PERFORMANCE OF PAŞAKÖY WASTEWATER TREATMENT PLANT

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ABSTRACT

PERFORMANCE OF PAŞAKÖY WASTEWATER TREATMENT PLANT

Ömerli Dam Reservoir is one of the major water resources supplying drinking water of Istanbul. Almost about 40% percentage of drinking water of Istanbul is supplied by Ömerli Dam Reservoir. Due to rapid urbanization, population growth, industrial development and insufficient infrastructure, Ömerli watershed is highly affected by the wastewater discharges from the residential areas and industrial plants. In order to protect the dam against pollution Paşaköy Wastewater Treatment Plant was constructed in 2000, and to extend the capacity, the second phase initiated in 2004 and completed in 2009. All of the process units are investigated for the whole plant, and the operation of the plant is acceptable due to advanced biological treatment plant standards. The influent and effulent parameters of the plant investigated and ploted. The parameters which are biological oxygen demand(BOD_5), suspended solids(SS), total nitrogen(TN), and total phosphorus are investigated and analysed between January 2007 and October 2010. The data indicated that the effluent parameters reach sufficient values according to design effluent parameters of the plant and they were under the limit values of Water Pollution and Control Regulations. After analyses and calculations, it can be said that removing load capacity of the plant is in high levels. Also a proposed model applied for the plant. There was a high correlation between the actual values of the laboratory analysis and predicted values of the model. This leads to use this model in order to forecast the performance of the plant under varying influent loads. Also it can be used as a tool for the plant operator to choose different operational modes without not spending much time. By Paşaköy WWTP ,the most polluted creek Paşaköy which used to flow untreated in to the dam, reaches the Riva Stream with a 6 km tunnel and discharged to the Black Sea. Thus, high amounts of nitrogen and phosphorus which are the nutrients mostly cause the eutrophication, is removed away. Now, Ömerli Dam is a first class potable water source after Paşaköy WWTP was put in to use.

ÖZET

PERFORMANCE OF PAŞAKÖY WASTEWATER TREATMENT PLANT

Ömerli Barajı İstanbul' a içme suyu sağlayan kaynakların başında gelir. İstanbul'un içme suyu ihtiyacının yaklaşık %40 tan fazlası bu baraj gölünden karşılanmaktadır. Çarpık kentleşme, nüfus artışı, endüstri sayısının artması ve alt yapı yetersiziliginden dolayı Ömerli Barajı endüstriyel ve kentsel atıksu deşarjına maruz kalarak kötü yönde etkilenmiştir. Ömerli Barajı'nı korumak adı altında Paşaköy Atıksu Arıtma Tesisi inşa edilmiş 2000 yılında inşa edilmiş ve kapasitesi artırımı için 2004 yılında çalışmalar başlanmış ve ikinci kapasite 2009 yılında bitirilmiştir. Tesisin tüm proses üniteleri tek tek incelenmiş ve ileri biyolojik atık su artma standartlara uygun olduğu görülmüştür. Giriş ve çıkış parametleri tek tek ele alınmış ve grafikler haline getirilmiştir. Biyolojik oksijen ihtiyacı($BOİ_5$), askıta katı madde(AKM), toplam azot(TN) ve toplam fosfor Ocak 2007 ve Ekim 2009 tarihleri arasında ele alınmıştır. Bunun sonucunda tesisin çıkış değerleri tesis dizayn çıkış kriterlerine göre yeterli olduğu anlaşılmış ve ayrıca Su Kirlilği Kontrol Yönetmeliği'nin öngördüğü çıkış değerlerine göre de yeterli seviyede olduğu görülmüştür. Değerlendirmeler sonucu tesisin yük giderme kapasitesinin yüksek seviyerlerde olduğu tespit edilmiştir. Ayrıca tesise bir model uygulanmış ve gerçek değerler ile modelin verdiği değerlerin yakın olduğu elde edilmiştir.Böylece bu model kullanılarak tesise gelen değişik yükler için gelecekte tesisin performansı hakkında yaklşımlarda bulunabilecektir ve tesis değişik durumlarda işletmenin de ne sonuçlar ortaya çıkaracağını vakit kaybetmeden erişilebilecektir. Tesisin inşa edilmesiyle Ömerli Barajı'nı en çok kirleten Paşaköy deresininde baraja girişi engellenmiş, Riva deresine verilen Paşaköy Karadeniz'e deşarj edilmiştir. Böylece ötrofikasyona en çok neden olan Paşaköy deresinden kaynaklı azot ve fosfor yüklerinin baraja girmesi sonlandırılmıştır. Şimdi, Ömerli Barajı içme suyu standartları açısından 1. niteliktedir.

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LIST OF SYMBOLS / ABBREVIATIONS

ATP	Adenozin Triphosphat
ADP	Adenozin Diphosphat
BOD_5	Five-Day Biochemical Oxygen Demand
BIO-P	Bio Phosphorus
COD	Chemical Oxygen Demand
EPDM	Ethylene Propylene Diene Monomer
PE	People Equivalent
SS	Suspended Solids
SSSP	Simulation of Single-Sludge Process
SRT	Sludge Retention Time
TN	Total Nitrogen
TP	Total Phosphorus
UV	Ultraviolet
WWTP	Wastewater Treatment Plant

1.INTRODUCTION

Ömerli Dam Reservoir is a major potable source of Istanbul and has an importance among other potable sources. The geographical features, technical charecterstics, population, climate, creeks feding the reservoir and economical state of the region is explained in a detailed way.

Paşaköy Wastewater Treatment plant was constructed in 2000 with a capacity 100.000 m^3/day to protect Ömerli Dam Reservoir against pollution. Paşaköy creek which was one of the most polluted creek used to flow in to Ömerli Dam is discharged to Riva Stream via 6 km tunnel and reaches to Black Sea. Due to increased pollution load of wastewater collection ares of the plant, the second phase initiated in 2007 and completed in 2009. The whole technical charecteristics of the plant are given and the new units of the plant are also mentioned unit by unit.

The infleunt and effluent parameters which are obtained from Paşaköy WWTP laboratory, are investigated and ploted. After analysis and evlautaions the efficiency of the plant is explained. Also the effluent parameters are taken in to account according to parameters of Water Pollution and Control Regulation (SKKY).

SSSP is one of the model that is applied to wastewater treatment plants to get the effluent parameters without spending much time. This model was proposed to Paşaköy WWTP. Both actual and modeled results compared. The correlation between the results was stated.

Paşaköy WWTP treats the wastewater coming from Paşaköy creek which was discharged to Ömerli Dam before. Thus, nutrient loading caused by Paşaköy creek was prevented to flow into the dam by the plant. The removed nitrogen and phosphorus loads were calculated between January 2007 and October 2009.

2. ÖMERLI DAM RESERVOIR

2.1. Genereal Overview of Water Suppliers of Istanbul

Istanbul has been a major urban center since the Hellenic period. Currently, Istanbul's population is approximately 12 million people, with an annual increase of 400,000. Daily domestic water requirements average 2.6 million m3 in 2000, while industrial demand is also increasing. Successive empires and administrations occupying Istanbul expanded extant infrastructure to meet these demands, a trend that continues today. To meet these needs, though, Istanbul today is forced to cast a wider net, proposing diversion from lakes across the Marmara Sea and from the Black Sea region. On-going urbanization and development continues to dominate the local landscape, requiring water resources and supply systems to keep apace for both domestic and industrial use (Demirci et al., 2001).

Istanbul is experiencing rapid urbanization, with high migration rates from throughout Turkey moving to the city. Official estimates place Istanbul's population in 2000 at 10.3 million. Unofficial estimates believe the number to be more accurately 12 million, with some estimates reaching 15 million. Annually, approximately 400,000 people migrate to the city . Given that the site of Istanbul has been an urban center since the Roman period during which is started its expansion, and its location on a peninsula, Istanbul has always had to face the issue of water supply to meet its ever increasing demands. Water supply has always been a critical issue in this region, which is dominated by limestone and karst topography. Groundwater reservoirs are scarce and unreliable. Thus, the water supply is heavily dependent on surface water supplies, which requires harnessing the surface runoff and safeguarding it against pollution. In this paper, we examine the historical development of water sources for Istanbul, from the Roman empire to proposed plans designed to meet the urban water crisis, in relationship to Istanbul's growing population (Demirci et al., 2001).

About 95% of the potable water demand of Istanbul is met from the surface waters. The rest is from ground water and from historical smalldams. The total area of the catchment basins is 6257 km². The protection of the water resources against pollution is a significant but also a difficult task considering the huge area to be protected. It is recognized that water resources are scarce and must be protected for sustainable water supply. It is also an agreed fact that "prevention is better than the treatment". Thus, we have to apply every possible means to maintain the quality and the quantity of our water resources (Akkoyunlu et al., 2001).

Because of this rapid growth it has become urgent to find new domestic water sources for the people. Istanbul was experienced with water scarcity in the history many times. Following the very severe water shortage problem in 1994, emphasis has been given to the development of water resources. This shortage problem had directed the administers and engineers to find new drinking water resources. Most of the surface water resources around the city were investigated for this purpose. As a result of these investigations some very vulnerable projects has been developed such as Melen Water Supply System and Yeşilçay System. Another attempt for finding new water sources was to renovate the existing structures. The aim of this attempt is to extend the capacity of the existing reservoirs around the Istanbul municipal area. Ömerli Dam is one of the most important water resources of the city. At the present it supplies approximately 40 percent of the total annual water demand of the city (Kılınç et al., 2008).

2.2. Background of Ömerli Dam

Ömerli Dam is a water conservation project planned and constructed by 14th Regional Directorate of DSI (National Water Resources Agency of Turkey) in Istanbul. Construction of the project was completed in 1972. The drainage area of the Project is 634 square kilometers. The dam is located on the Riva River approximately 25 kilometers upstream of the Black Sea. The dam embankment is located in Ömerli County, 28 kilometers east of the Istanbul city center. The Dam was planned to supply water demand for domestic use of Istanbul City. A map that is showing the place of Ömerli Dam is presented as below (Kılınç et al., 2008).

Table 2.1. Existing reservoirs of surface waters resources (Çodur et al., 2007).

Water Supply	**Date of Entering into Service**	**Annual Capacity (Million m3/year)**
Elmalı Dam I and II	1893-1950	15
Terkos Lake	1883	162
Alibeyköy Dam	1972	36
Ömerli Dam	1972	235
Darlık Dam	1989	97
Büyükçekmece Lake	1989	120
Yeşilvadi Conversion Structure	1992	10
Istranca Dams (I.Stage)	1995	44
Şile Caisson Wells	1996	30
Istranca Dams(II.Stage)	1997	191
Sazlıdere Dam	1998	85
Yeşilçay Regulator	2003	145
Total		1170

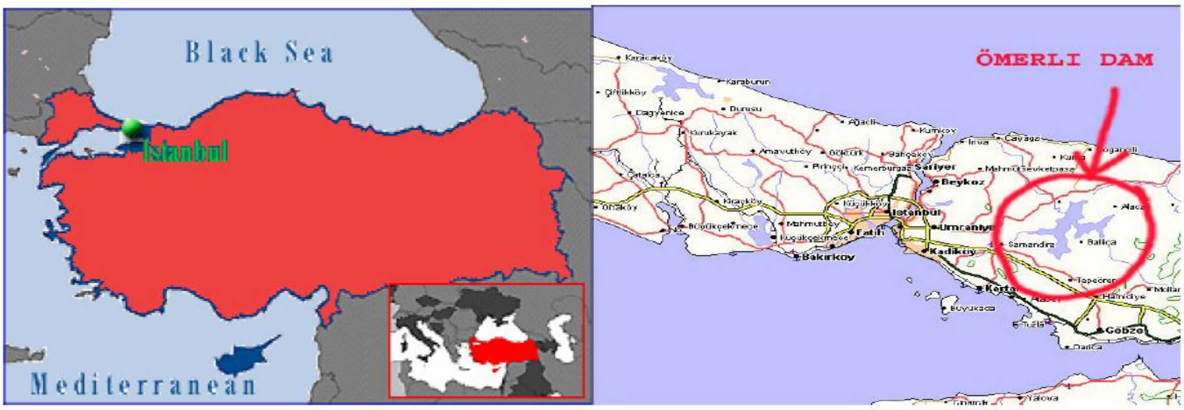


Figure 2.1. Place of Ömerli Dam (Kılınç et al., 2008)

Istanbul's drinking water is provided by surface water resources through seven water dams. Omerli Dam Water Basin is one of the major water dams which supplies 40% of the drinking water of the city (Göksel et al., 2008).

Ömerli Dam, constructed in 1972, is one of the most important reservoirs for the water supply of Istanbul. There is a rising need to water in Istanbul due to rapid population growth (Kılınç et al., 2008).

2.3. Geographical Features

Ömerli basin covers an area of 620 square kilometers. The basin is located Anatolian side of İstanbul and surrounded by Kartal, Ümraniye, Beykoz, Şile and Sultanbeyli districts (Göksel et al., 2000).

Table 2.2. Land Use in Ömerli Watershed (Hazırbaba, 1999)

Land Use Category	Area(km^2)
Urban	59
Agricultural/Rural	219
Forest	320
Surface area of the reservoir	23
TOTAL	621

The reservoir is approximately 50 m high over the sea levland in the eastern part of drainage area, due to step slopes, elevations change between 400 m and 500 m (Hazırbaba, 1999).

From geological point of view, the region is classed as Palazeoic.The formation of this region consists of arkose,sandstone and silt,conglomerate,clayey schist, and quartz (Hazırbaba, 1999).

In the watershed of the Ömerli Dam mean precipitation is approximately 800 mm per year and resulting mean runoff is 242.54 hm3 /year. The mean flow of the Riva creek is 7.78 m3/sec. The reservoir has a maximum volume of 435.54 hm3, an active volume of 240.85 hm3. Its dead volume was planned to be 127.3 hm3. More information related to the technical details of the Ömerli dam can be obtained from following table (Kılınç et al., 2008).

Table 2.3. Technical characteristics of Ömerli Dam and reservoir (Kılınç et al., 2008)

Drainage area	: 634	km^2
River	: Riva creek	
Mean precipitation	: 800	mm / year
Mean runoff	: 242.54	hm^3 /year
Maximum measured flow	: 85.69	m^3 / sec
Mean flow	: 7.78	m^3 / sec
Type of structure	: Earth fill with clayey zone	
Crest level	: 67.00	m
Crest width	: 10	m
Crest length	: 372.00	m
Height of structure	: 53.00	m
Max. water level	: 64.95	m
Max. volume of reservoir	: 435.54	hm^3
Max. Surface area	: 23.91	km^2
Normal water level	: 61.87	m
Volume at N.W.L	: 368.19	hm^3
Surface area at N.W.L	: 21.25	km^2
Min. Water Level	: 46.00	m
Surface area at Min.W.L	: 10.36	km^2
Dead Volume	: 127.3	hm^3
Active Volume	: 240.85	hm^3

2.4. Population of The Region

Population in the basin has shown a drastic increase since 1985(population: 36860) to 1997(population 257204). While the rate of population increase for Istanbul was around 5.3% the same rate for the total water basin were 26.76% for the period of 85-90. The forest area covers 320 square kilometers. The forest area are covered pine trees (18%) and leafed trees (generally oak trees) (82%). Existing forest area is not very efficient in terms of plant materials, therefore is open to erosion. The Watershed has been divided several administration zones. First of all the area located in two cities Istanbul and Kocaeli.

Istanbul Greater Municipality boundary has divided the area into two parts. Out of the Greater Istanbul Municipal boundary there are several independent municipalities (Sarıgazi, Yenidogan, Alemdag, Sultan Ciftligi and Samandra) and even a sub-districts (Sultanbeyli) . Industries are scattered and have not been segregated from the residential zones. 340 industries have located in the basin (Göksel et al., 2000).

2.5. Protection of The Region

The quality of water sources has been controlling and monitoring by the Water Board Authority of Greater İstanbul Municipality. The Water Board Authority of Istanbul applied a regulation for the whole drenage area. According to this regulation the watershed has been divided to protection zones such as Absolute (0-300m), Short (300-1000m) Medium (1000-2000m) and Long Range Protection zones (2000m-Borders of water basin area). Protection zones of Omerli Basin are shown in the following figure (Göksel et al., 2000).

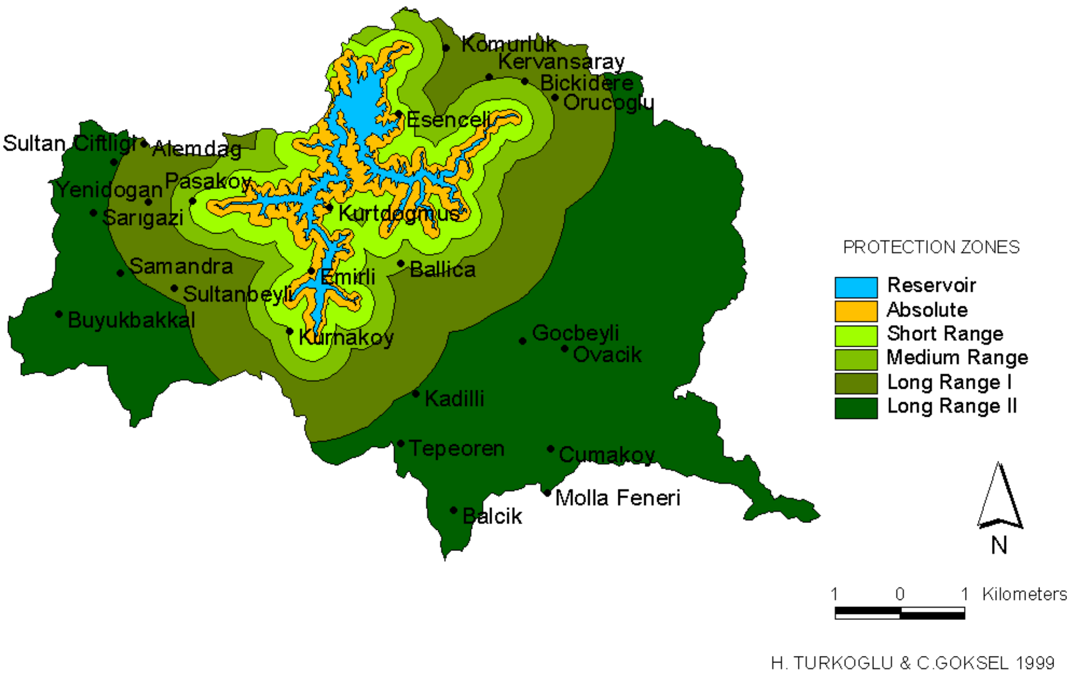


Figure 2.2. Protection zones of Omerli Water Basin Area (Göksel et al., 2000)

2.6. Climate of The Region

Ömerli drainage area and its environment is under the same climate conditions as Marmara Region. Since the region is near to Black Sea, summers are not so hot and dry. Temperature in winters changes between 5 ^{0}C and 8 ^{0}C where as in June,July and August the mean value is 22 ^{0}C.So far,the observed lowest and highest temperatures are -11 ^{0}C(in January) and +40 ^{0} C(in Ausgust) respectively. The mean temperature of the region is +13.2 ^{0}C (Hazırbaba, 1999).

2.7. The Economical State of The Region

The economy of the region depends on both agriculture and indsutry. Kurna Emirli, Paşaköy and Kurtdoğmuş are important villages in terms of agricultural activities. According to the personal communication with Directory of Agriculture in Kartal district,150 ton of fertilizer was distributed by the Directory of Agriculture to above mentioned villages in 1999. The widely produced agro-products of the region are barely and oats.Stock-breeding has no improtance in the drainage area; 2000 cows, 5000 sheep, and 10000 poultry are the figures reflecting the amount of farm animals in the region (Hazırbaba, 1999).

There are 388 industrial establishments in the region. Following table presents the types of industries located in the region.

2.8.Creeks Feding the Reservoir

Average annual inflow of Ömerli Dam is 7.5 m^3/s.For winters seasons the inflow changes between 15 m^3/s and 20 m^3/s whereas for summers the reservoir has only 1-2 m^3/s of inflow (Hazırbaba, 1999).

There are four main streams which are feeding the reservoir at present, namely; the Ozan, the Göçbeyli, the Köy and the Şalgam. The fifth stream named Paşaköy which was feeding the reservoir until 2004 has been connected to the Riva Stream via a tunnel (3 m diameter and 6 km length) by ISKI in order to prevent domestic and industrial wastewater inputs into the reservoir (Özuluğ et al., 2005).

