DOKUZ EYLÜL UNIVERSITY
GRADUATE SCHOOL OF NATURAL AND APPLIED SCIENCES

COMMISSIONING OF WASTEWATER TREATMENT PLANTS AND EVALUATION OF DIFFERENT STAGES OF OPTIONS

by
Erol BÖLEK

September, 2008
İZMİR

COMMISSIONING OF WASTEWATER TREATMENT PLANTS AND EVALUATION OF DIFFERENT STAGES OF OPTIONS

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M.Sc THESIS EXAMINATION RESULT FORM

We have read the thesis entitled **"COMMISSIONING OF WASTEWATER TREATMENT PLANTS AND EVALUATION OF DIFFERENT STAGES OF OPTIONS"** completed by **EROL BÖLEK** under supervision of **PROF. DR. AYŞE FİLİBELİ** and we certify that in our opinion it is fully adequate, in scope and in quality, as a thesis for the degree of Master of Science.

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COMMISSIONING OF WASTEWATER TREATMENT PLANTS AND EVALUATION OF DIFFERENT STAGES OF OPTIONS

ABSTRACT

Generally the wastewater treatment plants are designed for 20-25 years future expansion. Although designed values; when they are commissioning and at least first few years' incoming wastewater and pollution loads seem less than designed values. While first times and also according to the changes of flow and its characteristic or unexpected situations; the treatment plant needs to be run with different treatment options. According to these situations, the operator should find the optimum way and applied on plant.

In this study commissioning - in other words start-up – period is examined with the help of experiences of Piatra Wastewater Treatment Plant commissioning period. Actions before and during commissioning are examined with examples to give the view points to the engineers. Shatat Wastewater Treatment Plant is analyzed with different treatment options. These are carbon removal, carbon removal with nitrification, carbon removal with nitrification plus denitrification and extended aeration. The oxygen consumptions, sludge productions, necessary adjustments over the process and treatment efficiencies of the wastewater treatment plant are examined according to the German rules and standards for activated sludge plants (ATV-DVWK-A 131E, 2000). All the parameters for each process are calculated and summarized.

Keywords: wastewater, commissioning, treatment, nitrification, denitrification, oxygen consumption, sludge production

ATIKSU ARITMA TESİSLERİNİN İŞLETMEYE ALINMASI VE DEĞİŞİK ÇALIŞMA SEÇENEKLERİNİN DEĞERLENDİRİLMESİ

ÖZ

Arıtma tesisleri genellikle 20-25 senelik periyotlar icin inşaa edilmektedir. Dizayn değerlerinin aksine, arıtma tesisleri işletmeye alınırken ve ilk yıllarda, artıma tesisine dizayn değerlerinden debi ve kirlilik olarak çok farklı atıksu gelebilmektedir. Işletmeye alma esnasında ve debi, kirlilik yükünde beklenmeyen değişikliklerin görüldüğü durumlarda arıtma tesisi için değişik arıtma alternatifleri uygulamak gerekir. Bu değişik durumlara göre operatör, optimum çözümü bulmalı ve uygulamalıdır.

Bu çalışmada bir atıksu arıtma tesisinin işletmeye alınması için genel bir işletmeye alma periyodu Piatra Neamt Atıksu Arıtma tesisinin işletmeye alınması sürecinde elde edilen tecrübelerden yararlanarak incelenmiştir. İşletmeciye bakış açısı kazandırmak için işletmeye alma öncesi ve işletmeye alma sonrasındaki yapılacak uygulamalar örneklerle irdelenmiştir. Çalışmanın son bölümünde tipik bir atıksu arıtma tesisinin değişik arıtma seçenekleriyle işletilmesi analiz edilmiştir. Bu seçenekler sadece karbon giderimi, karbon giderimi ve nitrifikasyon, karbon giderimi nitrifikasyon ve denitrifikasyon; ve uzun havalandırma prosesleridir. Atıksu arıtma tesisinin arıtma seceneklerine göre oksijen tüketimi, üretilen çamur miktarı, arıtma tesisinin performansı ve her bir prosese gore proses üzerinde gerekli ayarlar Alman ATV-131 standartlarına göre incelenmiştir (ATV-DVWK-A 131E , 2000). Her bir proses icin gerekli parametreler hesaplanmıs ve özetlenmiştir.

Anahtar Kelimeler: atıksu, arıtma, işletmeye alma, nitrifikasyon, denitrifikasyon oksijen tüketimi, çamur üretimi

CONTENTS

CHAPTER ONE

INTRODUCTION

1.1 Introduction

Wastewater treatment becomes more important day by day that we are losing our fresh water sources. Wastewater treatment plants provide continuity on water cycle, helping nature defend water from pollution. Treating wastewater is started after the successful commissioning period and continues with applying the best effective treatment options to the plant. In commissioning period, wastewater treatment plant is setup ready to treat wastewater and process is optimized according to the conditions. Beside the design values of wastewater treatment plants, they are show different behaviors because of characteristics and amount of incoming water. Differences of incoming wastewater and unexpected situations cause to run the wastewater treatment plant with different treatment options. The operator should find and compare the different treatment options to find the optimum way for wastewater treatment according to the related legislation.

In chapter two; general background about wastewater treatment and activated sludge processes are given. German ATV rules and European Union Urban Wastewater Directive are expressed and compared to each other. Legislation about wastewater treatment and discharge in Turkey is mentioned.

In chapter three; the calculation principles of German rules and standards for activated sludge plants is given (ATV-DVWK-A 131E, 2000). The equations of the important parameters for the activated sludge processes are given. The relationship between the parameters according the ATV-131 is mentioned.

In the chapter four; commissioning period of wastewater treatment plant is examined with the experience of the Piatra Neamt Wastewater Treatment Plant commissioning activities. The plant design data and basic process flow is mentioned first. Then commissioning period is examined unit by unit. Preparations of commissioning, actions during commissioning, probable problems and their solutions are examined with the examples.

In chapter five of the thesis Shatat Wastewater Treatment Plant is analyzed with different treatment options according to the German rules and standards for activated sludge plants (ATV-DVWK-A 131E, 2000) mentioned in chapter three. The oxygen consumptions, sludge productions and treatment efficiencies of the wastewater treatment plant are calculated and summarized in the tables. Key points of each treatment option are discussed.

1.2 Scope and Research Objectives of the Thesis

The scope objective of the thesis was to explicate the commissioning steps of wastewater treatment plant and evaluation of different stages of options. The objectives were therefore:

- To give the view points to the engineers for the commissioning of wastewater treatment plant.
- To give the solutions for the possible problems that may happen during commissioning.
- To investigate the key point of different treatment options.
- To compare the different treatment options and find the liability of options for the typical wastewater treatment plant.
- To find the best treatment options for the real situations of the wastewater treatment plant.

CHAPTER TWO

GENERAL BACKGROUND

2.1 Introduction

In this chapter general information about wastewater treatment plant and activated sludge processes are explained. The standards which are using during the dimensioning and operating of the wastewater treatment plants are mentioned and compared in this chapter.

2.2 General Background

Most of treatment plants have physical, chemical and biological treatment units. Wastewater treatment is generally achieved in three steps: preliminary and primary treatment (generally physical treatment), secondary treatment (biological treatment of wastewater) and tertiary treatment (disinfection, advanced oxidation etc.).

Preliminary and primary treatment involves:

1. Screening and comminuting: removes large objects, such as stones or sticks that could plug pipes, block tank inlets or damage the equipments.
2. Grit chamber- removes grits
3. Primary sedimentation tank – removes settleable solids.

Secondary treatment can be achieved with different treatment options. The main three options include:

1. Activated Sludge- The activated sludge process is one of the most widespread biological wastewater treatment technologies currently use. The activated sludge processes are mentioned with details below.

2. Trickling Filters- The process is achieved by spraying the wastewater to plastic or stone coarse filter media those microorganisms attached on. Microorganism degrades the organic material and treated water flush away from the bottom of tank.

3. Lagoons- Lagoons are cheaper solutions according to the other treatment options but efficiency are very poor. Lagoons can be applied to the small communities where the large lands are available. In lagoons, wastewater is waited with interaction of sunlight, microorganisms and oxygen. Some lagoons are aerated and some of them closed to provide anaerobic medium.

Tertiary treatment in other word advanced treatment is usually applied where the treated wastewater is discharged to the sensitive zones or used for irrigation. In tertiary treatment, generally the wastewater is disinfected using chlorine, ozone or ultraviolet after secondary treatment. Sometimes membrane processes are used for tertiary treatment.

2.3 Activated Sludge

The activated sludge processes was discovered in 1914 at England (Ardern and Lockett, 1914). The activated sludge is normally thick brownish slurry that consists of the microorganisms that capable of aerobic degradation of organic matter and other particulate matter. It is mixed with the influent in the aerated activated sludge basin. Oxygen is provided to the activated sludge basin by aerators to provide oxygen for microorganisms and mixing the content of aeration tank. After aeration period, the wastewater with activated sludge is flows into settling tank. The sludge settles as sediment and cleaned effluent is withdrawn from the settler surface. The majority of the sludge is brought back as return sludge and the surplus is wasted as excess sludge. Crucial for good separation performance is that the sludge settles well. Basic schematic diagram of an activated sludge process is given Figure 2.1

The activated sludge units are designed according to pollution load. The food-to microorganism (F/M) ratio is a major design parameter. F and M can be suggested by influent biological oxygen demand (BOD) and suspended solids (SS) in the aeration tank (Mishoe G., 1999). The liquid and microorganisms in the aeration tank is called as mixed liquid and SS in the aeration tank are mixed liquor suspended solids (MLSS) (Weiner R.F., Matthews R.A., 2003).

The generally recommended nutrient supply to activated sludge are one kg of phosphorus and five kg of nitrogen for every 100 kg of BOD oxidized (a C:N:P ratio of 100:5:1) (Metcalf & Eddy, 2003). This ratio is calculated with 100 % of treatment efficiency and when observed yield is 0.41. In real conditions; C requirement is generally higher than the mentioned above. Because never 100 % C removal is reached and observed yield is generally lower than value 0.41. The necessary ratio should be calculated for the specific wastewater in concern instead of using a constant value (Ammary, B.Y 2004).

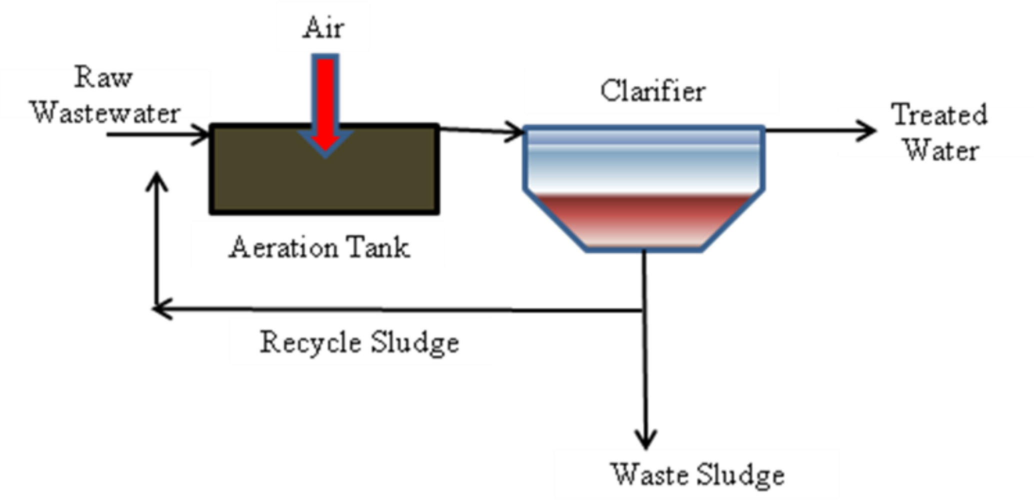

Figure 2.1 Basic schematic diagram of an activated sludge process.

At beginning activated sludge processes are used only carbon removal. Organic matter is absorbed and directly oxidized to CO_2 and H_2O. These compounds are then used for biomass synthesis. The rate of organic compounds removal by activated sludge results mainly causes from the intensity of new cell synthesis (Dobrzynska et al. 2003). During carbon removal a certain degree of nitrogen removal occurs in any biological wastewater treatment system due to the uptake of nitrogen into the waste sludge produced in the process. Nitrogen is a component of waste biomass produced as a result of biological treatment of carbonaceous organic matter. Organic nitrogen is also a component of the non-biodegradable particulate organic matter which is present in wastewaters. This material will generally be flocculated and incorporated into the biological treatment system mixed liquor and subsequently removed from the process with the waste sludge. Standard procedures are available to determine the quantity of nitrogen which will be removed by these mechanisms. Nitrogen removal will occur by this mechanism in biological nutrient removal (BNR) systems, just as it occurs in any biological wastewater treatment system. The difference between a typical biological wastewater treatment system and a BNR system is that, in a BNR system, additional nitrogen removal is achieved by the combined action of the two biological reactions:

a) nitrification
b) denitrification.

Nitrification is the biological conversion of ammonia-nitrogen to nitrate-nitrogen. It is accomplished by members of a group of bacteria called autotrophs. Autotrophic micro-organisms oxidize inorganic constituents to obtain energy for growth and maintenance, while they obtain carbon for the production of new biomass by the reduction of carbon dioxide. Notice that organic matter is not required for the growth of autotrophic bacteria. Nitrification is actually a two-step reaction. The first step is oxidation of ammonia-nitrogen to nitrite-nitrogen by bacteria of the genus Nitrosomonas.

The equation for this reaction, presented in simplified format, is as follows:

$$NH_4^+ + 1.5\ O_2 \rightarrow NO_3^- + 2H^+ + H_2O \quad (2.1)$$

The second step is the oxidation of nitrite-nitrogen to nitrate-nitrogen by bacteria of the genus Nitrospira. The simplified equation for this reaction is as follows:

$$NO_2^- + 0.5\ O_2 \rightarrow NO_3^- \quad (2.2)$$

Under steady-state conditions these two reactions will be in balance and the overall reaction will go essentially to completion. Including the synthesis of new biomass (expressed as the typical composition of biomass), the overall reaction is:

$NH_4^+ + 1.83\ O_2 + 1.98\ HCO_3^- \rightarrow 0.98\ NO_3^- + 0.021\ C_5H_7NO_2 + 1.88\ H_2CO_3 + 1.04\ H_2O$ (2.3)

Equation (2.3) illustrates the stoichiometry of the nitrification reaction. Oxygen is required to oxidize ammonia-nitrogen, and 4.6 mg of O_2 is required for each mg of NO_3^--N generated. Bicarbonate alkalinity is also consumed in the reaction to both neutralize the acid produced (i.e. ammonia-nitrogen is a base while nitrate-nitrogen is an acid) and as required for the synthesis of new biomass (from carbon dioxide which is present as bicarbonate alkalinity). The alkalinity requirement calculated from equation (2.3) is 7.2 mg of alkalinity as $CaCO_3$ for each mg of NO_3^--N produced. Biomass yield values are typically low for autotrophic bacteria, and the nitrification reaction is no exception. The yield value for the nitrifiers (both Nitrosomonas and Nitrospira) is 0.15 mg of bacteria as total suspended solids (TSS) per mg of nitrate-nitrogen generated (Gerardi, M., 2003).

