined as 0.35 kwh/m³. For 189,600 m³/d wastewater flow, 0.42kwh/m³ energy consumption and 0.33 kwh/m³ net energy consumption is defined. As can be seen in Table 5.2, 0.78 kwh/m³ energy consumption for 3,840

m³/d, drops to the value of 0.41 kwh/m³ as flow increases and for flow of 378,480 m³/d, net energy requirement drops to the value of 0.32 kwh/m³.

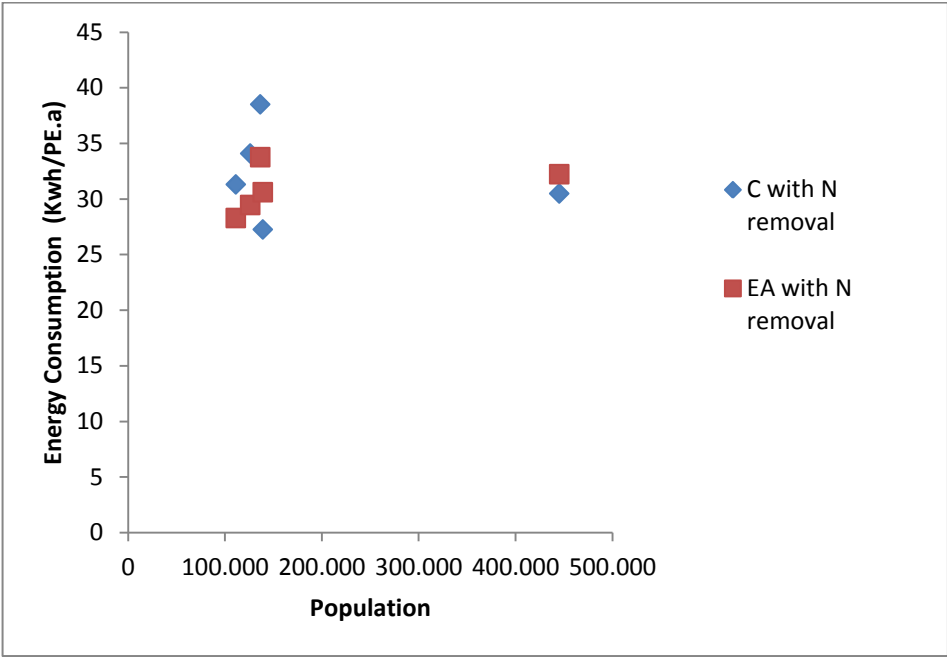


Figure 7.40 : Comparison of electricity requirements for C and N removal WWTP.

Also the plants that make only carbon removal and anaerobic and aerobic sludge stabilization, when population increases unit energy consumption (kwh/PE.a) decreases. As there are gas recovery units in Çarşamba, reduction depending on the population is not observed. As it can be seen in Figure 7.41, anaerobic sludge stabilization consumes less energy, therefore it is a more convenient solution.

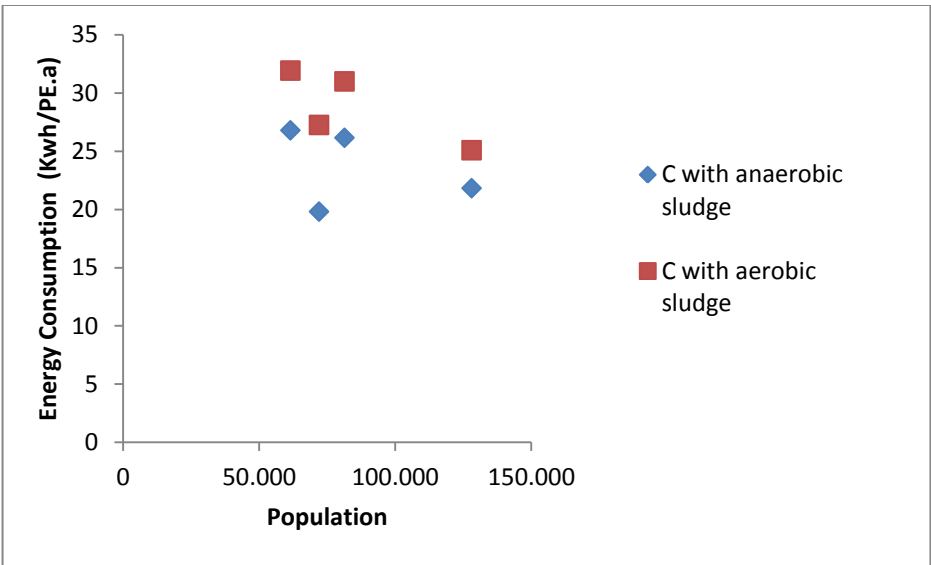


Figure 7.41 : Comparison of electricity requirements for C removal with sludge treatment alternatives.

Except the municipal AAT with gas utilization facilities, in all the chosen processes when population increases, energy consumption in kwh/PE.a decreases. When these processes are compared, EA process, in N removal as well as N and P removal, requires more energy compared with A²O and conventional activated sludge process.

The energy requirement of A²O process for population of 50,000, is calculated as 33.41 kwh/PE.a, and the recovery is found as 15.42 kwh/PE.a. Thus situated the net energy requirement is 17.99 kwh/PE.a. For a population of 200,000, this amount decreases to 9.04 kwh/PE.a. When compared with extended aeration process, the difference in energy requirement for a population of 200,000 is 19.08 kwh/PE.a (28.12 kwh/PE.a- 9.04 kwh/PE.a). In all the chosen processes as population increases unit energy consumption in kwh/PE.a reduces. When these processes are compared EA process both in N and N and P removal, requires more energy than A²O and conventional active sludge process.

In our country, there should be an energy focused approach in the running of the municipal treatment, energy production and energy requirement data should be measured in unit base and there should be a target for energy use reduction. In this context, it is necessary to share and compare the data of the municipality plants in our country, compare them with other countries, to put a target for use and regain of energy in the basis of the plant and continuous traceability must be ensured.

Energy potential is identified with Statistical analysis of the different wastewater treatment plants. Energy saving precaution can be developed. Monitoring and measuring of the parameters based on comparison are given in Table 7.33.

Table 7.33 : Treatment plants of energy efficiency benchmarking data table.

Parameter	Unit
Total specific energy consumption	kwh/(PE·a)
Specific energy consumption aerated tank	kwh/(PE·a)
Specific energy consumption dewatering	kwh/kg DS
Degree of gas reuse	%
Degree of gas conversion to power or electr.	%
Specific gas production per kg SS intake	L/kg SS
Anaerobic digester heat require	kwh/(PE·a)
Degree of self supply-Heat	%
Degree of self supply-Electricity	%

For energy optimization, a control and automation system including supervisory control and data acquisition (SCADA) must be created. With SCADA current status of processes and equipments can be monitored, alarms and work schedules can be controlled. By publishing the data in the digital media, persons concerned are provided access to these data.

8. CONCLUSION AND RECOMMENDATIONS

Within the study named Energy Based Approach in the Municipal Wastewater Treatment Plants, the energy requirement in the active sludge processes that are widely used in municipal wastewater treatment plants are evaluated depending on the population, an approach about energy potential depending on the population in municipal wastewater, and the energy requirement for the plant, also how much of this energy can be recovered by anaerobic sludge digestion is developed.

For this purpose, two parameters, wastewater flow-rate and pollutant load, that are important in the design of wastewater treatment plants, are evaluated in terms of municipal wastewater treatment plants, the studies in the influent of municipal wastewater treatment plants (15 different municipal wastewater treatment plants that are designed) and the values resulting in the treatment plants operated (15 different municipal wastewater treatment plants that are in use) in our country, are compared with the values from other countries and the results that are shown below, are reached.

As the result of the existing infrastructure in our country, in the determination of the influent values of wastewater treatment plants, the state of the infrastructure should be evaluated prior to measurements. In the assessment of the pollutant load of the wastewater treatment plants that are designed in different cities, it is observed that no long-term measurements but only short-term measurements or measurements on grab samples are made. When the short-term measurements are combined with the inadequacy of the infrastructure system, mostly the literature values are used for the influent concentrations during the design of treatment plants. The data from municipal wastewater treatment plants (influent values of 30 municipal wastewater treatment plant) evaluated in this study indicate a wide range for influent flowrate and pollutant loads. The statistical analysis of these data (Figure 3.10, Figure 3.11, Figure 3.12, Figure 3.13, Figure 3.14) shows that the influent flowrate is between 100 and 150 l/PE.d, COD is between 500 and 600 mg/L, SS is between 250 and 350 mg/L, TKN is between 55 and 65 mg/L and TP is between 12 and 15 mg/L.

Furthermore, according to data of TSI, the average wastewater generated in our country is between 100 and 150 L/PE day.

Therefore, in the determination of flowrate and influent pollutant loads of municipal treatment plants, first, the infrastructure should be adequate, then long-term measurements must be made, and at last, these values should be used for the design instead of literature values.

Problems faced in the operation and subjects that increase energy consumption should be analyzed, subjects like pumping stations and aeration which are the main source of energy consumption should be taken into consideration already at the design stage. This value which changes according to settlement conditions, changes in 14 different cities between the range of 0%-16.66%. Therefore during the planning of a wastewater treatment plant, the hydraulic compatibility of the area where the plant will be established, should taken into account.

In the design made in this study, A²O process, which is one of the processes for different populations (50,000-100,000-200,000-400,000 and 1,000,000) and used in nutrient removal , and also extended aeration process which is widely used in Turkey are taken as basis. As N and P limits are added to discharge standards in our country,by adding nutrient removal to extended aeration process, a third alternative is created. For these 3 design processes, reactor volume, amount of oxygen consumption and amount of sludge are calculated in ATV (A-131), which is a German standard active sludge design guide, and the results are compared. In addition, for A²O process conceptual calculation method is used and the results are compared with the values taken from ATV and the following results are obtained.

Whether there is nutrient removal or not, extended aeration process produces less sludge compared to A²O process. When the outputs of ATV program and conceptual design of A²O process are compared, ATV calculates approximately 10% less sludge.

Looking at oxygen requirement, although extended aeration process consumes less energy, as it requires more aeration, it also requires more energy. The oxygen requirement that is found in conceptual design is 55% less than the predenitrification with phosphorus removal (A²O) process that is found in ATV. When nutrient removal is made, extended aeration requires 20% less oxygen.

As extended aeration process works in high sludge age, reactor volume is approximately 4 times more. Reactor volume of A²O process calculated in ATV program is approximately the same value as in conceptual design. Extended aeration process with nutrient removal requires 2-4 %more reactor volume compared with carbon removal.

ATV-131 is a package program therefore provides practical measurement, also due to corrections in the check-points, it is convenient in taking results. On the other hand it does not allow alteration in management of system parameters like coefficients.

Assesment under 4 different titles have been made in terms of energy consumption and energy recovery in municipal wastewater treatment plants, which is the main subject in this study, and the following results are obtained.

Under the first title energy consumption of 20 biological treatment plants, 4 advanced biological treatment plants operated in İstanbul, are compared, according to their annual kwh energy consumption per population equivalent (kwh/PE.a) and their kwh energy consumption per m³ wastewater (kwh/m³).

Under the second title, energy consumption of municipal wastewater treatment plants in cities of Bursa, Fethiye, Siirt, Sivas and Balıkesir, annual kwh energy consumption per population equivalent and kwh energy consumption per m³ wastewater are compared.

Under the third title, energy consumption of cities of Seydişehir, Akşehir, Aksaray, Adıyaman and Diyarbakır where nutrient removal (A²O and extended aeration with nutrient removal) is made; Soma, Polatlı, Ceyhan, Lüleburgaz and Erzurum where C+N removal is made; Bartın, Merzifon, Çarşamba and Siverek where only carbon removal is made and where there is anaerobic sludge digester, are compared as annual kwh energy consumption per population equivalent (kwh/PE.a).

Under the fourth title, from the data from municipal wastewater treatment plants that are build in conceptual design in this study, energy consumption of A²O and extended aeration processes for population equivalent of 50,000 and 200,000, is calculated as annual kwh energy consumption per population equivalent (kwh/PE.a).

In the conceptual design study that is made in this study, for population of 50,000 and 200,000, the distribution of energy requirement of A²O process and extended

aeration activated sludge process according to units, is calculated and the ratio of energy requirement is defined in percentage. It is approximately calculated that in A²O process 5% of the energy consumption takes place in physical treatment unit, 60-63% in aeration unit, 10-12 % in final clarifier and sludge pumping (return and excess) system, and 15-17 % in sludge treatment. In the extended aeration process it is calculated that 3% of the energy consumption takes place in physical treatment unit, 70-75 % in aeration unit, 8-11% in final clarifier and sludge pumping system, and 5-7 % in the sludge treatment.

The energy requirement of A²O process for population of 50,000, is calculated as 33.41 kwh/PE.a, and the recovery is found as 15.42 kwh/PE.a. Thus situated the net energy requirement is 17.99 kwh/PE.a. For a population of 200,000, this amount decreases to 9.04 kwh/PE.a. When compared with extended aeration process, the difference in energy requirement for a population of 200,000 is 19.08 kwh/PE.a (28.12 kwh/PE.a- 9.04 kwh/PE.a). In all the chosen processes as population increases unit energy consumption in kwh/PE.a reduces. When these processes are compared EA process both in N and N and P removal, requires more energy than A²O and conventional active sludge process.

In İstanbul, in Ataköy and Paşaköy wastewater treatment plants where advanced treatment and energy recovery is done, unit energy consumptions per m³ are 0.38kwh/m³ and 0.37 kwh/m³. When these values are compared with evaluation results of WEF, year 2009, unit energy consumption is given as 0.41 kwh/m³ and net energy consumption is given as 0.32 kwh/m³.

Different design approaches have been evaluated in the context of design studies conducted by the Ministry of Environment and Urbanization. When the results of the A²O process and extended aeration activated sludge with nutrient removal process designs were compared for their energy requirement for 5 cities with populations ranging between 46,000 and 1,430,000, the per capita energy requirement increases due to the extended aeration cost, as provided in Figure 8.4. The increase is more pronounced for small population sizes. A²O process consumes less energy even without recovery, compared to extended aeration process. When their energy regain amount are taken into consideration, extended aeration requires 2.4-2.8 more energy.