Table 2.4. Industrial establishments located in Ömerli watershed (Hazırbaba, 1999)

INDUSTRY TYPE	NUMBER	LABOUR
Metal	72	1449
Glass	41	646
Fuel-Oil Station	14	91
Carpentry	28	272
Electronics	16	1056
Food	22	519
Farming	75	529
Textile	14	485
Plastic	21	236
Paper	3	57
Chemicals	29	354
Miscellanous	42	852
Uknown	11	163
TOTAL	388	6709

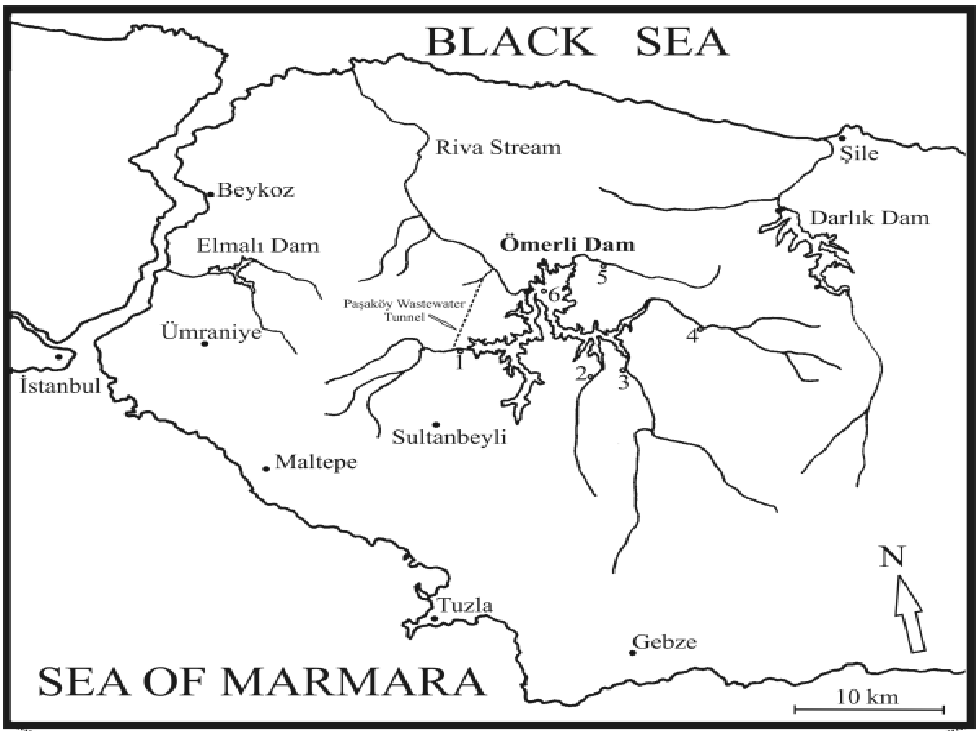


Figure 2.3. Paşaköy Wastewater Tunnel (Özuluğ et al., 2005)

Table 2.5. Creeks Feeding The Reservoir (Hazırbaba, 1999)

Creek	Annual Flowrate(m^3/s)
Ozan	2,69
Göçbeyli	1,43
Köy	0,47
Şalgam	0,22
Paşaköy	0,56

3. PASAKÖY WASTEWATER TREATMENT

3.1. The Importance and Aim of Paşaköy Wastewater Treatment Plant

Pasakoy Wastewater Treatment Plant has been commisioned in 2000 with capacity of 100.000 m^3/day in order to protect the Omerli Dam, one of the major potable sources of İstanbul, from the pollution of untreated wastewater. Pasakoy plant treats wastewater collected from Sancaktepe (Sarıgazi, Samandıra, Yenidoğan), Sultanbeyli, Alemdağ and Sultançiftliği distrcits near Omerli watershed which used to flow untreated into the Omerli Dam.

Wastewater treated according to advanced biological treatment reaches the Riva Stream via a 6 km tunnel and discharged to Black Sea. The ultimate treatment capacity of the plant is planned to be 2.500.000 PE and 500.000 m^3/day wastewater flow.

Figure 3.1. The Location of Paşaköy WWTP

The planning studies in 2004 - 2005 indicated the need of for construction of 2nd phase of the plant due to increased pollution load of wastewater collection area of the plant, so after a tender process the construction was initiated on 08 February,2007. In this second phase, the plant constructed has an additional capacity for a wastewater load of 500.000 PE and 100.000 m ^{3}/day. Thus, 2nd phase was completed in 2009 doubling the existing plant capacity.

Additionally the 2nd phase has the following units;

Sludge Drying, Cogeneration, Biofilter, Sand filtration and UV disinfection. By the help of Sludge Drying the sludge removal from the plant decreases and that sludge which is turned into a dry product 90% solids content and can be used as fertilizer or fuel. Darıca Aslan Cement Plant currently uses the dry product coming from Paşaköy WWTP as fuel.

Figure 3.2. First and Second Capacity of Paşaköy WWTP

3.2. Project Criteria

The plant process is based on biological nutrient removal, where carbon is removed from the wastewater as well as other nutrients such as nitrogen and phosphorus which also pose a pollution threat to Ömerli Dam Reservoir.

Table 3. 1. Paşaköy WWTP Project Criteria

	1.PHASE	2.PHASE
Maximum Flowrate	125.000 m3/d	125.000 m3/d
Project Flowrate	100.000 m3/d	100.000 m3/d
People Equivalent	500.000 PE	500.000 PE
Ultimate Flowrate	500.000 m3/d	
Ulitmate Population Equivalent	2.500.000 PE	
Project Area	507.000 m2	

3.3. Process Explanation

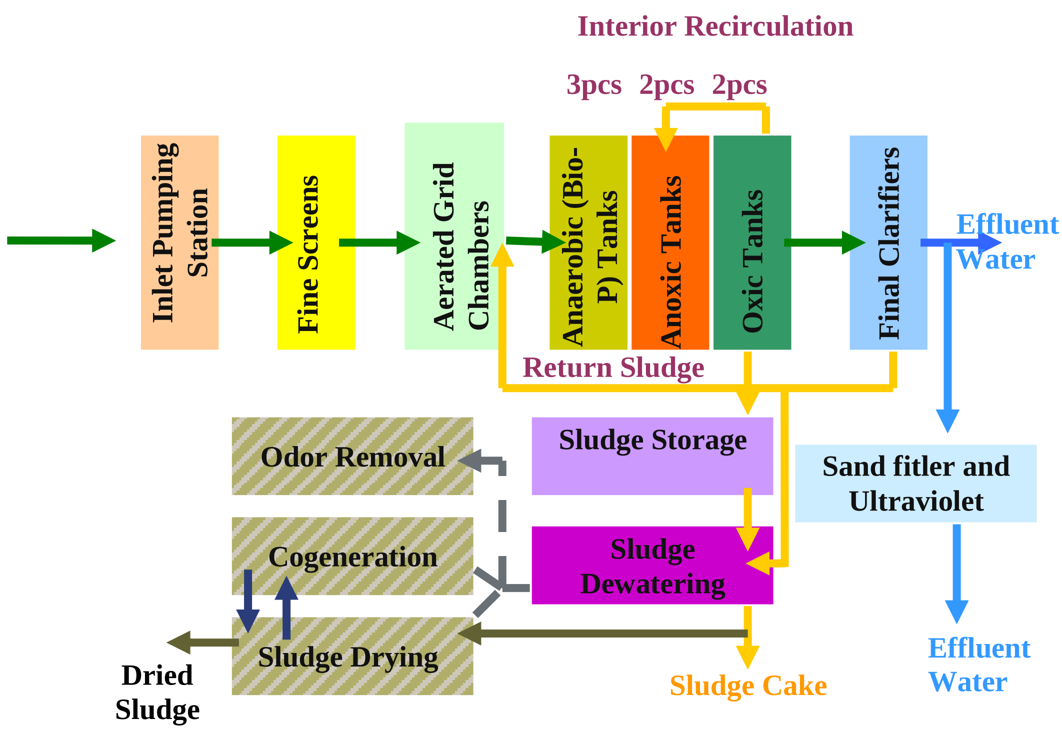


Figure 3.3. Process Schema of Paşaköy WWTP

Influent wastewater contains high levels nitrogen and suspended solids. The wastewater is treated by a physical and biological wastewater treatment plant. This process includes pre-treatment, biological phosphorus removal, denitrification, nitrification and final clarification. Required biological treatment consists of anaerobic, anoxic and aerated zones, sedimentation tanks, return and waste activated sludge pumping stations and blower units. Removal of waste activated sludge consists of direct dewatering and sludge drying units. In addtion, a cogeneration unit was installed in order to supply electric and thermal energy demand of the plant. Odor removal is activated by Biofiltration. And the effluent is disinfected by Sand Filitration and UV Disinfection.

Table 3. 2. Pasakoy Wastewater Treatment Plant Design Parameters

INFLUENT PARAMETERS	QUANTITY	UNIT
Suspended Solids (SS)	500	mg/L
Biological Oxygen Demand (BOD5)	325	mg/L
Chemical Oxygen Demand (COD)	600	mg/L
Total Kjeldahl Nitrogen (TKN)	70	mg/L
Total Phosphorus(TP)	8	mg/L
EFFLUENT PARAMETERS	QUANTITY	UNIT
Suspended Solids (SS)	35	mg/L
Biological Oxygen Demand (BOD5)	25	mg/L
Chemical Oxygen Demand (COD)	125	mg/L
Total Kjeldahl Nitrogen (TKN)	10	mg/L
Total Phosphorus(TP)	3	mg/L

The effluent design parameters issued due to Water Pollution and Control Regulation parameters (SKKY).

3.4. Intake Structure

Wastewater which is taken from Pasakoy stream and collector lines through the intake structure flows to inlet of the coarse screens. This structure is equipped with sliding gates to regulate the influent flow. In case of any failure in the plant,these gates are closed and wastewater is diverted to by pass line.

3.5. Inlet Pumping Station and Pumping Line

The incoming wastewater passes through coarse screens with 50 mm openings, in order to prevent any damage to pumps and other equipment caused by large pieces of solid waste. Coarse screens are automatically cleaned and waste collected on the screen is discharged to a container by a conveyor and sent to Kömürcüoda Solid Waste Sanitary Landfill.

The wastewater passing the screens, is pumped from the wet well to the plant by submersible pumps with a max. capacity of 650 l/sec each. The Variable flow is regulated by ultrasonic level sensors mounted in wet well. The pumping line is a 1.200 mm steel pipe and the pumping head is 39 meters.

Table 3.3. Technical Characterestics of Pumping Station

PUMPING STATION		
	1. PHASE	2. PHASE
Diameter of By Pass Line	2.200 mm	
Number of Coarse Screen/Bar Spacing	2 pieces / 50 mm	
Capacity of Pumps Wih Constant Flow	650 L/ sn	
Number of Pumps With Constant Flow/Power	3 pieces / 395 kW	
Capacity of Pumps With Variable Flow		300-650 L/ sec
Number of Pumps With Varible Flow /Power		3 pieces / 360 kW
Number and Diameter of Inlet Lines	2 lines / 1.200 mm	

Figure 3.4. Pumping Station of Paşaköy WWTP

3.6. Fine Screens and Aerated Grit Chambers

Wastewater coming from pumping station passes through fine screens which have 10 mm bar spacing and consist of 5 parallel lines. Here smaller solid waste particles which passes through the coarse screens are collected. Materials retained by fine screens are transferred with conveyors to screen press and discharged to containers. Wastewater coming from fine screens enters grit chamber which consist of coarse buble diffuser pipes. At this stage, air is supplied by blowers and the settled sand at the bottom of grit chamber, is pumped to gravity seperator by submersible pumps mounted on the mobile scraper bridges.

Accumulated oil and grease on the surface is also seperated by a surface scraper on the travelling bridge. Thus all floating solid waste, sand, grit and grease removed. At the discharge point of grit chamber, effluent flow rate is measured throug parshall flume. This measurement is used for adjusting capacities of equipment in rest of the plant.

Table 3.4. Technical Characterestics of Fine Screens and Grit Chamber

FINE SCREENS AND GRIT CHAMBER		
	1. PHASE	2. PHASE
Number of Fine Screens / Bar Spacing	2 pieces / 10 mm	3 pieces / 10 mm
Number of Grit Chamber Blowers / Power	3 pieces / 7,5 kW	3 pieces / 15 kW
Capacity of Grit Chamber Blowers	300 Nm3/sa, 500 mbar	545 Nm3/sa, 450 mbar
Number of Grit Chambers	2 pieces	2 pieces
Number of Grit Pumps / Power	2 pieces / 5,5 kW	2 pieces / 3,7 kW
Capacity of Grit Pumps	100 m3/sa, 8 mWC	108 m3/h 8 mSS
Number of Grease Pumps / Power	1 piece / 0,8 kW	4 pieces / 3,1 kW
Capacity of Grease Pumps	13,3 m3/sa, 5 mWC	36 m3/sa, 10 mWC
Number and Capacity of Drum Screens		2 pieces / 72 m3/h
Number and Capacity of Grit Seperator	2 pieces / 100 m3/h	2 pieces / 108 m3/h

Figure 3.5. Aerated Grit Chambers of Paşaköy WWTP

Figure 3.6. Grit Seperation of Paşaköy WWTP

3.7. Biological Phosphorus Removal Unit

Wastewater coming from fine screens and grit chamber is mixed with return activated sludge as it enters the biological phosphorus unit. At this stage, for the realization of phosphorus release, easily biodegradable soluble organic carbon in the influent wastewater must be stored in the cells;when an electron acceptor is found the stored carbon will be used for contunuity of microorganism activitivities and excess phosphorus will be stored for production for ATP.

The Bio-P tanks work in series. The motorized gates at the inlets of tanks and between tanks make it possible for the tanks to be used or closed invidually. Submersible mixers are installed to prevent settlements and redoxmeters and oxygenmeters are mounted in order to monitor continuity of desired process conditions. The process flow rate is monitored by a channel type flowmeter at the outlet.

It is possible to let only return activated sludge to 1st tank. Thus, nitrate which comes from retun activated sludge is removed by denitrification without using carbon in influent wastewater. So that, easily biodegrable organic carbon in the influent wastewater is not spent with nitrate and it can be fully used for removal of phosphorus. The influent wastewater is taken to 2nd tank, and is mixed with return activated sludge here. The second way is to mix return activated sludge and the influent wastewater at the first tank. Eventually, according to different conditions and needs biological phosphorus removal is flexible.

Figure 3.7. Bio-P Tanks of Paşaköy WWTP

Biological P Removal 3 tanks work in series.

Excess amounts of P removed incorporated into microorganism cells (P accumulating mo.)

Anaerobic environment } soluble org.C stored

$$ATP \leftrightarrow ADP + Energy + P \quad (3.1)$$

Oxidation Tanks } stored org.C consumed + new mo. cells

$$ADP + P \leftrightarrow ATP \quad (3.2)$$

Table 3.5. Technical Characterestics of Pumping Station

BIO-PHOSPHORUS TANKS		
	1.Phase	2.Phase
Number and volume of Bio-P Tanks	3 pieces/ total 8.910 m3	3 pieces/ 6.753 m3
Water Depth of Bio-p Tanks	5 m	5 m
Retention time of Bio-P Tanks	1,1 h	0,9 h
Number of Bio-P Mixers/Power	6 pieces / 2,3 kW	3 pieces / 3,1 kW
Speed/RPM of Bio-P Mixers	25 r/min	31 r/min

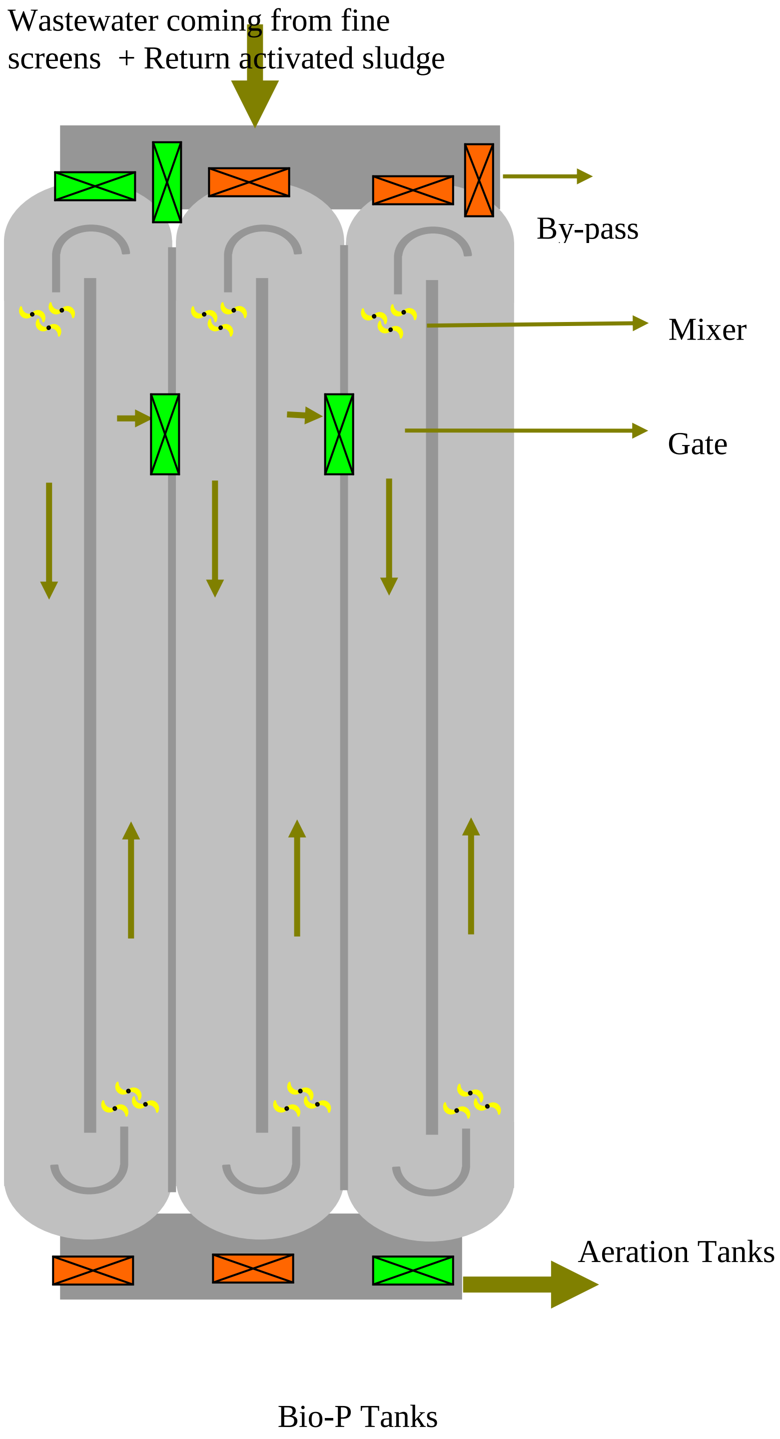


Figure 3.8. Working Principle of Bio-P Tanks

3.8. Aeration (Process Tanks)

Wastewater coming from Bio-P tanks flows to process(aeration) tanks for treatment. At this unit, biological treatment takes place in 2 stages nitrifcation and denitrication.

Nitrification is achieved by supplying enough amount of air, so that ammonia nitrogen in the influent water turns in to nitrate nitrogen and the generated nitrates are used as electron acceptors for the reduction for biological oxygen demand(carbon). In order to prevent sedimentation of suspended solids in each tank, to increase the aeration effiency by extending the path of air bubles and to provide homogenous mixing, banana type mixers of 2,500 mm diameter are mounted. Fine buble membrane diffusers are placed at the bottom of each aretion tank. The amount of air supplied to each channel is adjusted by motorized butterfly valves, so that the dissolved oxygen level in the tanks is kept a preset value.

Redox and oxygen meters were placed in the tanks in order to monitor anoxin and oxic conditions. Recirculation pumps were mounted in the mid-section of the tanks to provide internal recirculation and to pump the nitrate-rich water to anoxic region.