Denitrification is the utilization of carbonaceous organic matter by heterotrophic bacteria using nitrate-nitrogen as the terminal electron acceptor. Most of the heterotrophic bacteria activated sludge is capable of using either dissolved oxygen or nitrate-nitrogen as a terminal electron acceptor. Dissolved oxygen is used preferentially when both terminal electron acceptors are present. In anoxic medium nitrate-nitrogen serves as the terminal electron acceptor, in other words denitrification occurs; the nitrate-nitrogen is converted to nitrogen gas, which can then be liberated into the atmosphere (Metcalf & Eddy, 2003).

Denitrification significantly impacts the stoichiometry of a biological wastewater treatment system. Theoretically, 2.86 mg of carbonaceous oxygen demand is satisfied for each mg of NO_3^--N which is reduced to nitrogen gas. Denitrification also results in a reduction in process alkalinity consumption due to the removal of the acid nitrate. Theoretically, 3.6 mg of alkalinity as $CaCO_3$ is produced per mg of NO_3-N reduced to nitrogen gas (Metcalf & Eddy, 2003).

Anammox process can be alternative of nitrification/ denitrification process for strong the nitrogenous wastewaters. Anammox process is the process that oxidation of ammonia in anaerobic medium with the help of specific microorganism called anammox bacteria (Strous, M., et al, 1999).

2.2 German ATV-DVWK Rules and Standards

German ATV-DVWK rules and standard are published by German Associations for water, wastewater and waste which are a technical, scientific, politically and economically independent association (http://www.dwa.de/portale/dwahome).

These rules and standards generally used as the general basis for planning, construction and operation of water and wastewater treatment systems. The ATV-A-131 coded standards are used for wastewater treatment plants which uses empiric formulas that so close to real situations. The using of these standards is prerequisite in many design, build and/or operation tenders

The standard covers the selection of the most practical procedure for carbon, nitrogen and phosphorus removal, and for the dimensioning of the essential components and facilities of the plant basically applies for the dimensioning of single-stage activated sludge systems. The standard applies for domestic wastewaters. The selection and dimensioning of equipment is not dealt with in this standard (ATV-DVWK-A 131E. 2000).

The ATV-A-131 design rules gives very large volumes for wastewater plants over 100.000 population equivalent (PE), which is very likely to meet the treatment requirements, but also to be very expensive. Another crucial aspect of ATV-A 131 is that it cannot take into account local factors like climate, water quality standards (Benedetti L., 2006).

2.3 European Union Urban Wastewater Directive

European Union (EU) Urban Wastewater Directive (UWWD) was adopted in 1991 to protect the environment from the adverse effects of urban wastewater discharges and discharges from certain industrial sectors and concerns the collection, treatment and discharge of:

- Domestic waste water
- Mixture of waste water
- Waste water from certain industrial sectors

Specifically this directive requires:

- The Collection and treatment (secondary treatment) of wastewater in all places which has equal or more 2000 population equivalents (P.E.)
- More advanced treatment for wastewater that 10 000 (P.E.) in sensitive areas.
- Monitoring of the performance of wastewater treatment plants and receiving waters
- Controls of sewage sludge disposal and reuse
- Controls of treated wastewater discharge and reuse (CEC, 1991).

This directive was adapted to Turkish legislation in 2006 with the same content and name Urban Wastewater Treatment Legislation (Official Gazette of Turkey, No: 26047). This legislation accepts the wastewater treatment effluent values as European Union UWWD and for other situation that not mentioned in UWWD give reference the Water Pollution Control Regulation. Before 2006 Water Pollution Control Regulation was applied the for all water treatment activities(Official Gazette of Turkey, No: 2872).

The objectives of this regulation are to set out the legal framework for water pollution control in order to preserve the country's underground and surface water resources use, and ensure the best possible utilization and the prevention of water pollution in conformity with economic and social objectives.

The Water Pollution Control Regulation is divided eight sections:

1. Objective, legal rationale and definitions,
2. Quality classification of water environments (inland surface, underground marine and coastal waters),
3. Basis for water quality planning and prohibitions (protection areas, prohibition to pollute, control of oil discharge),
4. Principles for waste water discharge (sewerage, irrigation, industrial waste waters, sampling, dumping),
5. Basis for dumping permissions,
6. Implementation at waste water infrastructure facilities,
7. Miscellaneous provisions,
8. Twenty five tables list various chemical and biological parameters and standards.

UWWD is not complying with the ATV guidelines because German treatment discharge limits are stricter than UWWD (CEC, 1991). Comparison of EU Wastewater Directive Standards and German Standards is given in Table 2.1.

Table 2.1 Comparison of EU Urban Wastewater Directive Standards and German Standards (Benedetti L., 2006).

			BOD		COD		Total Nitrogen		Total Phosphorus		Ammonia	
Country	**Legislation**	**Category**	**General**	**Removal**	**General**	**Removal**	**General**	**Removal**	**General**	**Removal**	**General**	**Removal**
			[mg/L]	%	[mg/L]	%	[mg/L]	%	[mg/L]	%	[mg/L]	%
General	EU UWWTD 91/271/EEC	2000 -10,000 PE	25	70-90	125	75						
		10,000 - 100,000 PE	25	70-91	125	75	15	70-80	2	80		
		> 100,000 PE	25	70-92	125	75	10	70-80	1	80		
Germany	Wastewater Ordinance June 2004 (Federal Law Gazette 1 p. 1106)	Size Category 1 Less than 60kg/d BOD_5	40		150							
		Size Category 2 60 to 300 kg/d BOD_5	25		110							
		Size Category 3 300 to 600 kg/d BOD_5	20		90						10	
		Size Category 4 600 to 6,000 kg/d BOD_5	20		90		18		2		10	
		Size Category 5 larger than 6,000 kg/d BOD_5	15		75		10		1		10	

CHAPTER THREE

THE CALCULATION PRINCIPLES

The calculations principles for carbon removal, nitrification and denitrification systems are described in this chapter. In the fifth chapter; Shahat Wastewater Treatment Plant is analyzed with the rules and calculations method descried below.

3.1 Required Sludge Age

For the system with only carbon removal it is needed to know where the nitrification starts. The necessary sludge age which nitrification starts can be calculated with the equation 3.1. (ATV-DVWK-A 131E. 2000).

$$t_{SS,aerob} = SF \bullet 3{,}4 \bullet 1{,}103(T\text{-}15) \qquad (3.1)$$

Where:

$t_{SS,\ aerob}$: aerobic sludge age, day

SF: safety factor for nitrification

T: temperature in the aeration tank, ^oC

The value of 3.4 is found from the maximum net growth rate of the nitrosomonas at 15° C (2.13 d) and a factor of 1.6. Calculated sludge age via the formula is guaranteed enough nitrification can be developed if the necessary oxygen is provided and if there is no negative influence factors. Safety factor is used to taken into account negative influences such as small changes of pH or temperature.

In practical sludge age can be increased to the 40 days. After reaching MLSS 10,000 mg/L oxygen transfer may be problem. Using biological membrane system can provide zero sludge production. For nitrification and denitrification necessary sludge age can be calculated with equation 3.2.

$$t_{SS,dim} = t_{SS,aerob} \bullet \frac{1}{1-(V_D/V_{AT})} \text{ (day)} \qquad (3.2)$$

Where:

$t_{SS,dim}$: sludge age upon which dimensioning is based, day

V_D: volume of the aeration tank used for denitrification, m^3

V_{AT}: volume of aeration tank, m^3

Denitrification share can be changed according to the process conditions (temperature, inflow ammonia concentration etc.) by changing sludge age. At Figure 3.1 the effect of the sludge age over nitrification is shown.

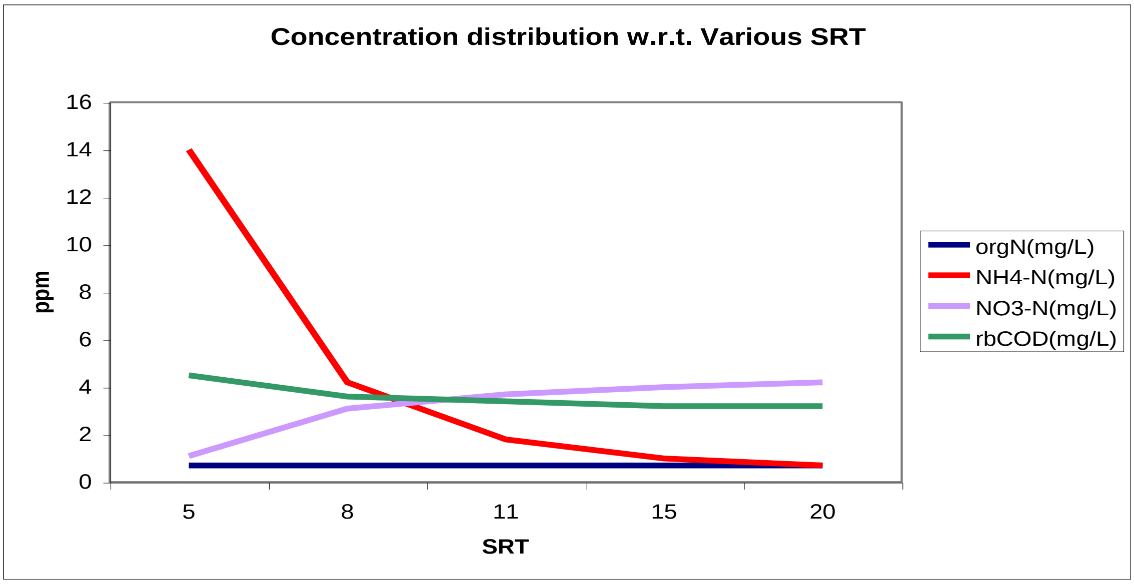

Figure 3.1 Effluent values for different sludge ages. Temperature is constant and 50 % of total volume is anoxic (Bolek, E. 2005).

3.2 Determination of Volume for Denitrification

Daily nitrate concentration that can be denitrified can be found with the equation 3.3.

$$S_{NO3,D} = C_{N,IAT} - S_{orgN,EST} - S_{NH4,EST} - S_{NO3,EST} - X_{orgN,BM} \quad [mg/l] \qquad (3.3)$$

Where:

$S_{NO3,D}$: concentration of nitrate nitrogen in the filtered sample, mg/L

$C_{N,IAT}$: concentration of the total nitrogen in the influent, mg/L

$S_{orgN,EST}$: concentration of the organic nitrogen in the effluent of the secondary settling tank, mg/L

$S_{NO3,EST}$: concentration of the nitrogen in the effluent of the secondary settling tank, mg/L

$X_{orgN,BM}$: concentration of organic nitrogen embedded in the biomass, mg/L

The daily nitrate concentration ($S_{NO3,EST}$) is generally negligible for domestic wastewater. In the calculations it is accepted zero. Chancing the anoxic /oxic volume ratio directly effect the effluent quality. During operation the wastewater treatment plant the operator may choose the change anoxic partition according to the plant capacity and find the best operating options for treatment. Effluent values for different Anoxic / Oxic volume ratios are given Figure 3.2.

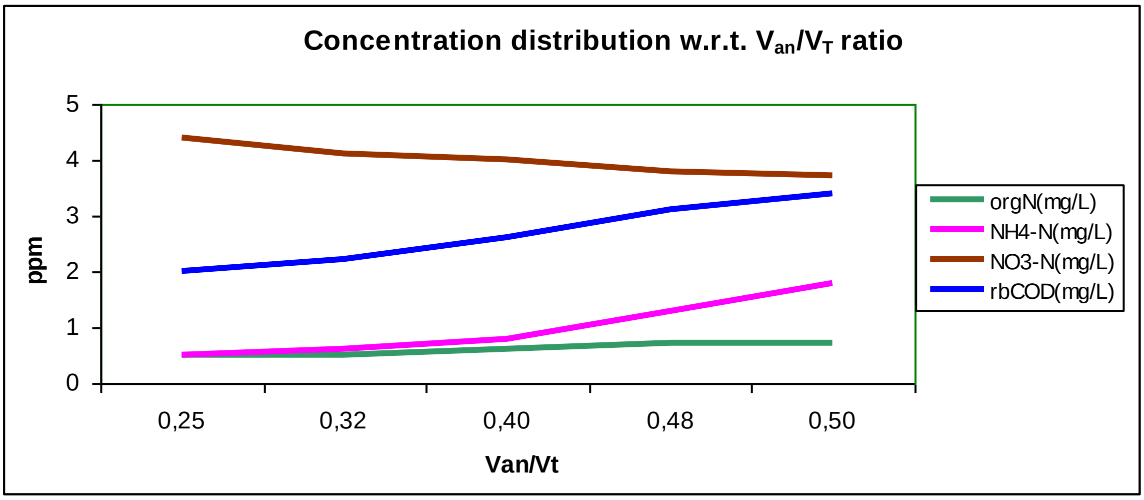

Figure 3.2 Effluent values for different Anoxic / Oxic volume (Bolek. E ,2005)

For denitrification processes the following equation 3.4 can be used to find denitrification volume.

$$\frac{S_{NO3,D}}{C_{BOD,IAT}} = \frac{0.75 OU_{C,BOD}}{2.9} \bullet \frac{V_D}{V_{AT}} \quad \text{(mg N /mg BOD}_5\text{)} \qquad (3.4)$$

Where:

$OU_{C,BOD}$: oxygen uptake for carbon removal, referred to BOD_5 mg/mg

$S_{NO3,D}$: concentration of nitrate nitrogen to be denitrified, mg/L

$C_{BOD,IAT}$: concentration of BOD_5 in the influent, mg/L

3.3 Required Recirculation and Cycle

Recirculation flow ratio is found using the ammonium concentration to be denitrified with the equation 3.5.

$$RC = \frac{S_{NH4,N}}{S_{NO3,EST}} - 1 \qquad (3.5)$$

Where:
RC: recirculation ratio, %
$S_{NH4,N:}$ concentration of ammonium nitrogen, mg/L

Internal recycle ratio is one of the other important operating parameter for the plants that uses denitrification process. Concentration distribution of effluent for with related to various international recirculation ratios is shown Figure 3.3. Internal Recirculation can be calculated with the equation 3.6.