In the context of design studies conducted by the Ministry of Environment and Urbanization when the results of conventional activated sludge with nitrogen and

extended aeration activated sludge with nitrogen removal were compared for their energy requirement for 5 cities with populations ranging between 111,000 and 4,450,000 (Figure 8.5), traditional activated sludge system is more convenient as the cost of the per capita energy of extended aeration is more. In both processes it is observed that as population increases energy consumption reduces. Similarly the kwh/m^3 energy consumption in wastewater treatment plants operated in different countries decrease with increasing population.

From the results obtained from advanced treatment plants that apply nitrification in United States, for $76,080\text{m}^3/\text{d}$ wastewater flow, energy use is defined as 0.44 kwh/m^3 , and net energy use is defined as 0.35 kwh/m^3 . For $189,600\text{ m}^3/\text{d}$ wastewater flow, 0.42kwh/m^3 energy consumption and 0.33 kwh/m^3 net energy consumption is defined. As can be seen in Table 5.2, 0.78 kwh/m^3 energy consumption for $3,840\text{ m}^3/\text{d}$, drops to the value of 0.41 kwh/m^3 as flow increases and for flow of $378,480\text{ m}^3/\text{d}$, net energy requirement drops to the value of 0.32 kwh/m^3 .

In the areas where only carbon removal is sufficient, for secondary treatment sludge stabilization, aerobic and anaerobic sludge stabilization are compared. Also in plants where only carbon removal is made together with aerobic and anaerobic sludge stabilization, as population increase unit energy consumption (kwh/PE.a) decreases.

From the researches in this study, the difficulties encountered, comparisons and analyses, the recommendations listed below are issued.

In our country, there should be an energy focused approach in the operation of the municipal treatment, energy production and energy requirement data should be measured in unit base and there should be a target for reduction in energy consumption. In this context, it is necessary to share and compare the data of the municipal plants in our country, compare them with other countries, to put a target for use and recovery of energy in the basis of the plant and continuous traceability must be ensured. Other facilities operated by the unit energy consumption benchmarking is gaining importance for energy optimization. Energy potential is identified with Statistical analysis of the different wastewater treatment plants. Energy saving precaution can be developed. Proposed monitoring and measurement parameters are given in Table 7.35.

In the application in different countries, applications in nutrient removal is made more often, discharge limits of N and P are lowered, on the other hand in our country, in the process of harmonization with EU, these parameters are set only in the sensitive areas. As in the municipal wastewater treatment direction of EEC, not only in sensitive areas but in all areas N and P limits should be added.

It is known that water consumption values used in calculation of wastewater that comes to municipal treatment plants, cannot reflect reality due to loss and leakage. In order to reduce the network loss which is calculated as approximately 51% in general in Turkey, priority must be given to necessary infrastructure plans.

In order to see all the direct and indirect energy consumption in wastewater treatment plants, energy requirements of activities such as use of chemicals, sludge transport must be taken into consideration.

Energy based approach should be planned in designing stage. The processes that are chosen must be evaluated in energy basis, the problems in the operation and the subjects that increase energy consumption must be analysed and must be taken into consideration in the future design. The researches prove that by taking measures in the main sources of energy consumption, specifically pumps and ventilation, an energy saving up to 30% can be made.

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APPENDICES

APPENDIX A: Influent Concentration data table of urban wastewater treatment plants in Turkey

APPENDIX B: Design and operation data table for biological nutrient removal systems

APPENDIX C: Energy consumption and recovery calculation tables

APPENDIX D: Output of the ATV program

Appendix A:Influent Concentration data table of urban wastewater treatment plants in Turkey.

Table A.1 : Influent concentration of Paşaköy Advanced wastewater treatment plant (Sözen,2008).

Sample time Parameter	09:00- 11:00	11:00- 13:00	13:00- 15:00	15:00- 17:00	17:00- 19:00	19:00- 21:00	21:00- 23:00	23:00- 01:00	01:00- 03:00	03:00- 05:00	05:00- 07:00	07:00- 09:00	Composite 9:00-9:00
TCOD	400	525	650	675	650	670	610	635	530	340	305	390	520
COD settled	270	260	455	350	325	325	260	290	290	185	130	190	300
COD filtered	135	190	255	275	265	260	195	250	215	135	120	130	150
UYA	4.9	11	13.8	18.9	19.7	22.5	14.1	13.4	8.7	1.3	1	1.8	4.9
P	7.3	9.1	10	11	10.7	9.7	9	9.4	9.2	7.1	7	9.3	7.3
TP	9.5	11.7	14.2	8.4	8.9	9.8	12.1	11.4	10.4	8	8.6	11.8	9.5
TKN	78	74	70	72	60	61	58	62	63	45	43	52	78
NH ₄ -N	57.6	52.1	48	48.7	38.9	37.5	39.8	41	43.7	32.4	28.6	41.2	57.6
NO ₃ N	0.13	0.33	0.11	0.2	0.2	0.24	0.11	0.1	0.21	0.15	0.12	0.13	0.13
SS	270	325	430	390	360	440	435	410	305	185	205	270	270
VSS	185	225	325	270	260	340	260	260	210	140	125	175	185
Alkalinity	439	435	425	434	396	376	384	386	392	346	336	372	439
pH	7.87	7.6	7.53	7.42	7.66	7.47	7.55	7.76	7.67	7.76	7.76	7.76	7.87
Chloride	133	132	125	148	131	139	149	138	194	119	128	120	133
Sulphate	99	113	124	129	126	120	125	119	123	105	110	103	99

Table A.2 : Evaluated minicipal wastewater characterization in Istanbul (Orhon, 2000)

Parameter (mg/l)	Kadıköy I- K.Çekmece -Baltalimanı			Ataköy		Kadıköy II		Tuzla	
	Mean	Standard deviation	Range	Mean	Range	Mean	Range	Mean	Range
COD Total	410	110	220-775	270	160-350	594	315-870	608	485-715
Settled	300	60	230-525			416	300-585		
Dissolved	140	30	85-210	114	60-180	188	130-240	215	175-235
BODs Total	180	42	73-410						
Settled	145	33	100-200						
Dissolved	68	14	43-170						
TKN Total	43	8.5	22-73	37	22-63	74	45-118	78	60-120
Settled	4.1	7.5	18-64			59	44-73		
Dissolved	38	7.5	8-57			57	38-68		
NH ₃ -N				23	12-40	45	32-61	52	41-62
TP Total	7.2	2	5-15	7	3-12	11	7-14	10.3	2-23
Settled	6	2.5	4.5-14.5			9.8	6.5-11.3		
Dissolved	4.5	2	2.2-10			8.1	5.1-8.5		
SS Total	210	105	85-930			417	220-504	323	240-400
Settled	115	15	95-132						
VSS Total	145	38	100-395			325	190-430	260	200-360
Settled	98	13	75-120						
Alkalinity				212	190-260	418	260-495		
pH				7.1	6.8-7.5	7.2	7.1-7.2	7.17	7.1-7.3

Table A.3 : Influent concentration of Sivas wastewater treatment plant (Sistemyapı,2009)

2009	FLOW (m ³ /day)	pH	°C	COD mg/l	Rb COD mg/l	FCOD mg/l	SS mg/l	BOD mg/l	TN mg/l	NH ₄ -N mg/l	TP mg/l	Alk. mg/l
Design	78,500	5-11	10-25	533	-	-	267	267	63	-	11	-
January	160,810	8		632		183	246	250	76	46	12	
February	50,358	8	9	558		145	310	242	49	46	7	444
March	72,235	8	10	431	183	120	304	223	41	27	6	401
April	66,505	8	11	280	152	112	224	179	28	21	4	377
Maı	65,623	8	11	280	152	112	224	179	28	21	4	377
June	59,385	8	16	330	192	125	204	198	27	22	4	413
July	46,406	8	18	371	221	156	202	216	36	30	5	424
August	46,288	8	18	403	252	174	239	258		40		430
September	48,036	8	18	422	251	190	237	262		31		495
October	47,097	8	16	413	209	168	245	245	48	33	6	485
November	52,049	8	13	415	223	157	268	213	40	29	6	473
December	41,814	8	12	387	223	146	209	208	48	32	6	448
Average	63,050	8	14	410	206	149	243	223	42	31	6	434

Table A.4 : Influent concentration of Siirt wastewater treatment plant (Sistemyapı,2009)

2009	Flow m3/day	oC	pH	conduct mS/cm	Raw Wastewater					
					COD mg/l	BOD mg/l	TN mg/l	TP mg/l	NH4-N	TSS mg/l
Design	18,686	8-26	6,0 - 9,0		632	271	86	14		310
January	1,716	12.9	8.1		663	288	67.8	8.0		347
February	10,109	12.6	8.0		722	210	68.5	8.7		283
March	9,318	13.1	8.0	785	650	218	71.1	7.5		337
April	13,335	14.8	8.0	655.8	584.6	199.6	63.9	7.0	28.8	333.5
May	15,494				604	259	58	8	44	320
June	14,811				589		47	6		288
July	14,715				481		65	7		380
August	13,805							7.5		456
September	13,473									
October	13,261				826	397	96	11.4	5	656
November	13,387				496	285	63	7.4		379
December	12,330				569	314	76	9.8		387
Average	12,146	13	8	720	618	271	68	8	26	379

Table A.5 : Influent concentrations of Balıkesir wastewater treatment plant (Sistemyapı,2009).

	Flow m ³ /d	Oil and grease mg/l	pH	SS mg/l	COD mg/l	F. COD mg/l	BOD ₅ mg/l
January	49,245	57	7.3	295	1,024	252	352
February	47,140	56	7.4	313	979	251	336
March	54,720	65	7.3	403	1,150	253	390
April	49,645	71	7.3	493	1,174	254	402
Mai	56,770	67	7.5	444	1,011	253	340
June	53,259	57	7.4	378	973	254	331
July	44,760	60	7.4	414	1,008	294	339
August	38,175	60	7.2	429	964	258	328
September	49,245	56	7.1	466	989	264	344
October	48,432	57	7.1	429	981	262	344
November	52,230	58	7.2	418	919	252	307
December	51,365	60	7.2	416	930	254	324
Average	49,582	60	7.0	408	1,009	258	345

Table A.6 : Influent concentrations of Fethiye wastewater treatment plant (Sistemyapı,2009).

	Flow m ³ /day	°C	COD mg/l	BOD mg/l	COD kg/d	TN mg/l	TP mg/l	NH ₄ -N mg/l	TSS mg/l
January	42,147	17	219	105	9,049	18	2	9	140
February	45,288	17	203	119	9,177	15	2	7	155
March	39,255	17	181	121	7,091	15	2	7	138
April	35,830	19	187	106	6,668	16	2	7	133
Mai	33,566	21	260	131	8,500	20	3	13	135
June	27,167	22	291		7,866	19	3	10	154
July	25,750	24	468	230	12,045	29	5	19	322
August	25,454	23	507	242	13,002	32	5	18	426
September	28,068	22	511	265	14,202	23	1	5	234
October	26,829	23	400	237	10,733	29	4	14	226
November	30,568	21	331	152	10,113	22	3	14	232
December	44,354	19	244	131	10,557	18	4	10	168
Average	33,690	20	317	167	9,917	21	3	11	205

Table A.7 : Influent concentrations of Bursa wastewater treatment plant (Sistemyapı,2009).

	Flow	pH	°C	COD mg/l	FCOD mg/l	SS mg/l	BOD ₅ mg/l	TN mg/l	NH ₄ - N mg/l	TP mg/l
Design	240,000	5-11	10-25	533	-	267	267	63	-	11
January	186,472	7.62	14.0	528.16	174	215	225	54	33	10.8
February	206,604	7.62	14.0	425.86	148	165	195	43	26	7.6
March	206,217	7.64	14.4	379.52	125	146	170	42	25	6.9
April	199,320	7.77	16.6	413.37	132	180	179	48	29	7.4
Mai	184,797	7.72	19.2	439.48	135	182	204.8	54	30	9.2
June	182,926	7.74	22.4	503.6	156	210	227.5	58	31	8.7
July	152,259	7.72	24.4	464.03	137.8	193	211	50.1	30	8.0
August	169,678	7.71	24.8	518.94	180.3	204	214	55.71	34	8.7
September	176,991	7.66	24.0	524	185	214	243	56.8	35	8.6
October	177,191	7.68	22.3	493.97	174	199	223	53.13	33	8.1
November	188,798	7.71	18.8	478	158	213	232	49.63	31	7.4
December	198,320	7.67	16.7	480.94	169	204	213	49.39	30	7.4
Average	185,798	7.69	19.3	471	156	194	211	51	31	8.2

Appendix B: Design and operation data table for biological nutrient removal systems.

Table B.1 : Advantages and limitations of nitrogen removal processes (Metcalf & Eddy,2003).