Figure 3.9. Aeration Tanks of Paşaköy WWTP

Table 3.6. Technical Characterestics of Aeration Tanks

AERATION TANKS		
	1st PHASE	2nd PHASE
Number and Volume of Aeration Tanks	4 pieces /total of 44.000 m3	4 pieces / total of 80.000 m3
Water Depth of Aeration Tanks	5,1 m	5,5 m
Number of Aeration Tanks Mixers	16 pieces	16 pieces
Power of Aeration Tanks Mixers	2,30 kW	6,03 kW
Speed/RPM of Aeration Tanks Mixers	25 r/min	47 r/min
Number and Diameter of Aeration Tanks Diffusers	7.852 pieces / 9 “	16.000 pieces / 9 “
Number of Aeration Tanks Recirculation Pumps / Power	4 pieces/ 25 kW	4 pieces / 16 kW
Capacity and Speed/RPM of Aeration Tanks Recirculation Pumps	4.032 m3/h	4.500 m3/h

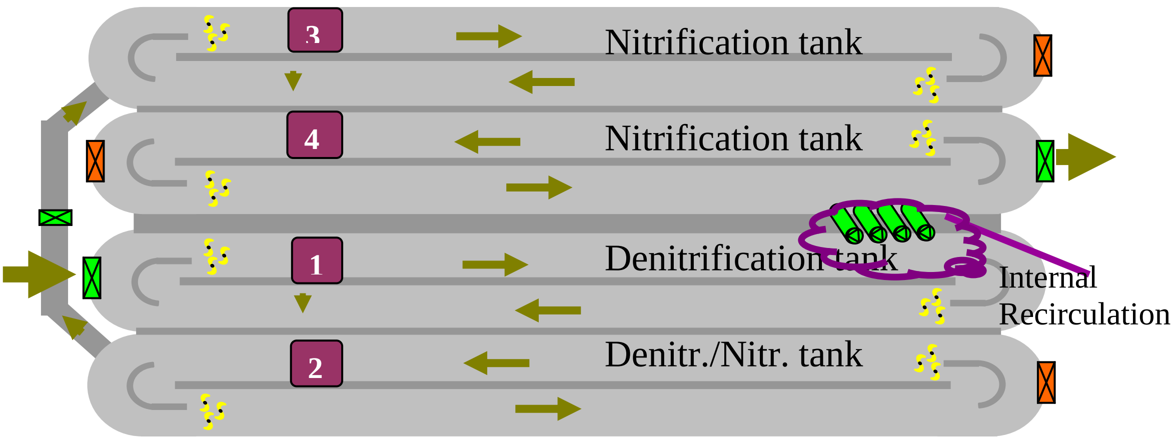


Figure 3.10. Working Principles of Aeration Tanks

Pre-denitrification with internal recirculation

Nitrification (org.N+ammoniumN oxidation):

org.N $\rightarrow$ NH4 + O2 $\rightarrow$ NO3 + mo. (free dissolved O2 used)

C oxidation (excess P removal takes simultaneously):

org.C + O2 $\rightarrow$ CO2 $\uparrow$ + mo. (free dissolved O2 used)

Denitrification (nitrate N reduction): save aeration energy

(excess P removal takes simultaneously)

org.C + NO3 $\rightarrow$ N2$\uparrow$ + mo. (bound oxygen of NO3 used)

3.9. Blower Building

In the 2nd phase, there are 5 blowers in blower building which supply air demand of the aeration tanks.

Table 3. 7. Technical Characterestics of Blower Unit

BLOWER UNIT		
	1st PHASE	2nd PHASE
Number and Capacity of Blowers	3 pieces / 4.950-11.000 Nm3/h 2 pieces / 3.000-9.000 m3/h	5 pieces / 7.000-20.000 Nm3/h
Power of Blowers	250 kW	450 kW
Pressure Values of Blowers	600 mbar	650 mbar

Figure 3.11. Blower Building of Paşaköy WWTP

3.10. Final Clarifiers

The final sedimentation process serves for the separation of the water from colloidal activated sludge generated in the aeration tank, and collection and disposal of the sludge which settles by gravity.

Wastewater flows from the aeration tanks into the final clarifiers which are equipped with rotating half scrapers. Settled sludge directed by bottom scrapers flows by gravity from sludge cone to return activated sludge pumping station and it is recycled to the inlet of biological phosphorus unit. Surface scrapers which were located on the bridges, transfer accumulated scum by gravity to scum collection basin. Scum is pumped from the basin to the sludge storage tank. While treated wastewater flows through one sided outlet weirs, cleaning of the channel is provided by a brush mounted on each rotating bridge.

Table 3.8. Technical Characterestics of Final Clarifiers

FINAL CLARIFIERS		
	1st PHASE	2nd PHASE
Number and Diameter of Final Clarifiers	4 pieces / 42 m 2 pieces / 42 m (addtional tanks)	4 pieces / 43 m
Water Depth of Final Clarifiers	2,8 m 3,5 m (additional tanks)	3,5 m
Hydraulic Retention Time	3,3 h	2,2 h

Figure 3.12. Final Clarifiers at Construction State

Figure 3.13. Final Clarifiers at Construction State

3.11. Final Clarifier Distrubition Chamber and Return Sludge Unit

Settled sludge, flowing by gravity from clarifiers to final clarifier distribution chamber and to the return sludge unit is recycled to the inlet of biological phosphorus unit by pumping. Recycle rate is approximately 80%.

Table 3.9. Technical Characterestics of Final Clarifiers Distrubition and Re-Circulation Chamber

FINAL CLARIFIERS DISTRIBUTION BOXES AND RE-CIRCULATION CHAMBER		
	1st PHASE	2nd PHASE
Number and Power of Recirculation pumps	10 pieces/ 22 kW	6 pieces / 75 kW
Capacity of Recirculation Pumps	540 m3, 10 mWC	1.875 m3/h, 10 mWC
Number and Power of Grease Pumps	2 pieces / 7,5 kW	4 pieces / 3,1 kW
Capacity of Grease Pumps	80 m3/h	20 m3/h, 21,5 mWC

3.12. Sand Filtration and Ultraviolet Disinfection Unit

The treated wastewater is disinfected by a Sand Filtration and UV Disinfection Unit in order to be used as industial and/or irrigation water. As a last step, treated wastewater coming from final clarifiers can be diverted to sand filtration unit which consist of 8 sand filters by a channel. The aim of this filtration is to decrease the suspended solids (SS) and turbidity of treated wastewater, so increase the effect of UV light to destroy bacteria and virus in the effluent. The type of sand filter is declining rate deep bed filter. Diameter of sand particles change between 0,8 mm and 1,2 mm. The uniformity coefficient equal or lower than 1,4.

After filtration and final SS removal wastewater flows through the UV channel. The design capacity of UV channel is 5.420 m3/h. In the channel, there are a total of 540 UV lamps in 3 modules and each module has 180 lamps. The lamps’ characteristics are high intensity, low pressure, variable outlet power and they can be cleaned automatically. After disinfection, the target fecal coliform value of treated wastewater will be 0-2 CFU/100 mL.

Table 3.10. Technical Characterestics of Sand Filter and Disinfection Unit

SAND FILTERS VE UV DISINFECTION UNIT		
	1st PHASE	2nd PHASE
Capacity of Sand Filters		100.000 m3 / d
Disinfection / Hydraulic Capacity (max)		125.000 m3 / d
Number and Area of Sand Filters		8 pieces / 60 m2
Influent / Effluent Turbidity		30 mg/L / < 10 NTU
UV Influent Fecal Coliform		max. 100.000 CFU / 100 mL
UV Effluent Fecal Coliform		< 2,2 CFU / 100 ml
		(30 days geometric environment)

Figure 3.14. Sand Filters of Paşaköy WWTP

3.13. Sludge Storage Tank

4,430 m3/d of waste activated sludge produced in the plant is taken from return activated sludge line; however when it is desired, piping is available to take the sludge from process tanks. Waste activated sludge taken from the plant, is mixed in sludge storage tank by 4 slow mixers in order to keep the sludge homogenous. Also to provide aerobic conditions, the tank is equipped with EPDM diffusers. The stored sludge in the storage tank is taken to sludge dewatering unit. Also sludge can be pumped directly to dewatering unit without the use of the storage tank.

Table 3.11. Technical Characterestics of Sludge Storage Tank

SLUDGE STORAGE TANK		
	1st PHASE	2nd PHASE
Volume of the Tank	2460 m3	
Diameter and Depth of the Tank	27m / 4,3m	
Number and speed RPM of mixer	4 pieces/ 480 d/d	
Number and Diameter of the Diffuser	340 pieces / 9”	
Number and Power of Blowers	2 pieces/40kW + 1 piece/90 kW	
Capacity of Blower	2 * 1.800 Nm3/h + 3.600 Nm3/h	

Figure 3.15. Sludge Storage Tank of Paşaköy WWTP

3.14. Sludge Dewatering Unit

Centrifuges are used in the plant as sludge dewatering equipment. Waste activated sludge of the system is directly dewatered without thickening. So, approximately 1% total solids containing sludge of the system is dewatered to become 25% sludge cake and conveyed to the sludge drying unit.

Table 3.12. Technical Characterestics of Dewatering Unit

DEWATERING UNIT		
	1st PHASE	2nd PHASE
Number and capacity of macerators	2 pieces/ 12 m3/h	1+1 162,5 m3/h
	2 pieces/ 35 m3/h	
	2 pieces/ 70 m3/h	
Number and Capacity of Sludge Feeding Pumps	2 pieces/ 4 -12 m3/h	5+1 65 m3/h
	2 pieces/ 17,5 - 35 m3/h	
	2 pieces/ 20 - 70 m3/h	
Number and Capacity of Polyelectrolyte Dosing Pumps	2 pieces/ 300 - 3.000 L/h	5+1 815 L/h
	2 pieces / 600 - 3.000 L/h	
	2 pieces / 500 - 3.000 L/h	
Polyelectrolyte Preperation Unit	2.000 L	6000 L
	3.000 L	
	6.000 L	
Number and Capacity of Centrifuges	2 pieces / 10 m3/h	5+1 65 m3/h
	2 pieces / 35 m3/h	Max.1.800 – 2.000 kg/h
	2 pieces / 70 m3/h	

Figure 3.16. Sludge Dewatering(Centrfuges)

3.15. Sludge Drying Unit

In sludge drying unit, biological sludge including 25% total solids, turns into a dry product of 90 % solids content. The excess moisture of the dewatered sludge is evaporated by heat transfer with convection through 2 pcs. sludge driers.

Each sludge dryer has a capacity drying 100 ton/d of wet sludge, and necessary heat for drying will supplied as 11.5 ton/h and 12 bar of vapour generated by the cogeneration unit. The dryers are pedalled type driers which are one of the most resistant and reliable equipments in the field. Wet sludge conveyed to each drier, is dried by blending with two pedalled shafts rotating in a channel, which has hot vapour in its jacket and inside the rotating paddles. Thereby sludge is dried indirectly without contact with heat source. By the help of indirect drying principle explosion risks are eliminated and air-vapour amount which is used and polluted in closed circuit drying decrease significantly.

Thanks to sludge drying, the environmentally and economically problematic issues of wet sludge transportation and storage are eliminated. Additionally 200 ton/day hazardous sludge turns into 50 ton/day dry product of economic value as it can be used as fertilizer or fuel.

Table 3.13. Technical Characterestics of Sludge Drying Unit

SLUDGE DRYING UNIT		
	1st PHASE	2nd PHASE
Influent Sludge Solid Content %		25%
Dried (Effluent) Sludge Solid Content %		92%
Number and Type of Dryers		2 pieces / Indirect Paddle Dryers
Capacity of the Dryers (wet cake)		100 ton / d
Steam Need of the Dryer (190 0C, 12 bar)		11,5 ton /h

Figure 3.17. Sludge Drying Unit of Paşaköy WWTP

3.16. Odor Removal Unit

The odor containing air is collected with 2 x 17,000 m3/h capacity fans from necessary places and odor removal is achieved by biofiltration through biomedia bed of 236 m3.

In the chemical pre-scrubber, odor containing air is washed with pH adjusted caustic (NaOH) – water solution, so that both removal of hydrogen sulfide (H2S) and acid components are enhanced and air is saturated with moisture prior to biofilter. Subsequently, conditioned air is diverted through CTP channels, underneath the biofilter bed, a special bacteria absorbed organic material, which provides biological odor removal by the help of bacteria activity inside.

3.17. Laboratory Building and Workshops

In the 2nd phase, an independent double-storey laboratory building was constructed. 1st level of the building consists of a chemistry department where gravimetric, volumetric, titrimetric and spectrophotometric analysis can be done, a bacteriology department for microbiological analysis, weighing room, chemical and glass materials storage room, offices and a gas room for gas cylinders. 2nd level consists of 3 analysis rooms each of where Atomic Absorption Spectrometer, Gas Chromatography and Total Organic Carbon instruments is placed along with offices.

In laboratories;

- Characterization wastewater which come to the plant from collectors, analysis of the parameters which take place in Regulations of ISKI on Wastewater Discharge to Sewerage,
- Control parameters about plant operation and lab-scale studies for process improvement.
- Analysis of discharged water through the Riva Stream as per the parameters indicated in quality classification of Ministry of Environment’s Water Pollution Control Regulations, also physical measurements, chemical analysis, heavy metal measurements and bacteriological analysis for recycling studies of treated wastewater can be done.

Also, in the 2nd phase, the existing workshop areas which were deficient for the plant were replaced by a new independent workshop building meeting the electrical-mechanical work requirements and needs of plant operation.

3.18. Energy Supply and Cogeneration

Gas Turbine type cogeneration unit is installed to meet the energy demand of the entire plant and also thermal energy demand of drying unit. To produce necessary electrical energy, a natural gas burning 4.6 MW Gas Turbine is used. Thereby, thermal energy is gained from wasted high temperature exhaust gas of Gas Turbine by heat exchanger and steam boiler systems, while electrical energy is produced by the turbine's generator. So, electricity is produced by burning natural gas and there is no additional expenditure for electricity. Also there isn't any cost of energy for the drying unit, since the waste exhaust heat is used for sludge drying. This system is especially advantageous when high amount of thermal energy is needed compared to the electrical energy demand, exactly like this application in the plant.

According to plant's electrical infrastructure, Uzundere pumping station is supplied by Ayedaş with 34.5 kV line and 2 units of 34.5/0.4 kV main substations (each 2000 kVA) are used. In Paşaköy plant 2 units of 34.5/6.3 kV 8000 kVA main substations, 1 unit of 34.5/6.3 kV 6750 kVA substation, 2 units of 6.3/0.4 kV 4000 kVA substations and 2 units of 6.3/0.4 kV 2500 kVA substations are used. (Paşaköy, 2009)

Table 3.14. Technical Characterestics of Co-Generation Unit

CO-GENERATION UNIT		
	1st PHASE	2nd PHASE
Type and Number of Generator		Gas Turbine / 1 piece
Fuel Type		Natural Gas
Electricity Production Rate		4.600 kW
Steam Production of Exhaust Gas		11,5 ton /h (190 0C, 12 bar)*

Figure 3.18. Co-Generation Unit of Paşaköy WWTP

3.19. Flow Schema and Hydraulic Profile of Paşaköy WWTP

The flow schema and hydraulic profile of Paşaköy WWTP are shown in Figure 3.20. and Figure 3.21.below. Direction of flow and hydraulic profile are stated unit by unit. Also in Figure 3.19. the process control diagram which operators of the plant follow whole processes from this diagram. If there is a wrong at any process, operators see the point which causes the fault and remove.