$$RC = \frac{Q_{RS}}{Q_{DW,h}} + \frac{Q_{IR}}{Q_{DW,h}} \qquad (3.6)$$

Where:

Q_{RS}: return sludge flow rate, m^3/h
$Q_{DW,h}$: inflow flow rate with dry weather, m^3/h
Q_{IR}: internal recirculation flow rate, m^3/h

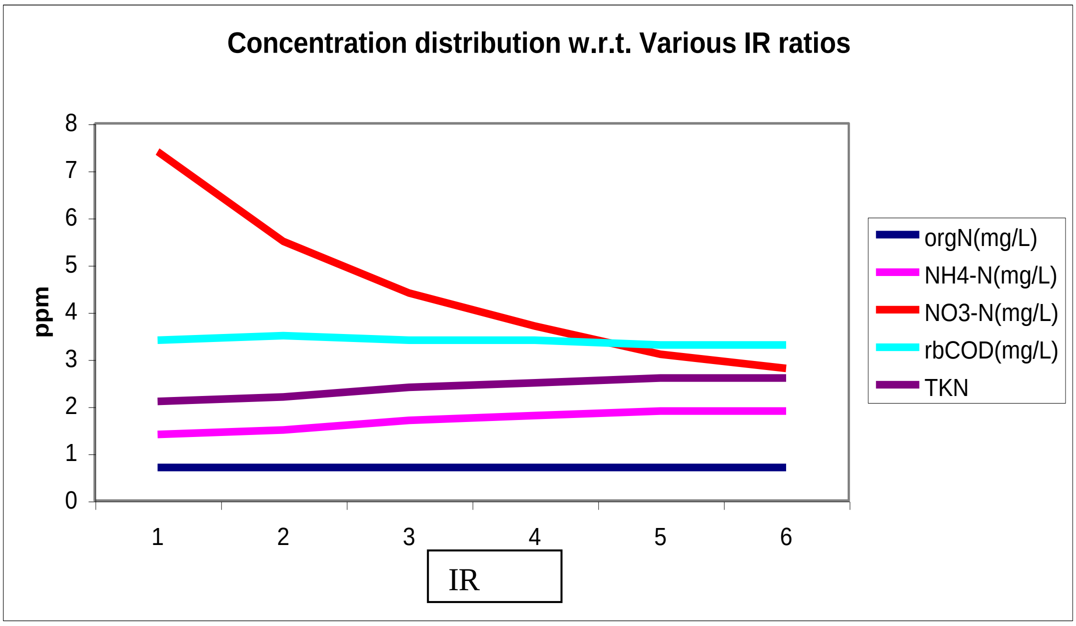

Figure 3.3 Effluent values for different internal recirculation (Bolek. E ,2005)

According to the Hatziconstantinou, G.J. ; Andreadakis, A., (2002), experimental study on pilot plants, showed that nitrogen removal activated sludge systems, operating under favorable conditions, seem to develop increased nitrification potential compared to fully aerobic systems under similar conditions. This potential difference can be explained with autotrophic populations that sustained more in anoxic reactors or phases.

3.4 Determination of Sludge Production

The produced sludge in wastewater treatment plant is made up from end products of biodegration and stored matter in microorganism or flocs. Chemical process like phosphorus removal should be added if applied.

In the calculations COD fractionation is accepted as reported in STOWA (1996) report with the ATV-131 standard. COD fractionation is shown on Figure 3.4.

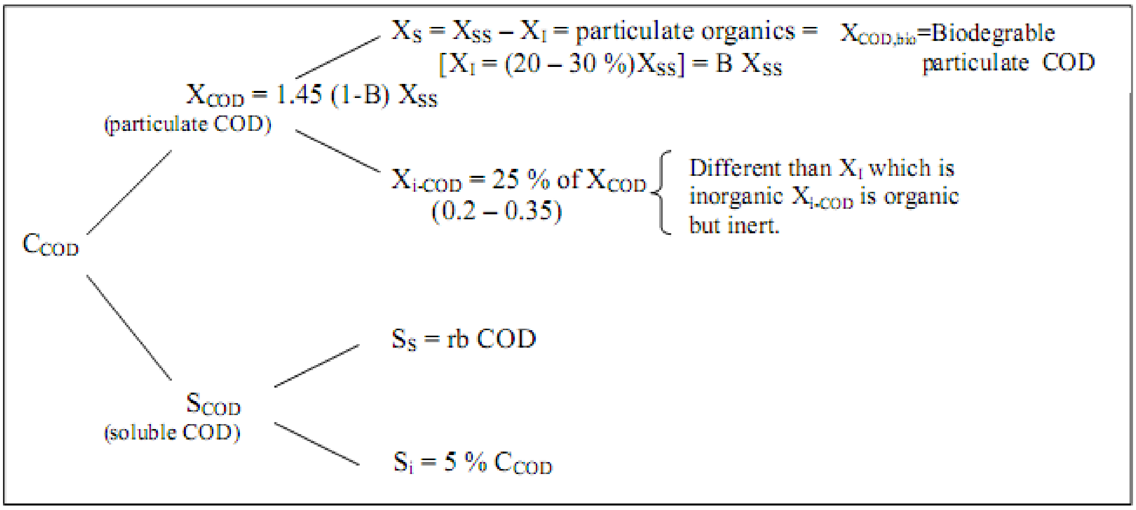

Figure 3.4 COD fractionation (STOWA, 1996)

COD in the raw wastewater can be divided two groups as soluble (S_{COD}) and particulate (X_{COD}) fraction. Each fraction has biodegradable part and inert part. Biodegradable COD can be found with equation 3.7.

$$b_{COD} = (S_S + X_S) = C_{COD} - (S_i + X_{i-COD}) \quad \text{(mg/L)} \qquad (3.7)$$

Where:
b_{COD}: biodegradable COD, mg/L

$S_{S:}$ readily biodegradable COD, mg/L

X_S: biodegradable particulate COD, mg/L

C_{COD}: total COD, mg/L

S_i: soluble inert COD, mg/L

$X_{i\text{-}COD}$: inert particulate COD, mg/L

Total filterable solid of the raw wastewater (X_{SS}) is given with the equation 3.8.

$$X_{SS} = X_S + X_I \quad \text{(mg/L)} \qquad (3.8)$$

Where:

X_I :Inert material such as grit, sand etc., mg/L

X_S : particulate organics, mg/L

Total inert particulates is defined as $X_I = B\ X_{SS}$

B = accepted as 25 % of total Suspended Solids (SS) entering the aeration tank. Generally organic dry matter in the inflow has accepted as 1.45 gram COD per gram organic SS. Particulate COD can be defined also like in the equation 3.9.

$$X_{COD}= C_{COD}-S_{COD}=1.45(1-B)X_{SS} \quad \text{(mg/L)} \tag{3.9}$$

After biological treatment, the waste activated sludge production is measured as COD ($X_{WAS\text{-}COD}$) is remaining. $X_{WAS\text{-}COD}$ is represented in the equation 3.10.

$$X_{WAS\text{-}COD}=X_{i\text{-}COD}+(X_{bH}+X_{bA})+X_p \quad \text{(mg/L)} \tag{3.10}$$

Where:

X_{bH}: concentration of COD in hetetrophic biomass, mg/L

X_{bA}: concentration of COD in autotrophic biomass, mg/L

X_P : particulate products = $X_{COD,\ Inert,\ Biomass}$, mg/L

Total COD in the biomass is defined as with the equation 3.11.

$$(X_{bH} + X_{bA}) = X_{COD,\ BM} \quad \text{(mg/L)} \tag{3.11}$$

Total COD in the biomass is the result of formation and the endogenous decay of biomass. The relations is given with the equation 3.12 and 3.13.

$$(X_{bH} + X_{bA}) = Y_{obsH}(S_S + X_S) - bF_T(SRT)(X_{bH} + X_{bA}), \quad \text{(mg/L)} \tag{3.12}$$

in which $F_T = 1.072^{(T-15)}$

$$(X_{bH} + X_{bA}) = \frac{Y_{obsH}}{1+b(SRT)F_T}(S_S + X_S) \tag{3.13}$$

Where:

Y_{obsH} : the assumed yield factor 0.67 g COD/ g COD_{deg}

b : the assumed decay coefficient;0.17, d^{-1}

The inert solid matter (Xp) remaining after endogenous decay is assumed 20 % of the decayed biomass. It is assumed that 80 % of WAS is organic. The daily total sludge production (SP_d) is given with the equation 3.14. Daily sludge production from carbon removal is given equation 3.15.

$$SP_d = SP_{d,C} + SP_{d,P} \quad \text{(kg/d)} \tag{3.14}$$

Where:

$SP_{d,C}$: daily sludge production from carbon removal, kg/day

$SP_{d,P}$: daily sludge production from phosphorus removal, kg/day

$$SP_{d,C} = Q_d.\left(\frac{X_{WAS\text{-}COD}}{0.8*1.45} + X_I\right)/1000 \quad \text{(kg/d)} \tag{3.15}$$

Phosphorus removal is take place via formation of biomass in the activated sludge that is accepted that 3 gram SS is reckoned per gram biologically removed phosphorus. The relationship for sludge production from phosphorus removal is given with the equations 3.16 and 3.17.

$$SP_{d,P} = Q_d \bullet 3 \bullet X_{PBioP} \quad \text{(kg/d)} \tag{3.16}$$

$$X_{P,PRec} = C_{P,IAT} - C_{P,EST} - X_{P,BM} - X_{PBioP} \quad \text{(mg/L)} \tag{3.17}$$

Where:

X_{PbioP}: assumed excess biological phosphorus removal, mg/L

$X_{P,BM}$: The phosphorus necessary for the build-up heterotrophic biomass, mg/L

$C_{P,EST}$: concentration of the phosphorus in the effluent, mg/L

$C_{P,IAT}$: concentration of the phosphorus in the influent, mg/L

$X_{P,Prec}$: concentration phosphorus removed by simultaneous precipitation, mg/L

The phosphorus concentration to build up hetetrophic biomass ($X_{P,BM}$) can be set between 0.01 and 0.005 b_{COD}.

Sludge Production and sludge age relationship is found with the equation 3.18.

$$t_{SS} = \frac{M_{SS,AT}}{SP_d} = \frac{V_{AT} \bullet SS_{AT}}{SP_d} = \frac{V_{AT} \bullet SS_{AT}}{Q_{WS,d} \bullet SS_{WS} + Q_d \bullet X_{SS,EST}} \text{ (day)} \qquad (3.18)$$

Where:

$M_{SS,AT}$: mass of suspended solids in the biological reactor, kg
SP_d : total sludge production, kg/d
SS_{AT} : suspended solids concentration in the aeration tank (MLSS), kg/m^3
SS_{WS} : suspended solids concentration in the effluent, kg/m^3
$X_{SS,EST}$: concentration of suspended solids of wastewater in the effluent, kg/m^3

For the sludge production from carbon removal can be found with the following empiric equation 3.19 using the Hartwig coefficients (Hartwig 1993).

$$SP_{d,C} = B_{d,BOD} \bullet (0.75 + 0.6 \bullet \frac{X_{SS,IAT}}{C_{BOD,IAT}} - \frac{(1-0.2) \bullet 0.17 \bullet 0.75 t_{SS} \bullet F_T}{1 + 0.17 \bullet t_{SS} * F_T}) \text{ (kg/d)} \qquad (3.19)$$

3.5 Required Oxygen Calculations

The oxygen requirement for the activated sludge processes is the sum of necessary oxygen for carbon removal and nitrification and sawing of oxygen from denitrification processes. For Carbon removal the required oxygen can be found with the equation 3.20.

$$OU_{d,C} = B_{d,BOD} \bullet 0.56 + \frac{0.15 \bullet t_{SS} \bullet F_T}{1 + 0.17 \bullet t_{SS} \bullet F_T} \text{ (kgO}_2\text{/d)} \qquad (3.20)$$

Where:

$OU_{d,C}$: daily oxygen uptake for carbon removal, kg/d
$B_{d,BOD}$: daily BOD_5 load, kg/d
F_T : temperature factor for endogenous respiration

For nitrification it is assumed that 4.3 g O_2 is required for per g oxidized nitrogen. The relationship between oxygen consumption and nitrification processes is given in the equation 3.21

$$OU_{d,N} = Q_d \bullet 4.3 \bullet (S_{NO3,D} - S_{NO3,IAT} + S_{NO3,EST})/1000 \quad (kg\ 0_2/d) \quad (3.21)$$

Where:

$OU_{d,N}$:Oxygen consumption for nitrification, kg O_2/d

Q_d : Daily flow , m^3/d

For denitrification it is assumed that 2.9 g O_2 is recovered for per g denitrified nitrate nitrogen. The relationship between oxygen consumption and denitrification process is given in the equation 3.22.

$$OU_{d,D} = Q_d \bullet 2.9 \bullet S_{NO3,D} \quad (kg\ 0_2/d) \quad (3.22)$$

Where:

$OU_{d,D}$:Oxygen consumption for denitrification, kg 0_2/d

Daily oxygen uptake can be found with the equation 3.23.

$$OU_h = \frac{f_C \bullet (OU_{d,C} - OU_{d,D}) + f_N \bullet OU_{d,N}}{24} \quad (kg\ O_2\ /h) \quad (3.23)$$

Where:

f_C : the peak factor that represents the ratio of oxygen uptake rate at peak to the average oxygen rate.

f_N : the peak factor that represents the ratio of TKN load in the 2 h peak to the 24 hour average load.

3.6 Volume of the Biological Reactor

According the equation 3.18 the required mass of suspended solids in the biological reactor can be found with equation 3.24

$$M_{SS,AT} = t_{SS,Dim} \bullet S_{Pd} \quad (kg) \quad (3.24)$$

The volume of biological reactor can be found with the equation 3.25.

$$V_{AT} = \frac{M_{SS,AT}}{SS_{AT}} \quad (m^3) \qquad (3.25)$$

3.7 Alkalinity

Alkalinity in raw wastewater is produced from hardness of drinking water plus ammonification of urea and of organic nitrogen. Alkalinity is decreased because of nitrification and phosphate precipitation. Some of alkalinity is recovered during denitrification process. Alkalinity is found with the equation 3. 26.

$$S_{ALK,EAT} = S_{ALK,IAT} – [\ 0{,}07 \bullet (\ S_{NH4,IAT} - S_{NH4,EST} + S_{NO3,EST} - S_{NO3,IAT}\ ) + 0{,}06 \bullet S_{Fe3} + 0{,}04 \bullet S_{Fe2} + 0{,}11 \bullet S_{AL3} – 0{,}03 \bullet X_{P,Prec}\ ] \quad [mmol/l] \qquad (3.26)$$

Where:

$S_{ALK,EAT}$: alkalinity in the effluent, mmol/L

$S_{ALK,IAT}$: alkalinity in the influent, mmol/L

S_{Fe3} : concentration of FE^{2+} in the filtered sample, mg/L

S_{Fe2} : concentration of FE^{3+} in the filtered sample, mg/L

S_{AL3} : concentration of AL^{3+} in the filtered sample, mg/L

$X_{P,Prec}$: concentration phosphorus removed by simultaneous precipitation, mg/L

CHAPTER FOUR

COMMISSIONING

4.1 Introduction

In the “Commissioning” part is formed with the experience of the commissioning activities of the related wastewater treatment plant. During commissioning all the activities, problems and solutions are taken into account of the part. In this chapter, “Commissioning” section, Piatra Neamt Wastewater Treatment Plant is taken into consideration. In the section 4.2 general information about the plant is given.