Process	Advantages	Limitations
Preanoxic-general	Saves energy; BOD is removed before aerobic zone. Alkalinity is produced before nitrification. Design includes an SVI selector.	
MLE	Very adaptable to existing activated-sludge processes. 5 to 8 mg/L TN is achievable.	Nitrogen-removal capability is a function of internal recycle. Potential Nocardia growth problem. DO control is required before recycle
Step feed	Adaptable to existing step-feed activated-sludge processes With internal recycle in last pass, nitrogen concentrations less than 5 mg/L are possible 5 to 8 mg/L TN is achievable.	Nitrogen-removal capability is a function of flow distribution. More complex operation than MLE; requires flow split control to optimize operation. Potential Nocardia growth problem. Requires DO control in each aeration zone
Sequencing batch reaktor	Process is flexible and easy to operate. Mixed-liquor solids cannot be washed out by hydraulic surges because flow equalization is provided. Quiescent settling provides low effluent TSS concentration. 5 to 8 mg/L TN is achievable.	Redundant units are required for operational reliability unless aeration system can be maintained without draining the aeration tank. More complex process design. Effluent quality depends upon reliable decanting facility. May need effluent equalization of batch discharge before filtration and disinfection.
Batch decant	5 to 8 mg/L TN is achievable Mixed-liquor solids cannot be washed out by hydraulic surges.	Less Flexible to operate than SBR Effluent quality depends upon reliable decanting facility.
Bio-denitro TM	5 to 8 mg/L TN is achievable Large reactor volume is resistant to shock loads.	Complex system to operate. Two oxidation ditch reactors are required; increases construction cost.
Nitrox TM	Large reactor volume is resistant to shock loads. Easy and economical to upgrade existing oxidation ditch processes. Provides SVI control	Nitrogen-removal capability is limited by higher influent TKN concentrations Process is susceptible to ammonia bleed-through. Performance is affected by influent variations.
Bardenpho (4-stage)	Capable of achieving effluent nitrogen levels less than 3 mg/L	Large reactor volumes required Second anoxic tank has low efficiency.
Oxidation ditch	Large reactor volume is resistant to load variations without affecting effluent quality significantly. Has good capacity for nitrogen removal; less than 10 mg/L effluent TN is possible	Nitrogen-removal capability is related to skills of operating staff and control methods
Postanoxic with carbon addition	Capable of achieving effluent nitrogen levels less than 3 mg/L May be combined with effluent filtration	Higher operating cost due to purchase of methanol. Methanol feed control required.
Simultaneous nitrification/denitrification	Low effluent nitrogen level possible (3mg/L lower limit). Significant energy savings possible. Process may be incorporated into existing facilities without new construction. SVI control enhanced Produces alkalinity	Large reactor volume; skilled operation also required. Process control system required.

Table B.2 : Advantages and limitations of phosphorus removal processes (Metcalf & Eddy,2003).

Process	Advantages	Limitations
Phoredox (A/O)	Operation relatively simple when compared to other processes. Low BOD/P ratio possible. Relatively short hydraulic retention time. Produces good settling sludge. Good phosphorus removal.	Phosphorus removal declines if nitrification occurs. Limited process control flexibility
A ² O	Removes both nitrogen and phosphorus. Provides alkalinity for nitrification. Produces good settling sludge. Operation is relatively simple. Saves energy.	RAS containing nitrate is recycled to anaerobic zone, thus affecting phosphorus removal capability. Nitrogen removal is limited by internal recycle ratio. Needs higher BOD/P ratio than the A/O process.
UCT	Nitrate loading on anaerobic zone is reduced, thus increasing phosphorus removal capability. For weaker wastewater, process can achieve improved phosphorus removal. Produces good settling sludge. Good nitrogen removal.	More complex operation. Requires additional recycle system.
VIP	Nitrate loading on anaerobic zone is reduced, thus increasing phosphorus removal capability. Produces good settling sludge. Requires lower BOD/P ratio than UCT.	More complex operation. Requires additional recycle system. More equipment required for staged operation.
Bardenpho (5-stage)	Can achieve 3 to 5 mg/l TN in unfiltered effluent. Produces good settling sludge.	Less efficient phosphorus removal. Requires larger tank volumes
SBR	Both nitrogen and phosphorus removal are possible. Process is easy to operate. Mixed liquor solids cannot be washed out by hydraulic surges. Quiescent settling may produce lower effluent TSS concentration. Flexible operation	More complex operation for N and P removal. Needs larger volume than SBR for N removal only. Effluent quality depends upon reliable decanting facility. Design is more complex. Skilled maintenance is required. More suitable for smaller flowrates.
PhoStrip	Can be incorporated easily into existing activated-sludge plants. Process is flexible; phosphorus-removal performance is not controlled by BOD/P ratio. Significantly less chemical usage than mainstream chemical precipitation process. Can achieve reliable effluent orthophosphate concentrations less than 1mg/l.	Requires lime addition for phosphorus precipitation. Requires higher mixed-liquor dissolved oxygen to prevent phosphorus release in final clarifier. Additional tank capacity required for stripping. Lime scaling may be a maintenance problem.

Table B.3 : Comparison of alternative methods for dewatering various types of sludge and biosolids (Metcalf & Eddy,2003).

Dewatering method	Advantages	Disadvantages
Solid-bowl centrifuge	Clean appearance, good odor containnet, fast startup and shutdown capabilites	Scrool ewar potentially a high maintenance problem
	Produces a relatively dry sludge cake	Requires grit removal and possibly a sludge grinder in the feed stream
	Low capital cost-to-capacity ratio	Skilled maintenance personnel required
Belt-filter press	High installed capacity to building area ratio	Moderately high suspended solids content in centrate
	Low energy requirements	High odor potential
	Relatively low capital and operating costs	Requires sludge grinder in feed stream
	Less complex mechanically and is easier to maintain	Very sensitive to incoming sludge feed characteristics
	High-pressure machines are capable of producing very dry cake	Automatic operation generally not advised
Recessed-plate filter press	Minimal effort required for system shutdown	
	Highest cake solids concentration	Batch operation
Sludge drying beds		High equipment cost
		High labor cost
		Special support structure requirements
		Large floor area required for equipment
		Skilled maintenance personnel required
		Additional solids due to large chemical addition require disposal
Sludge lagoons	Lowest capital cost method where land is readily available	Requires large area of land
	Small amount of operator altention and skill required	Requires stabilized stabilized sludge
	Low energy consumption	Design requires consideration of climatic affects
	Low to no chemical consumption	Sludge removal is labor-intensive
	Less sensitive to sludge variability	
	Higher solids content than mechanical methods	
Sludge lagoons	Low energy consumption	Potential for odor and vector problems
	No chemical consumption	Potential for groundwater pollution
	Organic matter is further stabilized	More land-intensive than mechanical methods
	Low capital cost where land is available	Appearance may be unsightly
	Least amount of skill required for operation	Design requires consideration of climatic affects

Appendix C: Energy consumption and recovery calculation tables

Table C.1 : Energy consumption profile for A²O processes (50,000 population).

Equipment Name	Amount	Power (Kw)	Time	Power kwh/d ⁽¹⁾	Unit power
Inlet Pumping Station	1	11.00	24	211	211
Coarse screen					30
Coarse Screens 40 mm bar space	2	1.00	6	10	
Coarse Screens 20 mm bar space	2	1.00	6	10	
Belt Conveyor for Coarse Screens	2	1.10	6	11	
Fine screen					21
Fine Screens	1	1.10	6	5	
Belt Conveyor for Fine Screens	1	1.10	6	5	
Screening Press for Fine Screens	1	2.20	6	11	
Grit and grease removal					104
Scrapers for Grit and Grease Chambers	2	0.55	8	7	
Grit Pumps	2	2.40	4	15	
Grit Classifier	1	0.75	4	2	
Grease Pumps	1	0.75	4	2	
Blowers for grid and grease chambers	1	4	24	77	
Primary Sedimentation					68
Scrapers for Primary Sedimentation Tanks	2	1.10	24	42	
Scum Pumps	2	1.50	4	10	
Primary Sludge Pumps	1	1.70	12	16	
Biological Treatment Units					2,766
Mixer for Anaerobic Tanks	1	1.10	24	21	
Blowers for Aeration Tanks	2	75.00	20	2,400	
Air Fans	2	3.00	3	14	
Mixer for Aeration Tanks	4	4.30	24	330	
Secondary Sedimentation					561
Scrapers for Secondary Sedimentation Tanks	2	0.55	24	21	
Return Sludge Pumps	2	13.50	24	518	
Excess Sludge Pumps	1	1.70	16	22	
Sludge treatment Units					815
Gravity Thickener for Primary Sludge	1	1.10	24	21	
Thickened sludge pumps	1	4.00	12	38	
Mechanical Thickener for Excess Sludge	2	2.20	12	42	
Poly Preparation Unit for Excess Sludge	1	1.50	12	14	
Poly Dosing Pumps for Excess Sludge	2	0.37	12	7	
Mixer for Sludge Blending Tank	1	1.10	24	21	
Digester Feed Pumps (from blender to dgester)	1	3.00	24	58	
Anaerobic Digester	2	6.00	24	230	
Sludge Recirculation Pumps	1	3.57	24	69	
Centrifugal Decanter	2	13.50	12	259	
Poly Preparation Unit for Dewatering	1	1.50	12	14	
Poly Dosing Pumps for Dewatering	1	0.25	12	2	
Supernatant Pumps	1	4.00	12	38	
Total (kwh/d)					4,576
Total (kwh/PE.a)					33.41

⁽¹⁾Multiplied by 0.8 for obtained energy

Table C.2 : Energy consumption profile for extended aeration processes (50,000 population).

Equipment Name	Amount	Power (Kw)	Time	Power kwh/d ⁽¹⁾	Unit power
Inlet Pumping Station	1	11.00	24	211	211
Coarse screen					30
Coarse Screens 40 mm bar space	2	1.00	6	10	
Coarse Screens 20 mm bar space	2	1.00	6	10	
Belt Conveyor for Coarse Screens	2	1.10	6	11	
Fine screen					21
Fine Screens	1	1.10	6	5	
Belt Conveyor for Fine Screens	1	1.10	6	5	
Screening Press for Fine Screens	1	2.20	6	11	
Grit and grease removal					104
Scrapers for Grit and Grease Chambers	2	0.55	8	7	
Grit Pumps	2	2.40	4	15	
Grit Classifier	1	0.75	4	2	
Grease Pumps	1	0.75	4	2	
Blowers for grid and grease chambers	1	4	24	77	
Biological Treatment Units					3,807
Mixer for Anaerobic Tanks	2	2.30	24	88	
Blowers for Aeration Tanks	3	75.00	20	3,600	
Mixer for Aeration Tanks	2	3.10	24	119	
Secondary Sedimentation					585
Scrapers for Secondary Sedimentation Tanks	2	0.37	24	14	
Return Sludge Pumps	2	13.50	24	518	
Excess Sludge Pumps	1	5.50	12	53	
Sludge treatment Units					363
Mixer for Sludge Buffer Tank	1	1.10	24	21.12	
Mechanical Thickener for Excess Sludge	1	3.00	12	28.80	
Mechanical Thickener Feed Pumps	2	2.20	12	42.24	
Poly Preparation Unit for Excess Sludge	1	1.50	12	14.40	
Poly Dosing Pumps for Excess Sludge	2	0.37	12	7.10	
Poly Preparation Unit for Dewatering	1	1.50	8	9.60	
Poly Dosing Pumps for Dewatering	2	0.25	8	3.20	
Sludge Dewatering Feed Pumps	2	3.00	8	38.40	
Centrifugal Decanter	2	13.50	8	172.80	
Supernatant Pumps	1	4.00	8	25.60	
Total (kwh/d)					5,121
Total (kwh/PE.a)					37.38

⁽¹⁾ Multiplied by 0.8 for obtained energy

Table C.3 : Energy consumption profile for A²O processes (200,000 population).

Equipment Name	Amount	Power (Kw)	Time	Power kwh/d ⁽¹⁾	Unit power
Inlet Pumping Station	2	30.00	24	1,152	1,152
Coarse screen					42
Coarse Screens 40 mm bar space	3	1.10	6	16	
Coarse Screens 20 mm bar space	3	1.10	6	16	
Belt Conveyor for Coarse Screens	2	1.10	6	11	
Fine screen					34
Fine Screens	3	1.27	6	18	
Belt Conveyor for Fine Screens	1	1.10	6	5	
Screening Press for Fine Screens	1	2.20	6	11	
Grit and grease removal					460
Scrapers for Grit and Grease Chambers	2	0.55	8	7	
Grit Pumps	2	2.40	4	15	
Grit Classifier	1	1.50	4	5	
Grease Pumps	2	1.70	4	11	
Blowers for grid and grease chambers	2	11	24	422	
Primary Sedimentation					111
Scrapers for Primary Sedimentation Tanks	2	1.10	24	42	
Scum Pumps	2	1.70	4	11	
Primary Sludge Pumps	2	3.00	12	58	
Biological Treatment Units					8,924
Mixer for Anaerobic Tanks	2	2.30	24	88	
Blowers for Aeration Tanks	2	250	20	8,000	
Air Fans	3	3.00	3	22	
Mixer for Aeration Tanks	8	5.30	24	814	
Secondary Sedimentation					1,298
Scrapers	4	1.10	24	84	
Return Sludge Pumps	2	30	24	1,152	
Excess Sludge Pumps	2	2.4	16	61	
Sludge treatment Units					1,979
Gravity Thickener for Primary Sludge	1	1.10	24	21	
Thickened sludge pumps	2	2.20	12	42	
Mechanical Thickener for Excess Sludge	2	3.00	12	58	
Poly Preparation Unit for Excess Sludge	1	1.70	12	16	
Poly Dosing Pumps for Excess Sludge	1	0.75	12	7	
Mixer for Sludge Blending Tank	2	5.50	24	211	
Digester Feed Pumps	2	3.00	24	115	
Anaerobic Digester	3	7.50	20	360	
Sludge Recirculation Pumps	3	9.00	24	518	
Centrifugal Decanter	2	27.50	12	528	
Poly Preparation Unit for Dewatering	1	2.30	12	22	
Poly Dosing Pumps for Dewatering	1	0.30	12	3	
Supernatant Pumps	2	4.00	12	77	
Total (kwh/d)					14,001
Total (kwh/PE.a)					25.55

⁽¹⁾ Multiplied by 0.8 for obtained energy

Table C.4 : Energy consumption profile for extended aeration processes (200,000 population).