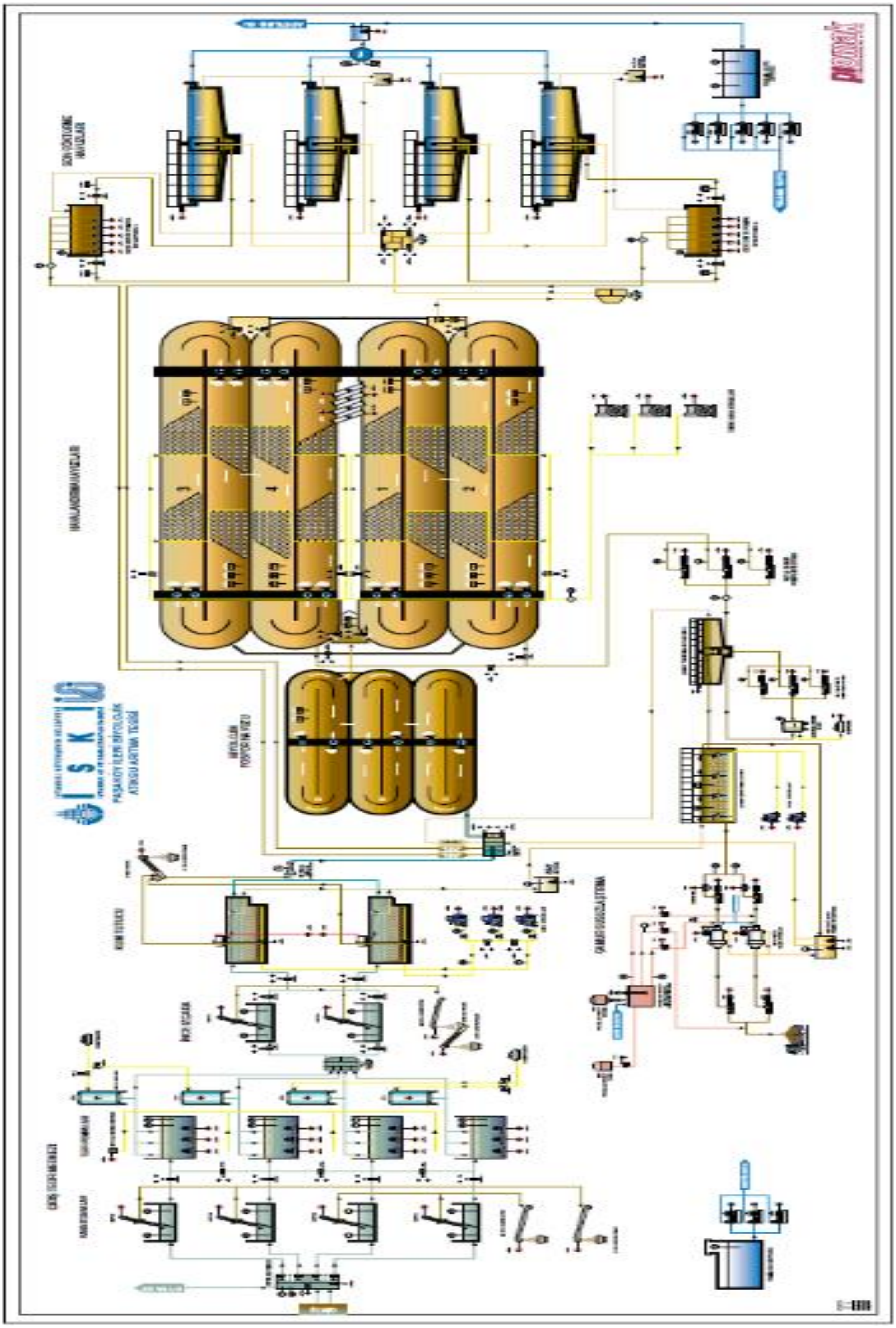


Figure 3.19. Process Control Diagram of Paşaköy WWTP

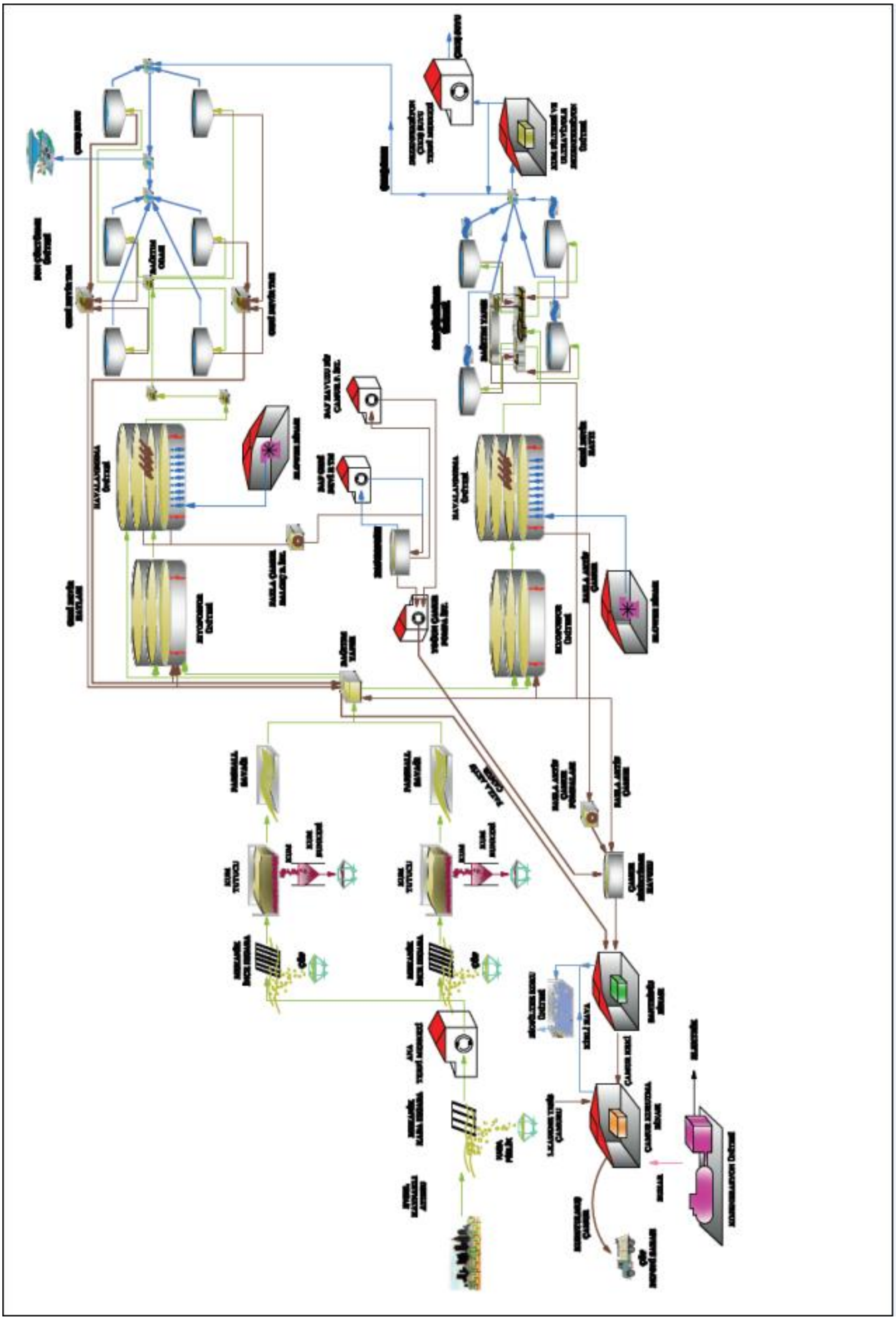


Figure 3.20. Process Flow Scheme of Paşaköy WWTP

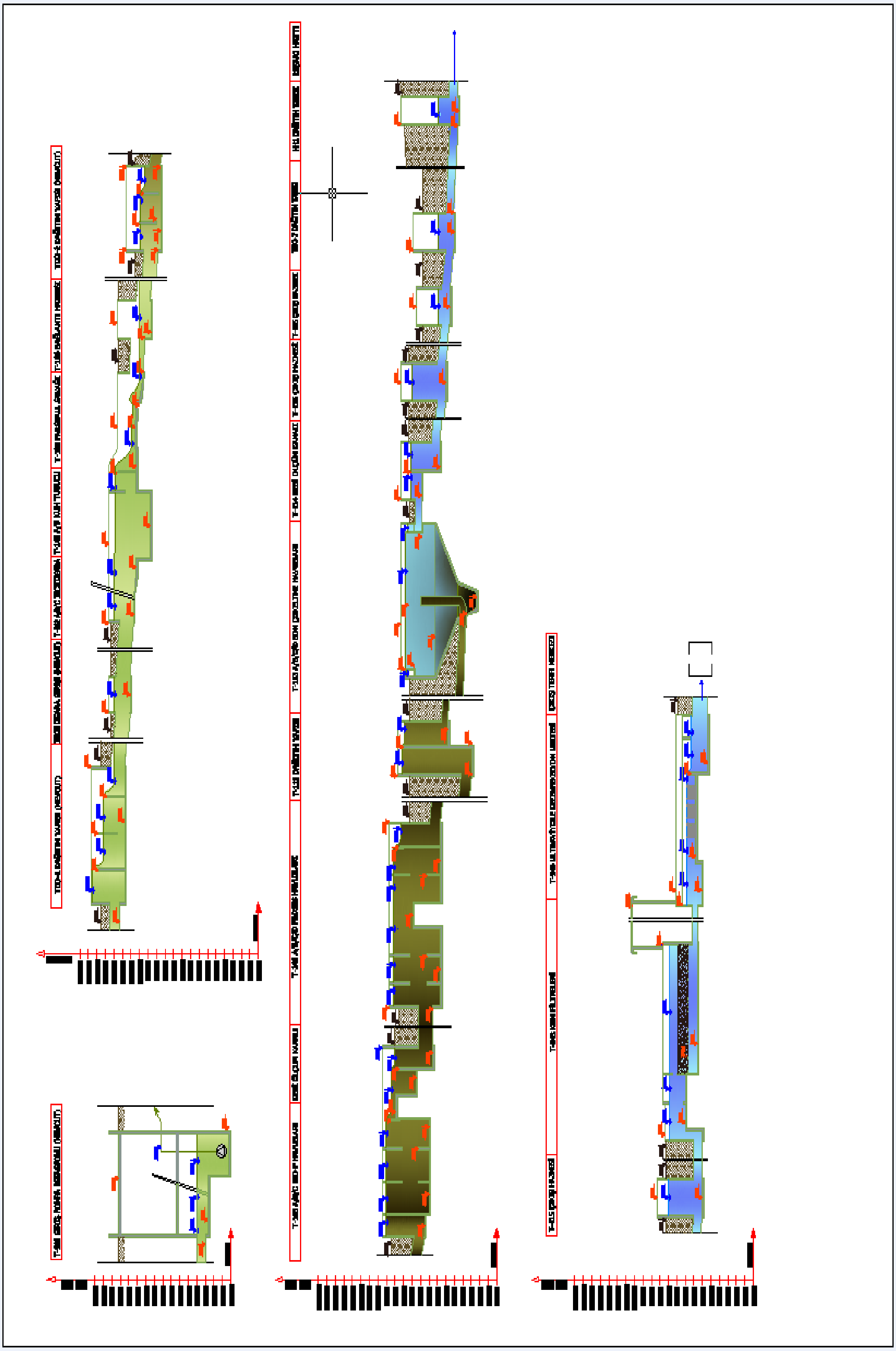


Figure 3.21. Hydraulic Profile of Paşaköy WWTP

4. EFFICIENCY OF PAŞAKÖY WWTP

In order to evaluate the treatment efficieny of the Paşaköy WWT Plant, the daily laboratory anaylsis results dating back to January 2007 till October 2009 were investgated. The data collected from the laboratory of the Paşaköy WWTP was analyzed and plotted against time to be able determine the changing trends within time. After regular analyses of influent and effluent wastewater from lab studies between January 2007 and October 2009, control parameters are compared with design parameters.

4.1. Suspended Solids(SS)

Between January 2007 and October 2009, monthly average influent concentration of suspended solids are given below (Table 4.2.). Taking into account the most highest concentration of SS which is 424 mg/l, was found in August 2008. Thus, treatment plant exceeds the design influent concentration.

Design Influent Concentration : 350 mg/l
Design Effluent Concentration : 30 mg/l
Design Flowrate : 80.000 m^3/day
Design Load : 28.000 kg/day
Removed Load : 20.545 kg/day

Q = 28.000 / 0,424 Q = 66.037 m^3/d

This means treatment plant is succesfull for 424 mg/l, if the flowrate is lower than 66.037 m^3/day. When we look at actual flowrates in August 2008 the average flowrate is 61.011 m^3/day (Table 4.2.). Thus, there is no obstacle for the treatment plant to reach suitable effluent concentration.

In dry conditions between May and October

Average Influent Concentration : 329 mg/l

Q = 28.000 / 0,316 Q = 88.607 m3/day

This indicates the treatmant plant can reach the suitable effluent values, if the flow rate is not higher than 88.607 m^3/day. In actual flowrate in dry sessions almost is 66.960 m^3/day (Table 4.2). Thus, the treatment plant is adequate.

Removal of SS in years 2007,2008 and 2009 can be seen in the figures.

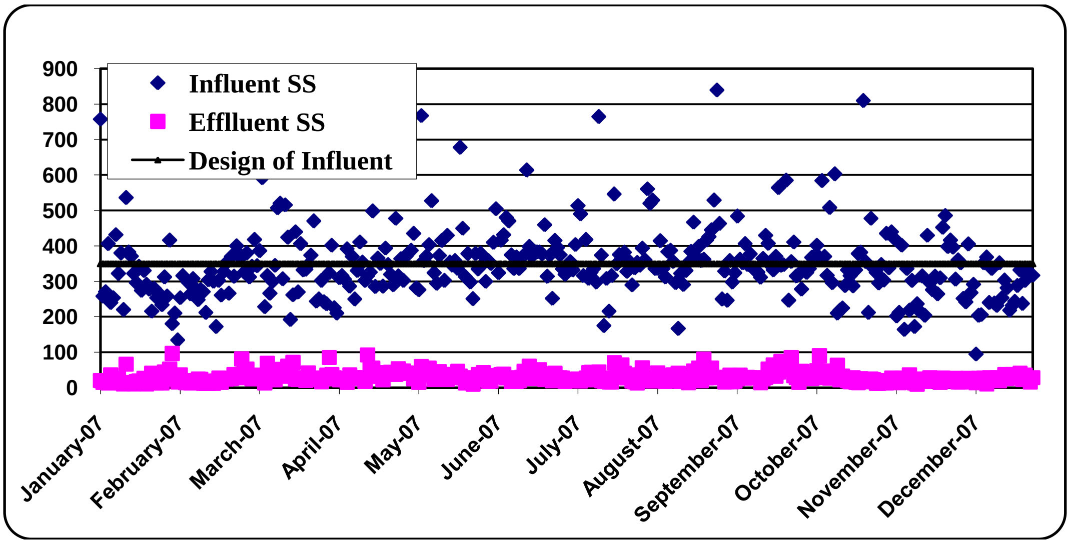


Figure 4.1. Removal of SS in 2007

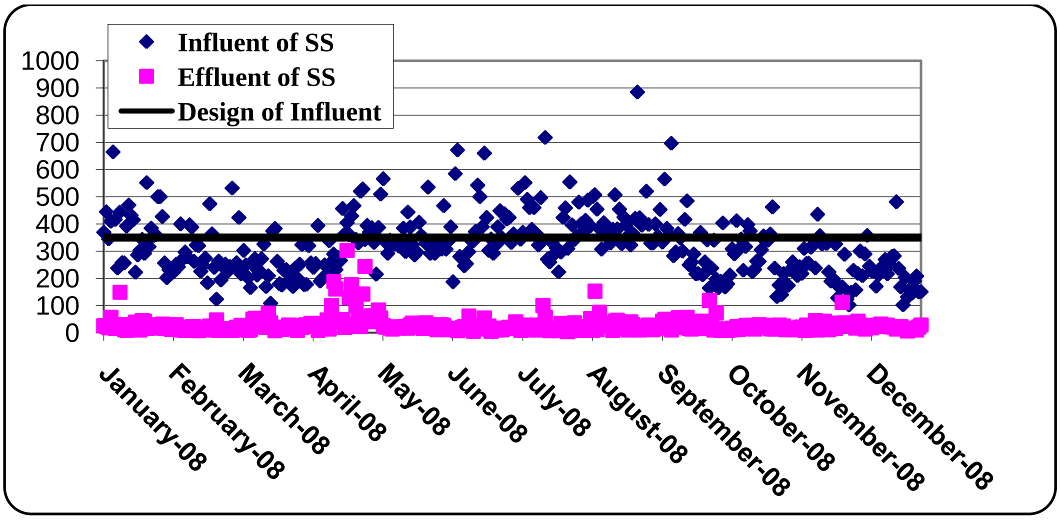


Figure 4.2. Removal of SS in 2008

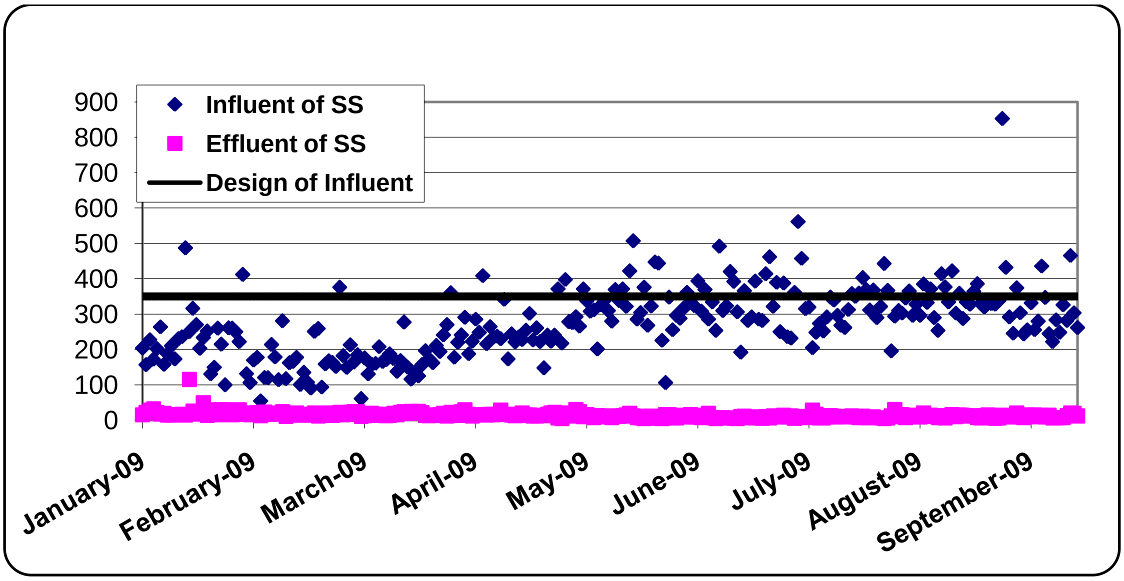


Figure 4.3. Removal of SS in 2009

From the graphs it can be seen that effluent SS values is lower than design effluent concentration which is 30 mg/l.It can be said that treatment flat is adequate in terms of SS removal.

4.2. Biological Oxygen Demand (BOD5)

When the values of BOD_5 between January 2007 and October 2009 are evaluated, monthly average concentration was observed as the highest value of 372 mg/l in August 2009 (Table 4.2.). With this treatment plant gets concentration which is over than design influent concentration.

Design Influent Concentration	: 300 mg/l
Design Effluent Concentration	: 20 mg/l
Design Flowrate	: 80.000 m3/day
Design Load	: 24.000 kg/day
Removed Load	: 19.561 kg/day

Q = 24.000 / 0,372 Q = 64.516 m3/day

When the treatment plant gets highest values of BOD_5 which is 372 mg/l, it can be adequate if the flowrate is lower than 64.516 m3/day. But the average flowrate is 67.865 mg/l in August 2009 (Table 4.2.). So the treatment plant is not adequate for this flowrate.

For dry seasons between May and October

Average Influent Concentration : 314,2 mg/l

Q = 24.000 / 0,3142 Q = 76.384 m3/day

It can be said for dry conditions, treatment plant is sufficient if the flowrate is lower than 76.384 m3/day. Actually the average flowrate between January 2007 and October 2009 is 66.960 m³/day. So, there is no obstacle for the treatment plant to remove BOD_5 sufficiently in dry sessions.

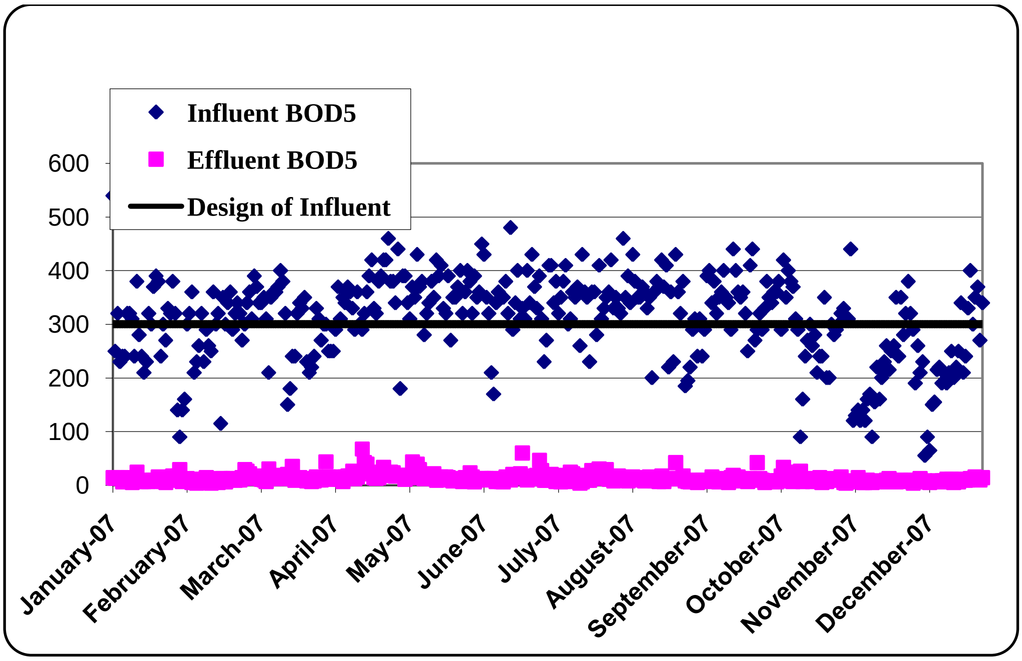


Figure 4.4. Removal of BOD5 in 2007

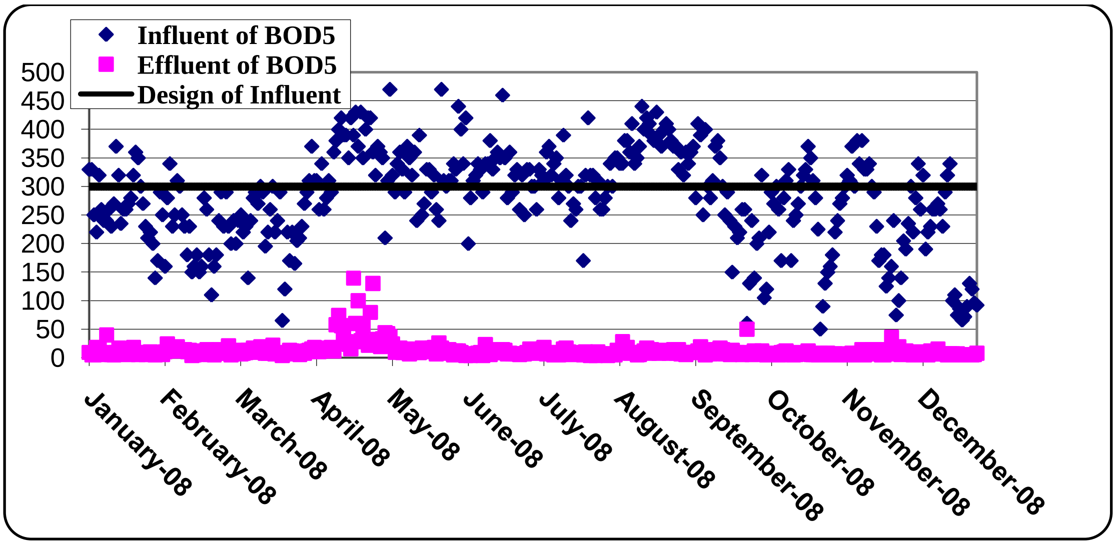


Figure 4.5. Removal of BOD5 in 2008

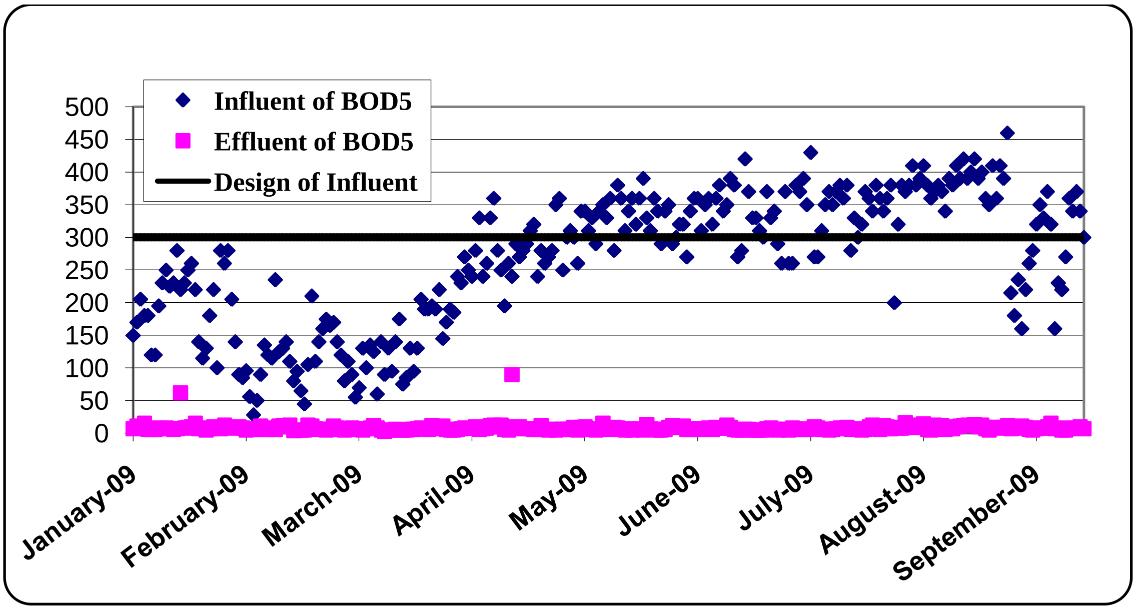


Figure 4.6. Removal of BOD5 in 2009

From the graphs it can be seen that effluent BOD_5 values is lower than design effluent concentration which is 20 mg/l. It can be said that treatment plant is adequate in terms of BOD_5 removal.