4.2 Piatra Neamt Wastewater Treatment Plant-Romania

The Piatra Neamt is a rehabilitation and extension project for the served population 230,000. The effluent parameters should be meet the effluent standards set out in the Directive 91/271/EEC according to the treatment plants with PE over 100,000. Design values and effluent requirements of the plant are given in Table 4.1, Table 4.2 and Table 4.3

The Piatra Neamt wastewater treatment plant comprises of four main sections:

- Preliminary Treatment
- Primary Treatment
- Secondary Treatment
- Sludge Treatment

Table 4.1 Influent Flows and Loads

Parameter	Symbol	Unit	Value
Dry weather Flow	Q_d	m^3/d m^3/h	27,000 1,100
Average Flow	Q_{ave}	m^3/d m^3/h	46,442 1,935
Peak Hourly Flow	Q_{peak}	m^3/h	2,800
Maximum Flow for Mechanical Stage	$Q_{max,\ mech}$	m^3/d m^3/h	90,000 6,000
Maximum Flow for Biological Stage	$Q_{max,\ bio}$	m^3/h	4,500
Biochemical Oxygen Demand (5 days)	BOD_5	mg/L	257
Suspended Solids	SS	mg/L	310
Ammonium Nitrogen (assumed 70% of Tot N)	NH_3-N	mg/L	30
Organic Nitrogen	Org. N	mg/L	12.3
Total Nitrogen	Tot N	mg/L	42.4
Total Phosphorus	Tot P	mg/L	8.4

Table 4.2 Effluent Requirements

Parameter	Symbol	Unit	Figure
Biochemical Oxygen Demand (5 days)	BOD_5	mg/L	$\leq$15
Chemical Oxygen Demand	COD	mg/L	$\leq$125
Suspended Solids	SS	mg/L	$\leq$15
Total Nitrogen	Tot N	mg/L	$\leq$10
Total Phosphorus	Tot P	mg/L	$\leq$1

Table 4.3 Design Parameters

	Unit	Figure
Minimum Design Temperature	^{o}C	10
Maximum Design Temperature	^{o}C	15
Sludge Age	kg BOD_5/kg MLSS.d	14
F/M	mg/L	0.077
MLSS concentration	mg/L	4,800

At preliminary treatment the wastewater is treated physically by coarse with 30 mm openings and fine screens with 5 mm openings and than by longitudinal aerated grit chamber. Collected screenings are conveyed to the skips located outside and extracted grit is separated by a grit classifier and conveyed to a skip for final disposal; furthermore the grease is pumped out and disposed of. These units are designed for maximum flow of mechanical stage, $q_{max, mech}$ which is 6,000 m^3/h.

Following the preliminary treatment, the wastewater enters the pre-treated wastewater pumping station and pumped to the distribution chamber for primary settlement tanks via 5 duty and 1 standby dry mounted centrifugal type pumps. There are two primary settling tanks with total 4,952 m^3 volume at the plant. Each tank has sludge and scum scrapers. Ferric chloride ($FeCl_3$) solution stored in Chemical Reagent Building will be dosed into the wastewater at two locations, firstly prior to the primary settling tanks and secondly after the activated sludge basins to remove phosphorus through sedimentation. $FeCl_3$ will additionally be dosed into the sludge feeding line to digesters to bind the sulphur.

At secondary treatment the flows above 4,500 m^3/h entering the chamber are diverted to the outlet of the plant and 4,500 m^3/h is distributed to each of the five anoxic zones proportionally. The anoxic zones consist of 2 radial tanks incorporating 2 submersible mixers in each and 3 rectangular basins incorporating 1 submersible mixer in each. Total volume of anoxic portion is 9,059 m^3 and total volume of aerobic portion is 13,589 m^3. Air required for the biological activity is provided by 3 duty and 1 standby blowers installed in the Blower Building. There are 3 radial secondary settling tank with total volume 16,700 m^3.

There are two lines of sludge treatment which are then combined into one following several steps. The first line is the primary sludge treatment, where the sludge extracted from the primary settling tanks is conveyed to the gravity sludge thickeners.

The second line is the mechanical thickening building where the excess sludge pumped from the biological pumping station is processed and then mixed with the gravity thickened primary sludge in the sludge mixing tank. The mixed sludge is pumped into the digesters. The digestion process is carried out in the mesophilic range (30 to 38°C) and biogas produced at the end of the process is collected and utilized for heating of the digesters. The digested sludge is ultimately stored in a basin and fed into the sludge building incorporating 4 sludge dewatering units (centrifuge) for dewatering and final disposal of the sludge. The collected gas through the digesters is to be treated by means of desulphurization. There are two gas holder units with volumes of 1,020 m^3 and 500 m^3.

4.3 Commissioning Procedures

One of the important steps to operate waste water treatment plant with maximum efficiency is commissioning. Commissioning include dry tests, wet tests and also training the operators. Although it seems that the commissioning period start after construction and mechanical installation, commissioning should be start at the same time mechanical installation start. While commissioning the treatment options can be evaluated for the best performance according to the inlet raw wastewater values. Typical flowchart for commissioning activities is shown Figure 4.1.

COMMISSIONING FLOW CHART

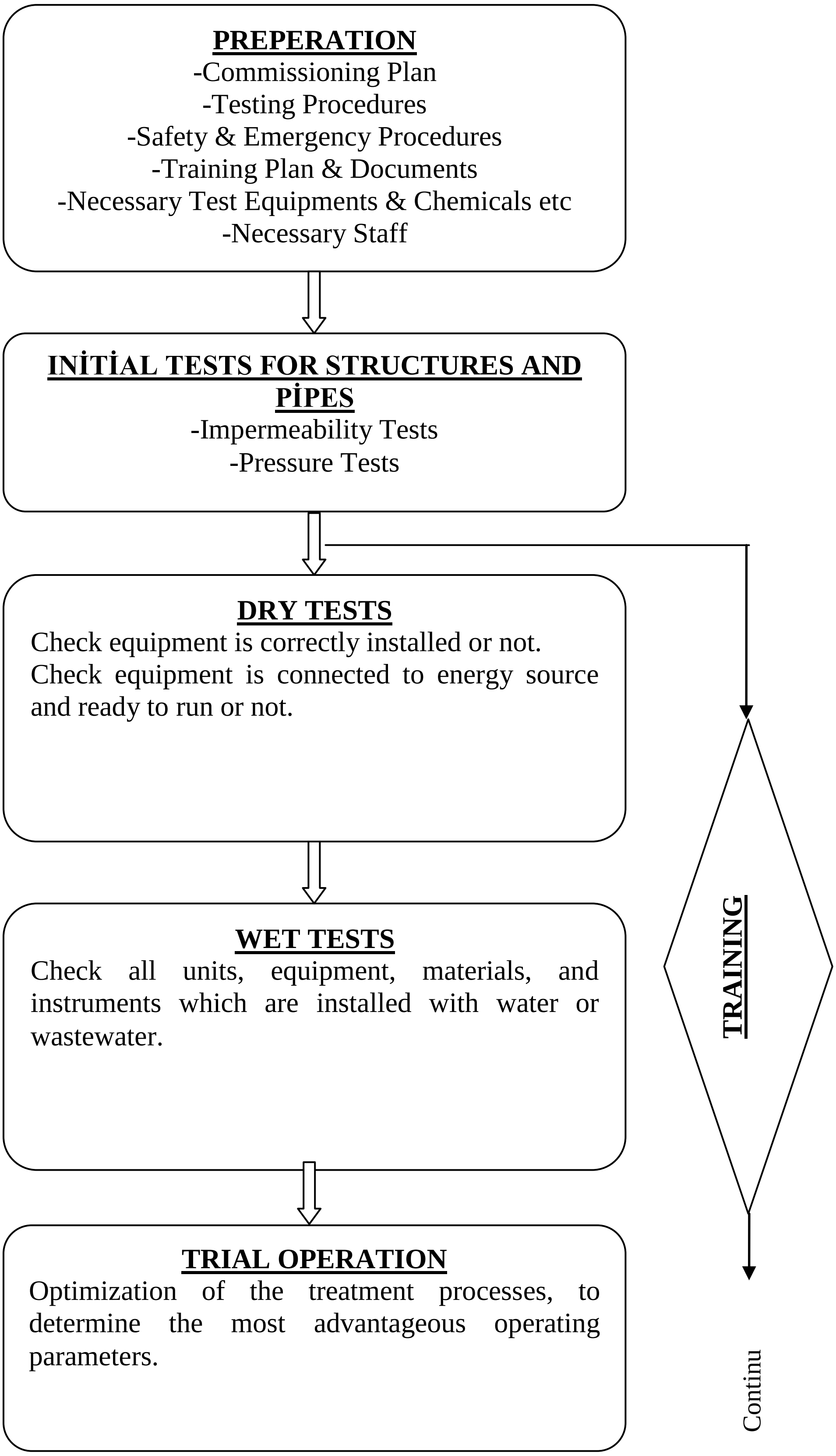

Figure 4.1 Typical flowchart for commissioning activities.

The commissioning of the wastewater treatment plant generally is carried out in 3 stages:

1. Dry tests
2. Wet Tests
3. Trial operation

Dry tests are the first stage to testify that wastewater treatment plant is ready to receive water. In dry test period, every equipment installed on site is tested for a short period in order to not to harm the equipment by running in dry conditions.

In dry test period following items are going to be checked;

- Equipment is correctly installed or not?
- Equipment is connected to energy source and ready to run or not?

For instruments, only electrical connections and availability on site is checked.

In wet testing period, all units, equipment, materials, and instruments which are installed or produced are tested with wastewater. Also wet test can be done with clean water. These tests are named as “Tests on Completion” or “Wet Tests”. Generally, this period will be done continuously and partially for some equipment, processes or units. And this period will be ended by operation of the plant in a routine way (start up of operation and trial operation period). In this period, some equipment is run in manual mode and some of them are run in automatic mode.

After the successful commissioning tests trial operation period is started. In this period, automation system, interlocks between equipment, instrument’s working conditions are controlled and adjustments are done if needed. Trial operation period serves for the optimization of the treatment process, to determine the most advantageous operating parameters. The trial operation is considered “successful” if in its third phase the final effluent quality systematic checks demonstrate that the effluent standards are met consistently and continuously.

4.4 Before Start-up

Before start-up a detailed commissioning plan should be prepared by commissioning engineer. Operation and maintenance manuals of each equipment and instrumentations should be prepared and examined by commissioning engineer and/or responsible person. All the necessary test equipments, chemicals or etc. should be prepared before commissioning. Testing procedures, safety procedures, emergency procedures should be prepared before tests. Some equipments should be commissioning by manufacturers such as blowers, scada etc. Manufacturer agents should be having enough experience. The agents commissioning work should be thought inside the plant commissioning period. Theoretical training should be done before tests. Inflow characteristics, pollutant loads should be examined and forecasting during commissioning period should be done. All staff positions should be clarified and be filled. The start-up operating shifts should be arranged as close as normal operating schedule as possible (Environmental Protection Agency [EPA], 1973).

4.5 Initial Tests for Structures and Pipes

Impermeability test can be done with potable water. All the tanks should be filled with potable water and should be waited at least 24 hours. A divided impermeable area should be filled water to determine the evaporation. Inlet, outlet and if exists other connections should be isolated. During test tanks should be checked visually that there is no leakage regularly. After test times up the water level should not be decreased after abstraction the evaporation height.

Pipe impermeability test can be done with potable water. Pipe entrance and exit should be closed at least one fixed closed valve. Other side can be closed with valve or test balloon. Figure 4.2 gives an example for the pipe impermeability test.

Figure 4.2 Pipe impermeability test. Installing the test valve and supporting it with wood blocks. Test valve was connected to hydrant water and other opening of pipeline is closed by test balloon. Then line was filled with water and waited definite time. Pressure is checked by manometer attached on test valve. Piatra Neamt Wwtp-Romania

4.6 Start-up of the Preliminary & Primary Treatment

Preliminary treatment units have essential importance for protection the equipments and carry on treatment. These facilities are screens, comminuting devices and grit chambers. Both facilities are generally run with on & off buttons that controlled by manual or scada systems. Before start-up these units must be inspected carefully and pre-tested that can eliminate many problems during commissioning.

4.6.1 Screens

Screens help us to remove solids that can cause damage on pumps such as debris plastics, woods and body of dead animals. Screens may be cleaned either manually or automatically.

Before start-up screens should be checked that they have been installed correct. For automatically cleaned screens the necessary operating interval should be adjusted. For mechanical parts necessary lubrication should be done. At pre-testing screen mechanism should be run to check it completes a cycle without any jam. If possible switches and electrical parts should be check for normal and overload situations (EPA, 1973).

While start-up screen large amount of debris may flow into the collection systems because of accumulated debris. After installation both units, they should be inspected for proper installation

4.6.2 Comminuting Devices

Comminuting devices in other word shredding devices are used for the cut in small pieces of debris that may damage the mechanical equipments and eliminate the screening problems. Although they are not common in Turkey; they impress themselves in wastewater treatment sector. Shredding equipments are expensive and needs more maintenance than other preliminary treatment equipments.

Shredding devices should be checked for proper installation according to the manufacturer instructions. Electrical connection and lubrications should be checked before testing. Alarms and protective instruments should be checked. Devices should complete at least one cycle without any jam, wrong noise or/and electrical problems in pre-testing period.

During start-up it must be controlled that unwanted materials such as stones or big glasses cannot reach the shredding device because they can damage the cutters of shredding devices. Shredding device cycle time should be calibrated according to the incoming wastewater characteristics.

