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Performance Evaluation of Gaza Waste water Treatment Plant

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ABSTRACT

*Waste*water *treatment plant at Gaza and its infiltration basins are considered the primary sources of pollution for the ground water and sea water. The historical records of Gaza Wastewater Treatment Plant from January 2000 to October 2004 were collected and evaluated to assess the system performance in term of removal efficiency for the Bio-Filter-Aerated Lagoon combined system. This study attempts to highlight the factors leading to the inadequate performance of GWWTP in removing organic matter, solids, nutrients and microorganisms. The efficiency of the different stages of the treatment process and the overall plant performance has been demonstrated.*

Collection and analysis of available historical data revealed a constantly decreasing removal efficiency of biochemical oxygen demand (BOD), chemical oxygen demand (COD), total suspended solids (TSS) and fecal coli form (FC). Analysis of data showed that the combined system removal reached 91%,87%,88%,97.5% and 21.5% of the influent BOD_5*,COD,TSS,FC and TKN respectively with an effluent concentration of 35.2 mg/l,110 mg/l,53 mg/l,7.73E+6 and 69.2 mg/l. The decrease in the removal efficiency of the pollutants appeared to be due to the increase in the hydraulic and organic load that exceeded treatment plant designed capacity, inaccurate design parameters and inadequate operation.*

Over hydraulic and organic loading, increasing values of TSS than normal values, inadequate design of the plant and inadequate plant management are the main factors behind the drop in the treatment system performance. The study presents proposals like the construction of another anaerobic lagoon and a grit chamber and splitting the flow equally between the trickling filters and the aeration lagoon along with the construction of another final settling tank which can be introduced as short, simple and economic solutions to overcome and improve the inefficient and poor treatment plant performance.

الخلاصة

تعتبر محطة معالجة المياه العادمة لمدينة غزة وأحواض الترسيب الملحقة بها المصدر الرئيسي لتلوث مياه البحر والخزان الجوفي. لقد تم تجميع وتحليل السجلات التاريخية السابقة لمحطة معالجة المياه العادمة من يناير 2000 وحتى أكتوبر 2004 وذلك لعمل دراسة لكفاءة أداء المحطة من حيث كفاءة إزالة الملوثات للنظام المركب من الأحواض الهوائية والمصفى (الفلتر) البيولوجي. وتسلط هذه الدراسة الضوء على العوامل التي تؤدي إلى خفض كفاءة تشغيل محطة معالجة المياه العادمة لمدينة غزة من حيث المواد العضوية، والمواد الصلبة، والمغذيات والأحياء الدقيقة. وقد تم استعراض كفاءة كل مرحلة من مراحل المعالجة المختلفة على حدة وكذلك الكفاءة الكلية للمحطة ككل.

أظهرت نتائج تجميع وتحليل البيانات التاريخية المتوفرة تراجعا ثابتا في كفاءة إزالة الأكسجين المستهلك حيوياً والأكسجين المستهلك كيماويا والمواد الصلبة العالقة بالإضافة إلى البكتيريا القولونية. ومن خلال تحليل البيانات، أزال نظام المعالجة في محطة غزة 91% من الأكسجين المستهلك حيوياً، و87% من الأكسجين المستهلك كيماوياً، و 88% من المواد الصلبة المعلقة، و 97.5% من البكتيريا القولونية و 21.5% من نيتروجين كلدال الكلي، وقد كان تركيز هذه المواد في المياه الخارجة من المحطة بعد عملية المعالجة خلال فترة الدراسة على الترتيب كالتالي: 35.2 ملجم/لتر ، 115 ملجم/لتر ، 53 ملجم/لتر ، 73 X 106 وحدة / 100 مل ، 69.2 ملجم/ لتر. ويعود السبب في انخفاض كفاءة إزالة الملوثات إلى زيادة الحمل الهيدروليكي والعضوي اللذان يتعديان قدرة المحطة، بالإضافة إلى الأخطاء في معطيات التصميم والتشغيل.

وقد خلصت الدراسة إلى أن زيادة الحمل الهيدروليكي، وازدياد قيم المواد الصلبة العالقة عن حدودها المعتادة، وعدم دقة التصميم وكذلك الأخطاء في إدارة وتشغيل المحطة هي العوامل الرئيسة التي أدت إلى انخفاض أداء نظام المعالجة. وقد قدمت الدراسة بعض الاقتراحات مثل إنشاء حوض آخر لاهوائي، وإنشاء قناة إزالة أتربة وتوزيع المياه بشكل متساو بين الفلاتر العضوية وحوض المعالجة الهوائية مع إنشاء حوض ترسيب نهائي جديد كحلول اقتصادية بسيطة وقصيرة المدى لرفع كفاءة المحطة وتحسين أدائها.