Equipment Name	Amount	Power (Kw)	Time	Power kwh/d ⁽¹⁾	Unit power
Inlet Pumping Station	2	30	24	1,152	1,152
Coarse screen					42
Coarse Screens 40 mm bar space	3	1.1	6	16	
Coarse Screens 20 mm bar space	3	1.1	6	16	
Belt Conveyor for Coarse Screens	2	1.1	6	11	
Fine screen					34
Fine Screens	3	1.27	6	18	
Belt Conveyor for Fine Screens	1	1.1	6	5	
Screening Press for Fine Screens	1	2.2	6	11	
Grit and grease removal					460
Scrapers for Grit and Grease Chambers	2	0.55	8	7	
Grit Pumps	2	2.4	4	15	
Grit Classifier	1	1.5	4	5	
Grease Pumps	2	1.7	4	11	
Blowers for grid and grease chambers	2	11	24	422	
Biological Treatment Units					11,768
Mixer for Anaerobic Tanks	2	2.3	24	88	
Blowers for Aeration Tanks	2	300	20	9,600	
Air Fans	3	3	3	22	
Mixer for Aeration Tanks	16	6.7	24	2,058	
Secondary Sedimentation					1,275
Scrapers for Secondary Sedimentation Tanks	4	1.1	24	84	
Return Sludge Pumps	2	30	24	1,152	
Excess Sludge Pumps	2	2	12	38	
Sludge treatment Units					679
Mixer for thickened Excess Sludge Buffer Tank	1	4.3	24	82.56	
Mechanical Thickener for Excess Sludge	2	3	12	57.60	
Mechanical Thickener Feed Pumps	2	3	12	57.60	
Poly Preparation Unit for Excess Sludge	1	1.8	12	17.28	
Poly Dosing Pumps for Excess Sludge	2	0.37	12	7.10	
Poly Preparation Unit for Dewatering	1	3.25	8	20.80	
Poly Dosing Pumps for Dewatering	2	0.55	8	7.04	
Sludge Dewatering Feed Pumps	2	4	8	51.20	
Centrifugal Decanter	2	27.5	8	352.00	
Supernatant Pumps	1	4	8	25.60	
Total (kwh/d)					15,411
Total (kwh/PE.a)					28.12

⁽¹⁾ Multiplied by 0.8 for obtained energy

Table C.5 : Calculated amount of energy recovery for İstanbul Wastewater Treatment Plant designed as advanced treatment.

Process	unit	Value
Anaerobic Digester operating time	h/d	24
Primary Sludge density	kg/m ³	1020
Primary Sludge	kg/d	513.339
Excess Sludge	kg/d	513338,5
Flow, sludge	m ³ /d	12.582
Total sludge production	kg/d	1.026.677
Primary Sludge DS-Load	kg/d	513.339
VSS content of primary sludge	%	0,65
Primary sludge VSS, inlet digester	kg/d	333.670
Degradation	%	50
Primary sludge VSS, degraded	kg/d	166.835
Total primary sludge amount out of digester	kg/d	346.503
Excess Sludge DS-Load	kg/d	513.339
VSS content of excess activated sludge	-	0,85
Excess sludge VSS, inlet digester	kg/d	436.338
Degradation	%	50
Excess sludge VSS, degraded	kg/d	218.169
Excess sludge VSS, degraded, based on operation period	kg/h	9.090
Total excess sludge amount out of digester	kg/d	295.170
Total Sludge DS-Load at inlet	kg/d	1.026.677
Total VSS, inlet digester	kg/d	770.008
Total sludge VSS, degraded	kg/d	385.004
Solids content at inlet	%	4,0
Inlet Sludge flow	m ³ /d	25.164
Specific gas Production	Nm ³ gas/VSS	0,935
Gas production from primary sludge	m ³ /d	155991
Gas production from biological sludge	m ³ /d	203988
Total Gas Production	m ³ /d	359979
Specific gas Production	lt/kgVSS	450
Gas production from primary sludge	m ³ /d	150.152
Gas production from biological sludge	m ³ /d	196.352
Total Gas production	m ³ /d	346.503
Average gas production	m ³ /h	14.438
Calorific value of digester gas	kJ/m ³	22.400
		8.063.521
Total calorific value of digester gas	kJ/d	.158
Total calorific value of digester gas ¹	kwh/d	2.239.867
Total calorific value of digester gas based on 24 h/d operation	kwh	93.328
Percent of digester gas recovered as electr. energy	%	35
Percent of digester gas recover.as heat energy	%	50
Electrical energy recovered	kwh/d	783.953
Heat energy recovered	kwh/d	1.119.933

¹ kwh:3600 kj

Table C.6 : Energy recovery design calculation table for different cities.

Process/Province	unit	Erzurum	Diyarbakır	Aksaray	Adıyaman
Primary Sludge					
Primary Sludge	kg/d	10,699	27,599	6,570	5,668
VSS content of primary sludge	%	0.65	0.65	0.65	0.65
Primary sludge VSS, inlet digester	kg/d	6,954	17,939	4,271	3,684
Degradation	%	50	50	50	50
Primary sludge VSS, degraded	kg/d	3,477	8,970	2,135	1,842
Total primary sludge amount out of digester	kg/d	7,222	18,629	4,435	3,826
Excess Sludge					
Excess Sludge	kg/d	12,520	42,124	10,221	7,954
Chemical sludge for phosphorus removal	kg/d	0	5,898	1,221	1,400
VSS content of excess activated sludge	-	0.85	0.85	0.85	0.85
Excess sludge VSS, inlet digester	kg/d	10,642	30,792	7,650	5,571
Degradation	%	50	50	50	50
Excess sludge VSS, degraded	kg/d	5,321	15,396	3,825	2,785
Total excess sludge amount out of digester	kg/d	7,199	26,728	6,396	5,169
Biogas Production					
Specific gas Production	Nm ³ gas /VSS	0.9350	0.9350	0.9350	0.9350
Gas production from primary sludge	m ³ /d	3,251	8,387	1,996	1,722
Gas production from biological sludge	m ³ /d	4,975	14,395	3,576	2,604
Total Gas Production	m ³ /d	8,226	22,782	5,573	4,327
Calorific value of digester gas	kJ/m ³	22,400	22,400	22,400	22,400
Total calorific value of digester gas	kJ/d	184,268,977	510,315,744	124,831,476	96,919,407
	kwh/d	51,186	141,754	34,675	26,922
Design of CHP Units					
Percent of electr. Energy	%	35	35	35	35
Percent of heat.energy	%	50	50	50	50
Electr.energy recovered	kwh/d	17,915	49,614	12,136	9,423
Heat energy recovered	kwh/d	25,593	70,877	17,338	13,461
Population	PE	444,934	1,430,000	201,782	245490
Electrical energy recovered kwh/PE.d		14.70	12.66	14.55	14.01

¹ kwh:3600 kj

Appendix D: Output of the ATV Program.

Project: POPULATION :50.000, BNR

Calculated by: AYSEGUL NUHOGLU

Date: 21.07.2010

Summary of results (Biological reactor)

CONFIGURATION OF PLANT:

- ☐ Primary settling
- ☐ Anaerobic mixing tank
- ☐ Activated sludge tank(s)
- ☐ Secondary settling

TREATMENT OBJECTIVES:

- ☐ Removal of org. carbon
- ☐ Nitrification
- ☐ Denitrification
- ☐ Phosphorus-precipitation

Denitrification process: Pre-anoxic zone denitrification/td>

Precipitant: Iron (trivalent)

Secondary settling: Type of tank(s) Circular tank, Flow characteristics transition horiz./vertical, Blade scraper/td>

SIZE CLASS AND LOAD CASES:

Size class: 1125 kg BOD₅/d

Calculated load cases:

- ☐ Load case 1: Dimensioning
- ☐ Load case 2: Proof of nitrification with lowest temperature
- ☐ Load case 3: Calculation of oxygen uptake with highest temperature

Calculation based on COD

	Load case	1	2	3
INFLOW:				
Daily dry weather flow rate	Q _{DW,d}	5000	5000	5000 m³/d
Hourly dry weather flow as 2hr mean	Q _{DW,h}	208	208	208 m³/h
Load:				
COD	B _{d,COD}	2340	2340	2340 kg/d
Dissolved COD	B _{d,SCOD}	935	935	935 kg/d
BOD ₅	B _{d,BOD}	1125	1125	1125 kg/d
Filterable solids	B _{d,XSS}	665	665	665 kg/d
Total Kjeldahl-Nitrogen	B _{d,TKN}	300,0	300,0	300,0 kg/d
Ammonia nitrogen	B _{d,NH4}	195,0	195,0	195,0 kg/d
Nitrate nitrogen	B _{d,NO3}	0,0	0,0	0,0 kg/d
Phosphorus	B _{d,P}	50,0	50,0	50,0 kg/d

EFFLUENT CONCENTRATIONS:

Ammonia nitrogen	SNH4,EST	0,0	0,0	0,0 mg/l
Nitrate nitrogen	SNO3,EST	12,6	12,6	11,6 mg/l
Phosphorus	CP,EST	2,0	2,0	2,0 mg/l
Alkalinity	SALK,EST	6,1	6,1	6,1 mmol/l

BIOLOGICAL REACTOR:

Volume of biological reactor	V _{AT}	2755	2755	2755 m ³
Required safety factor	SF req.	1,80	1,20	1,80 -
Existing safety factor	SF	1,80	1,80	5,52 -
Denitrification share	V _D /V	40	40	40 %
Temperature	T	15,00	15,00	25,00 °C
Dry matter suspended solids	SS _{AT}	4,07	4,07	4,07 kg/m ³
Sludge age	t _{SS}	10,2	10,2	11,7 d
Aerobic sludge age	t _{SS,aer.}	6,1	6,1	7,0 d
Recirculation	RC	300	300	300 %
Nitrogen embedded in biomass	X _{N,BM}	11,7	11,7	11,7 mg/l
Nitrogen nitrified	S _{NH4,N}	46,3	46,3	46,3 mg/l
Nitrogen denitrified	S _{NO3,D}	33,7	33,7	34,7 mg/l
Phosphorus embedded in biomass	X _{P,BM} +X _{P,BIOP}	5,6	5,6	5,6 mg/l
Phosphorus precipitated	X _{P,Prec}	2,4	2,4	2,4 mg/l
Demand of precipitant	FM	*****	*****	***** kg Metal/d

Sludge production:

Daily sludge production	SP _d	1098	1098	955 kg/d
... from P-removal	SP _{d,P}	130	130	130 kg/d
... from ext. carbon source	SP _{d,ext}	0	0	0 kg/d

Oxygen uptake:

... for carbon removal	OU _{d,C}	1254	1254	1420 kg/d
... for nitrification	OU _{d,N}	995	995	995 kg/d
... carbon removal by denitrification	OU _{d,D}	-489	-489	-504 kg/d
Average daily oxygen uptake	OU _d	1761	1761	1912 kg/d
Average hourly oxygen uptake	OU _h	73,4	73,4	79,7 kg/h
Peak factor carbon respiration	f _C	1,20	1,20	1,20 -
Peak factor ammonia oxidation	f _N	2,00	2,00	2,00 -
Maximum hourly oxygen uptake	OU _h	114,9	114,9	121,2 kg/h
Required oxygen transfer	alpha*OC _h	143,3	143,3	159,8 kg/h

ANAEROBIC MIXING TANK:

Volume	V _{BioP}	208 m ³		
Contact time (with Q _{DW,h} , RS=1)	t _{BioP}	0,5	0,5	0,5 h

Project: POPULATION : 100.000 BNR

Calculated by: AYSEGUL NUHOGLU

Date: 21.07.2010

Summary of results (Biological reactor)

CONFIGURATION OF PLANT: TREATMENT OBJECTIVES:

- | | |
|---|--|
| <ul style="list-style-type: none"> ○ Primary settling ○ Anaerobic mixing tank ○ Activated sludge tank(s) ○ Secondary settling | <ul style="list-style-type: none"> ○ Removal of org. carbon ○ Nitrification ○ Denitrification ○ Phosphorus-precipitation |
|---|--|

Denitrification process: Pre-anoxic zone denitrification/td>

Precipitant: Iron (trivalent)

Secondary settling: Type of tank(s) Circular tank, Flow characteristics horizontal, Blade scraper/td>

SIZE CLASS AND LOAD CASES:

Size class: 2250 kg BOD₅/d

Calculated load cases:

- Load case 1: Dimensioning
- Load case 2: Proof of nitrification with lowest temperature
- Load case 3: Calculation of oxygen uptake with highest temperature

Calculation based on COD

	Load case	1	2	3
INFLOW:				
Daily dry weather flow rate	Q _{DW,d}	10000	10000	10000 m ³ /d
Hourly dry weather flow as 2hr mean	Q _{DW,h}	417	417	417 m ³ /h
Load:				
COD	B _{d,COD}	4680	4680	4680 kg/d
Dissolved COD	B _{d,SCOD}	1870	1870	1870 kg/d
BOD ₅	B _{d,BOD}	2250	2250	2250 kg/d
Filterable solids	B _{d,XSS}	1330	1330	1330 kg/d
Total Kjeldahl-Nitrogen	B _{d,TKN}	600,0	600,0	600,0 kg/d
Ammonia nitrogen	B _{d,NH4}	390,0	390,0	390,0 kg/d
Nitrate nitrogen	B _{d,NO3}	0,0	0,0	0,0 kg/d
Phosphorus	B _{d,P}	100,0	100,0	100,0 kg/d

EFFLUENT CONCENTRATIONS:

Ammonia nitrogen	SNH4,EST	0,0	0,0	0,0 mg/l
Nitrate nitrogen	SNO3,EST	12,6	12,6	11,6 mg/l
Phosphorus	CP,EST	2,0	2,0	2,0 mg/l
Alkalinity	SALK,EST	6,1	6,1	6,1 mmol/l

BIOLOGICAL REACTOR:

Volume of biological reactor	V _{AT}	5318	5318	5318 m ³
Required safety factor	SF req.	1,72	1,20	1,72 -
Existing safety factor	SF	1,72	1,72	5,29 -
Denitrification share	V _D /V	40	40	40 %
Temperature	T	15,00	15,00	25,00 °C
Dry matter suspended solids	SS _{AT}	4,07	4,07	4,07 kg/m ³
Sludge age	t _{SS}	9,8	9,8	11,3 d
Aerobic sludge age	t _{SS,aer.}	5,9	5,9	6,8 d
Recirculation	RC	300	300	300 %
Nitrogen embedded in biomass	X _{N,BM}	11,7	11,7	11,7 mg/l
Nitrogen nitrified	S _{NH4,N}	46,3	46,3	46,3 mg/l
Nitrogen denitrified	S _{NO3,D}	33,7	33,7	34,7 mg/l
Phosphorus embedded in biomass	X _{P,BM} +X _{P,BIOP}	5,6	5,6	5,6 mg/l
Phosphorus precipitated	X _{P,Prec}	2,4	2,4	2,4 mg/l
Demand of precipitant	FM	*****	*****	***** kg Metal/d

Sludge production:

Daily sludge production	SP _d	2214	2214	1921 kg/d
... from P-removal	SP _{d,P}	260	260	260 kg/d
... from ext. carbon source	SP _{d,ext}	0	0	0 kg/d

Oxygen uptake:

... for carbon removal	OU _{d,C}	2488	2488	2827 kg/d
... for nitrification	OU _{d,N}	1991	1991	1991 kg/d
... carbon removal by denitrification	OU _{d,D}	-977	-977	-1007 kg/d
Average daily oxygen uptake	OU _d	3502	3502	3811 kg/d
Average hourly oxygen uptake	OU _h	145,9	145,9	158,8 kg/h
Peak factor carbon respiration	f _C	1,20	1,20	1,20 -
Peak factor ammonia oxidation	f _N	2,00	2,00	2,00 -
Maximum hourly oxygen uptake	OU _h	228,9	228,9	241,7 kg/h
Required oxygen transfer	alpha*OC _h	285,5	285,5	318,9 kg/h

ANAEROBIC MIXING TANK:

Volume	V _{BioP}	417 m ³		
Contact time (with Q _{DW,h} , RS=1)	t _{BioP}	0,5	0,5	0,5 h

Project: POPULATION : 200.000 BNR

Calculated by: AYSEGUL NUHOGLU

Date: 21.07.2010

Summary of results (Biological reactor)

CONFIGURATION OF PLANT:

- Primary settling
- Anaerobic mixing tank
- Activated sludge tank(s)
- Secondary settling

TREATMENT OBJECTIVES:

- Removal of org. carbon
- Nitrification
- Denitrification
- Phosphorus-precipitation

Denitrification process: Pre-anoxic zone denitrification/td>

Precipitant: Iron (trivalent)

Secondary settling: Type of tank(s) Circular tank, Flow characteristics horizontal, Blade scraper/td>

SIZE CLASS AND LOAD CASES:

Size class: 4875 kg BOD₅/d

Calculated load cases:

- Load case 1: Dimensioning
- Load case 2: Proof of nitrification with lowest temperature
- Load case 3: Calculation of oxygen uptake with highest temperature

Calculation based on COD

		Load case	1	2	3
INFLOW:					
Daily dry weather flow rate	Q _{DW,d}	25000	25000	25000	m ³ /d
Hourly dry weather flow as 2hr mean	Q _{DW,h}	1042	1042	1042	m ³ /h
Load:					
COD	B _{d,COD}	10075	10075	10075	kg/d
Dissolved COD	B _{d,SCOD}	4025	4025	4025	kg/d
BOD ₅	B _{d,BOD}	4875	4875	4875	kg/d
Filterable solids	B _{d,XSS}	2900	2900	2900	kg/d
Total Kjeldahl-Nitrogen	B _{d,TKN}	1275,0	1275,0	1275,0	kg/d
Ammonia nitrogen	B _{d,NH4}	825,0	825,0	825,0	kg/d
Nitrate nitrogen	B _{d,NO3}	0,0	0,0	0,0	kg/d
Phosphorus	B _{d,P}	200,0	200,0	200,0	kg/d

EFFLUENT CONCENTRATIONS:

Ammonia nitrogen	SNH4,EST	0,0	0,0	0,0	mg/l
Nitrate nitrogen	SNO3,EST	7,8	7,8	7,8	mg/l
Phosphorus	CP,EST	1,0	1,0	1,0	mg/l
Alkalinity	SALK,EST	6,9	6,9	6,9	mmol/l

BIOLOGICAL REACTOR:

Volume of biological reactor	V_{AT}	12137	12137	12137 m ³
Required safety factor	SF req.	1,53	1,20	1,53 -
Existing safety factor	SF	1,53	1,53	4,69 -
Denitrification share	V_D/V	50	50	50 %
Temperature	T	15,00	15,00	25,00 °C
Dry matter suspended solids	SS _{AT}	4,07	4,07	4,07 kg/m ³
Sludge age	t _{SS}	10,4	10,4	12,0 d
Aerobic sludge age	t _{SS,aer.}	5,2	5,2	6,0 d
Recirculation	RC	400	400	400 %
Nitrogen embedded in biomass	$X_{N,BM}$	10,1	10,1	10,1 mg/l
Nitrogen nitrified	$S_{NH_4,N}$	38,9	38,9	38,9 mg/l
Nitrogen denitrified	$S_{NO_3,D}$	31,1	31,1	31,1 mg/l
Phosphorus embedded in biomass	$X_{P,BM}+X_{P,BIOP}$	4,8	4,8	4,8 mg/l
Phosphorus precipitated	$X_{P,Prec}$	2,2	2,2	2,2 mg/l
Demand of precipitant	FM	*****	*****	***** kg Metal/d

Sludge production:

Daily sludge production	SP _d	4736	4736	4126 kg/d
... from P-removal	SP _{d,P}	578	578	578 kg/d
... from ext. carbon source	SP _{d,ext}	0	0	0 kg/d

Oxygen uptake:

... for carbon removal	OU _{d,C}	5421	5421	6128 kg/d
... for nitrification	OU _{d,N}	4184	4184	4184 kg/d
... carbon removal by denitrification	OU _{d,D}	-2258	-2258	-2258 kg/d
Average daily oxygen uptake	OU _d	7348	7348	8055 kg/d
Average hourly oxygen uptake	OU _h	306,2	306,2	335,6 kg/h
Peak factor carbon respiration	f _C	1,20	1,20	1,20 -
Peak factor ammonia oxidation	f _N	1,90	1,90	1,90 -
Maximum hourly oxygen uptake	OU _h	463,1	463,1	492,6 kg/h
Required oxygen transfer	alpha*OC _h	577,7	577,7	649,7 kg/h

ANAEROBIC MIXING TANK:

Volume	V_{BioP}	1042 m ³		
Contact time (with Q _{DW,h} , RS=1)	t _{BioP}	0,5	0,5	0,5 h

Project: POPULATION : 400.000 BNR
Calculated by: AYSEGUL NUHOGLU
Summary of results (Biological reactor)

Date: 21.07.2010

CONFIGURATION OF PLANT:	TREATMENT OBJECTIVES:
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- ☐ Primary settling
- ☐ Anaerobic mixing tank
- ☐ Activated sludge tank(s)
- ☐ Secondary settling

- ☐ Removal of org. carbon
- ☐ Nitrification
- ☐ Denitrification
- ☐ Phosphorus-precipitation

Denitrification process: Pre-anoxic zone denitrification/td>

Precipitant: Iron (trivalent)

Secondary settling: Type of tank(s) Circular tank, Flow characteristics horizontal, Blade scraper/td>

SIZE CLASS AND LOAD CASES:

Size class: 10290 kg BOD₅/d

Calculated load cases:

- ☐ Load case 1: Dimensioning
- ☐ Load case 2: Proof of nitrification with lowest temperature
- ☐ Load case 3: Calculation of oxygen uptake with highest temperature

Calculation based on COD

	Load case	1	2	3
INFLOW:				
Daily dry weather flow rate	Q _{DW,d}	56000	56000	56000 m ³ /d
Hourly dry weather flow as 2hr mean	Q _{DW,h}	2333	2333	2333 m ³ /h
Load:				
COD	B _{d,COD}	21560	21560	21560 kg/d
Dissolved COD	B _{d,SCOD}	8624	8624	8624 kg/d
BOD ₅	B _{d,BOD}	10304	10304	10304 kg/d
Filterable solids	B _{d,XSS}	6104	6104	6104 kg/d
Total Kjeldahl-Nitrogen	B _{d,TKN}	2800,0	2800,0	2800,0 kg/d
Ammonia nitrogen	B _{d,NH4}	1792,0	1792,0	1792,0 kg/d
Nitrate nitrogen	B _{d,NO3}	0,0	0,0	0,0 kg/d
Phosphorus	B _{d,P}	448,0	448,0	448,0 kg/d

EFFLUENT CONCENTRATIONS:

Ammonia nitrogen	SNH4,EST	0,0	0,0	0,0 mg/l
Nitrate nitrogen	SNO3,EST	8,5	8,5	7,7 mg/l
Phosphorus	CP,EST	1,0	1,0	1,0 mg/l
Alkalinity	SALK,EST	6,8	6,8	6,9 mmol/l

BIOLOGICAL REACTOR:

Volume of biological reactor	V_{AT}	25063	25063	25063 m ³
Required safety factor	SF req.	1,45	1,20	1,45 -
Existing safety factor	SF	1,45	1,45	4,44 -
Denitrification share	V_D/V	50	50	50 %
Temperature	T	15,00	15,00	25,00 °C
Dry matter suspended solids	SS_{AT}	4,07	4,07	4,07 kg/m ³
Sludge age	t_{SS}	9,9	9,9	11,3 d
Aerobic sludge age	$t_{SS,aer.}$	4,9	4,9	5,7 d
Recirculation	RC	400	400	400 %
Nitrogen embedded in biomass	$X_{N,BM}$	9,6	9,6	9,6 mg/l
Nitrogen nitrified	SNH_4,N	38,4	38,4	38,4 mg/l
Nitrogen denitrified	SNO_3,D	29,8	29,8	30,7 mg/l
Phosphorus embedded in biomass	$X_{P,BM}+X_{P,BIOP}$	4,6	4,6	4,6 mg/l
Phosphorus precipitated	$X_{P,Prec}$	2,4	2,4	2,4 mg/l
Demand of precipitant	FM	*****	*****	***** kg Metal/d

Sludge production:

Daily sludge production	SP_d	10332	10332	8995 kg/d
... from P-removal	$SP_{d,P}$	1356	1356	1356 kg/d
... from ext. carbon source	$SP_{d,ext}$	0	0	0 kg/d

Oxygen uptake:

... for carbon removal	$OU_{d,C}$	11485	11485	13036 kg/d
... for nitrification	$OU_{d,N}$	9241	9241	9241 kg/d
... carbon removal by denitrification	$OU_{d,D}$	-4846	-4846	-4986 kg/d
Average daily oxygen uptake	OU_d	15880	15880	17291 kg/d
Average hourly oxygen uptake	OU_h	661,7	661,7	720,5 kg/h
Peak factor carbon respiration	f_C	1,20	1,20	1,20 -
Peak factor ammonia oxidation	f_N	1,80	1,80	1,80 -
Maximum hourly oxygen uptake	OU_h	969,7	969,7	1028,5 kg/h
Required oxygen transfer	$\alpha \cdot OC_h$	1209,8	1209,8	1356,6 kg/h

ANAEROBIC MIXING TANK:

Volume	V_{BioP}	2333 m ³		
Contact time (with $Q_{DW,h}$, RS=1)	t_{BioP}	0,5	0,5	0,5 h

Project: POPULATION : 1.000.000, BNR

Calculated by: AYSEGUL NUHOGLU

Date: 21.07.2010

Summary of results (Biological reactor)

CONFIGURATION OF PLANT:

- Primary settling
- Anaerobic mixing tank
- Activated sludge tank(s)
- Secondary settling

TREATMENT OBJECTIVES:

- Removal of org. carbon
- Nitrification
- Denitrification
- Phosphorus-precipitation

Denitrification process: Pre-anoxic zone denitrification/td>

Precipitant: Iron (trivalent)

Secondary settling: Type of tank(s) Circular tank, Flow characteristics horizontal, Blade scraper/td>

SIZE CLASS AND LOAD CASES:

Size class: 27600 kg BOD₅/d

Calculated load cases:

- Load case 1: Dimensioning
- Load case 2: Proof of nitrification with lowest temperature
- Load case 3: Calculation of oxygen uptake with highest temperature

Calculation based on COD

		Load case	1	2	3
INFLOW:					
Daily dry weather flow rate	Q _{DW,d}	160000	160000	160000 m ³ /d	
Hourly dry weather flow as 2hr mean	Q _{DW,h}	6667	6667	6667 m ³ /h	
Load:					
COD	B _{d,COD}	57600	57600	57600 kg/d	
Dissolved COD	B _{d,SCOD}	23040	23040	23040 kg/d	
BOD ₅	B _{d,BOD}	27680	27680	27680 kg/d	
Filterable solids	B _{d,XSS}	16320	16320	16320 kg/d	
Total Kjeldahl-Nitrogen	B _{d,TKN}	7360,0	7360,0	7360,0 kg/d	
Ammonia nitrogen	B _{d,NH4}	4800,0	4800,0	4800,0 kg/d	
Nitrate nitrogen	B _{d,NO3}	0,0	0,0	0,0 kg/d	
Phosphorus	B _{d,P}	1120,0	1120,0	1120,0 kg/d	

EFFLUENT CONCENTRATIONS:

Ammonia nitrogen	SNH _{4,EST}	0,0	0,0	0,0 mg/l
Nitrate nitrogen	SNO _{3,EST}	7,8	7,8	7,8 mg/l
Phosphorus	CP,EST	1,0	1,0	1,0 mg/l
Alkalinity	SALK,EST	7,1	7,1	7,1 mmol/l

BIOLOGICAL REACTOR:

Volume of biological reactor	V _{AT}	65530	65530	65530 m ³
Required safety factor	SF req.	1,45	1,20	1,45 -
Existing safety factor	SF	1,45	1,45	4,46 -
Denitrification share	V _D /V	50	50	50 %
Temperature	T	15,00	15,00	25,00 °C
Dry matter suspended solids	SS _{AT}	4,07	4,07	4,07 kg/m ³
Sludge age	t _{SS}	9,9	9,9	11,4 d
Aerobic sludge age	t _{SS,aer.}	4,9	4,9	5,7 d
Recirculation	RC	350	350	350 %
Nitrogen embedded in biomass	X _{N,BM}	9,0	9,0	9,0 mg/l
Nitrogen nitrified	S _{NH4,N}	35,0	35,0	35,0 mg/l
Nitrogen denitrified	S _{NO3,D}	27,2	27,2	27,2 mg/l
Phosphorus embedded in biomass	X _{P,BM} +X _{P,BIOP}	4,3	4,3	4,3 mg/l
Phosphorus precipitated	X _{P,Prec}	1,7	1,7	1,7 mg/l
Demand of precipitant	FM	*****	*****	***** kg Metal/d

Sludge production:

Daily sludge production	SP _d	27016	27016	23430 kg/d
... from P-removal	SP _{d,P}	3031	3031	3031 kg/d
... from ext. carbon source	SP _{d,ext}	0	0	0 kg/d

Oxygen uptake:

... for carbon removal	OU _{d,C}	30683	30683	34842 kg/d
... for nitrification	OU _{d,N}	24080	24080	24080 kg/d
... carbon removal by denitrification	OU _{d,D}	-12631	-12631	-12631 kg/d
Average daily oxygen uptake	OU _d	42132	42132	46291 kg/d
Average hourly oxygen uptake	OU _h	1755,5	1755,5	1928,8 kg/h
Peak factor carbon respiration	f _C	1,20	1,20	1,20 -
Peak factor ammonia oxidation	f _N	1,80	1,80	1,80 -
Maximum hourly oxygen uptake	OU _h	2558,2	2558,2	2731,5 kg/h
Required oxygen transfer	alpha*OC _h	3191,5	3191,5	3603,0 kg/h

ANAEROBIC MIXING TANK:

Volume	V _{BioP}	6667 m ³		
Contact time (with Q _{DW,h} , RS=1)	t _{BioP}	0,5	0,5	0,5 h

Project: EA, POPULATION 50.000

Calculated by: AYSEGUL NUHOGLU

Date: 21.07.2010

Summary of results (Biological reactor)

CONFIGURATION OF PLANT:

- ☐ Activated sludge tank(s)
- ☐ Secondary settling

TREATMENT OBJECTIVES:

- ☐ Removal of org. carbon
- ☐ Simultaneous aerobic sludge stabilization

Secondary settling: Type of tank(s) Circular tank, Flow characteristics horizontal, Blade scraper/td>

SIZE CLASS AND LOAD CASES:

Size class: 36800 kg BOD₅/d

Calculated load cases:

- ☐ Load case 1: Dimensioning
- ☐ Load case 2: Calculation of oxygen uptake with highest temperature

Calculation based on COD

		Load case	1	2
INFLOW:				
Daily dry weather flow rate	Q _{DW,d}		5000	5000 m ³ /d
Hourly dry weather flow as 2hr mean	Q _{DW,h}		208	208 m ³ /h
Load:				
COD	B _{d,COD}		3250	3250 kg/d
Dissolved COD	B _{d,SCOD}		1300	1300 kg/d
BOD ₅	B _{d,BOD}		1500	1500 kg/d
Filterable solids	B _{d,XSS}		1950	1950 kg/d
Total Kjeldahl-Nitrogen	B _{d,TKN}		325,0	325,0 kg/d
Ammonia nitrogen	B _{d,NH4}		210,0	210,0 kg/d
Nitrate nitrogen	B _{d,NO3}		0,0	0,0 kg/d
Phosphorus	B _{d,P}		50,0	50,0 kg/d

EFFLUENT CONCENTRATIONS:

Ammonia nitrogen	S _{NH4,EST}	0,0	0,0 mg/l
Nitrate nitrogen	S _{NO3,EST}	46,8	46,8 mg/l
Phosphorus	C _{P,EST}	6,8	6,8 mg/l
Alkalinity	S _{ALK,EST}	8,8	8,8 mmol/l

BIOLOGICAL REACTOR:

Volume of biological reactor	V_{AT}	9205	9205 m ³
Required safety factor	SF req.	1,45	1,45 -
Existing safety factor	SF	5,88	17,07 -
Denitrification share	V_D/V	0	0 %
Temperature	T	15,00	25,00 °C
Dry matter suspended solids	SSAT	3,43	3,43 kg/m ³
Sludge age	t _{SS}	20,0	21,8 d
Aerobic sludge age	t _{SS,aer.}	20,0	21,8 d
Recirculation	RC	300	300 %
Nitrogen embedded in biomass	$X_{N,BM}$	16,3	16,3 mg/l
Nitrogen nitrified	SNH_4,N	46,8	46,8 mg/l
Nitrogen denitrified	SNO_3,D	0,0	0,0 mg/l
Phosphorus embedded in biomass	$X_{P,BM}+X_{P,BIOP}$	3,3	3,3 mg/l

Sludge production:

Daily sludge production	SP _d	1579	1448 kg/d
... from P-removal	SP _{d,P}	0	0 kg/d
... from ext. carbon source	SP _{d,ext}	0	0 kg/d

Oxygen uptake:

... for carbon removal	OU _{d,C}	1935	2086 kg/d
... for nitrification	OU _{d,N}	1005	1005 kg/d
... carbon removal by denitrification	OU _{d,D}	0	0 kg/d
Average daily oxygen uptake	OU _d	2940	3091 kg/d
Average hourly oxygen uptake	OU _h	122,5	128,8 kg/h
Peak factor carbon respiration	f _C	1,10	1,10 -
Peak factor ammonia oxidation	f _N	1,50	1,50 -
Maximum hourly oxygen uptake	OU _h	143,4	149,7 kg/h
Required oxygen transfer	$\alpha \cdot OC_h$	179,0	197,5 kg/h

Project: EA, POPULATION 100.000

Calculated by: AYSEGUL NUHOGLU

Date: 21.07.2010

Summary of results (Biological reactor)

CONFIGURATION OF PLANT:

- ☐ Activated sludge tank(s)
- ☐ Secondary settling

TREATMENT OBJECTIVES:

- ☐ Removal of org. carbon
- ☐ Nitrification
- ☐ Simultaneous aerobic sludge stabilization

Secondary settling: Type of tank(s) Circular tank, Flow characteristics horizontal, Blade scraper/td>

SIZE CLASS AND LOAD CASES:

Size class: 1500 kg BOD₅/d

Calculated load cases:

- ☐ Load case 1: Dimensioning
- ☐ Load case 2: Calculation of oxygen uptake with highest temperature

Calculation based on COD

		Load case	1	2
INFLOW:				
Daily dry weather flow rate	Q _{DW,d}	10000	10000 m ³ /d	
Hourly dry weather flow as 2hr mean	Q _{DW,h}	417	417 m ³ /h	
Load:				
COD	B _{d,COD}	6500	6500 kg/d	
Dissolved COD	B _{d,SCOD}	2600	2600 kg/d	
BOD ₅	B _{d,BOD}	3000	3000 kg/d	
Filterable solids	B _{d,XSS}	3900	3900 kg/d	
Total Kjeldahl-Nitrogen	B _{d,TKN}	650,0	650,0 kg/d	
Ammonia nitrogen	B _{d,NH4}	420,0	420,0 kg/d	
Nitrate nitrogen	B _{d,NO3}	0,0	0,0 kg/d	
Phosphorus	B _{d,P}	100,0	100,0 kg/d	

EFFLUENT CONCENTRATIONS:

Ammonia nitrogen	S _{NH4,EST}	0,0	0,0 mg/l
Nitrate nitrogen	S _{NO3,EST}	46,8	46,8 mg/l
Phosphorus	C _{P,EST}	6,8	6,8 mg/l
Alkalinity	S _{ALK,EST}	8,8	8,8 mmol/l

BIOLOGICAL REACTOR:

Volume of biological reactor	V_{AT}	18410	18410 m ³
Required safety factor	SF req.	1,74	1,74 -
Existing safety factor	SF	5,88	17,07 -
Denitrification share	V_D/V	0	0 %
Temperature	T	15,00	25,00 °C
Dry matter suspended solids	SS _{AT}	3,43	3,43 kg/m ³
Sludge age	t _{SS}	20,0	21,8 d
Aerobic sludge age	t _{SS,aer.}	20,0	21,8 d
Recirculation	RC	300	300 %
Nitrogen embedded in biomass	$X_{N,BM}$	16,3	16,3 mg/l
Nitrogen nitrified	S _{NH4,N}	46,8	46,8 mg/l
Nitrogen denitrified	S _{NO3,D}	0,0	0,0 mg/l
Phosphorus embedded in biomass	$X_{P,BM}+X_{P,BIOP}$	3,3	3,3 mg/l

Sludge production:

Daily sludge production	SP _d	3157	2896 kg/d
... from P-removal	SP _{d,P}	0	0 kg/d
... from ext. carbon source	SP _{d,ext}	0	0 kg/d

Oxygen uptake:

... for carbon removal	OU _{d,C}	3870	4173 kg/d
... for nitrification	OU _{d,N}	2010	2010 kg/d
... carbon removal by denitrification	OU _{d,D}	0	0 kg/d
Average daily oxygen uptake	OU _d	5880	6183 kg/d
Average hourly oxygen uptake	OU _h	245,0	257,6 kg/h
Peak factor carbon respiration	f _C	1,10	1,10 -
Peak factor ammonia oxidation	f _N	1,70	1,70 -
Maximum hourly oxygen uptake	OU _h	303,6	316,2 kg/h
Required oxygen transfer	alpha*OC _h	378,8	417,1 kg/h

Project: EA, POPULATION 200.000

Calculated by: AYSEGUL NUHOGLU

Date: 21.07.2010

Summary of results (Biological reactor)

CONFIGURATION OF PLANT:

- ☐ Activated sludge tank(s)
- ☐ Secondary settling

TREATMENT OBJECTIVES:

- ☐ Removal of org. carbon
- ☐ Nitrification
- ☐ Simultaneous aerobic sludge stabilization

Secondary settling: Type of tank(s) Circular tank, Flow characteristics horizontal, Blade scraper/td>

SIZE CLASS AND LOAD CASES:

Size class: 6500 kg BOD₅/d

Calculated load cases:

- ☐ Load case 1: Dimensioning
- ☐ Load case 2: Calculation of oxygen uptake with highest temperature

Calculation based on COD

		Load case	1	2
INFLOW:				
Daily dry weather flow rate	Q _{DW,d}		25000	25000 m ³ /d
Hourly dry weather flow as 2hr mean	Q _{DW,h}		1042	1042 m ³ /h
Load:				
COD	B _{d,COD}		14000	14000 kg/d
Dissolved COD	B _{d,SCOD}		5600	5600 kg/d
BOD ₅	B _{d,BOD}		6500	6500 kg/d
Filterable solids	B _{d,XSS}		8500	8500 kg/d
Total Kjeldahl-Nitrogen	B _{d,TKN}		1400,0	1400,0 kg/d
Ammonia nitrogen	B _{d,NH4}		900,0	900,0 kg/d
Nitrate nitrogen	B _{d,NO3}		0,0	0,0 kg/d
Phosphorus	B _{d,P}		225,0	225,0 kg/d

EFFLUENT CONCENTRATIONS:

Ammonia nitrogen	S _{NH4,EST}	0,0	0,0 mg/l
Nitrate nitrogen	S _{NO3,EST}	40,0	40,0 mg/l
Phosphorus	C _{P,EST}	6,2	6,2 mg/l
Alkalinity	S _{ALK,EST}	9,7	9,7 mmol/l

BIOLOGICAL REACTOR:

Volume of biological reactor	V_{AT}	54643	54643 m ³
Required safety factor	SF req.	1,45	1,45 -
Existing safety factor	SF	5,88	17,06 -
Denitrification share	V_D/V	0	0 %
Temperature	T	15,00	25,00 °C
Dry matter suspended solids	SS_{AT}	2,50	2,50 kg/m ³
Sludge age	t_{SS}	20,0	21,8 d
Aerobic sludge age	$t_{SS,aer.}$	20,0	21,8 d
Recirculation	RC	433	433 %
Nitrogen embedded in biomass	$X_{N,BM}$	14,0	14,0 mg/l
Nitrogen nitrified	$S_{NH_4,N}$	40,0	40,0 mg/l
Nitrogen denitrified	$S_{NO_3,D}$	0,0	0,0 mg/l
Phosphorus embedded in biomass	$X_{P,BM}+X_{P,BIOP}$	2,8	2,8 mg/l

Sludge production:

Daily sludge production	SP_d	6830	6268 kg/d
... from P-removal	$SP_{d,P}$	0	0 kg/d
... from ext. carbon source	$SP_{d,ext}$	0	0 kg/d

Oxygen uptake:

... for carbon removal	$OU_{d,C}$	8335	8987 kg/d
... for nitrification	$OU_{d,N}$	4300	4300 kg/d
... carbon removal by denitrification	$OU_{d,D}$	0	0 kg/d
Average daily oxygen uptake	OU_d	12635	13287 kg/d
Average hourly oxygen uptake	OU_h	526,5	553,6 kg/h
Peak factor carbon respiration	f_C	1,10	1,10 -
Peak factor ammonia oxidation	f_N	1,50	1,50 -
Maximum hourly oxygen uptake	OU_h	616,0	643,2 kg/h
Required oxygen transfer	$\alpha \cdot OC_h$	768,5	848,4 kg/h

Project: EA, POPULATION 400.000

Calculated by: AYSEGUL NUHOGLU

Date: 21.07.2010

Summary of results (Biological reactor)

CONFIGURATION OF PLANT:

- Activated sludge tank(s)
- Secondary settling

TREATMENT OBJECTIVES:

- Removal of org. carbon
- Nitrification
- Simultaneous aerobic sludge stabilization

Secondary settling: Type of tank(s) Circular tank, Flow characteristics horizontal, Blade scraper/td>

SIZE CLASS AND LOAD CASES:

Size class: 13720 kg BOD₅/d

Calculated load cases:

- Load case 1: Dimensioning
- Load case 2: Calculation of oxygen uptake with highest temperature

Calculation based on COD

		Load case	1	2
INFLOW:				
Daily dry weather flow rate	Q _{DW,d}	56000	56000 m ³ /d	
Hourly dry weather flow as 2hr mean	Q _{DW,h}	2333	2333 m ³ /h	
Load:				
COD	B _{d,COD}	29960	29960 kg/d	
Dissolved COD	B _{d,SCOD}	11984	11984 kg/d	
BOD ₅	B _{d,BOD}	13720	13720 kg/d	
Filterable solids	B _{d,XSS}	17920	17920 kg/d	
Total Kjeldahl-Nitrogen	B _{d,TKN}	3024,0	3024,0 kg/d	
Ammonia nitrogen	B _{d,NH4}	1960,0	1960,0 kg/d	
Nitrate nitrogen	B _{d,NO3}	0,0	0,0 kg/d	
Phosphorus	B _{d,P}	504,0	504,0 kg/d	

EFFLUENT CONCENTRATIONS:

Ammonia nitrogen	S _{NH4,EST}	0,0	0,0 mg/l
Nitrate nitrogen	S _{NO3,EST}	38,6	38,6 mg/l
Phosphorus	C _{P,EST}	6,3	6,3 mg/l
Alkalinity	S _{ALK,EST}	9,8	9,8 mmol/l

BIOLOGICAL REACTOR:

Volume of biological reactor	V _{AT}	96906	96906 m ³
Required safety factor	SF _{req.}	1,45	1,45 -
Existing safety factor	SF	5,88	17,07 -
Denitrification share	V _D /V	0	0 %
Temperature	T	15,00	25,00 °C
Dry matter suspended solids	SS _{AT}	3,00	3,00 kg/m ³
Sludge age	t _{SS}	20,0	21,8 d
Aerobic sludge age	t _{SS,aer.}	20,0	21,8 d
Recirculation	RC	415	415 %
Nitrogen embedded in biomass	X _{N,BM}	13,4	13,4 mg/l
Nitrogen nitrified	S _{NH₄,N}	38,6	38,6 mg/l
Nitrogen denitrified	S _{NO₃,D}	0,0	0,0 mg/l
Phosphorus embedded in biomass	X _{P,BM} +X _{P,BIOP}	2,7	2,7 mg/l

Sludge production:

Daily sludge production	SP _d	14536	13333 kg/d
... from P-removal	SP _{d,P}	0	0 kg/d
... from ext. carbon source	SP _{d,ext}	0	0 kg/d

Oxygen uptake:

... for carbon removal	OU _{d,C}	17837	19232 kg/d
... for nitrification	OU _{d,N}	9301	9301 kg/d
... carbon removal by denitrification	OU _{d,D}	0	0 kg/d
Average daily oxygen uptake	OU _d	27137	28533 kg/d
Average hourly oxygen uptake	OU _h	1130,7	1188,9 kg/h
Peak factor carbon respiration	f _C	1,10	1,10 -
Peak factor ammonia oxidation	f _N	1,50	1,50 -
Maximum hourly oxygen uptake	OU _h	1324,5	1382,6 kg/h
Required oxygen transfer	alpha*OC _h	1652,4	1823,8 kg/h

Project: EA, POPULATION 1.000.000

Calculated by: AYSEGUL NUHOGLU

Date: 21.07.2010

Summary of results (Biological reactor)

CONFIGURATION OF PLANT:

- ☐ Activated sludge tank(s)
- ☐ Secondary settling

TREATMENT OBJECTIVES:

- ☐ Removal of org. carbon
- ☐ Nitrification
- ☐ Simultaneous aerobic sludge stabilization

Secondary settling: Type of tank(s) Circular tank, Flow characteristics horizontal, Blade scraper/td>

SIZE CLASS AND LOAD CASES:

Size class: 36800 kg BOD₅/d

Calculated load cases:

- ☐ Load case 1: Dimensioning
- ☐ Load case 2: Calculation of oxygen uptake with highest temperature

Calculation based on COD

Load case 1 2

INFLOW:

Daily dry weather flow rate	Q _{DW,d}	160000	160000 m ³ /d
Hourly dry weather flow as 2hr mean	Q _{DW,h}	6667	6667 m ³ /h

Load:

COD	B _{d,COD}	80000	80000 kg/d
Dissolved COD	B _{d,SCOD}	32000	32000 kg/d
BOD ₅	B _{d,BOD}	36800	36800 kg/d
Filterable solids	B _{d,XSS}	48000	48000 kg/d
Total Kjeldahl-Nitrogen	B _{d,TKN}	8000,0	8000,0 kg/d
Ammonia nitrogen	B _{d,NH4}	5280,0	5280,0 kg/d
Nitrate nitrogen	B _{d,NO3}	0,0	0,0 kg/d
Phosphorus	B _{d,P}	1280,0	1280,0 kg/d

EFFLUENT CONCENTRATIONS:

Ammonia nitrogen	SNH _{4,EST}	0,0	0,0 mg/l
Nitrate nitrogen	SNO _{3,EST}	35,5	35,5 mg/l
Phosphorus	C _{P,EST}	5,5	5,5 mg/l
Alkalinity	SALK _{EST}	10,2	10,2 mmol/l

BIOLOGICAL REACTOR:

Volume of biological reactor	V_{AT}	226583	226583 m ³
Required safety factor	SF req.	1,45	1,45 -
Existing safety factor	SF	5,88	17,07 -
Denitrification share	V_D/V	0	0 %
Temperature	T	15,00	25,00 °C
Dry matter suspended solids	SS_{AT}	3,43	3,43 kg/m ³
Sludge age	t_{SS}	20,0	21,8 d
Aerobic sludge age	$t_{SS,aer.}$	20,0	21,8 d
Recirculation	RC	300	300 %
Nitrogen embedded in biomass	$X_{N,BM}$	12,5	12,5 mg/l
Nitrogen nitrified	$S_{NH_4,N}$	35,5	35,5 mg/l
Nitrogen denitrified	$S_{NO_3,D}$	0,0	0,0 mg/l
Phosphorus embedded in biomass	$X_{P,BM}+X_{P,BIOP}$	2,5	2,5 mg/l

Sludge production:

Daily sludge production	SP_d	38859	35647 kg/d
... from P-removal	$SP_{d,P}$	0	0 kg/d
... from ext. carbon source	$SP_{d,ext}$	0	0 kg/d

Oxygen uptake:

... for carbon removal	$OU_{d,C}$	47628	51354 kg/d
... for nitrification	$OU_{d,N}$	24424	24424 kg/d
... carbon removal by denitrification	$OU_{d,D}$	0	0 kg/d
Average daily oxygen uptake	OU_d	72052	75778 kg/d
Average hourly oxygen uptake	OU_h	3002,2	3157,4 kg/h
Peak factor carbon respiration	f_C	1,10	1,10 -
Peak factor ammonia oxidation	f_N	1,50	1,50 -
Maximum hourly oxygen uptake	OU_h	3511,0	3666,2 kg/h
Required oxygen transfer	$\alpha \cdot OC_h$	4380,2	4836,0 kg/h

**Project: EA WITH NUTRIENT REMOVAL
POPULATION 50.000**

Calculated by: AYSEGUL NUHOGLU

Date: 21.07.2010

Summary of results (Biological reactor)

CONFIGURATION OF PLANT:

- Anaerobic mixing tank
- Activated sludge tank(s)
- Secondary settling

TREATMENT OBJECTIVES:

- Removal of org. carbon
- Nitrification
- Denitrification
- Simultaneous aerobic sludge stabilization
- Phosphorus-precipitation

Denitrification process: Pre-anoxic zone denitrification/td>

Precipitant: Iron (trivalent)

Secondary settling: Type of tank(s) Circular tank, Flow characteristics horizontal, Blade scraper/td>

SIZE CLASS AND LOAD CASES:

Size class: 1500 kg BOD₅/d

Calculated load cases:

- Load case 1: Dimensioning
- Load case 2: Calculation of oxygen uptake with highest temperature

Calculation based on COD

	Load case	1	2
INFLOW:			
Daily dry weather flow rate	QDW,d	5000	5000 m ³ /d
Hourly dry weather flow as 2hr mean	QDW,h	208	208 m ³ /h
Load:			
COD	B _{d,COD}	3250	3250 kg/d
Dissolved COD	B _{d,SCOD}	1300	1300 kg/d
BOD ₅	B _{d,BOD}	1500	1500 kg/d
Filterable solids	B _{d,XSS}	1950	1950 kg/d
Total Kjeldahl-Nitrogen	B _{d,TKN}	325,0	325,0 kg/d
Ammonia nitrogen	B _{d,NH4}	210,0	210,0 kg/d
Nitrate nitrogen	B _{d,NO3}	0,0	0,0 kg/d
Phosphorus	B _{d,P}	50,0	50,0 kg/d

EFFLUENT CONCENTRATIONS:

Ammonia nitrogen	SNH4,EST	0,0	0,0 mg/l
Nitrate nitrogen	SNO3,EST	12,6	12,0 mg/l
Phosphorus	CP,EST	2,0	2,0 mg/l
Alkalinity	SALK,EST	11,1	11,2 mmol/l

BIOLOGICAL REACTOR:

Volume of biological reactor	V_{AT}	8862	8862 m ³
Required safety factor	SF req.	1,74	1,74 -
Existing safety factor	SF	4,95	14,30 -
Denitrification share	V_D/V	17	17 %
Temperature	T	15,00	25,00 °C
Dry matter suspended solids	SS_{AT}	3,78	3,78 kg/m ³
Sludge age	t_{SS}	20,3	22,0 d
Aerobic sludge age	$t_{SS,aer.}$	16,8	18,3 d
Recirculation	RC	290	290 %
Nitrogen embedded in biomass	$X_{N,BM}$	16,3	16,3 mg/l
Nitrogen nitrified	SNH_4,N	46,8	46,8 mg/l
Nitrogen denitrified	SNO_3,D	34,1	34,8 mg/l
Phosphorus embedded in biomass	$X_{P,BM}+X_{P,BIOP}$	7,8	7,8 mg/l
Phosphorus precipitated	$X_{P,Prec}$	0,2	0,2 mg/l
Demand of precipitant	FM	*****	***** kg Metal/d

Sludge production:

Daily sludge production	SP_d	1651	1522 kg/d
... from P-removal	$SP_{d,P}$	75	75 kg/d
... from ext. carbon source	$SP_{d,ext}$	0	0 kg/d

Oxygen uptake:

... for carbon removal	$OU_{d,C}$	1938	2088 kg/d
... for nitrification	$OU_{d,N}$	1005	1005 kg/d
... carbon removal by denitrification	$OU_{d,D}$	-495	-504 kg/d
Average daily oxygen uptake	OU_d	2449	2589 kg/d
Average hourly oxygen uptake	OU_h	102,0	107,9 kg/h
Peak factor carbon respiration	f_C	1,10	1,10 -
Peak factor ammonia oxidation	f_N	1,70	1,70 -
Maximum hourly oxygen uptake	OU_h	131,3	137,2 kg/h
Required oxygen transfer	$\alpha \cdot OC_h$	163,9	180,9 kg/h

ANAEROBIC MIXING TANK:

Volume	V_{BioP}	208 m ³	
Contact time (with $Q_{DW,h}$, RS=1)	t_{BioP}	0,5	0,5 h

**Project: EA WITH NUTRIENT REMOVAL
POPULATION 100.000**

Calculated by: AYSEGUL NUHOGLU

Date: 21.07.2010

Summary of results (Biological reactor)

CONFIGURATION OF PLANT:

- Anaerobic mixing tank
- Activated sludge tank(s)
- Secondary settling

TREATMENT OBJECTIVES:

- Removal of org. carbon
- Nitrification
- Denitrification
- Simultaneous aerobic sludge stabilization
- Phosphorus-precipitation

Denitrification process: Pre-anoxic zone denitrification

Precipitant: Iron (trivalent)

Secondary settling: Type of tank(s) Circular tank, Flow characteristics horizontal, Blade scraper

SIZE CLASS AND LOAD CASES:

Size class: 3000 kg BOD₅/d

Calculated load cases:

- Load case 1: Dimensioning
- Load case 2: Calculation of oxygen uptake with highest temperature

Calculation based on COD

		Load case	1	2
INFLOW:				
Daily dry weather flow rate	$Q_{DW,d}$	10000	10000 m ³ /d	
Hourly dry weather flow as 2hr mean	$Q_{DW,h}$	417	417 m ³ /h	
Load:				
COD	$B_{d,COD}$	6500	6500 kg/d	
Dissolved COD	$B_{d,SCOD}$	2600	2600 kg/d	
BOD ₅	$B_{d,BOD}$	3000	3000 kg/d	
Filterable solids	$B_{d,XSS}$	3900	3900 kg/d	
Total Kjeldahl-Nitrogen	$B_{d,TKN}$	650,0	650,0 kg/d	
Ammonia nitrogen	B_{d,NH_4}	420,0	420,0 kg/d	
Nitrate nitrogen	B_{d,NO_3}	0,0	0,0 kg/d	
Phosphorus	$B_{d,P}$	100,0	100,0 kg/d	

EFFLUENT CONCENTRATIONS:

Ammonia nitrogen	SNH_4,EST	0,0	0,0 mg/l
Nitrate nitrogen	SNO_3,EST	12,6	12,0 mg/l
Phosphorus	C_P,EST	2,0	2,0 mg/l
Alkalinity	$SALK,EST$	11,1	11,2 mmol/l

BIOLOGICAL REACTOR:

Volume of biological reactor	V _{AT}	17723	17723 m ³
Required safety factor	SF _{req.}	1,59	1,59 -
Existing safety factor	SF	4,95	14,30 -
Denitrification share	V _D /V	17	17 %
Temperature	T	15,00	25,00 °C
Dry matter suspended solids	SS _{AT}	3,78	3,78 kg/m ³
Sludge age	t _{SS}	20,3	22,0 d
Aerobic sludge age	t _{SS,aer.}	16,8	18,3 d
Recirculation	RC	290	290 %
Nitrogen embedded in biomass	X _{N,BM}	16,3	16,3 mg/l
Nitrogen nitrified	S _{NH4,N}	46,8	46,8 mg/l
Nitrogen denitrified	S _{NO3,D}	34,1	34,8 mg/l
Phosphorus embedded in biomass	X _{P,BM} +X _{P,BIOP}	7,8	7,8 mg/l
Phosphorus precipitated	X _{P,Prec}	0,2	0,2 mg/l
Demand of precipitant	FM	*****	***** kg Metal/d

Sludge production:

Daily sludge production	SP _d	3301	3044 kg/d
... from P-removal	SP _{d,P}	150	150 kg/d
... from ext. carbon source	SP _{d,ext}	0	0 kg/d

Oxygen uptake:

... for carbon removal	OU _{d,C}	3877	4175 kg/d
... for nitrification	OU _{d,N}	2010	2010 kg/d
... carbon removal by denitrification	OU _{d,D}	-990	-1008 kg/d
Average daily oxygen uptake	OU _d	4897	5177 kg/d
Average hourly oxygen uptake	OU _h	204,1	215,7 kg/h
Peak factor carbon respiration	f _C	1,10	1,10 -
Peak factor ammonia oxidation	f _N	1,50	1,50 -
Maximum hourly oxygen uptake	OU _h	245,9	257,6 kg/h
Required oxygen transfer	alpha*OC _h	306,8	339,8 kg/h