4.6.3 Grit Chambers

Grit chambers are used to remove of grit, sand, cinders and other heavy solids. Grit chambers are provided to (1) protect moving mechanical equipment from abrasion and accompanying abnormal wear; (2) reduce formation of heavy deposits in pipelines, channels and conduits; (3) reduce the frequency of digester cleaning caused by excessive accumulation of grit (Metcalf & Eddy, 2003).

There are three types of grit chamber commonly used: gravity type, aerated and vortex. Gravity type grit chambers are generally long horizontal channels where the flow speed is arranged that the inorganic settled down while organics stayed in suspension. In aerated grit chambers water flow is circulated as spiral by aeration. Aerated grit chambers use less space than gravity types. Vortex type grit chamber uses centrifugal and gravitational forces to separate grits.

Before commissioning; all the mechanical equipments should be inspected for proper installation, tight mounting and lubricating. All the data should be recorded for future maintenance.

During start-up the grit chambers should be checked that the grit is removed properly. Mechanical equipments run properly. For vortex types grit classifier and grit pump run cycle should be arranged. Figure 4.3 gives an example for grit classifier run cycle arrangement during commissioning.

Figure 4.3 Grit classifier run cycle arrangement during commissioning.

4.6.4 Primary Sedimentation Tank

Primary sedimentation tanks are used for remove easily settleable solids and floating materials. It is advisable that primary sedimentation is required when raw sewage water contents high suspended solid. For systems include digester, need primary sedimentation to feed digesters. Primary sedimentation tanks remove 50-70 % of suspended solids and 25 to 40 % of biological oxygen demand (BOD) (Metcalf & Eddy, 2003).

Because of major units and mechanical equipments working submerged during operation, the inspection and pretesting before filling the tanks is very important. All the gates, sludge collection mechanism and scraper mechanism should be checked for proper operation. Scraper should be checked that it completes at least one cycle without any jam.

During start-up the raw sludge removed from sedimentation tank should be checked that dry solid content is normal. Sludge removal rates, primary sludge pumps working cycle and time should be settled according to the wastewater characteristics. If sludge is too thin pumping should be stopped. If dry solid mater content is installed this control is become easy and primary sludge pumps can be controlled according to the dry solid meter content.

Scum scraper and collection mechanism should be checked that working proper or not. Scum pumps working time and schedule should be arranged according to the scum level at scum collection chamber.

If pumps are working irregular, probably there is a problem because of thick sludge. Operator should be increase the primary sludge pumps operation time and cycle. If nothing changes, all the equipments should be checked that they have no damage. This checking may be required emptying the tanks.

If the tanks have odor like “rotten egg”; possibly septic conditions occur at sedimentation tank. Sludge should be removed more often. Beside bad odor, septic conditions can make harder to primary sludge dewatering process and increase polymer demand.

Another problem that can be seen primary sedimentation tanks is short circuiting. Short circuiting decreased the solid removal efficiencies. Short circuiting seems more in rectangular tanks than circular ones. It is difficult to determine short circuiting in circular tanks. There are a lot of reasons of short circuiting such as high velocities, uneven distributed weirs, and inadequate baffles. To check the short circuiting, it is important to control inlet velocity, distribution structure and flows over weirs.

4.7 Start-up of the Secondary Treatment

The secondary treatment facilities are aeration tanks consists of biological systems that help us to treating wastewater. Hence secondary treatment include biological process combined with hydraulic process, it is more complex to commissioning than primary treatment units. It is important to set balance between microorganisms and food to reach the maximum efficiency during commissioning. If the wastewater treatment plant consist more than one aeration tank, the tanks should be commissioning in order.

Before biological process developed; the units’ structures, water and air piping systems must be checked. The piping system both air and water should be checked for leaks. The safety instruments and equipments should be checked. Blower pressure, amperage should be checked if necessary adjustments should be done and values should be recorded.

The aeration tanks should be filled with clean water about 30 cm over the diffuser systems to control aeration systems. Then blowers should be tested and diffuser installation should be checked. If any diffusers found which are not working or installed false should be fixed or changed by technicians. Figure 4.4 gives an example during diffuser system check.

Oxygen transfer test is a common test applicable at commissioning period to determine the oxygen transfer efficiency. This test can be done with chemicals or nitrogen gas (BS EN 12555-15, 2003). Although most tests are done with sodium sulfite or cobalt chloride, usage of nitrogen gas became more popular day by day. Nitrogen gas has more deoxygenating ability, more economic according to the chemicals.

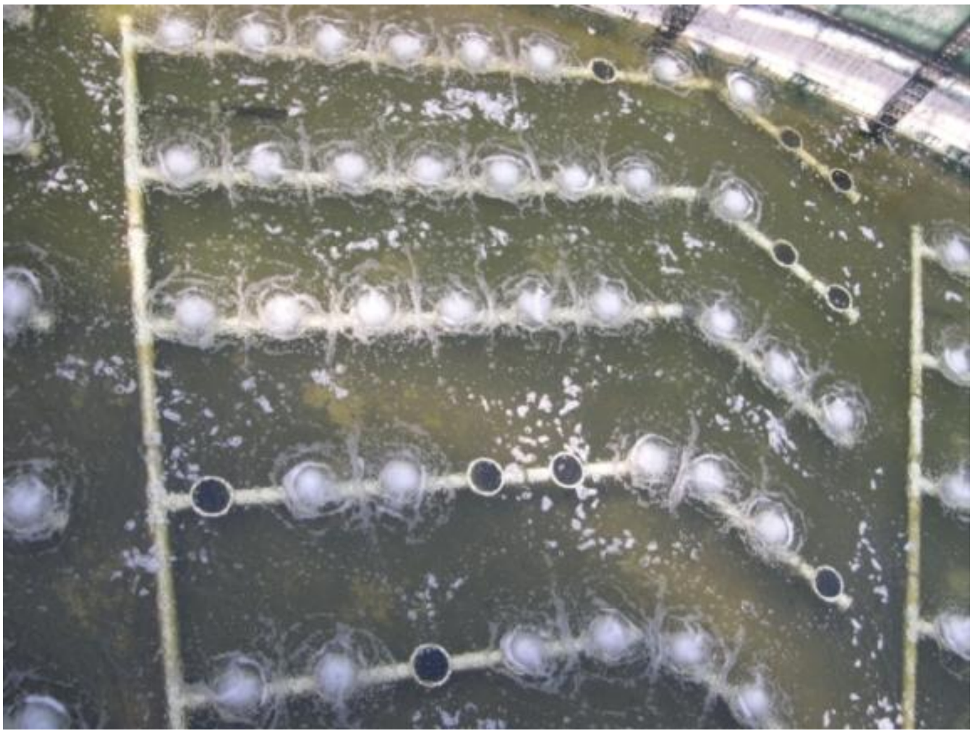

Figure 4.4 Working and non-working diffusers are seen during diffuser system check.

The test method is based upon removal of dissolved oxygen from the water by addition of chemicals followed by reaeration to near the saturation level. The dissolved oxygen inventory of the water volume is monitored during the reaeration period by measuring dissolved oxygen concentrations at several determination points selected to best represent the tank contents. The data obtained at each determination point are then analyzed by a simplified mass transfer model to estimate the apparent mass transfer coefficient , K_{La}, and steady state dissolved oxygen saturation concentration C*. There is no limitation dissolved solids and electrical conductivity if deoxygenation is achieved by nitrogen gas injection. (Stenstrom, M, et al. 2006)

If DO probes are not enough to all tanks, the test can be proceeding partially. As the N_2 flow rate increased, the time required to deplete DO is reduced. The water level in the tanks should be constant during test and checked before and after test. Quality of the water to be used for testing should be determined prior the test. If installed, mixers should be operated for at least 24 h before the start of testing in order to drain the pipes and to clean the diffusers. After installation of the DO probes, the aeration and if applicable the mixers should be operated at the lowest setting to be tested. The readings of the oxygen concentration should be indicated whether the turbulence at the probes is sufficient. If by the movement of a probe a higher value reading is indicated, the turbulence is insufficient. Agitators should then be employed to increase the turbulence at the probe. Nitrogen gas can be injected through the air aeration system and measure gas flow rate. DO concentrations should be recorded during test and when DO concentration reduced the nitrogen gas injection should be stopped and the water should be reaerated close to oxygen saturation.

After equipment tests, commissioning with wastewater can be started. Hence raw wastewater does not include necessary microorganism population to degrade the organic matter, it is necessary to develop sufficient biomass called as actived sludge. At normal operation biomass increase by feeding organic matter then they are settled at secondary settling tanks. Necessary percent of biomass is recirculated to aeration tank to provide desired biomass to efficient treatment. Excess biomass is sent to sludge treatment units.

While start-up operator can develop necessary activated sludge or can seed with activated sludge from another wastewater treatment plant. Seeding sludge generally is taken from other plant excess sludge line and carried by sewage trucks then discharged into the new plant aeration tank.

Until reach to calculated MLSS the sludge should not dispose from secondary sedimentation tanks and should be recirculate the aeration tanks. After desired MLSS is establish than sludge can be wasted from secondary sedimentation tanks. MLSS concentration can be calculated according to the raw water and required effluent standards. And also the selected treatment process directly affects the MLSS concentration. MLSS value for different treatment process is shown at Table 4.4 (Metcalf & Eddy, 2003).

Table 4.4 The MLSS values for different treatment options.

Process Name	SRT,d	MLSS, mg/L
High-rate Aeration	0.5-2	200-1,000
High Purity Oxygen	1-4	2,000-5,000
Conventional Plug Flow	3-15	1,000-3,000
Step Feed	3-15	1,500-4,000
Complete Mix	3-15	1,500-4,000
Extended Aeration	20-40	2,000-5,000
Oxidation ditch	15-30	3,000-5,000
Batch Decant	12-25	2,000-5,000
Sequencing batch reactor	10-30	2,000-5,000

Laboratory analysis is necessary to control activated sludge process during start-up because nothing will be same with designed values at start-up period. The commissioning engineer should be obtained sludge age and MLSS concentration according to the raw wastewater flow rate, pollutants concentration, temperature; etc. Actual parameters should be determined before start-up and controlled during start-up. It can be easily produced correction factor between actual and design values.

The minimum MLSS concentration should be determined and no activated sludge wasted until reach this value. After reaching the minimum MLSS concentration, MLSS concentration can be controlled according to the treatment efficiency and physical factors by wasting or returning more or less sludge. The optimum MLSS concentration is the value when the final sedimentation tanks effluent pollutions are minimum values.

Seeding method is more safe and easy than using only raw wastewater. Seed sludge must contain at least 500 mg/L of MLSS. (EPA, 1973) After filling the aeration basin with seed sludge it should be provided oxygen concentration in the tank minimum 1.5 mg/L concentration. The raw water must be taken into the plant from low to max. In first day it will be better taking 10 % of raw water. And increase 10 % daily is good choice to arrange the necessary MLSS concentration easily.

Using raw sludge is more complicated and difficult to arrange than seeding. The aeration tank should be filled with raw wastewater. It is advisable that by passing primary clarifiers first time (EPA, 1973). The oxygen concentration should be minimum 1 mg/L inside the tank. The oxygen concentration should be controlled periodically during start-up. The submersible mixers should be run to prevent the sedimentation and so clogging the diffusers. After filling the tank it should be waiting at least 8 hours without taking wastewater inside the tank. After 8 hours the mixers and aeration should be stopped. It should be waited 60 minutes. During this time produced sludge and supernatant bed is formed. After waiting raw wastewater that can be filled 1/3 of the tank should be taking into aeration tank. Then the tank should be aerated and mixed for 8 hours again. At the end of the each circle Imhoff cone measurement should be done. This circle should be continued till reaching the optimum MLSS. After reaching the optimum MLSS concentration raw wastewater make take into tank continuously. This circle time can be adjustable according to the site conditions and tanks volumes.

The produced sludge should not be wasted during start-up and returned the inlet of aeration tank. The sludge should be returned at a rate that no sludge blanket will develop in the secondary settling tanks (EPA, 1973). This provides the rapid development of MLSS concentration.

After reaching the required MLSS concentration for full flow, the sludge returning rate can be adjustable. Necessary amount sludge that returned the aeration basin can be determined by SVI experiment. An example for determining required MLSS is given below:

Example: Determining required MLSS

Design Conditions:

Flow = 100,000 m^3/d

BOD_5 =250 mg/L

MLSS: 3,000 mg/L

Actual Conditions:

Flow = 65,000 m^3/d

BOD_5 =150 mg/L

Minimum MLSS Concentration

$$= 3{,}000 mg/l x \frac{250 mg/L}{150 mg/L} x \frac{65{,}000 m^3/d}{100{,}000 m^3/d} = 3{,}250 mg/L$$

If any conditions that the operator want to less process line than all lines the calculated value above must multiply with:
number of used basin (or volume) / total number of designed basin. (or volume)

An example for determining of required return sludge rate is given below:

Example: Determining required Return Sludge Rate (RAS)

Flow = 65,000 m^3/d

RAS=65,000 m^3/d

SVI: sludge volume after 30 minutes of settling (SV) = 0.40

Therefore
Adjusted RAS = 0.40 x (65,000+65,000)=52,000 m^3/d

RAS flow rate have to be adjusted 52,000m^3/d to maintain the proper MLSS in the aeration tank. An example for determining of required waste activated sludge rate is given below:

Example: Determining required Waste Activated Sludge Rate (Q_w)

Q_w can be calculated using following equation:

Q_w. = V.X / X_w. t_{ss}

Volume of Aeration Tank (V): 20,000 m^3

MLSS: 3,500 mg/L

Sludge age (t_{ss}) : 20 days

Concentration of sludge (X_w)= 8kg/m^3

$$= \frac{20{,}000\ m^3 x 3{,}500\ mg/L}{20d\ x\ 8kg/m^3}$$

$$= 437.5\ m^3/d$$

If the sludge age was less than the design values, no wasting should be done. Under normal conditions, the sludge age will indicate when to reduce or increase the wasting rate.

The waste activated sludge and return sludge pumping rates have to be adjusted if the characteristic of inflow change. After the steady state conditions obtained, a good sludge settle rapidly leaving a clear, odorless and stable supernatant.

During start-up, when the MLSS are low, the aeration basins may experience severe foaming. Foaming can be seen in Figure 4.5. Foaming is believed to occur because of synthetic detergents and other surfactants in conjunction with high aeration and low MLSS. The foam contains sludge solids, grease and bacteria and should be brought under control as quickly as possible (EPA, 1973). The operator may use defoaming agents, water nozzles to control foam. The operator may be decreased aeration rate until the required MLSS building up.

Sludge bulking may occur during start-up due to overloading the basin. The bulking sludge cannot settle and compact and rises over the weirs in final sedimentation tank. At bulking sludge filamentous organisms are attaching themselves from one floc to another and stopping the compaction of sludge particles. Low pH, low DO, low nitrogen concentration, high F/M ratio, industrial discharge or septic sludge is the major causes of the bulking sludge (Metcalf & Eddy, 2003).