DEDICATION

*I would like to dedicate this **thesis** to my parents to whom I owe everything since I was born.*

*Also, this **thesis** is dedicated to my wife who supported and encouraged me at all stages of my study, and for my beloved suns: Khalid and Basil, and beloved daughters Asil and Lama who have been a great source of motivation and inspiration.*

*Finally, this **thesis** is dedicated to my brothers, sisters, friends, colleagues at the Coastal Municipalities Water Utility, and all those who believe in the richness of learning.*

Majed M.Ghannam

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LIST OF ABBREVIATIONS AND ACRONYMS

^0C	**Degrees Celsius.**
Cl	**Chloride**
BOD_5	**5 days Biochemical Oxygen Demand.**
CAMP	**Coastal Aquifer Management Program.**
cfu	**Colony Forming Units**
COD	**Chemical Oxygen Demand**
Conc.	**Concentrated**
EQA	**Environment Quality Authority.**
FC	**Faecal Coliform.**
GIS	**Geographic Information System**
GWWTP	**Gaza Wastewater Treatment Plant**
h	**Hour**
hp	**Horse Power**
KfW	**The German Bank of International Development**
KW	**Kilo Watt**
L	**Liter**
L_v	**Volumetric Loading**
Lab	**Laboratory**
l/c/d	**Liter Per Capita Per Day**
m^3	**Cubic meter**
mm	**Millimeter**
MCM	**Million Cubic Meter**
m^3/h	**Cubic Meter Per Hour**

mg/l	**Milligrams per Liter**
MOG	**Municipality Of Gaza**
MOPIC	**Ministry Of Planning and International Corporation.**
NH_3	**Ammonia**
NO_3	**Nitrates**
PCBS	**Palestinian Central Bureau of Statistics.**
ppm	**Parts Per Million**
P.S.	**Pumping Station**
PWA	**Palestinian Water Authority**
TDS	**Total Dissolved Solids**
Temp.	**Temperature**
TKN	**Total Kjeldhal Nitrogen**
TSS	**Total Suspended Solids**
UNDP	**United Nations Development Program**
UNRWA	**United Nations Relieve and Work Agency**
U.S $	**United States Dollar**
USAID	**Unites States Agency for International Development**
WHO	**World Health Organization**
WWTP	**Wastewater Treatment Plant**

CHAPTER (1): INTRODUCTION

1.1 Background

Gaza Strip (GS) has a coastline of 40 km at the eastern extreme of the Mediterranean and on the edge of the Sinai Desert. GS has a total area of 365 square kilometers (MOPIC, 1998) and the population is estimated to be around 1,500,000 people.

There are more than 7000 wells (PWA, 2004) existing in Gaza strip most of which is privately owned and used for agricultural purposes. In most regions, over pumping has resulted in extractions greatly exceeding replenishment which in turn has caused the saltwater interface to move inland (CAMP,2001). Continuing over pumping for agricultural and domestic use will deplete the fresh water aquifer resulting in its replacement with sea water. The extractions for agricultural purposes now exceed 80 million cubic meters (MCM) per year, while rainfall replenishment is only about 40-50 Mcm per year (Al-Jamal, K and Yaqubi, 2003) Agricultural return flows are about 20 MCM and often carry pesticides and nitrates, a major pollutant in the upper levels of the aquifer (CAMP, 2001). Also, sea water intrusion has caused the closing of many wells (CAMP, 2001)

Surface water in Gaza strip consists mainly of wadi Gaza, which originates in the Negev Desert, is the major one. Its catchments area is about 3500 km^2 (CAMP, 2001). The estimated average annual flow of Wadi Gaza is 20 to 30 MCM (PWA, 1997). When surface runoff occurs, it occurs during a limited number of days. Presently, surface water resources are not anymore available in the Gaza Strip due to Israeli violations.

Groundwater in the Gaza Strip is a confined Pleistocene age costal aquifer and divided into three sub-aquifers composed mainly of sandstone and pebbles (PWA, 1997). The sub-aquifers overlie each other and are separated by impervious and/or semi-pervious clay and clayey layers. The thickness of the aquifer in eastern boundary is about 10 m and increasing gradually to about 150 m at the cost. The pumping test results indicate that the aquifer is highly permeable with a transmissivity of about 1000 m^2/day and an average porosity of 25% (PWA, 1997). Depth to the water level ranges from 8 to 90 meters in West and the East of the Gaza Strip respectively (PWA, 1997).