4.3 Total Nitrogen(TN)

Between January 2007 and October 2009 the average highest value of TN is 79,6 mg/l in July 2007. It can be seen in Table 4.2.. It is higher than the design influent concentration.

Design Influent Concentration	: 60 mg/l
Design Effluent Concentration	: 10 mg/l
Design Flowrate	: 80.000 m3/day
Design Load	: 4.800 kg/day
Removed Load	: 3.699 kg/day

Q = 4800 / 0,0796   Q = 60. 301 m3/day

For sufficient treatment according to design influent values, the flowrate should be lower than 60.301 m3/day for the highest value which is 79,6 mg/l. The average flowrate is 54.333 m3/day in July 2009 (Table 4.2.). So the treatment plant is adequate for this flowrate.

For dry seasons between May and October

Average Influent Concentration : 60,93 mg/l

Q = 4.800 / 0,06093   Q = 78.779 m3/day

This shows that the treatmant plant can reach the suitable effluent values, if the flow rate is not higher than 78.779 m^3/day. The average flowrate in dry sessions is 66.960 m^3/day, thus treatment plant is succesfull in terms of removing TN.

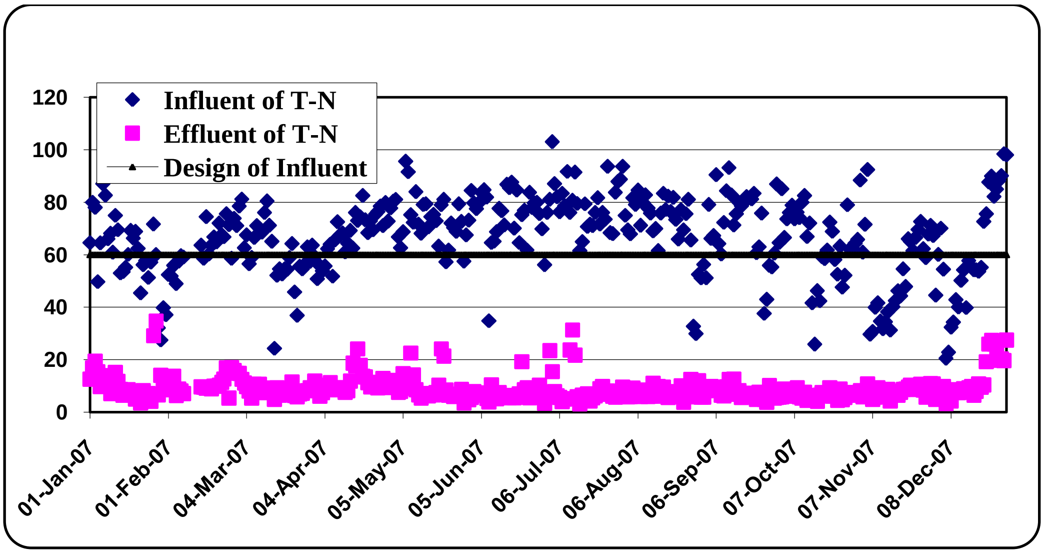


Figure 4.7. Removal of TN in 2007

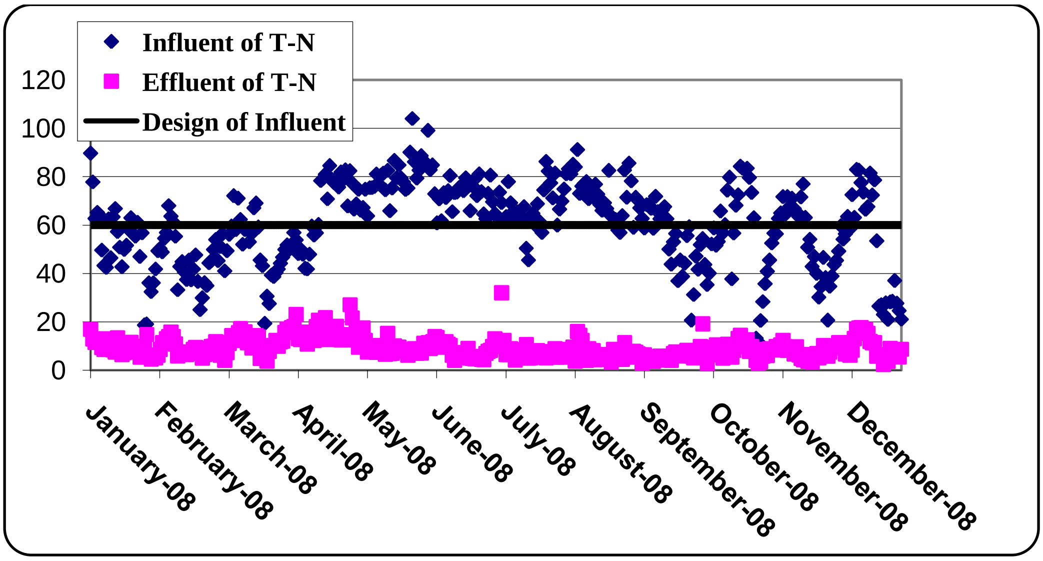


Figure 4.8. Removal of TN in 2008

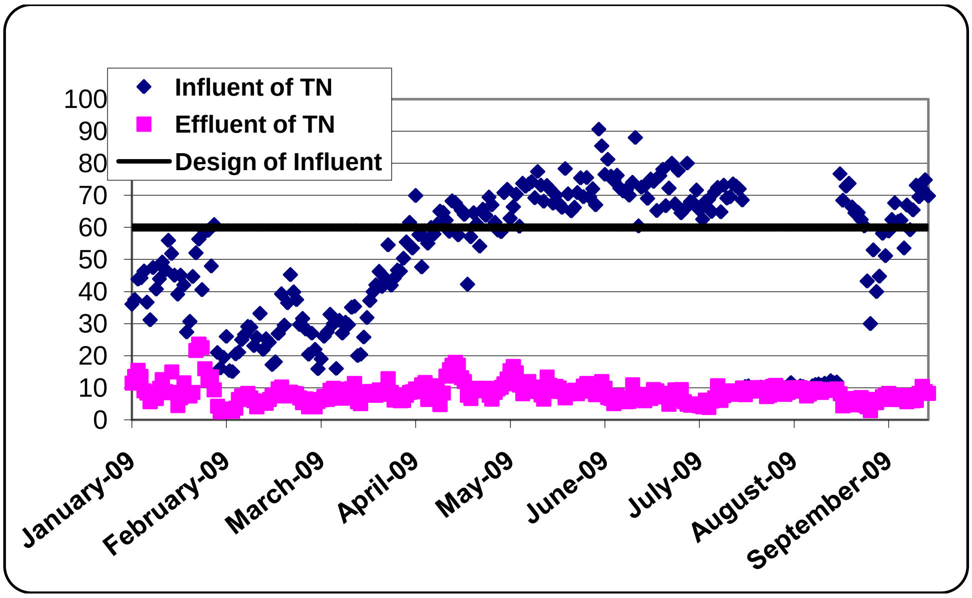


Figure 4.9. Removal of TN in 2009

From the graphs it can be seen that effluent T-N values is lower than design effluent concentration which is 10 mg/l.It can be said that treatment flat is adequate in terms of T-N removal.

4.4 Total Phosphorus(TP)

Between January 2007 and October 2009, monthly average influent concentrations of total phosphorus are in Table 4.2.. Taking into account the most highest concentration of TP which is 11,5mg/l, was get in May 2008.

Design Influent Concentration	: 5,4 mg/l
Design Effluent Concentration	: 3 mg/l
Design Flowrate	: 80.000 m3/day
Design Load	: 432 kg/day
Removed Load	: 279 kg/day

Q = 432 / 0,0115 Q = 37. 565 m3/day

For sufficient phosphorus removal with a highest value of 11,5 mg/l in May 2008, flowrate has to be lower than 37. 565 m³ day. In table 4.2, it can be seen that flowrate is 68.232 m³/ day. Thus, treatment plant is not adequate such highest phosphorus load. When we look at the influent values of TP it can be seen that at almost infleunt concentration exceeds the design influent concentration. The removal of phosphorus at almost about 52 percentage. Phosphorus removal is affected by the concentration of Volatile Fatty Acids(VFA). The lack of VFA badly affect the phosphorus removal.

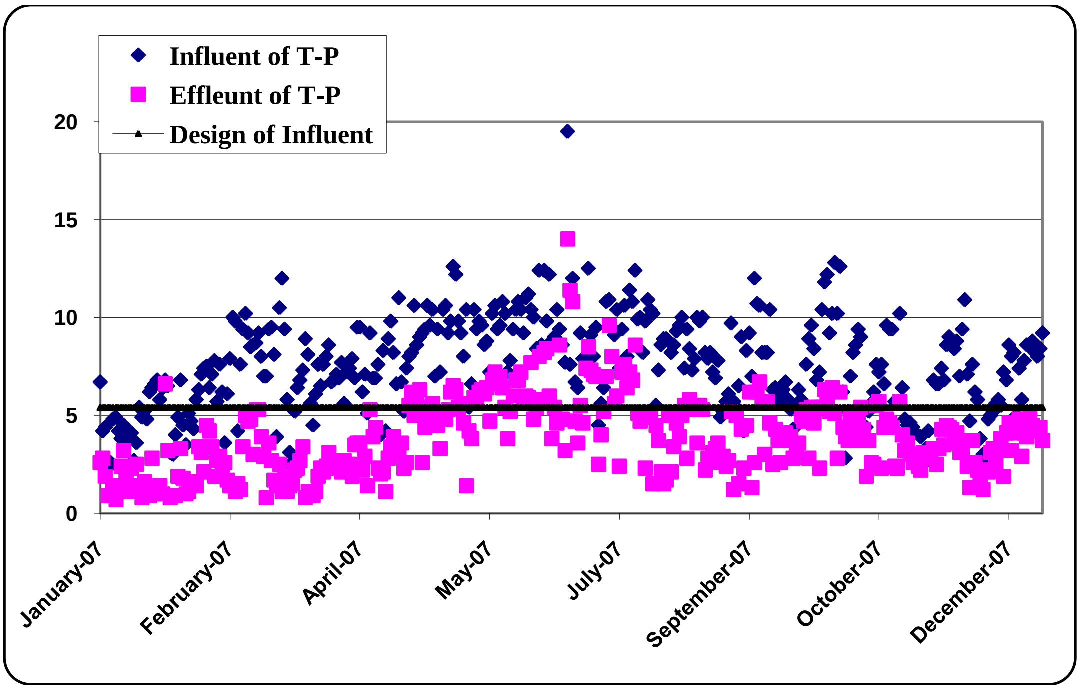


Figure 4.10.Removal of TP in 2007

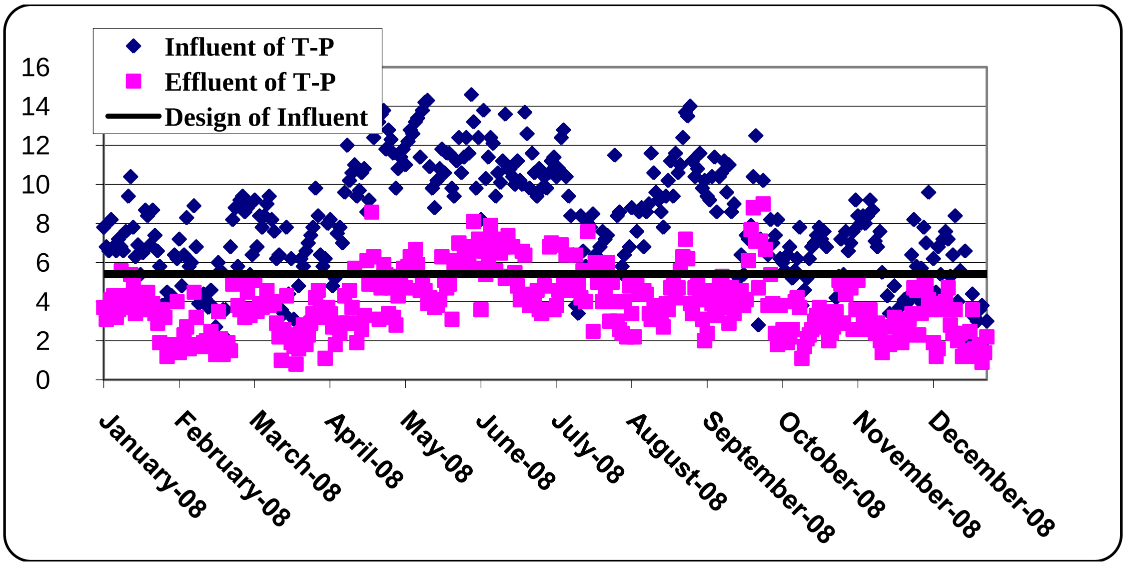


Figure 4.11. Removal of TP in 2008

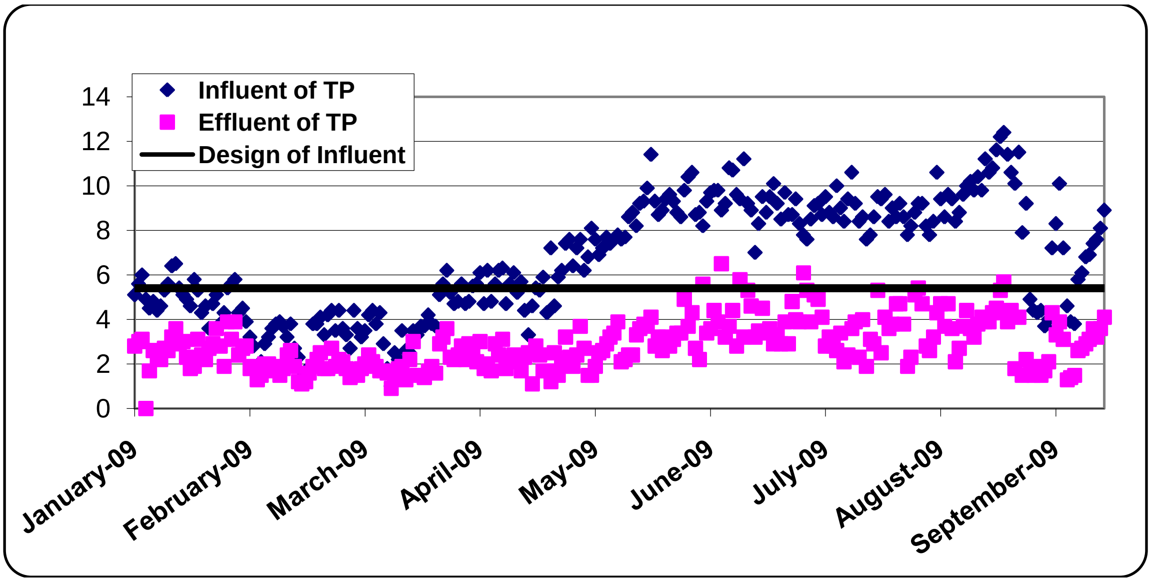


Figure 4.12. Removal of TP in 2008

4.5. Changes in Flowrate and Temperature

Change in flowrate and change in temperature of the wastewater is shown below. Flowrate and the temperature of the wastewater are the ones that affect the effluent parameters. It is important know their values. Flowrate changes affect the load of nitruent. Besides, temperature has an effect on synthesis and growth of bacterias in the wastewater, dissolved oxygen, BOD_5 and decompostion.