Figure 4.5 Foam Formation during start-up.

Increasing sludge age or decreasing the F/M ratio is the primary object to control bulking sludge. The DO, pH and F/M in the aeration tank should be checked regularly. If DO is not enough, the capacity of blowers should be increased. If pH is low, lime may be used to raise pH to improve sludge settling characteristics. If F/M ratio high; increasing return sludge rate or decreasing wasting rate are the solutions to reduce the F/M.

If the wastewater is commissioning in winter season, it will take longer to build up necessary MLSS concentration. In winter the loadings, air rates are changed. The operator should check the parameters and calculations again against the cold air conditions.

The operator should avoid the extreme change on the process to control the any problems. Changes should be done step by step.

4.8 Start-up of the Sludge Treatment Units

Sludge treatment is the one of the most important part of the wastewater treatment. Sludge treatment is the most expensive and difficult part of the treatment plant. Proper start-up and operation of sludge treatment directly affect the plant efficiency and operation costs. Goals of the sludge treatment are stabilizing the sludge and minimize the volume. Major sludge treatment processes are thickening and digesting. In small wastewater treatment plants especially in hot climatic conditions, sludge drying beds are the common method to dewater the sludge.

4.8.1 Digesters

Digesters are the complex structures that used for the stabilizing sludge. In digesters the sludge is digested at anaerobic medium. Anaerobic bacteria break down the complex molecules; produce methane gas and odor-free stabilized sludge. Operating digester needs more care than other units. pH and temperature are very important factors to control digestion. The raw sludge must contain necessary volatile solids. The plants without primary sedimentation tanks cannot feed the digesters. BOD / COD ratio must enough to feed digesters. It is not applicable where low BOD produced countries like Turkey. Although generally plants are designed and constructed with 300 mg/L BOD design value, the real BOD is 100-150 mg/L at Turkey. Nowadays most of the plant cannot run their digesters in Turkey.

The aim of the commissioning digestion is provide suitable environmental for the bacteria. During the commissioning feeding, volatile acid concentration, mixing rates, temperature and pH should be taken under control.

Because the digesters are complex structures; the inspection and pretesting of digester is more difficult than other units. The operator should check the all the process and safety equipment and instruments. Before start-up the remaining debris should be removed completely and that the inside bottom and walls will be cleaned thoroughly. During start-up digestion process should proceed similar to Figure 4.6.

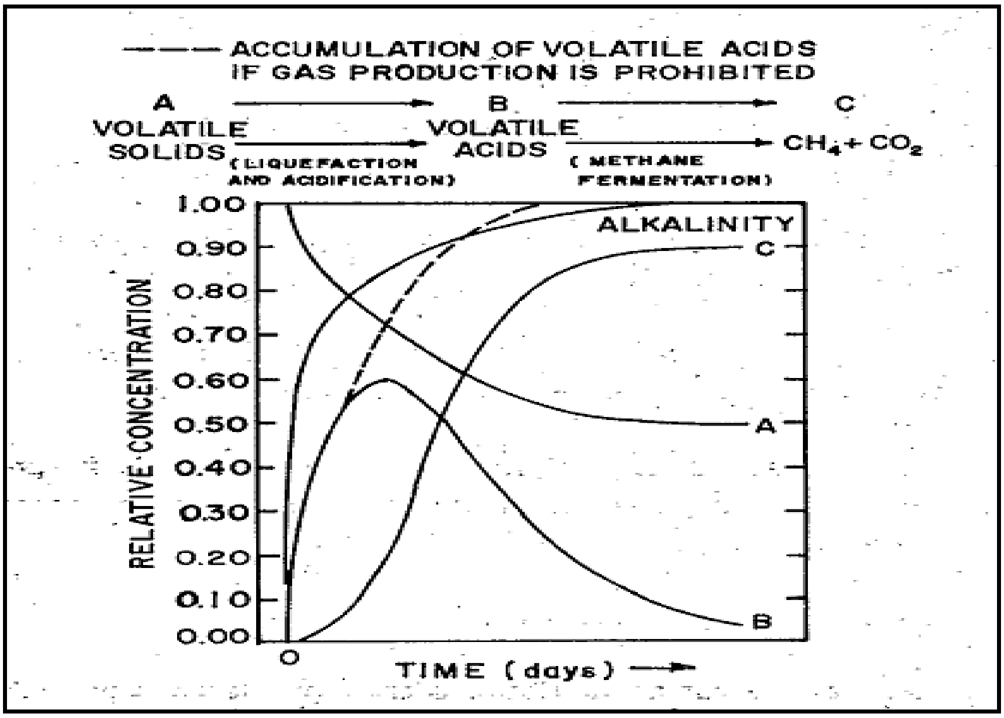

Figure 4.6 Sequential Mechanism of Anaerobic Sludge Digestion
(Courtesy of Texas Water Utilities Association, 1971)

There are two ways to start-up digester with seed sludge or without seed sludge. For commissioning with seed sludge digester should be filled with water (not chlorinated water) then the water will be heated up to 38 ^{o}C. Digester mixers should be started. After the water heated the digester will be started to fill with seed sludge. Seed sludge will be used to feed the digester. Seed sludge volume should be around 40 % of total digester volume. Before commencing the seed sludge pumping, the mixer should be switched off and the water is allowed to overflow from the top take off point as the seed sludge is pumped in. The mixer will be switched on when the seed sludge transfer is completed. The heating will continue to keep the temperature in the range of 34 – 36 °C.

The pH should be checked. If it is > 6.8 feeding can be started with small dosing of raw sludge 1/30 of digester volume daily. It is advised that pH should be adjusted with lime to 7.5 -7.8 (Isık, H, 2008). Upon completion of the seeding, raw sludge feeding can start incrementally. To seed an anaerobic digester with an adequate population of facultative anaerobes and anaerobes including methane-forming bacteria, a ratio of 1:10 of secondary sludge to primary sludge should be used (Gerardi, M. 2003).

Raw sludge feed should be no more than 10 percent of the anticipated ultimate daily load each day. Approximately 5 % of the design sludge feed rate should be fed into the digester in 4 equal portions each day. When the gas produced is about 50 percent of that calculated to be available normally from the volatile solids added and the volatile acids do not rise sharply; the feed can be increased by 50 to 100 % of the initial daily feed until the ultimate design load is reached (EPA, 1973).

To avoid the displacement of seed sludge during feeding, the mixer should be switched off one hour before pumping the raw sludge portion to allow settlement of the seed sludge.

pH measurement should be made 4 hours after each batch of feed. If sharp falls are observed, feeding should be postponed. If it falls below 6.8, correction must be made with lime to raise it above 7. The volatile acids should be measured and maintained it below 1,000 mg/L. Samples should be taken at about 1 hour before next batch of raw sludge feed. If it increases; raw sludge feeding rate should be decreased or alkali agent (NaOH) should be added. When digester pH and alkalinity started to increase (pH= 7, > 2,000 mg/L alkalinity), the raw sludge dosing rate can be increased (Gerardi, M. 2003).

The sludge feed batches can be increased after 3rd day if digestion progresses successfully. A careful assessment must be made before such action is taken.

The listed control parameters should be measured until the design load is reached.

- Temperature

  (The temperature should not be increased more than 2 ^{o}C daily.)
- Volatile Acids (VA)
- Total Alkalinity (Alk)

(VA /Alk ratio should be 0.5)

- pH

  (pH will be maintained within the optimum range of 6.8 to 7.2)
- Gas analysis for methane and CO_2
- Volatile solids (VS)

Gas production is generally expected to develop in the second week of start-up. It is a quick indicator of the digestion process performance. Any sudden fall in methane concentration indicates the failure of methane forming bacteria Gas production should be measured and recorded daily. CO_2 content of the gas should be checked daily. The volume of gas collected should be about 0.6 m^3/kg added in raw feed per day. The CO_2 ultimately should be between 30 and 35 % by volume (Gerardi, M. 2003).

After one digester is operating properly at its design load, digested sludge and overflow liquor can be used to seed other tanks.

Using thermal hydrolyzed sludge, at 165°C for 30 minutes, as seed sludge has marked advantage according to the other (Potts, L.G.A., Jolly M., 2002).

If there is no source of sludge for digester feeding start-up will be slower than seeding method. 80 % of digester should be filled with water and water should be heated up to 38 ^oC. After that sludge feeding can be started. While feeding NaOH should be added to control pH. Alkalinity, volatile acids and gas quantity should be checked regularly. Chemical additions can be arranged according to the Volatile acid concentration and alkalinity. Volatile acids / Alkalinity ratio should be 0.5 to avoid toxic levels adding of cations to the digester (EPA, 1973). When digester becomes stable, digester can be feed with design rate. The required alkalinity can be calculated as following example.

Example: Chemical Addition for digester control.

Determined Concentration:

Volatile Acids (VA): 1,200 mg/L

Alkalinity (Alk) = 540 mg/L

The required alkalinity to ensure a favorable environment for the methane bacteria

$$= \frac{\text{VA concentration in the sludge}}{\text{VA/Alk ratio} = 0.5} = \frac{1{,}200}{0.5} = 2400 mg/L$$

At start-up period volatile acid concentration is going to be decreased and methane production is going to be increased; therefore chemical necessary will be decreased. Methane formers are beginning to feed and reproduce at around seventh day of the start-up. They use volatile acids and produce methane gas and alkalinity. Signs of stabilization should be seen after ten or fourteen days the digester if there is no sign to stabilization, the operator should control the feed sludge for toxicity and control parameters (EPA, 1973).

Methane content of gas should be monitored daily. It should be increased the 60 - 70 % of total gas at the end of 30-40 days and sings that digester start-up finished and digester can feed with design values. During start-up if foaming occurs, the sludge feeding should be stopped and the sludge inside the digester should be mixed completely.

4.8.2 Sludge Dewatering Equipments

Most common methods for sludge dewatering sludge are gravity filter, belt filter press, centrifuges and sludge drying beds. Except sludge drying beds the others are mechanical equipments. They should be pretested and commissioned according to their operation and maintenance manuals.

For belt filter press; before start-up tension of the belts, water systems, air systems and polyelectrolyte dosing systems should be checked.

Because of polyelectrolyte used in sludge dewatering equipments; the engineer should made arrangement of poly dose according to the sludge features and amounts. The arrangement can be done by using laboratory scale experiments or method of trial and error. If the sludge is squeezed out of the belt onto the belt; the operator should increased the polymer dosing or decreased the sludge feeding. If the sludge is sticks on the belt; the operator should increased the sludge feeding or decreased the polymer dosing.

Summarized control parameters for commissioning period are given Table 4.5.

Table 4.5 Control parameters for commissioning period.

	CONTROL PARAMETERS	
	DRY TESTS	**WET TESTS/TRIAL OPERATION**
Pipes& Structures		Visual Check Pressure Loose
Screens	Visual Check for mechanical equipments Sound, Vibration check of running mechanical equipments	
Communiting Devices		
Grit Chamber	Check installation ,electrical connection and lubrications of mechanical equipments.	Aeration amount and distribution Grit removal efficiency
Sedimentation Tanks	Visual Check for mechanical equipments Sound, Vibration check of running mechanical equipments	Dry solid content of raw sludge removed from sedimentation tank.(for primary sludge) Sludge removal rates. Scum collection efficiency. Scum pumps working time and schedule. Odor (possibly septic conditions). Polymer consumption (if) Short circuiting.
Aeration Tanks	Vısual Check (installation of units, mechanical equipments etc) Sound, Vibration check of running mechanical equipments The safety instruments and equipments. Blower pressure, amperage.	***1-O2 test*** -Check oxygen transfer efficiency sludge age and MLSS concentration according to the raw wastewater flow rate, pollutants concentration, temperature; etc. ***2-Start-up*** -Raw water characteristics (SS, BOD,COD,NH_4 etc.) -Foaming -Oxygen concentration in the tank & ORP -SVI -RAS -SAS
Digesters	Vısual Check (installation of units, mechanical equipments etc) The safety instruments and equipments.	-Volatile solids content of raw sludge -BOD / COD ratio of raw sludge -pH -Temperature -Alkalinity -Volatile acid -Mixing rates -Produced Gas amount -CO2 content of the gas -Methane content of gas
Sludge Dewatering Systems	-Tension of the belts (for belt filter press) -Water systems -Air systems -Polyelectrolyte dosing systems	-Polyelectrolyte dose -Dewatering efficiency

CHAPTER FIVE

MATERIAL & METHOD

5.1 Introduction

In this chapter Shahat Wastewater Treatment Plant is taken into consideration because it is easy to run the plant with different treatment options. The plant is built just for carbon removal but it is easy the applicate different treatment options for better treatment effluent if the other conditions are proper. Different treatment options are examined beside the carbon removal according the ATV-131 rules. It is investigated that necessary process parameters values for different treatment options and results are compared to each other.

5.2 Plant Presentation

5.2.1 Shatat Wastewater Treatment Plant-Libya

The design population is defined as 33,722 for the year 2015. The plant principally will operate as a conventional activated sludge treatment plant for carbon removal; the plant is also equipped to nitrogen removal because of hot climatic conditions. Treatment requirements for the effluent of the plant must be obey EU Directive 91/271/EEC according to the treatment plants with P.E. between 10,000-100,000. The standards applied only for Carbon removal. Design values of the plant are given in Table 5.1.

Table 5.1 Design Flows and Loads

Parameter	Unit	Figure
<u>Wastewater flow rates:</u>		
Mean Daily Flow	m^3/d	6,744
Maximum Hourly Flow	m^3/h	422
Influent parameters		
COD	mg/L	600
BOD_5	mg/L	300
Ammonia Nitrogen	mg/L	45
Nitrate Nitrogen	mg/L	0
Phosphorus	mg/L	12
SS Concentration	mg/L	350
Alkanity	mmol/L	7.10
Effluent parameters		
BOD_5	mg/L	25
SS	mg/L	35

The compact plant comprises of four main sections:

- Inlet Works and Raw Sewage Pumping Station
- Preliminary Treatment
- Biological Treatment
- Sludge Treatment (Belt press)

The sewage comes to Inlet Works and Raw Water Pumping Station from the main collection chamber by gravity. The first unit is manually cleaned coarse screen with 50 mm bar spacing. The sewage then passes into the sump. There are 2 duties and 1 standby submersible pump. Each pump has 211 m^3/h flow pumping capacity. The flow rate of the sewage is to be measured by electromagnetic flow meter device installed after the pumping. Lifted sewage comes into the Fine Screen & Grit Removal Unit. There are 2 channels in the screen facility and each one equipped with automatically cleaned fine screen with 15 mm bar spacing.