Thirty years of deteriorating infrastructure and negligence, over the period 1967-1994, lead to inadequate investment in the various environmental sectors, particularly water, wastewater, sanitation and solid waste (Kelly & Dioxin, 1995). Overpopulation is also a major challenge that creates additional pressure, especially on the limited natural resources in the area and has a profound impact on the quality of health and social life of people (Coad, 1997; MOH, 2002). During the period some of existing infrastructure deteriorated while the population and their needs rapidly increased. This leads to environmental degradation on almost every aspect (El-Hawi, M. and Hamilton, A. 2001). Quality of the groundwater is a major problem in Gaza strip. Access to sewerage facilities, at present, varies from areas to areas. On average, it is estimated that about 60% of the population is connected to a sewerage network (UNEP, 2003). Cesspits and boreholes are the other wastewater disposal systems in the area.

Gaza Governorate is one of the five GS Governorates with a current population around 530,000 (PCBS, 2005) people. The Municipality of Gaza (MOG) jurisdictions comprises about 35square kilometers (USAID, 1996). The population growth has been rapid in Gaza strip and particularly in the governorate of Gaza. The annual water production from the municipal water wells of Gaza is 30,000,000 m^3 (PWA, 2005), and the average daily flow of waste water is estimated to be 55,000 m^3 (DORSCH CONSULT, 2005).

1.2 Wastewater treatment facilities in the Gaza Strip

There are three wastewater disposal and treatment facilities in Gaza strip, Beit Lahia (BLWWTP), Gaza City (GWWTP) and Rafah (RWWTP), but none is functioning effectively (MOPIC, 1998). They are not sophisticated treatment technology, its consist of anaerobic lagoons, aerated lagoons and maturation ponds. GWWTP is the only treatment facility which has trickling filters. The effluent from Gaza and Rafah treatment plants is mostly discharged into the Mediterranean Sea. In the case of the Beit Lahia wastewater treatment plant, a substantial quantity of wastewater infiltrates into the ground, contaminating soil and groundwater in the area. High level of nitrate has recently been detected from the aquifer, and it is most likely that the excess effluent is responsible for the deterioration of the water quality of the aquifer (Abu-Jalalah, 1999).

The GWWTP is located in Sheikh Ejleen Area southern Gaza city; this area is known to produce the most famous grapes in Gaza strip and may be in whole Palestine. The area, where the plant is located is owned by the MOG totaling around 120 dunums including the infiltration basins and most of this area is bought by the MOG from private owners. Before the Israeli disengagement in 12/09/05, there was an Israeli settlement just 100m from the treatment plant called Netzarim, and now after the disengagement this area is expected to be developed for the harbor stores **.**

As the level of treatment in GWWTP continued to deteriorate, the pollution in Wadi Gaza became a problem. This was worsened by wastewater flows from the middle area. To alleviate the problem in 1994, UNRWA and the municipality of Gaza upgraded the treatment plant by removing sand and sludge from the lagoons, upgrading their physical conditions and altering the treatment process to adequately treat around 12,000m^3 of influent daily.

In 1998, the United State Agency for International development (USAID) upgraded GWWTP to receive influent quantity up to 32,000 m^3 daily in 2005 (USAID, MOG, 1997) However, the system is overloaded and the hydraulic flows exceed the planning schedule. The GWWTP receive new an estimated effluent of more than 55,000 m^3/day (DORSCH CONSULT, 2005).

1.3 Problem Identification

The upgrading of GWWTP conducted by the USAID in 1998 increased the capacity of the plant to receive 32,000m^3/day by 2005. The records of the plant showed that the influent quantity reached the maximum hydraulic designed loads as the plant started to operate after the upgrading in 1998 (MOG lab, 1999). Due to the natural increase in population and connecting the non sewered areas to the sewage network, the effluent reached more than 55,000m^3 in the year 2004 (MOG lab, 2004). This was reflected negatively on the treatment capacity of the system and as a result the quality of the effluent deteriorated with BOD_5 effluent of more than 60mg/l (DORSCH CONSULT, 2005). The final discharge points of the treatment plant effluent are the sea beach and the infiltration ponds south of Gaza City. The environment and public health has been affected by the current trends and practices of wastewater disposal and treatment due to the poor quality of the effluent. The biological seawater quality exceeds the WHO standard in most locations of Gaza beach (Afifi, 1998) and health impact indications were observed on beach users

(Elmanama,A,2004). In addition, the water quality of agricultural and municipal wells located nearby the infiltration ponds showed a great increase in nitrates concentration (PWA, 2004).