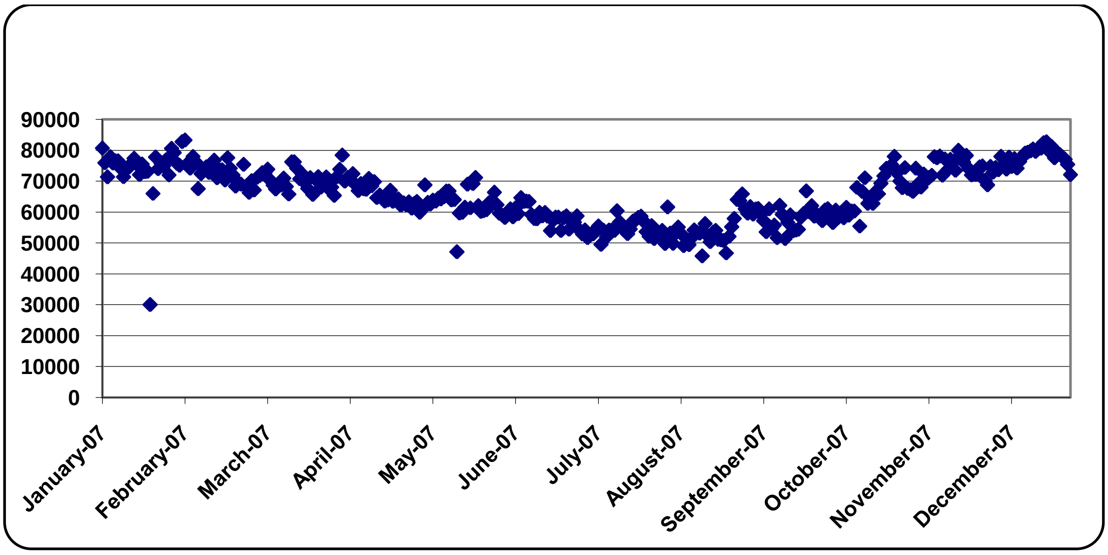


Figure 4.13. Flowrate change in 2007

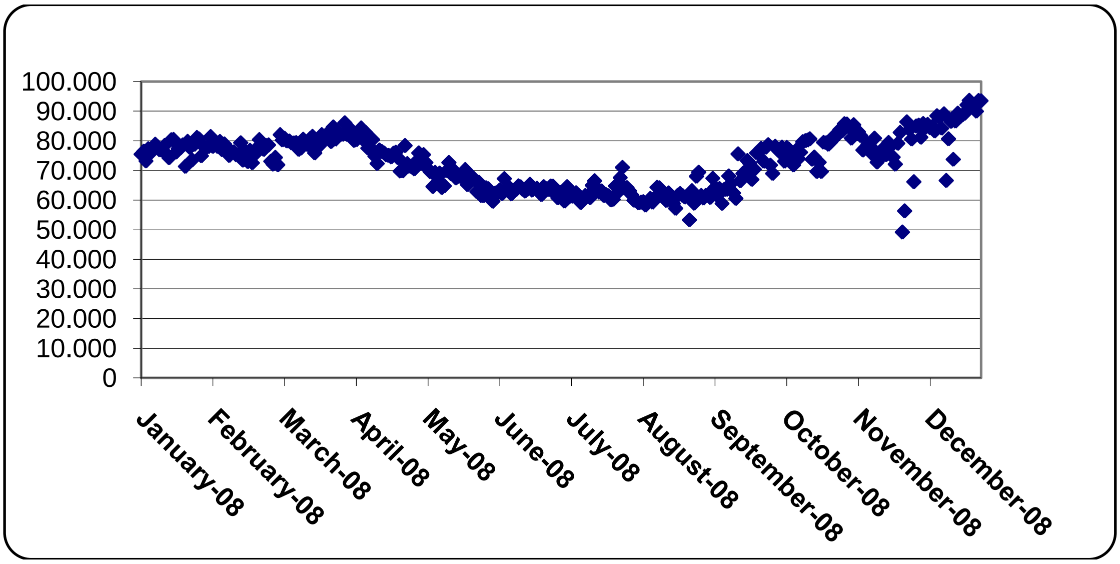


Figure 4.14. Flowrate change in 2008

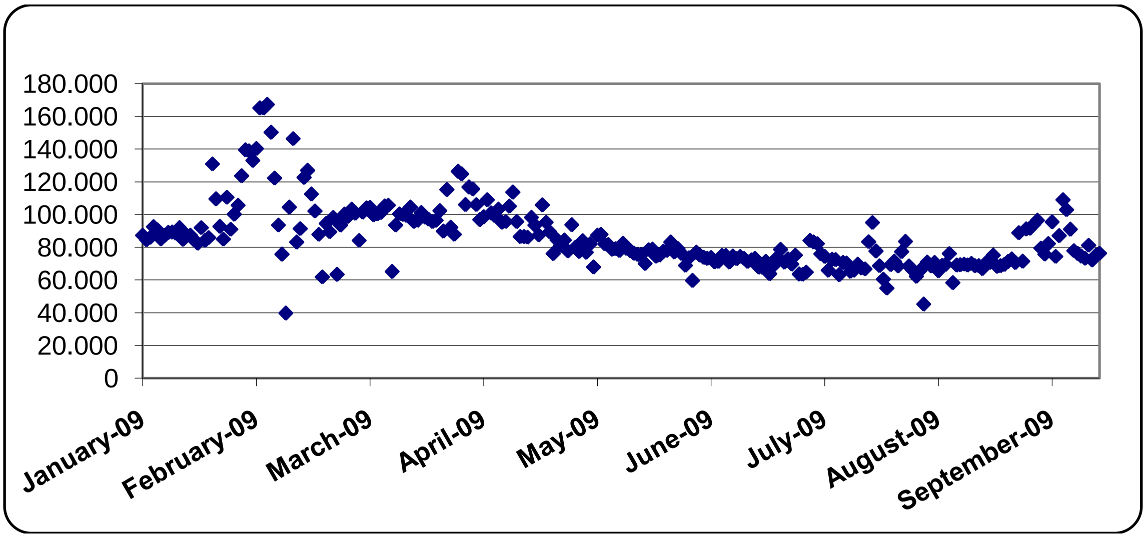


Figure 4.15. Flowrate change in 2009

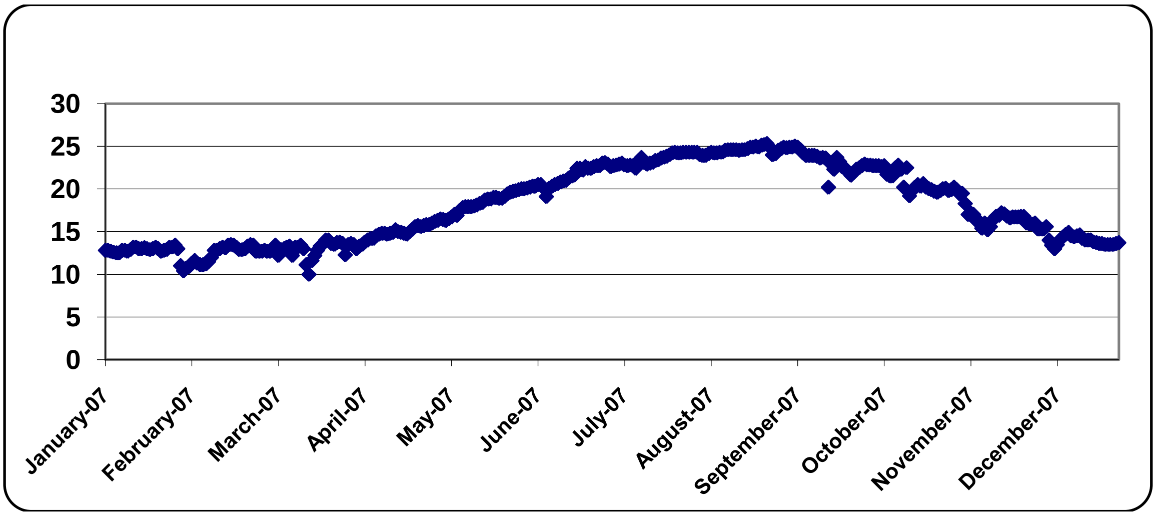


Figure 4.16. Temperature Change in 2007

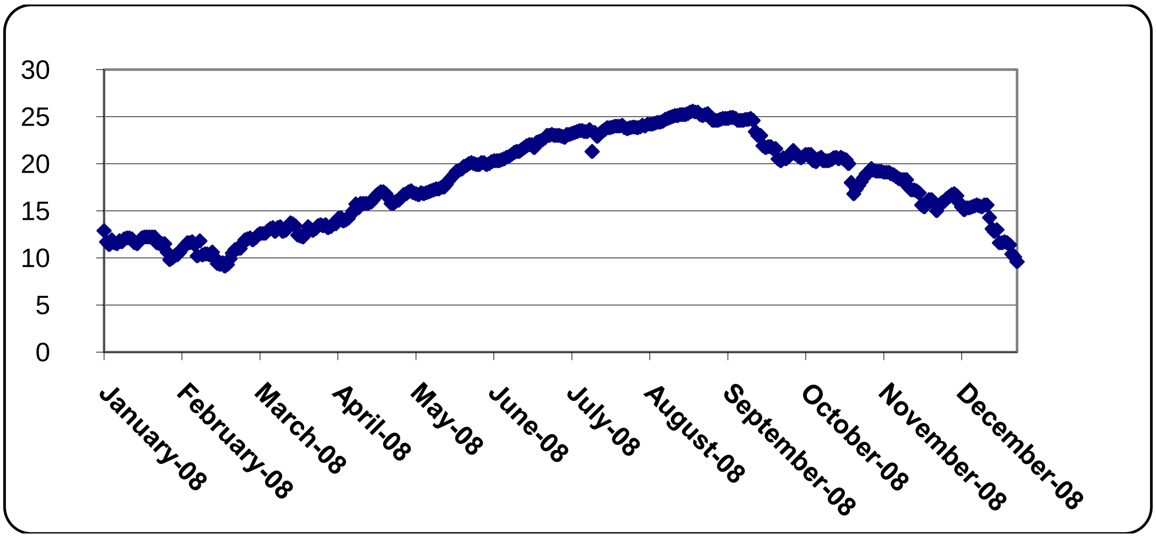


Figure 4.17. Temperature Change in 2008

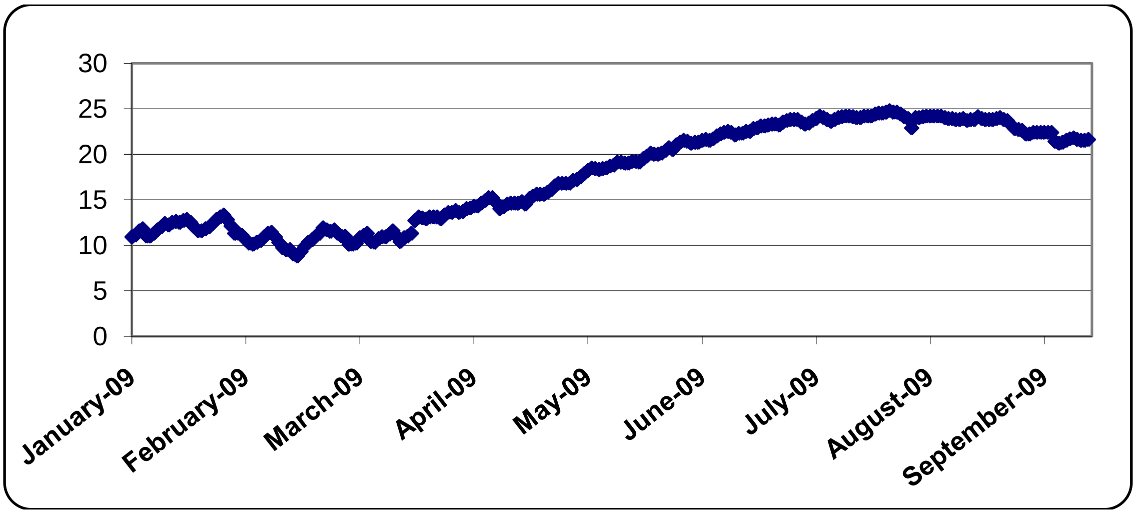


Figure 4.18. Temperature Change in 2009

4.6. Evaluation The Efficiency of The Paşaköy WWTP

After looking at the all concentrations of the parameters day by day, the effiency of all parameter is calculated due to monthly average concentrations which are shown below.

It can be seen that for BOD_5 effluent value is lower than 20 mg/l which is the effluent design parameter of Paşaköy WWTP. That indicates plant is adequate due to its construction aim.When we look at its efficiency, it is almost about %96.On the other hand if we compare the results with Water Pollution and Control Regulations parameters, it is also suitable.The value of BOD_5 at Paşaköy WWTP is lower than 35 mg/l.

Effluent of SS is also adeqaute both design criteria of Paşaköy WWTP,which is 30 mg/l and Water Pollution and Control Regulations parameters, which is 25 mg/l.Our effleunt parameter for SS is about 25 mg/l.Removed SS has a percantage of % 92 and it is efficient.

Total Nitrogen(TN) has a efficieny of %84 due to removing capacity,and its effluent value is adequate compared to Paşaköy WWTP design effluent concentration,which is 10 mg/l.Average monthly concentrations of TN is about 9 mg/l.

Total Phosphorus is removed with a efficiency of %52 percentage.It is almost good if it is thought that influent of phosphorus exceeds the design parameter of Paşaköy WWTP.

When we look at overall performance of Paşaköy WWTP is succesfull.The plant works well and with the second phase it will have beter results.

Table 4.1. Municipal Effleunt Wastewater Parameters, Due to Water Pollution and Control Regulations
(Population >100.000) (SKKY, 2004)

Parameter	Unit	Composit Sample for 24 hour
Biochemical Oxygen Demand (BOD_{5})	mg/L	35
Suspended Solids (SS)	mg/L	25

Table 4.2. Monthly Average Infleunt and Effluent Concentrations and Efficencies

Date	Flowrate	BOD_5(mg/L)			SS(mg/L)			TN(mg/L)			TP(mg/L)		
		Inf.	Effl.	Effic. (%)	Inf.	Effl.	Effic. (%)	Inf.	Effl.	Effic. (%)	Inf.	Effl.	Effic. (%)
January-07	73.759	284	11	0,96	314	25	0,92	61,0	11,4	0,81	4,7	1,7	0,64
February-07	72.755	297	11	0,96	319	26	0,92	63,4	10,6	0,83	6,8	2,5	0,63
March-07	70.421	304	15	0,95	350	35	0,90	60,7	8,8	0,86	7,3	2,3	0,68
April-07	66.316	346	21	0,94	336	36	0,89	68,2	11,1	0,84	7,3	2,9	0,60
May-07	62.969	354	17	0,95	382	33	0,91	73,3	9,5	0,87	9,3	5,1	0,45
June-07	58.561	346	16	0,95	390	31	0,92	74,9	6,9	0,91	10,2	6,4	0,37
July-07	54.333	348	14	0,96	372	30	0,92	79,6	9,2	0,88	9,1	6,3	0,31
August-07	54.126	338	12	0,96	397	32	0,92	71,0	7,5	0,90	8,2	3,6	0,56
September-07	58.314	327	11	0,97	364	35	0,90	68,7	8,0	0,88	7,2	4,0	0,44
October-07	65.749	296	11	0,96	363	31	0,91	64,1	6,5	0,90	8,1	4,4	0,46
November-07	73.777	236	8	0,97	322	21	0,93	54,3	7,8	0,86	6,5	3,4	0,48
December-07	77.503	238	9	0,96	283	25	0,91	62,1	12,8	0,79	6,6	3,3	0,50
January-08	77.139	270	11	0,96	369	29	0,92	55,0	10,0	0,82	7,0	4,0	0,43
February-08	76.821	226	11	0,95	148	12	0,92	41,2	15,9	0,61	6,0	3,0	0,50
March-08	80.056	225	10	0,96	243	24	0,90	51,0	11,0	0,78	7,0	3,0	0,57
April-08	76.835	357	39	0,89	321	74	0,77	65,0	15,8	0,76	9,9	3,9	0,61
May-08	68.232	322	17	0,95	353	29	0,92	78,5	9,2	0,88	11,5	5,0	0,57
June-08	63.455	333	9	0,97	382	19	0,95	74,7	8,0	0,89	11,2	5,8	0,48
July-08	62.675	307	9	0,97	391	22	0,94	66,8	8,3	0,88	8,3	5,0	0,40
August-08	61.011	370	10	0,97	424	29	0,93	73,0	7,1	0,90	9,3	4,1	0,56
September-08	67.627	289	11	0,96	342	31	0,91	57,0	5,8	0,90	9,0	4,7	0,48
October-08	76.755	237	8	0,97	276	21	0,92	55,0	8,7	0,84	6,1	3,1	0,49
November-08	77.958	236	8	0,97	244	26	0,89	54,0	7,4	0,86	6,1	3,0	0,51
December-08	85.582	202	7	0,97	224	22	0,90	51,0	9,8	0,81	5,3	2,8	0,47
January-09	88.286	156	10	0,94	201	27	0,87	35,0	9,9	0,72	4,5	2,3	0,49
February-09	115.678	132	8	0,94	173	19	0,89	31,0	8,4	0,73	3,5	2,2	0,37
March-09	95.171	120	7	0,94	173	19	0,89	29,0	7,7	0,73	3,4	1,9	0,44
April-09	100.904	242	11	0,95	243	17	0,93	55,0	10,0	0,82	5,1	2,3	0,55
May-09	81.523	316	7	0,98	302	14	0,95	66,2	10,2	0,85	7,4	2,6	0,65
June-09	73.539	338	8	0,98	325	10	0,97	73,0	8,2	0,89	9,3	3,8	0,59
July-09	72.179	333	6	0,98	327	10	0,97	71,0	7,0	0,90	8,9	3,6	0,60
August-09	67.865	372	10	0,97	335	12	0,96	69,0	7,0	0,90	9,3	3,7	0,60
September-09	81.118	312	8	0,97	334	11	0,97	62,0	6,6	0,89	7,5	2,9	0,61
Average	**73.909**	**285**	**12**	**0,96**	**313**	**25**	**0,92**	**61**	**9**	**0,84**	**7**	**4**	**0,52**

Table 4.3. Monthly Average Infleunt and Effluent Loads and Efficencies

Date	Flowrate	BOD_5(kg/day)			SS(kg/day)			TN(kg/day)			TP(kg/day)		
		Inf.	Effl.	Effic. (%)	Inf.	Effl.	Effic. (%)	Inf.	Effl.	Effic. (%)	Inf.	Effl.	Effic. (%)
January-07	73.759	20.938	840	0,96	23.160	1.844	0,92	4.499	841	0,81	347	125	0,64
February-07	72.755	21.608	800	0,96	23.209	1.892	0,92	4.613	771	0,83	495	182	0,63
March-07	70.421	21.408	1.056	0,95	24.647	2.465	0,90	4.275	620	0,86	514	162	0,68
April-07	66.316	22.945	1.393	0,94	22.282	2.387	0,89	4.523	736	0,84	484	192	0,60
May-07	62.969	22.291	1.070	0,95	24.054	2.078	0,91	4.616	598	0,87	586	321	0,45
June-07	58.561	20.262	937	0,95	22.839	1.815	0,92	4.386	404	0,91	597	375	0,37
July-07	54.333	18.908	761	0,96	20.212	1.630	0,92	4.325	500	0,88	494	342	0,31
August-07	54.126	18.295	650	0,96	21.488	1.732	0,92	3.843	403	0,90	441	195	0,56
September-07	58.314	19.069	641	0,97	21.226	2.041	0,90	4.006	467	0,88	420	233	0,44
October-07	65.749	19.462	723	0,96	23.867	2.038	0,91	4.215	427	0,90	533	289	0,46
November-07	73.777	17.411	590	0,97	23.756	1.549	0,93	4.006	575	0,86	480	251	0,48
December-07	77.503	18.446	698	0,96	21.933	1.938	0,91	4.813	992	0,79	512	256	0,50
January-08	77.139	20.828	849	0,96	28.464	2.237	0,92	4.243	771	0,82	540	309	0,43
February-08	76.821	17.362	845	0,95	11.370	922	0,92	3.165	1.221	0,61	461	230	0,50
March-08	80.056	18.013	801	0,96	19.454	1.921	0,90	4.083	881	0,78	560	240	0,57
April-08	76.835	27.430	2.997	0,89	24.664	5.686	0,77	4.994	1.214	0,76	761	300	0,61
May-08	68.232	21.971	1.160	0,95	24.086	1.979	0,92	5.356	628	0,88	785	341	0,57
June-08	63.455	21.152	550	0,97	24.265	1.225	0,95	4.740	508	0,89	711	368	0,48
July-08	62.675	19.241	564	0,97	24.506	1.379	0,94	4.184	518	0,88	520	313	0,40
August-08	61.011	22.594	628	0,97	25.869	1.769	0,93	4.454	433	0,90	567	250	0,56
September-08	67.627	19.544	744	0,96	23.128	2.096	0,91	3.855	392	0,90	609	318	0,48
October-08	76.755	18.191	614	0,97	21.184	1.612	0,92	4.222	668	0,84	468	238	0,49
November-08	77.958	18.398	624	0,97	19.022	2.027	0,89	4.210	577	0,86	476	234	0,51
December-08	85.582	17.288	599	0,97	19.170	1.883	0,90	4.365	839	0,81	454	240	0,47
January-09	88.286	13.773	883	0,94	17.745	2.384	0,87	3.090	874	0,72	397	203	0,49
February-09	115.678	15.269	925	0,94	20.012	2.198	0,89	3.586	972	0,73	405	254	0,37
March-09	95.171	11.421	666	0,94	16.465	1.808	0,89	2.760	733	0,73	324	181	0,44
April-09	100.904	24.419	1.110	0,95	24.520	1.715	0,93	5.550	1.009	0,82	515	232	0,55
May-09	81.523	25.761	571	0,98	24.620	1.141	0,95	5.397	832	0,85	603	212	0,65
June-09	73.539	24.856	588	0,98	23.900	735	0,97	5.368	603	0,89	684	279	0,59
July-09	72.179	24.036	433	0,98	23.603	722	0,97	5.125	505	0,90	642	260	0,60
August-09	67.865	25.246	679	0,97	22.735	814	0,96	4.683	475	0,90	631	251	0,60
September-09	81.118	25.309	649	0,97	27.093	892	0,97	5.029	535	0,89	608	235	0,61
Average	**73.909**	**20.398**	**837**	**0,96**	**22.380**	**1.835**	**0,92**	**4.381**	**682**	**0,84**	**534**	**255**	**0,52**

5. SIMULATION OF SINGLE-SLUDGE PROCESSES FOR CARBON OXIDATION NITRIFICATION AND DENITRIFICATION

5.1. SSSP Model

This part only explains the process of the model and not related to Paşaköy Wastewater Treatment Plant.