After the fine screen, sewage passes into the grit removal section by gravity. There are two circular vortex type grit removal unit and two Cyclone Sand Classifiers.

After the fine screen, sewage passes into the grit removal section by gravity. There are two circular vortex type grit removal unit and two Cyclone Sand Classifiers. Physically treated sewage passes by gravity into the Biological Oxidation Tanks. Each Biological oxidation tank combines aerobic + anoxic part + return sludge (RAS)/Surplus Sludge (SAS) sections and settlement tank. Volume of Aeration Tank (outer) is 1,361 m^3 and settlement tank (inner) is 896 m^3. The designed sludge age is 4 days and MLSS concentration is 3.7 kg/m^3. In the tank, 1 submersible mixer, 1 recycle pump, 2 RAS pumps (1 stand by and 1 duty), 1 SAS pumps,1 sludge scraper and diffuser system are installed. Air required for the biological activity provided by 2 duties and 1 standby blowers installed in the Operation Building, Blower Room. The air requirement for the maximum flow is 1,414 m^3/h. The treated effluent goes by gravity into the effluent chamber connected to Biological Oxidation Tank. The final effluent discharges from a chamber adjacent to the Biological Oxidation Tank. Before discharge, chlorination is applied to the treated effluent.

The return sludge (RAS) collected by gravity under the settlement tanks comes into the RAS/SAS section. Biological Oxidation Tanks are fed by one duty pump. The excess sludge is taken from the RAS/SAS sections by SAS pumps and transferred into the Sludge Holding Tank. The daily sludge production is 304 m^3/d.

Surplus sludge from biological treatment is collected in a sludge holding tank where it is continuously mixed with a submersible mixer, pumped over a belt type mechanical gravity thickener. Thickened sludge flows by gravity on to a belt press for dewatering.

Thickened sludge is dewatered in a belt press with dosing of flocculants prepared by a package type polyelectrolyte dissolving and dosing unit. Sludge cake from belt press is loaded into skips by a belt conveyor. Filtrate is drained back into the raw sewage pump sump by gravity.

5.3 Evaluation of The Different Treatment Options

In this section Shatat Wastewater Treatment Plant is analyzed according to four different treatment objectives. These are:

- Carbon removal
- Carbon removal + Nitrification
- Carbon removal + Nitrification + Denitrification
- Extended Aeration

All the calculations and assumptions are made according to the ATV standards (ATV-DVWK-A 131E, 2000). The standard provides a logical role to determine the dimension. In this thesis constructed plant with fixed dimensions is checked how the plant behavior according to the treatment options. Design parameters for the wastewater treatment plant given in Table 5.1 are used for the calculations.

ATV rules are applied for constant volume. Parameters such as sludge age and MLSS concentration are changed according to the conditions to give best treatment result for the each treatment options. Equations given in Section 3 are used to calculate the process parameters and results. In the calculations it is assumed that flow is equal the design flow and both two aeration tanks are used. It is assumed that all pollutant parameters are same for minimum, average and maximum flowrate. Only temperature is different for calculations because it is directly effect on the processes such as nitrification. Optimum operations parameters are found with trial and error and summarized in tables.

5.3.1 Carbon Removal

For the system only carbon removal the sludge age should be less than minimum sludge age where nitrification starts. The required sludge age is calculated as mentioned in section 3.

Minimum aerobic sludge age required for nitrification can be calculated using equation 3.1.

$Tss,aerob=SFx3.4x1.103^{(15-T)}$

Assuming minimum temperature (T) = 15 ^{o}C
Assuming safety factor (SF) =1.5
Minimum aerobic sludge age was calculated as
Tss, aerob:=5.1 days

Therefore it is need to keep the sludge age below 5.1 days through wasting more activated sludge than designed value. Biological tank configuration for carbon removal is given in Table 5.2 according the calculations.

Table 5.2 Biological tank configuration for carbon removal

BIOLOGICAL REACTOR:					
		Average	Lowest	Highest	Unit
Total volume	V_{AT}	2698	2698	2698	m^3
Denitrification share	V_D/V	0	0	0	%
Nitrification volume	V_N	2698	2698	2698	m^3
Denitrification volume	V_D	0	0	0	m^3
Dry matter suspended solids	SS_{AT}	3.00	3.00	2.90	kg/m^3
Temperature	T	20	15	25	°C
BOD_5 sludge load	$B_{SS,BOD}$	0.250	0.250	0.259	kg/(kg*d)
Sludge age	t_{SS}	4.8	4.5	5.0	d
Nitrogen nitrified	S_{NN}	0	0	0	mg/L
Nitrogen denitrified	S_{ND}	0	0	0	mg/L
Recirculation	RC	0	0	0	%

Total volume specified in Table 5.2 is the existing tank volume. Because of aim is only carbon removal all the tank volume is used with aeration and no recirculation is needed. Dry suspended solids concentration is found with the equations 3.24. According to the calculations keeping MLSS concentration under 3000 mg/L is provide the sludge age below 5.1 days.

Sludge production of the carbon removal system is calculated with the equation 3.15. Oxygen consumption of the system is calculated with the equation 3.20. Because of only carbon removal aimed there is no oxygen need for nitrification. The peak factor for carbon respiration is taken account 1.25 as specified for 6 days sludge age in the ATV-131 standard (ATV-DVWK-A 131E, 2000). The peak factor ammonium oxidation is taken account as standard value 2. Sludge production amount and oxygen consumption amount for the system are given in Table 5.3 and table 5.4.

Table 5.3 Sludge production from carbon removal

Sludge production:		Average	Lowest	Highest	Unit
...by carbon removal	$S_{Pd,C}$	1674	1801	1561	kg/d
...by biolog. P-removal	$S_{Pd,P}$	0	0	0	kg/d
...by extra C addition	$S_{Pd,x}$	0	0	0	kg/d
…by P-Precipitation	$S_{Pd,Prec}$	0	0	0	kg/d
Total	S_{Pd}	1674	1801	1561	kg/d

Table 5.4 Oxygen consumption from carbon removal

Oxygen uptake:		Average	Lowest	Highest	Unit
...for carbon removal	$OU_{d,C}$	2093	1906	2259	kg/d
...for nitrogen removal	$OU_{d,ND}$	0	0	0	kg/d
Total	OU_d	2093	1906	2259	kg/d
Peak factor C-respiration	fC	1.25	1.25	1.25	
Peak factor ammonium oxidation	fN	2	2	2	
Hourly uptake rate	OU_h	100.9	99.3	117.6	kg/h

5.3.2 Carbon Removal with Nitrification

Carbon removal with nitrification requires more oxygen than only carbon removal. Discharges limits can be reaching by differentiate the sludge age, recirculation factors for different temperatures. Sometimes it is difficult to escape denitrification especially in hot climatic conditions. Necessary sludge ages may decrease to 4 days in hot climatic conditions. The minimum sludge age which nitrification starts was calculated 5.1 days with equation 3.1.

Biological tank configuration to operate the plant for carbon removal with nitrification is given Table 5.5.

Table 5.5 Biological tank configuration for carbon removal with nitrification

BIOLOGICAL REACTOR:					
		Average	Lowest	Highest	Unit
Total volume	V_{AT}	2698	2698	2698	m^3
Denitrification share	V_D/V	0	0	0	%
Nitrification volume	V_N	2698	2698	2698	m^3
Denitrification volume	V_D	0	0	0	m^3
Dry matter suspended solids	SS_{AT}	3.70	3.70	3.70	kg/m^3
Temperature	T	20	15	25	°C
BOD_5 sludge load	$B_{SS,BOD}$	0.203	0.203	0.203	kg/(kg*d)
Sludge age	t_{SS}	6.2	6.1	6.7	d
Nitrogen nitrified	S_{NN}	43.0	43.0	43.0	mg/L
Nitrogen denitrified	S_{ND}	0	0	0	mg/L
Recirculation	RC	0	0	0	%

Because of nitrification occur in aerobic medium all the tanks volume is used for nitrification with carbon oxidation. Dry suspended solids concentration (MLSS) is found with the equations 3.24. The MLSS is higher than only carbon removal treatment options due to decreasing F/M ratio. If the MLSS is kept same as carbon removal options, we need more aeration volume. The founded MLSS is in the normal range that used for activated sludge processes. The sludge age is kept over 5.1 days with decreasing the waste activated sludge amount that can be calculated via using equation 3.18.

Sludge production is less than only carbon removal. Sludge treatment may be cause problem. Sludge may be denitrificate in any where anoxic conditions occur in treatment plant such as secondary settling tanks or sludge holding tank. Sludge rising can be seen because of uncontrolled sludge. Nitrogen gas is formed in sludge and rises the sludge. Rising sludge can be determined by watching the small gas bubbles on floating solids. Oxygen consumption of the system is calculated with the equations 3.20 and 3.21. Total oxygen consumption is the sum of oxygen requirement for nitrification and for carbon removal. The peak factor for carbon respiration is taken account 1.25 as specified for 6 days sludge age in the ATV-131 standard (ATV-DVWK-A 131E, 2000). The peak factor ammonium oxidation is taken account as standard value 2. Oxygen consumption is more than only carbon removal systems. Sludge production amount and oxygen consumption amount for the system are given in Table 5.6 and table 5.7.

Table 5.6 Sludge production from carbon removal with nitrification

Sludge production:		Average	Lowest	Highest	Unit
...by carbon removal	$S_{Pd,C}$	1598	1708	1481	kg/d
...by biolog. P-removal	$S_{Pd,P}$	0	0	0	kg/d
...by extra C addition	$S_{Pd,x}$	0	0	0	kg/d
…by P-Precipitation	$S_{Pd,Prec}$	0	0	0	kg/d
Total	S_{Pd}	1598	1708	1481	kg/d

Table 5.7 Oxygen consumption from carbon removal with nitrification

Oxygen uptake:		Average	Lowest	Highest	Unit
...for carbon removal	$OU_{d,C}$	2205	2043	2377	kg/d
...for nitrogen removal	$OU_{d,ND}$	1247	1247	1247	kg/d
Total	OU_d	3452	3290	3624	kg/d
Peak factor C-respiration	fC	1.25	1.25	1.25	
Peak factor ammonium oxidation	fN	2.00	2.00	2.00	
Hourly uptake rate	OU_h	195.8	189.0	202.9	kg/h

Because of nitrification consumes alkalinity; alkalinity decreases to less than 1 mmol/l during the treatment. The effluent alkalinity is calculated with equation 3.26. Alkalinity addition with alkali neutralization agents such as lime should be added if these processes selected. Organic nitrogen in the effluent is set 2 mg/L as rule value. Ammonia nitrogen in the effluent value is set 0 mg/L as rule value. Nitrate nitrogen set point in the effluent is selected 10 mg/L. Effluent values for the treatment options are given Table 5.8.

Table 5.8 Effluent values of systems for carbon removal with nitrification

EFFLUENT					
		Average	Lowest	Highest	Unit
Ammonia nitrogen	$S_{NH4,EST}$	0.0	0.0	0.0	mg/L
Nitrate nitrogen	$S_{NO3,EST}$	43.0	43.0	43.0	mg/L
Organic Nitrogen		2.0	2.0	2.0	mg/L
Phosphorus	$C_{P,EST}$	9.0	9.0	9.0	mg/L
Alkalinity	$S_{ALK,EST}$	0.94	0.94	0.94	mmol/l

5.3.3 Carbon Removal with Nitrification and Denitrification

Carbon removal, nitrification and denitrification processes are applied generally together in wastewater treatment plants where the discharge standards quality is more districts. In the Shahat Wastewater Plant pre-anoxic zone for denitrification can be made up easily for denitrification processes. The anoxic zone can be created by closing the some diffuser groups in the oxidation tank. Sludge age is an important factor for the process. Using very short sludge age will decreased the treatment efficiency and increases the sludge production. The required sludge age is calculated with equation 3.1. The safety factor is calculated wit the equation 3.2. According the standard safety factor is high and may cause problems during treatment (ATV-DVWK-A 131E. 2000). If denitrification is not completed during treatment the operator can overcome the problem by increasing the MLSS concentration or internal recirculation ratios. Amount of nitrogen nitrified and denitrified is calculated with the equation 3.3. Recirculation ratio is calculated with equation 3.6. Biological tank configuration for nitrogen removal options is given Table 5.9.

Table 5.9 Biological tank configuration of systems for carbon removal with nitrification and denitrification

BIOLOGICAL REACTOR:					
		Average	Lowest	Highest	Unit
Total volume	VAT	2698	2698	2698	m^3
Denitrification share	VD/V	40	30	40	%
Nitrification volume	VN	1619	1889	7200	m^3
Denitrification volume	VD	1079	809	4800	m^3
Dry matter suspended solids	SSAT	3.70	3.70	3.70	kg/m^3
Temperature	T	20	15	25	°C
BOD_5 sludge load	BSS,BOD	0.203	0.203	0.203	kg/(kg*d)
Sludge age	tSS	6.1	6.0	6.6	d
Safety factor	SF	1.76	1.23	3.09	
Nitrogen nitrified	SNN	43.0	43.0	43.0	mg/L
Nitrogen denitrified	SND	33.4	33.4	33.4	mg/L
Recirculation	RC	350	350	350	%

Sludge production of the system is calculated with the equations 3.16 and 3.17. The system needs less oxygen consumption than just nitrification and carbon removal and the discharged water is better. The oxygen consumptions for carbon removal, nitrification and recovery from denitrification are calculated with the equations 3.20 and 3.21. Sludge production amount and oxygen consumption amount for the system are given in Table 5.10 and table 5.11

Table 5.10 Sludge production of systems for carbon removal with nitrification and denitrification

Sludge production:		Average	Lowest	Highest	Unit
...by carbon removal	$S_{Pd,C}$	1604	1715	1487	kg/d
...by biolog. P-removal	$S_{Pd,P}$	30	30	30	kg/d
...by extra C addition	$S_{Pd,x}$	0	0	0	kg/d
…by P-Precipitation	$S_{Pd,Prec}$	0	0	0	kg/d
Total	S_{Pd}	1634	1745	1517	kg/d

Oxygen consumption is less about % 20 of carbon removal with nitrification process because some part of oxygen is to regain. If we think that in long time construction of more spaces is become cheaper than using extra oxygen and also gives us high quality treated water.

Table 5.11 Oxygen consumption of systems for carbon removal with nitrification and denitrification.