The limited hydraulic capacity of plant has forced the Municipality of Gaza (MOG) to divert the flow from two pumping stations (P.S2 & P.S3) directly to the beach; pumping raw sewage into the seashore which was has resulted in increase the environmental and health impacts in the discharge points. In addition, the overflow of the initial sedimentation ponds, due to over hydraulic loading, has caused a tremendous damage to the agriculture areas around the plant leading to great economic losses for the farmers.

The study seeks to identify and evaluate the limiting treatment factors of GWWTP to produce effluent that reduce the environmental and health effects on the area. The overall aim of the study is to evaluate the current performance of GWWTP and to highlight the problems hindering GWWTP to produce the best possible effluent quality and to propose solutions for these problems that will be reflected positively on environment, health, economy and the service level.

1.4 Objectives

The objectives of this study are:

- Evaluate and document the treatment system performance in respect to the removal efficiency of biochemical, chemical and biological parameters.
- Determine the limiting factors leading to inadequate treatment system performance
- Determine and recommend economic and short term solution needed for optimal operation and maximum waste removal

1.5 Methodology

1.5.1 Data Collection

The monthly and daily reports of GWWTP laboratory were collected for the years: 2000,2001,2002,2003 and 2004. The laboratory conducts physical, chemical and biological parameters on the influent and effluent of water received by GWWTP through an automatic refrigerated sampler for 24 hours daily. The following parameters were performed:

1. Temperature which was measured by a thermometer at the sampling point.

2. pH which was measured by a pH meter at various points throughout the treatment plant.
3. Settleable solids (SS) which was measured using an Imhoff cone.
4. Total Solids (TS) which was measured using dry oven at 105 C^o.
5. Volatile Total Solids (VTS) which was measured using burning oven at 550 C^o.
6. Suspended Solids (SS) which was measured using dry oven at 105 C^o.
7. Volatile Suspended Solids (VSS) which was measured using burning oven at 550 C^o.
8. Dissolved Solids (DS) which was measured using dry oven at 105 C^o.
9. Chemical Oxygen Demand (COD) which was measured by micro COD apparatus using potassium dichromate.
10. Biochemical Oxygen Demand (BOD_5) was measured with OxiTop measuring system.
11. Ortho-phosphorous which was determined using ascorbic acid colorimetric method.
12. Total Coli form TC: For estimation of FC bacterial populations, the Membrane Filtration (MF) technique is performed.

1.5.2 Analysis of Data

All the data obtained from monthly and daily reports were entered as Microsoft Excel sheets and results were arranged in tables or figures.

1.5.3 Assessment of Data Reliability

The data collected from the records of GWWTP have three sources of error; human error, instrument accuracy and sampling procedure. The data reliability was examined and the sources of error were highlighted. In the following points, the main sources of error are discussed briefly. .

1.5.4 Human Error

The sources of this type of error are the laboratory technicians who are collecting, transporting and analyzing the samples. As mentioned before the composite samples are collected by an automatic refrigerated sampler through 24 hours a day, but the calibration of the sampler on the level of the water on which the samples are collected is performed by the operators and sometimes when the level is too low the values for BOD and TSS are very high reflecting a misleading indication for the water sample and the contrary is correct, when the sampler is raised too much, the values are too low for BOD, and TSS. Some times the samples are collected at a point where the sample stream is not well mixed. The samples are not mixed through the collection and testing which leads to some error in the results. The human error arises also in taking the readings and weights during performing the tests inside the laboratory.

1.5.5 Instruments' Accuracy

The age of GWWTP laboratory now is ten years and these instruments were maintained more than once and calibrated, but when operated it indicates some errors. Recent cross checking for the laboratory results with other laboratories showed deviations in the results which questions the accuracy of these instruments.

1.5.6 Sampling Procedure

The collected composite samples were time composite of a fixed volume, but taking in consideration the night flow for quality and quantity wise is much different than the morning flow. A variable volume technique for forming the composite sample is the most practical with a manual sampling method.