The first step in applying this model is to translate the reactor configuration of the wastewater treatment system you wish to simulate into one which casn be interpreted by the computer.For example,a real system such as shown in Figure 5.1 could be modeled as a chain of 3 completely mixed reactors with all feed and all sludge recycle to the first basin with sludge wastage from reactor as shown in Figure 5.2. (Bidstrup, et al., 1987)

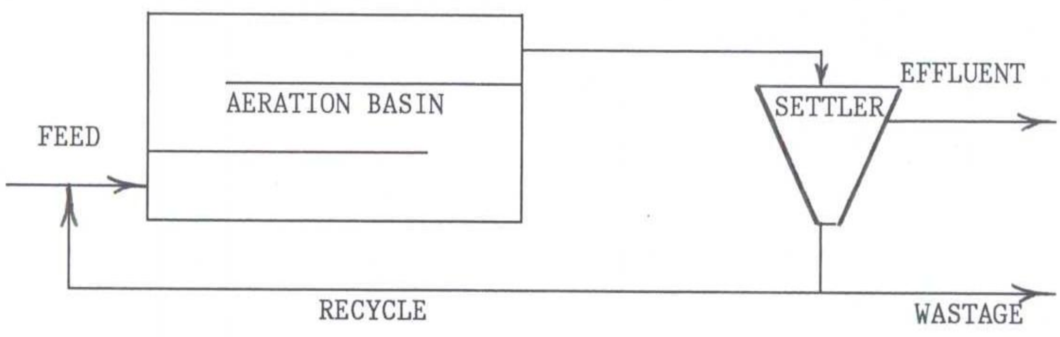


Figure 5.1. Typical Wastewater Treatment System (Bidstrup, et al., 1987)

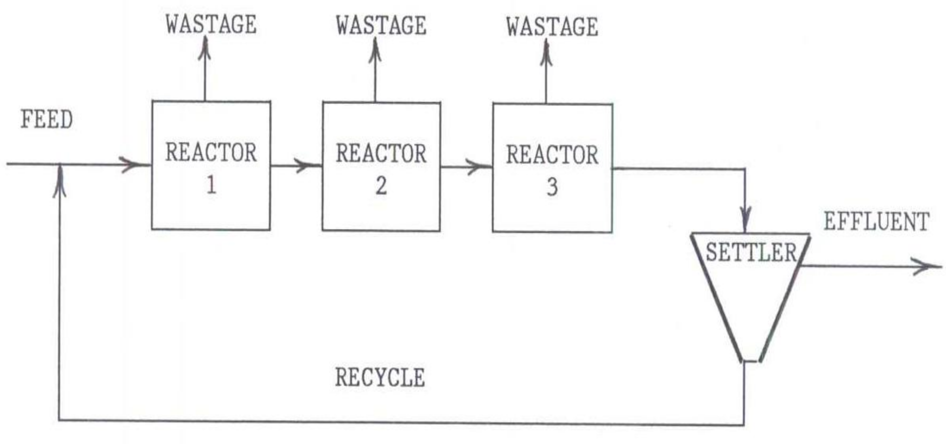


Figure 5.2. Typical Wastewater Treatment System (Bidstrup, et al., 1987)

Once the reactor configuration and the necessary system paramters have been defined and entered into the program, mass balance equations are generated for all relevant components. Then, the computer uses numerical subroutines to solve the mass balances(Bidstrup, et al., 1987).

A number of simplifying assumptions were made during the formulation of the mass balances. First, the sludge is wasted from each reactor in proportion to the reactor's volume. You only need to enter the desired SRT, from which the computer calculates the amount of sludge wasted from each reactor. This allows independent control of the SRT and the recycle flowrate, unlike many wastewater treatment plants which waste sludge from the settler underflow. This simplification has a negligible effect on the solids concentrations in the basins (Bidstrup, et al., 1987).

Secondly, SSSP does not calculate a true dynamic solution. To do so, the program would have to be modified to account for the dynamic behavior of the solids within the settler. However, it does compute reasonable values for the time variation in the rate of oxygen uptake and the various concentrations şn the aeration basins for a system in which the clarifier is not overloaded (Bidstrup, et al., 1987).

Third, it was assumed that no reactions occur in the settling basin. Consequently, the concentrations of soluble components in the clarifier effluent and underflow streams are the same as those in the last aeration basin. One expection to this is the concentration of dissolved oxygen in the clarifier, which is zero. Therefore, its concentration in the return sludge line is also zero (Bidstrup, et al., 1987).

A few of the finer points in the application of the model to real system can best be explained by examining a system designed to perform simultaneous carbon oxidation, nitrifacation and denitrification in a single sludge. Figure 5.3. is a typical example of one of these systems (Bidstrup, et al., 1987).

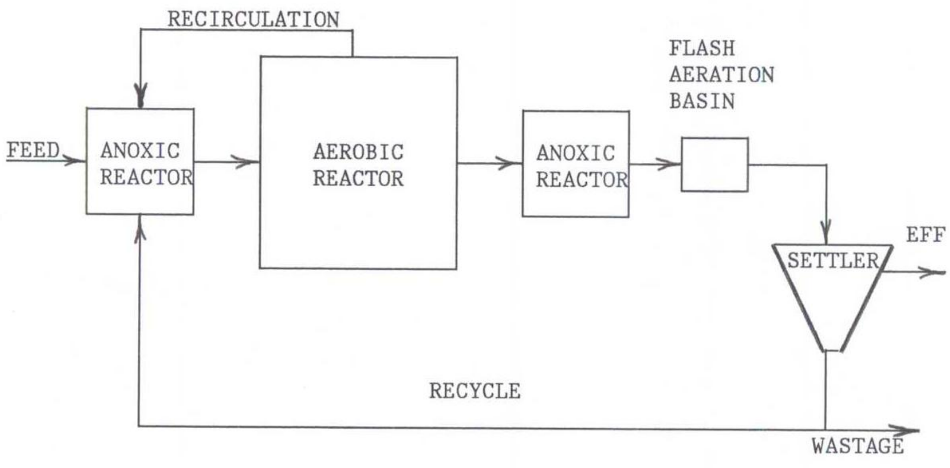


Figure 5.3. Bardenpho Process (SSSP, 1987)

Note that in Figure 5.3. there are two streams which flow back to the firs reactor. The recycle stream carries the settled MLSS from the underflow of the secondary clarifier. Its purpose is to concentrate the biomass in the system so that effective biological treatment can occur within a reasonable hydraulic retention time. The recirculation stream carries unsettled MLSS from the second reactor (the first aerobic basin), back to the first anoxic reactor. Its purpose to carry nitrate formed in the aerbic reactor back to the first anoxic reactor so that denitrification can occur. It is important that you recognize the distinction made between the terms recycle and recirculation because they are used in the foregoing manner throughout this manual and for data entry in the program (Bidstrup, et al., 1987).

In single sludge systems undergoing carbon oxidation, nitrification and denitrification there are two types of reactors: anoxic and aerobic. Anoxic basin refer to those which are not aerated but which contain nitrate nitrogen; aerobic basins are ones which are aerated. In the anoxic zones the heterotrophic biomass oxidizes organic material as it reduces nitrate nitrogen to nitrogen gas, yielding energy and carbon for cell synthesis. In the aerobic zones the autotrophic biomass oxidizes amonia nitrogen to nitrate nitrogen for energy and uses carbon dioxide as its source of carbon for synthesis, while the heterotrophic biomass oxidizes organic material for energy and cell synthesis (Bidstrup, et al., 1987).

The program may be used in two different ways with respect to the specification of the dissolved oxygen(DO) concentrations in a reactor. In the first method the DO concentarion is fixed at a constant value despite variations in oxygen uptake rate. In the second, the mass transfer coefficient for oxygen is constant, thereby making the DO concentration a variable which depends on the relative rates of utilization and transfer.Each method has advantages under certain cirumstances and eache has distinct charecteristics which should be recognized (Bidstrup, et al., 1987).

First consider the case of constant DO concentration. The major advantage associated with this method is that the oxygen mass balance equation is eliminated from the system of equations describing the process. This reduces the required computation time for the time-variant solution. Furthermore, you can stil query the program for oxygen consumption rate. However, one disadvantage of this method is that the extent of denitrification may be over-estimated in some situations. For example, referring to Figure 5.3., if recirculation flow is large with respect to the influent flow, and DO concentration in the aerobic reactor is high, then an appreciable mass of oxygen will enter the first anoxic reactor. Its effect should be to reduce the amount of denitrification occcuring in the basin. However, this effect will not be evident in the solution because the DO concentration in the anoxic reactor was specified as zero and the mass balance for oxygen was not solved. Nevertheless, as lon as this limitation is recognized, situations leading to appreciable error can be avoided (Bidstrup, et al., 1987).

Specification of the required oxygen mass transfer coefficient eliminates the error mentioned above but increases the time for the computer to solve the mass balance equations for time-variant inputs. Furthermore, in order to simplifty the solution of the steady-state mass balance equations, it was necessary to set the oxygen half-saturation coefficients for the autotrophic and the heterotrophic biomass equal to their average. Thus the two parameters are equal, which is not necessarily true. The value of autotrophs is generally larger. This coul lead to overestimation of the degree of nitrfication during low DO conditions. Again, recognition of this limitation can minimize its impact (Bidstrup, et al., 1987).

One will obtain the most realitsic solutions by utlizing time-variant inputs with specification of the oxygen mass transfer coefficients. In this case it was not necessary to average the values of the oxygen half-saturation coefficeints. Consequently, the limitaions mentioned in the preceding two paragraphs are eliminated. However, since these require the most time to compute and because setting the DO concentration at a fixed value will be adequate for the majority of cases, it is recommended that the fixed DO condition be used for routine work. Then after the process configuration has been fine-tuned for a given input situation, you can query the program to determine the mass transfer coefficient required to maintain the desired DO level in the reactors and use those values as input to the other solution technique, thereby giving more exact results (Bidstrup, et al., 1987).

The computer solves the material balances for the most important constituents. These constituents and the processes acting upon them are shown in Table 5.1. The observed reaction rate for any constituent may be determined by moving down the column representing it and summing the products of the stoichiometric coeeficients in the column times the process rate experssions on the right side of the rows containing coeffcients. For example, if you examine the column for constituent number 9, soluble nitrate and nitrite nitrogen, it is evident that two processes affect their fate; the anoxic growth of heterotrophs and the aerobic growth of autotrophs. Following the method described above, the equation descrbing the observed rate of reaction of the nitrate and nitrite is:

$$r_{S_{NO}} = -\left[\frac{1-Y_H}{2.86Y}\right] \hat{\mu}_H \left[\frac{S_S}{K_S+S_S}\right] \left[\frac{K_{O,H}}{K_{O,H}+S_O}\right] \left[\frac{S_{NO}}{K_{NO}+S_{NO}}\right] \eta_g X_{B,H}$$

$$+ \left[\frac{1}{Y_A}\right] \hat{\mu}_A \left[\frac{S_{NH}}{K_{NH}+S_{NH}}\right] \left[\frac{S_O}{K_{O,A}+S_O}\right] X_{B,A} .$$

Table 5.1. Process Kinetics and Stoichiometry for Carbon Oxidation,Nitrification and Denitrification

Table 5. Process Kinetics and Stoichiometry for Carbon Oxidation, Nitrification, and Denitrification

j	Component → i / Process ↓	1 S_I	2 S_S	3 X_I	4 X_S	5 $X_{B,H}$	6 $X_{B,A}$	7 X_P	8 S_O	9 S_{NO}	10 S_{NH}	11 S_{ND}	12 X_{ND}	13 S_{ALK}	Process Rate, ρ_j, $ML^{-3}T^{-1}$
1	Aerobic growth of heterotrophs		$-\frac{1}{Y_H}$			1			$-\frac{1-Y_H}{Y_H}$		$-i_{XB}$			$\frac{-i_{XB}}{14}$	$\hat{\mu}_H\left(\frac{S_S}{K_S+S_S}\right)\left(\frac{S_O}{K_{O,H}+S_O}\right)X_{B,H}$
2	Anoxic growth of heterotrophs		$-\frac{1}{Y_H}$			1				$-\frac{1-Y_H}{2.86Y_H}$	$-i_{XB}$			$\frac{1-Y_H}{14\cdot 2.86Y_H}-i_{XB}/14$	$\hat{\mu}_H\left(\frac{S_S}{K_S+S_S}\right)\left(\frac{K_{O,H}}{K_{O,H}+S_O}\right)\left(\frac{S_{NO}}{K_{NO}+S_{NO}}\right)\eta_g X_{B,H}$
3	Aerobic growth of autotrophs						1		$-\frac{4.57-Y_A}{Y_A}$	$\frac{1}{Y_A}$	$-i_{XB}-\frac{1}{Y_A}$			$-\frac{i_{XB}}{14}-\frac{1}{7Y_A}$	$\hat{\mu}_A\left(\frac{S_{NH}}{K_{NH}+S_{NH}}\right)\left(\frac{S_O}{K_{O,A}+S_O}\right)X_{B,A}$
4	"Decay" of heterotrophs				$1-f_P$	-1		f_P					$i_{XB}-f_P i_{XP}$		$b_H X_{B,H}$
5	"Decay" of autotrophs				$1-f_P$		-1	f_P					$i_{XB}-f_P i_{XP}$		$b_A X_{B,A}$
6	Ammonification of soluble organic nitrogen										1	-1		$\frac{1}{14}$	$k_a S_{ND} X_{B,H}$
7	"Hydrolysis" of entrapped organics		1		-1										$k_h \frac{X_S/X_{B,H}}{K_X+(X_S/X_{B,H})}\left[\left(\frac{S_O}{K_{O,H}+S_O}\right)+\eta_h\left(\frac{K_{O,H}}{K_{O,H}+S_O}\right)\left(\frac{S_{NO}}{K_{NO}+S_{NO}}\right)\right]X_{B,H}$
8	"Hydrolysis" of entrapped organic nitrogen											1	-1		$\rho_7(X_{ND}/X_S)$
	Observed Conversion Rates, $ML^{-3}T^{-1}$	$r_i = \sum_j \nu_{ij}\rho_j$													
	Stoichiometric Parameters: Heterotrophic yield: Y_H; Autotrophic yield: Y_A; Fraction of biomass yielding particulate products: f_P; Mass N/Mass COD in biomass: i_{XB}; Mass N/Mass COD in products from biomass: i_{XP}	Soluble inert organic matter - $M(COD)L^{-3}$	Readily biodegradable substrate - $M(COD)L^{-3}$	Particulate inert organic matter - $M(COD)L^{-3}$	Slowly biodegradable substrate - $M(COD)L^{-3}$	Active heterotrophic biomass - $M(COD)L^{-3}$	Active autotrophic biomass - $M(COD)L^{-3}$	Particulate products arising from biomass decay $M(COD)L^{-3}$	Oxygen (negative COD) $M(-COD)L^{-3}$	Nitrate and nitrite nitrogen - $M(N)L^{-3}$	$NH_4^+ + NH_3$ nitrogen - $M(N)L^{-3}$	Soluble biodegradable organic nitrogen - $M(N)L^{-3}$	Particulate biodegradable organic nitrogen - $M(N)L^{-3}$	Alkalinity - Molar units	Kinetic Parameters: Heterotrophic growth and decay: $\hat{\mu}_H, K_S, K_{O,H}, K_{NO}, b_H$; Autotrophic growth and decay: $\hat{\mu}_A, K_{NH}, K_{O,A}, b_A$; Correction factor for anoxic growth of heterotrophs: η_g; Ammonification: k_a; Hydrolysis: k_h, K_X; Correction factor for anoxic hydrolysis: η_h

A brief description of all the components of relevance as follows:

- Readily biodegradable COD: Soluble organic substrate which can be used directly for maintaning life functions of the heterotrophic biomass and for synthesis of new biomas.

- Slowly biodegradable COD: Particulate and high molecular weight organic material which is hydrolyzed extracellularly into readily biodegradable COD.The rate of hydrolysis is lower than the rate of oxygen uptake of readily biodegradable substrate.It will be referred to as particulate organic material throuhout the manual.

- Heterotrophic biomass: Biomass which uses readily biodegradable substrate as both its source of carbon for synthesis and its source of energy for maintining life functions.It grows under aerobic and anoxic conditions,but not under anaerobic conditions (neither oxygen or nitrate present).Decay results in the conversion of biomass into slowly biodegradable substrate and into particulate products which are not biodegradable.Heterotrophs take up ammonia for cell synthesis under aerobic and anoxic conditions,and convert nitrate nitrogen to gaseous N_2 under anoxic conditions.

- Autotrophic biomass: Biomass which uses carbon dioxide as its carbon source for synthesis, and converts ammonia into nitrate for energy.It grows only under aerobic conditions.Decay results in the conversion of biomass into slowly biodegradable substrate and particulate products which are not bidegradable.Autotrophs also take up ammonia nitrogen for cell synthesis during aerobic growth.This biomass includes both Nitrobacter and Nitrosomonas bacteri which are grouped together for the purposes of simulation.

- Particulate products: Particulate organic matter arising from the decay of autotrophs and heterotrophs.Its rate of degradation is so low that for all practical purposes it is inert.

- Soluble nitrate nitrogen: It serves as the terminal electron acceptor of the heterotrophic biomass when oxygen is not present.It is produced by the converison of ammonia during aerobic growth of autotrophs and it is removed as gaseous nitrogen during anoxic growth of heterotrophs.

- Soluble ammonia nitrogen: Ammonia is formed by the ammonification of soluble biodegradable organic nitrogen.It is used for cell synthesis of both the heterotrophic and the autotrophic biomass.It is also converted into nitrate while serving as the energy source for the aerobic growth of autotrophs.

- Soluble organic nitrogen: It is formed by the hydrolysis of particulate organic nitrogen and it is transformed into ammonia nitrogen by ammonification.

- Particulate biodegradable organic nitrogen: It is formed by the decay of the autotrophic and heterotrophic biomass.Some of the particulate organic nitrogen which is formed decay becomes part of the inert particulate products and so becomes unavailable to the biomass.The rest is slowly converted into soluble organic nitrogen by hydrolysis.

- Dissolved oxygen: It serves as the primary electron acceptor during aerobic growth of the biomass.It is transferred into solution by mass diffusion.The dissolved oxygen is utilized during oxidation of soluble substrate by the heretrophic biomass and during oxidation of ammonia nitrogen to nitrate nitrogen by the autotrophic biomass.

- Bicarbonate Alkalinity: Significant changes in the alkalinity result changes in pH.Altough the effects of changes in pH during the treatment process are not included in the model,it is desirable to include alkalinity in the model so that one can predict deleterious changes in the pH and the effect of low alkalinity discharges on receiving waters.Alkalinity is consumed with the conversion of ammonia nitrogen to amino acids during synthesis of heterotrophic and autotrophic biomass and is produced by the reverse of the process during ammonification.However the most significant change in alkalinity during the treatment process is the loss associated with the oxidation of ammonia nitrogen to nitrate nitrogen during nitrification.One of the benefits associated with the denitrification process is the restoration of some of the alkalinity consumed during the nitrification process.

- Inert particulate organic material. : Inert suspended organic material present in the influent which,like all particulate material,is concentrated in the reactor due to the hydraulics associated with the recycle stream.It is removed only by sludge wastage.

The model doesn’t contain a term for inert organic particulates. If you desire to determine their concentration in each reactor simply multiply their concentration in the feed by the ratio of the concentration of inert particulate organic matter in each reactor to its concentration in the feed. Although shown in Table 5.1., SSSP doesn’t calculate the concentration of soluble inert organic matter (nonbiodegradable soluble COD). Such material will pass through the system unchanged and consequently its concentration in each reactor will be equal to it concentration in the feed.

In the model all organic components are in units of grams of COD per cubic meter and nitrogen containing components are in units of grams of nitrogen per cubic meter. Note that one gram per cubic meter is equivalent to one miligramper liter. The fact that COD units are are used for all organic materials simplifies the computation of a COD balance and the specification of all parameters. It requires, however, that you multiply by an appropriate factor to convert all solids concentrations to a mass basis. For example, biomass is normally considered to have 1.41 g COD/g volatile solids.

5.2. Using SSSP

5.2.1 COD Fraction

The concentration of all componentsin the feed stream are needed. Because of the fact that all organic components are in units of grams of COD per cubic meter, some tests are done to get the values of components for Paşaköy Wastewater Treatment Plant.

Procedure is made in Pasaköy Wastewater Treatment Plant, on 23.05.2009.

- COD is passed through 0.1 μ filter paper and filtered and not filtered COD is measured.

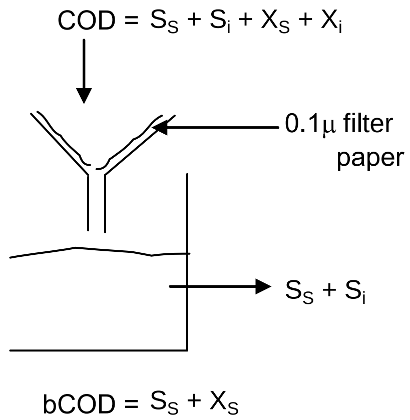


Figure 5.4. COD Fraction

COD =	437	mg/L
Filtered COD =	119	mg/L

- Effluent COD is filtered and the filtered part is called as S_i which is soluble inert organic matter.

Si=	35,5	mg/L

- Ultimate BOD is calculated by the help of Thomas Method.

Table 5.2. Calculation of BOD ultimate by Thomas Method (Metcalf, et. al, 1972)
.