Oxygen uptake:		Average	Lowest	Highest	Unit
...for carbon removal	$OU_{d,C}$	2195	2033	2367	kg/d
...for nitrogen removal	$OU_{d,ND}$	593	593	593	kg/d
Total	OU_d	2788	2626	2960	kg/d
Peak factor C-respiration	fC	1.25	1.25	1.25	
Peak factor ammonium oxidation	fN	2.00	2.00	2.00	
Hourly uptake rate	OU_h	168.1	161.4	175.3	kg/h

Organic nitrogen in the effluent is set 2 mg/L as rule value. Ammonia nitrogen in the effluent value is set 0 mg/L as rule value. Nitrate nitrogen set point in the effluent is selected 10 mg/L. The plant capacity is not enough to denitrify all the nitrate nitrogen. Effluent values for the treatment options are given Table 5.12. . The effluent alkalinity is calculated with equation 3.26.

Table 5.12 Effluent values of systems for carbon removal with nitrification and denitrification

EFFLUENT					
		Average	Lowest	Highest	Unit
Ammonia nitrogen	$S_{NH4,EST}$	0	0	0	mg/L
Nitrate nitrogen	$S_{NO3,EST}$	9.6	9.6	9.6	mg/L
Organic Nitrogen		2	2	2	mg/L
Phosphorus	$C_{P,EST}$	7.5	7.5	7.5	mg/L
Alkalinity	$S_{ALK,EST}$	3.28	3.28	3.28	mmol/l

Comparison of the three treatment options for average temperature is given Table 5.13.

Table 5.13 Comparison of treatment options for average temperature

	Options	**Denitrification Volume m^3**	**Oxygen Consumption kg/d**	**Sludge Production kg/d**
Case 1a	C-Removal	0	2093	1674
Case 2	C-Removal + Nitrification	0	3452	1598
Case 3	C-Removal + Nitrification+Denitrification	1619	2788	1634

5.3.4 Extended Aeration

The extended aeration options can be applied only flow less than designed values because of it require more volume than other wastewater treatment options. In extended aeration systems F/M ratio is generally between 0.01 and 0.07 (Metcalf & Eddy, 2003). In Shatat Wastewater Treatment plant can be operated as extended aeration while the flow is equal or under 2810 m^3/d. To provide the low F/M ratio MLSS concentration increases to 4.5 kg/m^3. Sludge age is calculated after determination of MLSS concentration with the equation 3.18. Amount of nitrogen nitrified and denitrified is calculated with the equation 3.3. Recirculation ratio is calculated with equation 3.6. Biological tank configuration for extended aeration options when flow is 2810 m^3/d is given Table 5.14.

Table 5.14 Biological tank configuration of systems for extended aeration

BIOLOGICAL REACTOR:					
		Average	Lowest	Highest	Unit
Total volume	V_{AT}	2698	2698	2698	m^3
Denitrification share	V_D/V	20	30	30	%
Nitrification volume	V_N	2158	1889	1889	m^3
Denitrification volume	V_D	540	809	809	m^3
Dry matter suspended solids	SS_{AT}	4.50	4.50	4.50	kg/m^3
Temperature	T	20	15	25	°C
BOD_5 sludge load	$B_{SS,BOD}$	0.069	0.069	0.069	kg/(kg*d)
Sludge age	t_{SS}	21.8	20.6	22.7	d
Safety factor	SF	1.0	1.0	1.0	
Nitrogen nitrified	S_{NN}	43.0	43.0	43.0	mg/L
Nitrogen denitrified	S_{ND}	33.4	33.4	33.4	mg/L
Recirculation	RC	350	350	350	%

Organic nitrogen in the effluent is set 2 mg/L as rule value. Ammonia nitrogen in the effluent value is set 0 mg/L as rule value. Nitrate nitrogen set point in the effluent is selected 10 mg/L. Nitrate nitrogen denitrify capacity of the plan can be increase via increasing the anoxic partition and / or internal recirculation ratio. Effluent values for the treatment options are given Table 5.12. The effluent alkalinity is calculated with equation 3.26. Sludge production is about 33% less than of other treatment plants. Sludge production of the system is calculated with the equations 3.16 and 3.17. The oxygen consumptions for carbon removal, nitrification and recovery from denitrification are calculated with the equations 3.20 and 3.21. Sludge production amount and oxygen consumption amount for the system are given in Table 5.15 and Table 5.16.

Table 5.15 Sludge production of systems for carbon removal with nitrification and denitrification

Sludge production:		Average	Lowest	Highest	Unit
...by carbon removal	$S_{Pd,C}$	545	576	521	kg/d
...by biolog. P-removal	$S_{Pd,P}$	13	13	13	kg/d
...by extra C addition	$S_{Pd,x}$	0	0	0	kg/d
…by P-Precipitation	$S_{Pd,Prec}$	0	0	0	kg/d
Total	S_{Pd}	557	589	534	kg/d

Table 5.16 Oxygen consumption of systems for carbon removal with nitrification and denitrification.

Oxygen uptake:		Average	Lowest	Highest	Unit
...for carbon removal	$OU_{d,C}$	1097	1051	1131	kg/d
...for nitrogen removal	$OU_{d,ND}$	247	247	247	kg/d
Total	OU_d	1344	1298	1378	kg/d
Peak factor C-respiration	fC	1.15	1.10	1.10	
Peak factor ammonium oxidation	fN	1.60	1.60	1.60	
Hourly uptake rate	OU_h	69.0	69.2	70.4	kg/h

Effluent values for the treatment options are given Table 5.17.

Table 5.17 Effluent values of systems for carbon removal with nitrification and denitrification

EFFLUENT					
		Average	Lowest	Highest	Unit
Ammonia nitrogen	$S_{NH4,EST}$	0	0	0	mg/L
Nitrate nitrogen	$S_{NO3,EST}$	9.6	9.6	9.6	mg/L
Organic Nitrogen		2	2	2	mg/L
Phosphorus	$C_{P,EST}$	7.5	7.5	7.5	mg/L
Alkalinity	$S_{ALK,EST}$	3.30	3.30	3.30	mmol/l

CHAPTER SIX

CONCLUSIONS and RECOMMENDATIONS

6.1 Conclusions

In this study commissioning of the wastewater treatment units are examined with samples to provide for putting new wastewater treatment plant into operation, new lines into operation or changing the mode of plant's operation. During commissioning most of the parameters may be different than the designed or expected values. The possible problems and solutions that may happen during commissioning are examined in this study with the experience that owned on the commissioning period of the Piatra Neamt Wastewater Treatment Plant. The commissioning engineer must adopt the procedures according to the site conditions. The engineer must capable the change the current procedures and also find the true practical way to overcome the problems.

In the fifth chapter of the thesis Shahat Wastewater Treatment Plant, which is operated for only carbon removal in real, is simulated with four different treatment options. For each treatment options necessary parameters and results of treatment options are calculated and summarized.

These options are:

1. Carbon removal
2. Carbon removal + Nitrification
3. Carbon removal + Nitrification + Denitrification
4. Extended Aeration

The option 1 is aimed to remove only carbon. The option requires less oxygen than the other treatment options and produces less sludge than other treatment options. Although C removal uses less oxygen than other, it is not proper for most of countries because of strict discharge limits. Nitrogen compounds and phosphorus are not treated in the system. The sludge age should kept under 5.1 days to prevent nitrification but according the climate keeping sludge age is difficult because of hot temperatures up to 45 ^{0}C. Because of the required sludge age for nitrification decreasing to 3 days in summer times; escaping nitrification may become impossible. The operator may face up with sludge problem. If the operator decreases the sludge age below 3, carbon removal become insufficient and partial nitrification occurs. The sludge may denitrificate in the sludge tank. The effluent quality decreases and rising sludge problem occurs.

In options two; carbon removal with nitrification can be achieved with keeping the sludge age over 5.1 and MLSS concentration 3.7 kg/m^3. Like only carbon removal options; this treatment options has also cause sludge problems. The sludge must not keep waiting in the sludge holding tank and send to sludge treatment. Otherwise rising sludge problems occurs. The oxygen consumption of this treatment option is more than only carbon removal because oxygen consumes also for nitrification. Sludge amount is less than only carbon removal options. The alkalinity decreases during nitrification. If this options selected additional alkalinity should be added.

In option three, carbon removal with nitrification and denitrification can be achieved with sludge age 6.1 days and internal recirculation that 350 % of inlet flow. The MLSS concentration for this option is 3.7 kg/m^3. Sludge age should be more to keep denitrification in efficient point. According the calculations the capacity of the plant is not enough for full denitrification. As given in Table 5. 2 some of nitrate nitrogen is discharged. The options three requires less oxygen than carbon removal with nitrification and gives better quality. This option also provides some of phosphorus removal. This is the best treatment options for that can be applied for the plant.

In options four, extended aeration is applied to the treatment plant. In extended aeration is the best treatment option and gives best effluent quality. However these advantages it can apply to plant until the incoming raw water increases to 50 % of the design flow. Till the amount of incoming raw wastewater reach the design values, extended aeration can be applied completely or partially. The option requires less oxygen and produces less sludge than other systems. The sludge may not cause problems because it is stabilized. The plant is designed for year 2025 and now the inlet raw wastewater flow is less than half of the design flow. Extended aeration options can be used now with providing the necessary parameters of the options are given in Table 6.1. The operation parameters and calculated results according to the average temperature for four treatment options can be summarized in Table 6.1.

Table 6.1 The operation parameters and results for four treatment options that can applied to the plant.

Biological Reactor Configuration		Option 1	Option 2	Option 3	Option 4	Unit
Total volume	V_{AT}	2698	2698	2698	2698	m^3
Denitrification share	V_D/V	0	0	40	20	%
Nitrification volume	V_N	**2698**	**2698**	**1619**	**2158**	m^3
Denitrification volume	V_D	**0**	**0**	**1079**	**540**	m^3
Dry matter suspended solids	SS_{AT}	**3**	**3.7**	**3.7**	**4.5**	kg/m^3
BOD_5 sludge load	$B_{SS,BOD}$	0.25	0.203	0.203	0.069	kg/(kg*d)
Sludge age	t_{SS}	**4.8**	**6.2**	**6.1**	**21.8**	d
Nitrogen nitrified	S_{NN}	0	43	43	43	mg/L
Nitrogen denitrified	S_{ND}	0	0	33.4	33.4	mg/L
Recirculation	RC	0	0	**350**	**350**	%
Sludge production:						
...by carbon removal	$S_{Pd,C}$	1674	1598	1604	545	kg/d
...by biolog. P-removal	$S_{Pd,P}$	0	0	30	13	kg/d
…by P-Precipitation	$S_{Pd,Prec}$	0	0	0	0	kg/d
Total	S_{Pd}	**1674**	**1598**	**1634**	**557**	kg/d
Oxygen uptake:						
...for carbon removal	$OU_{d,C}$	2093	2205	2195	1097	kg/d
...for nitrogen removal	$OU_{d,ND}$	0	1247	593	247	kg/d
Total	OU_d	**2093**	**3452**	**2788**	**1344**	kg/d
Peak factor C-respiration	fC	1.25	1.25	1.25	1.15	
Peak factor ammonium oxidation	fN	2	2	2	1.6	
Hourly uptake rate	OU_h	100.9	195.8	168.1	69	kg/h
Alkalinity	$S_{ALK,EST}$	3.5	**0.94**	3.28	3.3	mmol/L

6.2 Recommendations

Although wastewater treatment plants different from each other their basic principles are same. In Turkey there is not enough information about commissioning of wastewater treatment plant. It will be useful a published detailed commissioning guidelines in mother tongue which prepared with experience of many plants.

To better understand the advantages and disadvantages of treatment options a full scale treatment plant may be operated with different treatment options. To reduce to oxygen consumption, sludge treatment costs and give the best treatment efficiency a balance can be found while commissioning of wastewater treatment plants with trying the treatment options.

The standards for wastewater treatment effluents are become strict. The existing plants may operate better treatment options. Operators should try the capabilities of the plants. Wastewater treatments plants capacities, equipments should be checked for better treatment options. Necessary revisions for better effluent should be determined. Until the wastewater treatment plants reached their design capacity, they should check for better operations and changed their process for better treatment efficiency.

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APPENDIX

ABBREVIATIONS

BOD	Biological oxygen demand
COD	Chemical oxygen demand
F/M	Food to Microorganism loading
HRT	Hydraulic Retention Time (process volume / input flow)
Qw	Waste sludge flow, wasting from aerator mixed liquor
SRT	Solids Retention Time (day)
TKN	Total Kjeldahl Nitrogen
TOC	Total organic carbon
VSS	Volatile suspended solids
WAS	Waste Activated Sludge
RAS	Return activated sludge
MLSS	Mixed-liquor suspended solids
SVI	Sludge volume index
ц	M□ □ □ □ □ aximum specific growth rate
b	decay rate coefficient for
Y	yield factor for
r	reaction rate factor for
ц$_{H}$	maximum specific growth rate for hetetrophic biomass
b_H	decay rate coefficient for hetetrophic biomass
Y_H	yield factor for hetetrophic biomass
r_H	reaction rate factor for hetetrophic biomass
ц$_{A}$	maximum specific growth rate for autotrophic biomass
b_A	decay rate coefficient for autotrophic biomass
Y_A	yield factor for autotrophic biomass
r_A	reaction rate factor for autotrophic biomass
K_S	organic substrate half-saturation coefficient
K_{NO}	nitrate half-saturation coefficient
K_{NH}	ammonia half-saturation coefficient
K_{OH}	oxygen half-saturation coefficient

k_h	maximum specific hydrolysis rate
k_a	ammonification rate
K_X	half-saturation coefficient for hydrolysis of slowly biodegradable substrate
V_{AT}	Total volume of aeration tanks
V_D/V	Denitrification share
V_N	Nitrification volume
V_D	Denitrification volume
S_{SAT}	Dry matter suspended solids
T	Temperature
$B_{SS,BOD}$	BOD5 sludge load
tSS	Sludge age
SF	Safety factor
S_{NN}	Nitrogen nitrified
S_{ND}	Nitrogen denitrified
RC	Recirculation
$S_{Pd,C}$	Sludge production by carbon removal
$S_{Pd,P}$	Sludge production by biological P-removal
$S_{Pd,x}$	Sludge production by extra C addition
$S_{Pd,Prec}$	Sludge production by P-Precipitation
$O_{Ud,C}$	Oxygen uptake for carbon removal
$O_{Ud,ND}$	Oxygen uptake for nitrogen removal
O_{Ud}	Total oxygen uptake
f_C	Peak factor C-respiration
f_N	Peak factor ammonium oxidation
O_{Uh}	Hourly uptake rate
$S_{NH4,EST}$	Ammonia nitrogen
$S_{NO3,EST}$	Nitrate nitrogen
$C_{P,EST}$	Phosphorus
$S_{ALK,EST}$	Alkalinity