ANAEROBIC MIXING TANK:

Volume	V _{BioP}	417 m ³	
Contact time (with Q _{DW,h} , RS=1)	t _{BioP}	0,5	0,5 h

**Project: EA WITH NUTRIENT REMOVAL
POPULATION 200.000**

Calculated by: AYSEGUL NUHOGLU

Date: 21.07.2010

Summary of results (Biological reactor)

CONFIGURATION OF PLANT:

- Anaerobic mixing tank
- Activated sludge tank(s)
- Secondary settling

TREATMENT OBJECTIVES:

- Removal of org. carbon
- Nitrification
- Denitrification
- Simultaneous aerobic sludge stabilization
- Phosphorus-precipitation

Denitrification process: Pre-anoxic zone denitrification/td>

Precipitant: Iron (trivalent)

Secondary settling: Type of tank(s) Circular tank, Flow characteristics horizontal, Blade scraper/td>

SIZE CLASS AND LOAD CASES:

Size class: 6500 kg BOD₅/d

Calculated load cases:

- Load case 1: Dimensioning
- Load case 2: Calculation of oxygen uptake with highest temperature

Calculation based on COD

		Load case	1	2
INFLOW:				
Daily dry weather flow rate	Q _{DW,d}	25000	25000 m ³ /d	
Hourly dry weather flow as 2hr mean	Q _{DW,h}	1042	1042 m ³ /h	
Load:				
COD	B _{d,COD}	14000	14000 kg/d	
Dissolved COD	B _{d,SCOD}	5600	5600 kg/d	
BOD ₅	B _{d,BOD}	6500	6500 kg/d	
Filterable solids	B _{d,XSS}	8500	8500 kg/d	
Total Kjeldahl-Nitrogen	B _{d,TKN}	1400,0	1400,0 kg/d	
Ammonia nitrogen	B _{d,NH4}	900,0	900,0 kg/d	
Nitrate nitrogen	B _{d,NO3}	0,0	0,0 kg/d	
Phosphorus	B _{d,P}	225,0	225,0 kg/d	

EFFLUENT CONCENTRATIONS:

Ammonia nitrogen	S _{NH4,EST}	0,0	0,0 mg/l
Nitrate nitrogen	S _{NO3,EST}	7,5	7,5 mg/l
Phosphorus	C _{P,EST}	1,0	1,0 mg/l
Alkalinity	S _{ALK,EST}	11,8	11,8 mmol/l

BIOLOGICAL REACTOR:

Volume of biological reactor	V_{AT}	51996	51996 m ³
Required safety factor	SF req.	1,45	1,45 -
Existing safety factor	SF	4,71	13,58 -
Denitrification share	V_D/V	21	21 %
Temperature	T	15,00	25,00 °C
Dry matter suspended solids	SS_{AT}	2,86	2,86 kg/m ³
Sludge age	t_{SS}	20,3	21,9 d
Aerobic sludge age	$t_{SS,aer.}$	16,0	17,3 d
Recirculation	RC	433	433 %
Nitrogen embedded in biomass	$X_{N,BM}$	14,0	14,0 mg/l
Nitrogen nitrified	$S_{NH_4,N}$	40,0	40,0 mg/l
Nitrogen denitrified	$S_{NO_3,D}$	32,5	32,5 mg/l
Phosphorus embedded in biomass	$X_{P,BM}+X_{P,BIOP}$	6,7	6,7 mg/l
Phosphorus precipitated	$X_{P,Prec}$	1,3	1,3 mg/l
Demand of precipitant	FM	*****	***** kg Metal/d

Sludge production:

Daily sludge production	SP_d	7328	6775 kg/d
... from P-removal	$SP_{d,P}$	511	511 kg/d
... from ext. carbon source	$SP_{d,ext}$	0	0 kg/d

Oxygen uptake:

... for carbon removal	$OU_{d,C}$	8350	8991 kg/d
... for nitrification	$OU_{d,N}$	4300	4300 kg/d
... carbon removal by denitrification	$OU_{d,D}$	-2355	-2356 kg/d
Average daily oxygen uptake	OU_d	10295	10935 kg/d
Average hourly oxygen uptake	OU_h	429,0	455,6 kg/h
Peak factor carbon respiration	f_C	1,10	1,10 -
Peak factor ammonia oxidation	f_N	1,50	1,50 -
Maximum hourly oxygen uptake	OU_h	518,6	545,2 kg/h
Required oxygen transfer	$\alpha \cdot OC_h$	646,9	719,2 kg/h

ANAEROBIC MIXING TANK:

Volume	V_{BioP}	1042 m ³	
Contact time (with $Q_{DW,h}$, $RS=1$)	t_{BioP}	0,5	0,5 h

**Project: EA WITH NUTRIENT REMOVAL
POPULATION 400.000**

Calculated by: AYSEGUL NUHOGLU

Date: 21.07.2010

Summary of results (Biological reactor)

CONFIGURATION OF PLANT:	TREATMENT OBJECTIVES:
○ Anaerobic mixing tank ○ Activated sludge tank(s) ○ Secondary settling	○ Removal of org. carbon ○ Nitrification ○ Denitrification ○ Simultaneous aerobic sludge stabilization ○ Phosphorus-precipitation
Denitrification process: Pre-anoxic zone denitrification/td>	
Precipitant: Iron (trivalent)	
Secondary settling: Type of tank(s) Circular tank, Flow characteristics horizontal, Blade scraper/td>	

SIZE CLASS AND LOAD CASES:
Size class: 13720 kg BOD ₅ /d
Calculated load cases:
○ Load case 1: Dimensioning ○ Load case 2: Calculation of oxygen uptake with highest temperature
Calculation based on COD

		Load case	1	2
INFLOW:				
Daily dry weather flow rate	Q _{DW,d}	56000	56000 m ³ /d	
Hourly dry weather flow as 2hr mean	Q _{DW,h}	2333	2333 m ³ /h	
Load:				
COD	B _{d,COD}	29960	29960 kg/d	
Dissolved COD	B _{d,SCOD}	11984	11984 kg/d	
BOD ₅	B _{d,BOD}	13720	13720 kg/d	
Filterable solids	B _{d,XSS}	17920	17920 kg/d	
Total Kjeldahl-Nitrogen	B _{d,TKN}	3024,0	3024,0 kg/d	
Ammonia nitrogen	B _{d,NH4}	1960,0	1960,0 kg/d	
Nitrate nitrogen	B _{d,NO3}	0,0	0,0 kg/d	
Phosphorus	B _{d,P}	504,0	504,0 kg/d	

EFFLUENT CONCENTRATIONS:				
Ammonia nitrogen	S _{NH4,EST}	0,0	0,0 mg/l	
Nitrate nitrogen	S _{NO3,EST}	7,6	7,5 mg/l	
Phosphorus	C _{P,EST}	1,0	1,0 mg/l	
Alkalinity	S _{ALK,EST}	11,8	11,8 mmol/l	

BIOLOGICAL REACTOR:

Volume of biological reactor	V_{AT}	91240	91240 m ³
Required safety factor	SF req.	1,45	1,45 -
Existing safety factor	SF	4,71	13,57 -
Denitrification share	V_D/V	21	21 %
Temperature	T	15,00	25,00 °C
Dry matter suspended solids	SS_{AT}	3,50	3,50 kg/m ³
Sludge age	t_{SS}	20,3	21,9 d
Aerobic sludge age	$t_{SS,aer.}$	16,0	17,3 d
Recirculation	RC	415	415 %
Nitrogen embedded in biomass	$X_{N,BM}$	13,4	13,4 mg/l
Nitrogen nitrified	$S_{NH_4,N}$	38,6	38,6 mg/l
Nitrogen denitrified	$S_{NO_3,D}$	31,0	31,1 mg/l
Phosphorus embedded in biomass	$X_{P,BM}+X_{P,BIOP}$	6,4	6,4 mg/l
Phosphorus precipitated	$X_{P,Prec}$	1,6	1,6 mg/l
Demand of precipitant	FM	*****	***** kg Metal/d

Sludge production:

Daily sludge production	SP_d	15736	14554 kg/d
... from P-removal	$SP_{d,P}$	1229	1229 kg/d
... from ext. carbon source	$SP_{d,ext}$	0	0 kg/d

Oxygen uptake:

... for carbon removal	$OU_{d,C}$	17869	19241 kg/d
... for nitrification	$OU_{d,N}$	9301	9301 kg/d
... carbon removal by denitrification	$OU_{d,D}$	-5039	-5055 kg/d
Average daily oxygen uptake	OU_d	22131	23487 kg/d
Average hourly oxygen uptake	OU_h	922,1	978,6 kg/h
Peak factor carbon respiration	f_C	1,10	1,10 -
Peak factor ammonia oxidation	f_N	1,50	1,50 -
Maximum hourly oxygen uptake	OU_h	1115,9	1172,4 kg/h
Required oxygen transfer	$\alpha \cdot OC_h$	1392,1	1546,4 kg/h

ANAEROBIC MIXING TANK:

Volume	V_{BioP}	2333 m ³	
Contact time (with $Q_{DW,h}$, RS=1)	t_{BioP}	0,5	0,5 h

**Project: EA WITH NUTRIENT REMOVAL,
POPULATION 1.000.000**

Calculated by: AYSEGUL NUHOGLU

Date: 21.07.2010

Summary of results (Biological reactor)

CONFIGURATION OF PLANT:

- Anaerobic mixing tank
- Activated sludge tank(s)
- Secondary settling

TREATMENT OBJECTIVES:

- Removal of org. carbon
- Nitrification
- Denitrification
- Simultaneous aerobic sludge stabilization
- Phosphorus-precipitation

Denitrification process: Pre-anoxic zone denitrification/td>

Precipitant: Iron (trivalent)

Secondary settling: Type of tank(s) Circular tank, Flow characteristics horizontal, Blade scraper/td>

SIZE CLASS AND LOAD CASES:

Size class: 36800 kg BOD₅/d

Calculated load cases:

- Load case 1: Dimensioning
- Load case 2: Calculation of oxygen uptake with highest temperature

Calculation based on COD

		Load case	1	2
INFLOW:				
Daily dry weather flow rate	Q _{DW,d}		160000	160000 m ³ /d
Hourly dry weather flow as 2hr mean	Q _{DW,h}		6667	6667 m ³ /h
Load:				
COD	B _{d,COD}		80000	80000 kg/d
Dissolved COD	B _{d,SCOD}		32000	32000 kg/d
BOD ₅	B _{d,BOD}		36800	36800 kg/d
Filterable solids	B _{d,XSS}		48000	48000 kg/d
Total Kjeldahl-Nitrogen	B _{d,TKN}		8000,0	8000,0 kg/d
Ammonia nitrogen	B _{d,NH4}		5280,0	5280,0 kg/d
Nitrate nitrogen	B _{d,NO3}		0,0	0,0 kg/d
Phosphorus	B _{d,P}		1280,0	1280,0 kg/d

EFFLUENT CONCENTRATIONS:

Ammonia nitrogen	S _{NH4,EST}	0,0	0,0 mg/l
Nitrate nitrogen	S _{NO3,EST}	7,8	7,1 mg/l
Phosphorus	C _{P,EST}	1,0	1,0 mg/l
Alkalinity	S _{ALK,EST}	12,0	12,1 mmol/l

BIOLOGICAL REACTOR:

Volume of biological reactor	V _{AT}	240897	240897 m ³
Required safety factor	SF _{req}	1,45	1,45 -
Existing safety factor	SF	4,83	13,93 -
Denitrification share	V _D /V	19	19 %
Temperature	T	15,00	25,00 °C
Dry matter suspended solids	SS _{AT}	3,50	3,50 kg/m ³
Sludge age	t _{SS}	20,3	22,0 d
Aerobic sludge age	t _{SS,aer}	16,4	17,8 d
Recirculation	RC	400	400 %
Nitrogen embedded in biomass	X _{N,BM}	12,5	12,5 mg/l
Nitrogen nitrified	S _{NH4,N}	35,5	35,5 mg/l
Nitrogen denitrified	S _{NO3,D}	27,8	28,4 mg/l
Phosphorus embedded in biomass	X _{P,BM} +X _{P,BIOP}	6,0	6,0 mg/l
Phosphorus precipitated	X _{P,Prec}	1,0	1,0 mg/l
Demand of precipitant	FM	*****	***** kg Metal/d

Sludge production:

Daily sludge production	SP _d	41547	38388 kg/d
... from P-removal	SP _{d,P}	2764	2764 kg/d
... from ext. carbon source	SP _{d,ext}	0	0 kg/d

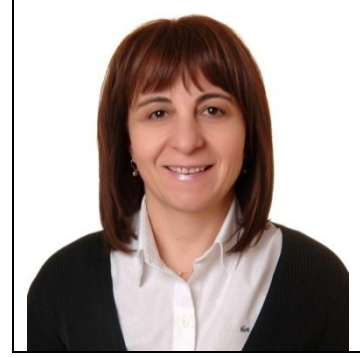
Oxygen uptake:

... for carbon removal	OU _{d,C}	47715	51380 kg/d
... for nitrification	OU _{d,N}	24424	24424 kg/d
... carbon removal by denitrification	OU _{d,D}	-12876	-13178 kg/d
Average daily oxygen uptake	OU _d	59263	62627 kg/d
Average hourly oxygen uptake	OU _h	2469,3	2609,4 kg/h
Peak factor carbon respiration	f _C	1,10	1,10 -
Peak factor ammonia oxidation	f _N	1,50	1,50 -
Maximum hourly oxygen uptake	OU _h	2978,1	3118,3 kg/h
Required oxygen transfer	alpha*OC _h	3715,4	4113,2 kg/h

ANAEROBIC MIXING TANK:

Volume	V _{BioP}	6667 m ³	
Contact time (with Q _{DW,h} , RS=1)	t _{BioP}	0,5	0,5 h

CURRICULUM VITAE



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Publications:

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