Ultimate BOD (Thomas Method)					
t	**y(BODt)**	**(t/y)^(1/3)**	**t*(t/y)^(1/3)**	**t2**	**[(t/y)^(1/3)]^2**
1	100	0,2154	0,2154	1	0,04642
2	170	0,2274	0,4549	4	0,05173
3	230	0,2354	0,7062	9	0,05541
4	250	0,2520	1,0079	16	0,06350
5	270	0,2646	1,3228	25	0,07000
15	1.020	1,1948	3,7073	55	0,28705

- By the help of Thomas Method, ultimate BOD_5 is found as 332,2 mg/L

- Biodegradable COD is calculated.

$$bCOD = BOD_{ultimate}/(1\text{-}fBOD)$$

fBOD=	0,15	
bCOD=	390,77	mg/L

- Soluble organics are calculated.

$$S_s = \text{Influent filtered COD} - \text{Effluent filtered COD}$$

S_s =	83,5	mg/L

- Particulate organics are calculated.

$$X_s = bCOD - S_s$$

X_s =	307,27	mg/L

- Inert particulates are calculated.

$$X_i = \text{Influent COD} - X_s - S_s - S_i$$

X_i =	10,73	mg/L

5.2.2. Entering the Concentrations of Components in SSSP

After calculation concentration of compnents are entered as follows.

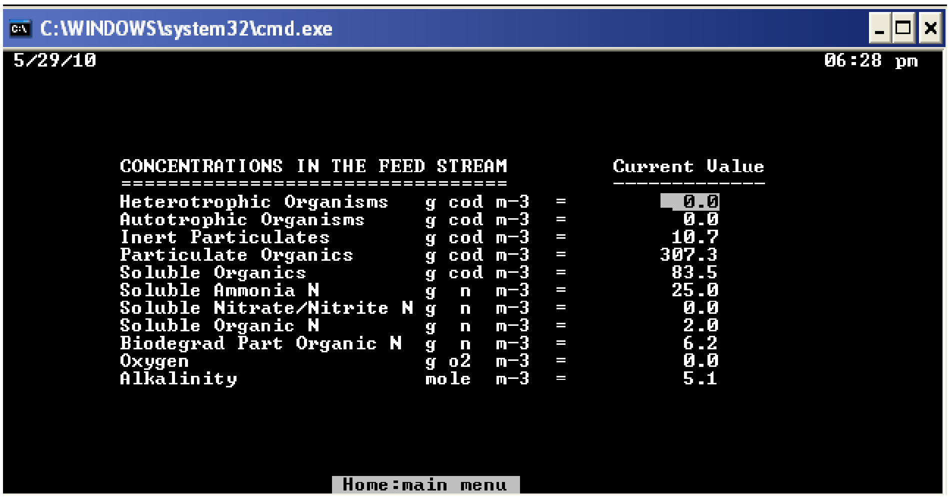


Figure 5.5. Concentrations in The Feed Stream

5.2.3. Entering the Process Flow Scheme

Firstly number of reactors which are 4 pieces are entered.Then the solid retention time(SRT) is put as 18 days.After that average flowrate is entered as 100.000 m^3/day.

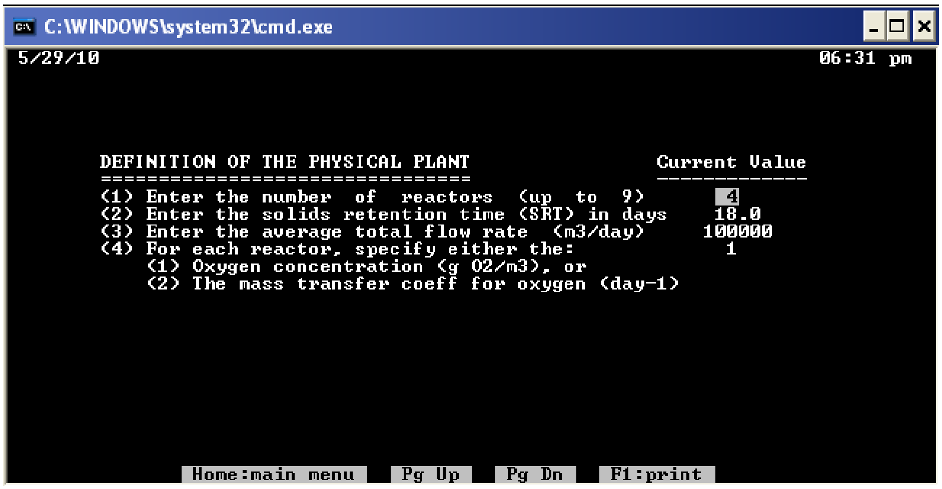


Figure 5.6. Definiton of The Physical Plant

5.2.4. Arrangement of Reactors

In process flow scheme 4 reactors are entered and it is time to arrenge the reactors. Reactor volumes are 10.000 m³ and all of them are same. The process works in series,so the fraction of the total flowrate is entered as 1 in first reactor and zero for the rest.Paşaköy WWTP works with the ratio of % 50 in terms of oxic and anoxic state. First and second reactor are chosen as anoxic and third and forth reactor are oxic due to working principle of Paşaköy WWTP. Recycle input is 100.000 m³/day for the first reactor. Wastewater recirculates from forth reactor to first reactor. Also recirculation input is entered as 400.000 m³/day for the first reactor. After entering the inputs program runs with no fatal errors.

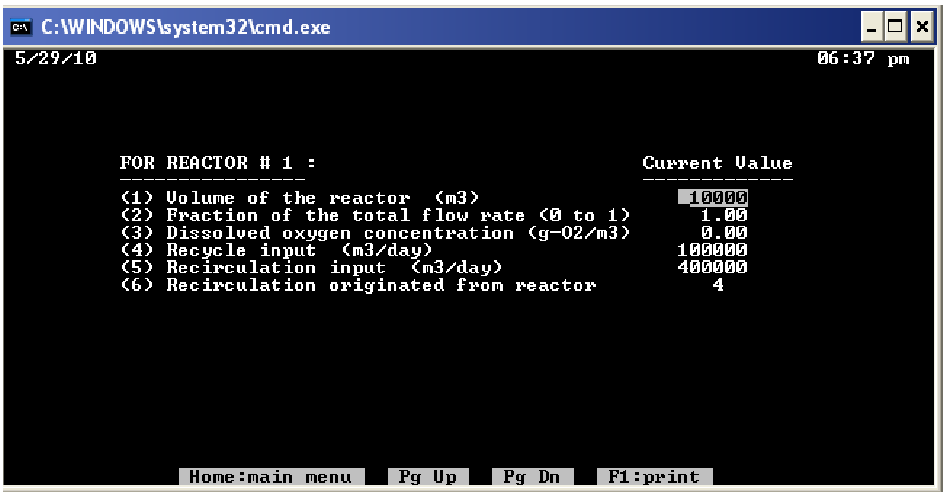


Figure 5.7. Arrangement of Reactor 1

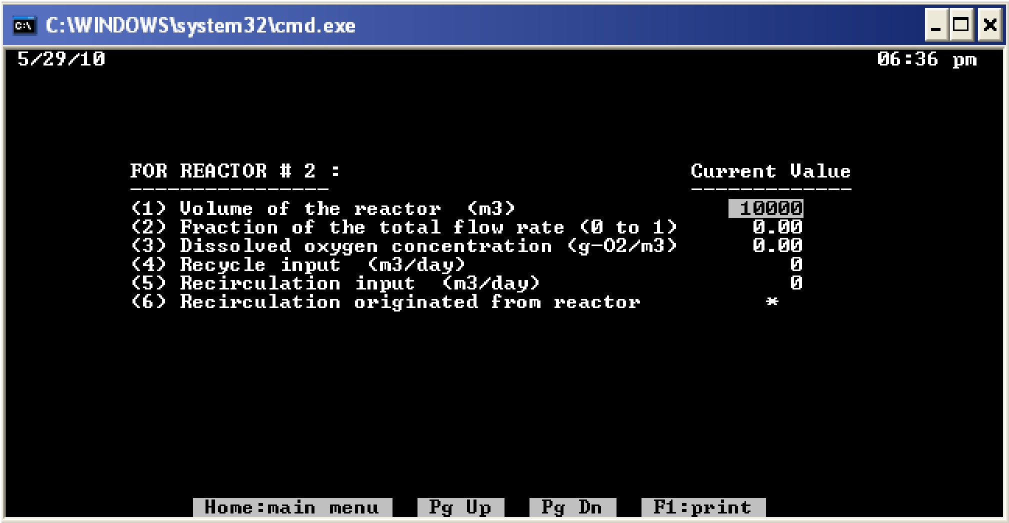


Figure 5.8. Arrangement of Reactor 2

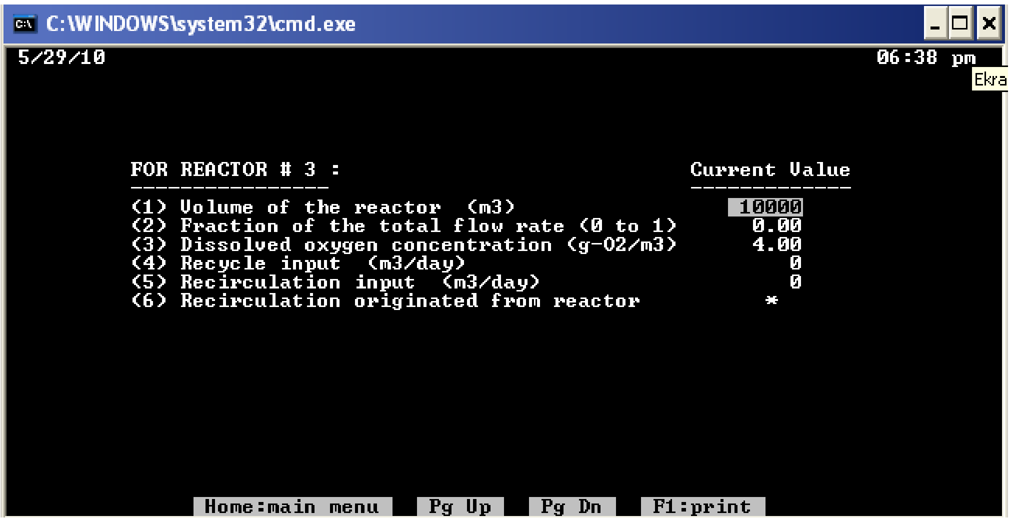


Figure 5.9. Arrangement of Reactor 3

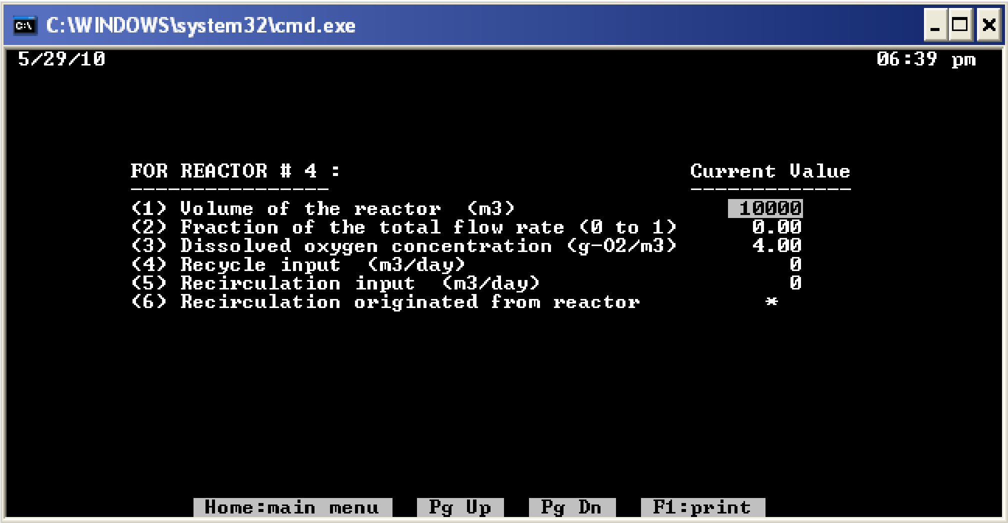
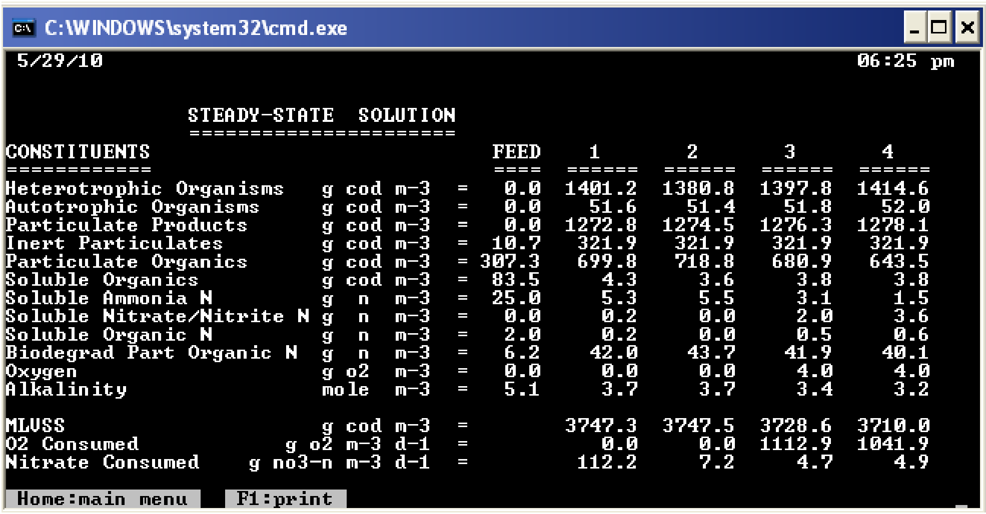


Figure 5.10. Arrangement of Reactor 4

5.2.5. Steady-State Solutions

After entering all the parameters in the feed stream and arrangement of all reactors according to Paşaköy WWTP, steady-state solutions are get.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STEADY-STATE SOLUTION

CONSTITUENTS			FEED	1	2	3	4
Heterotrophic Organisms	g cod m-3	=	0.0	1401.2	1380.8	1397.8	1414.6
Autotrophic Organisms	g cod m-3	=	0.0	51.6	51.4	51.8	52.0
Particulate Products	g cod m-3	=	0.0	1272.8	1274.5	1276.3	1278.1
Inert Particulates	g cod m-3	=	10.7	321.9	321.9	321.9	321.9
Particulate Organics	g cod m-3	=	307.3	699.8	718.8	680.9	643.5
Soluble Organics	g cod m-3	=	83.5	4.3	3.6	3.8	3.8
Soluble Ammonia N	g n m-3	=	25.0	5.3	5.5	3.1	1.5
Soluble Nitrate/Nitrite N	g n m-3	=	0.0	0.2	0.0	2.0	3.6
Soluble Organic N	g n m-3	=	2.0	0.2	0.0	0.5	0.6
Biodegrad Part Organic N	g n m-3	=	6.2	42.0	43.7	41.9	40.1
Oxygen	g o2 m-3	=	0.0	0.0	0.0	4.0	4.0
Alkalinity	mole m-3	=	5.1	3.7	3.7	3.4	3.2
MLVSS	g cod m-3	=		3747.3	3747.5	3728.6	3710.0
O2 Consumed	g o2 m-3 d-1	=		0.0	0.0	1112.9	1041.9
Nitrate Consumed	g no3-n m-3 d-1	=		112.2	7.2	4.7	4.9

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Figure 5.11. Steady-State Solutions

After getting steady-state results, modeled and actual MLVSS values it is found that the results are so close to each other. If we take the first data, modeled and actual MLVSS values are 3710 g COD/m^3 and 3619 g COD/m^3 respectively. Soluble ammonia is 1,5 g/m^3 in the model and it is 0,5 g/m^3 in actual. The proposed model was iterated due to different influent and effluent values and the high correlation with the actual values of laboratory analysis is found. Similarity of the results of modeled and actual lets us to forecast the performance of the plant in the future under different influent loads. Besides, it is also possible to use this model in order to choose the best mode to reach the efficient results without not consuming much time. Also the process parameters can be changed due to results of model.For instance, if the amount of nitrite and nitrate is high in the model, oxic and anoxic ratio can be changed to reach the optimal effluent parameters.

Table 5.3. Comparision of Modeled Effluent and Actual Effluent

	MLVSS, g COD/m^3		**NH_4-N ,g/m^3**	
Date	**MODEL**	**ACTUAL**	**MODEL**	**ACTUAL**
23.05.2009	3710	3619	1,5	0,5
11.08.2009	3850	3776	1,3	0,7
12.09.2009	3833	3752	1,2	0,8

6. LOAD REMOVAL BY PAŞAKÖY WWTP

Paşaköy is the most polluted creek that feeds Ömerli Dam (Akkoyunlu, 2002). In order to protect Ömerli dam, Paşaköy WWTP is constructed in 2000. The plant collects the Paşaköy creek which is discharged to Ömerli Dam, and give the flow through Paşaköy Tunnel to the Riva Stream, and it joins to the Black Sea. Therefore Paşaköy creek is prevented to reach Ömerli Dam, also nutrient loading which are caused by this creek is not discharged to Ömerli Dam. To evaluate how much load is prevented after Paşaköy WWTP, the influent parameters of the plant are taken, because of the fact that if the plant did not exist, that load would be discharged to Ömerli Dam.

We can start with Total Nitrogen which has a major role causing eutrophication (Ceyhan, 1999). When we look at the datas between January 2007 and October 2009 the average influent concentration of TN is 61 mg/l. The average flowrate between those dates is 73.909 m^3/day (Table 4.2.). To evaluate the total load, average concentration and average flowrate are multiplied.

Load of nitrogen removed 61 mg/l x 73.909 m^3/day x 10^{-3} = 4.508 kg/day

To evaluate annual load removal

4.508 kg/day x 365 x 10^{-3} =1.646 tons/yr

Another element which leads to eutrophication highly is phosphorus.The average of effluent value of phosphorus is 7 mg/l between January 2007 and October 2009 (Table 4.2.). By the Paşaköy WWTP this eutrophication leading nutrient is prevented to reach Ömerli Dam.The average flowrate is 73.909 m^3/day.

Load of phosphorus removed daily 7 mg/l x 73.909 m^3/day x 10^{-3} = 517 kg/day

To evaluate annual load removal

517 kg/day x 365 x 10^{-3} =189 tons/yr

After calculations of load values it can be said that large amount of nutrients caused by Paşaköy creek is removed by Paşaköy WWTP. Thus, it improves the water quality of the dam. It reveals that construction of Paşaköy WWTP plays key role protecting the dam and its effect for reducing the nutrient load.

7. CONCLUSIONS

Omerli Reservoir is one of the major drinking water reservoirs of Greater Metropolis Istanbul, providing 40% of the overall water demand. İstanbul where is one of the greatest metropolitan areas of the world with a population over 10 million and a rate of population increase about twice that of Turkey. As a result of population growth and industrial development, Omerli watershed is highly affected by the wastewater discharges from the residential areas and industrial plants (Coşkun, et al., 2008).

Paşaköy WWTP is a very critical plant protecting Ömerli Dam against pollution.Its 1st phase commisioned in 2000 and the second phase was initiated in 2007 due to increased pollution load of wastewater collection area of the plant and construction completed in 2009.The plant has advanced wastewater treatment plant standards.The second phase added new units such as, sludge drying,cogeneration,biofilter,sand filitration and UV disinfection.In the presence of sludge drying unit the sludge is turned in to a dry product 90% solids content which can be used as fertilizer or fuel.

The influent and effulent data investigated and ploted showed the plant performance for the criteria of BOD_5, SS, TN and TP is adequate. Removing load capacity of the plant is in high levels. After analysis and evalutions we can say that Paşaköy WWTP is sufficient and the quality of treated water which is discharged to Riva has the requirement of both the plant design criteria and Water Pollution and Control Regulations.

The proposed model was iterated for many different effulent and influent values and found to be highly correlated with the actual values of the laboratory analysis. The high correlation between the actual measured and predicted values by the model showed that this model is a suitable tool in foreseeing plant's performance under varying influent loads. This also makes it possible to use this model in order to assist the plant operator to choose between different operational modes without the need for time consuming laboratory analysis and easily predict the effulent parameters at an early stage. Thus the operator can change the process parameters according to this calibrated model and ensure higher plant performance.

After construction of Paşaköy WWTP, the most polluted creek which is Paşaköy is discharged through a tunnel to Riva. Thus, the paşaköy creek doesn’t reach to the dam and the nutrients are also prevented to flow in to the dam. Therefore high amounts of N and P loads are removed away, and the quality of Ömerli is in the first degree in terms of drinking water qualities after the construction of the plant.

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