1.5.7 Reliability of Data

Based on the data analysis, it has been observed that the average values were in general accepted. Some of the data is subject to the errors mentioned before such as the values of the BOD which indicates a grip sample or some error in taking the sample by the sampler. Some error arises in the values of the influent parameters is due to the dilution of storm water connected to the sewerage system which runs to GWWTP. The relation between the average values of BOD, TSS and COD is identical to the theoretical relations between these parameters which indicate a strong reliability of the GWWTP lab analysis.

1.5.8 Data Filtration

During the processing of the data collected throughout the five years, some values of the parameters highlighted by this research were much higher or lower than the average or expected values. The average monthly values for the BOD, COD, TSS and TKN are shown in table 4.1 below. Values less than the average minimum mentioned in the table are not considered in this research since their reliability are in question and the same applies for values greater than double the values of the average monthly values of the studied parameters. The values which are considered to be lower than minimum are replaced by the minimum average value for that parameter and the values greater than the double average are replaced by the double the average values.

CHAPTER (2): STUDY AREA

2.1. Location and Population

The Municipality of Gaza (MOG) is located in the mid-north of Gaza strip and it is a separate Governorate as shown in map (1), with a total area of 40 square kilometers (MOPIC, 1998). The Gaza City population is estimated to be around 500,000 people, about two third of them are refugees. All governmental institutes and international organizations headquarters are located in Gaza city.

2.2 Administration

Prior to the year 1948, the MOG had a population of about 75,000 inhabitants with a limited infrastructure **(USAID, 1997).** The mass of refugees resulting from the 1948 war caused crowded housing conditions, and thus massive drainage problems to the city. After the war, the Egyptian Government Administration took responsibility for the Gaza strip, who undertook limited sewer construction activities within the area of MOG in an attempt to keep up with the rapidly growing population. The 1967 war brought Israeli occupation and control to Gaza strip. Infrastructure control, including planning and development, was under Israeli Civil Administration. The population of MOG was estimated to be about 117,000 by the year 1967. After the establishment of the Palestinian Authority in 1994, the first most accurate survey for population was performed by the Palestinian Central Bureau of Statistics **(PCBS)** which showed the population of Gaza governorate was 357,768 in mid 1994, and according to the PCBS report on December 1999 the population projection for the year 2005 is 516,882 inhabitants and it is expected to reach 650,033 by the year 2010 **(PCBS, 1999)**.

Nowadays, the Governorate of Gaza is divided into nine main areas: Turkman, Judeidah, Al-Daraj, Al-Sabrah, Northern Remal, Southern Remal, Tal El-hawa, al-Zaitoon, and the Beach Camp. The Ministry of Local Governorates is responsible for the administration of the local Municipalities within the territories of the Palestinian Authority (PA) which was created after Oslo Agreement in 1994, which signed between the Palestinian Liberation Organization (PLO) and Israeli. The Municipality has municipal counsels, which formed by Presidential Decrees to supervise the administration of municipalities. The municipal counsel of Gaza municipality consisted of seven members.

The responsibility for the development, operation and maintenance of all wastewater and storm water drainage system within the MOG is fully under the direction of the Mayor. Similarly, the Municipality is also responsible for water supply. Operation and maintenance costs are covered by the MOG annual budget which is supplemented by user fees assessed on water use. Such fees include a surcharge for wastewater services. Funding support from donor program is being obtained directly through National Ministries and used primarily for new projects and for rehabilitation of existing works. However, some efforts are being dedicated to operations, maintenance and training activities.

The department of water and wastewater in the MOG consists of four main sub-departments: Water (wells and networks), Wastewater (networks and pumping stations), Treatment (wastewater treatment plant- GWWTP) and Maintenance (electrical and mechanical). The GWWTP consists of two sections: operation and laboratory. The operation section is responsible for the daily operations of the plant and to monitor the performance of the different mechanical facilities in the plant and to record the daily activities while the laboratory is responsible for monitoring the quality of influent and effluent coming to the plant or discharging to the sea or infiltration ponds.

In 2000, the MOG along with 25 Municipal Counsels signed a memorandum of understanding with the Palestinian Water Authority to consolidate the water services in all the 26 municipalities in one single water utility called Coastal Municipalities Water Utility (CMWU). The establishment of the CMWU was one of the major reforms adopted by Palestinian Water Authority (PWA) in water sector and it became a major demand from the donors to cooperate with the PA. The Board of the Utility has also been nominated. The Minister of Local Government, as part of his mandate, issued a Decree of CMWU establishment under the Local Government Law (CMWU Quarter Report, 2005)

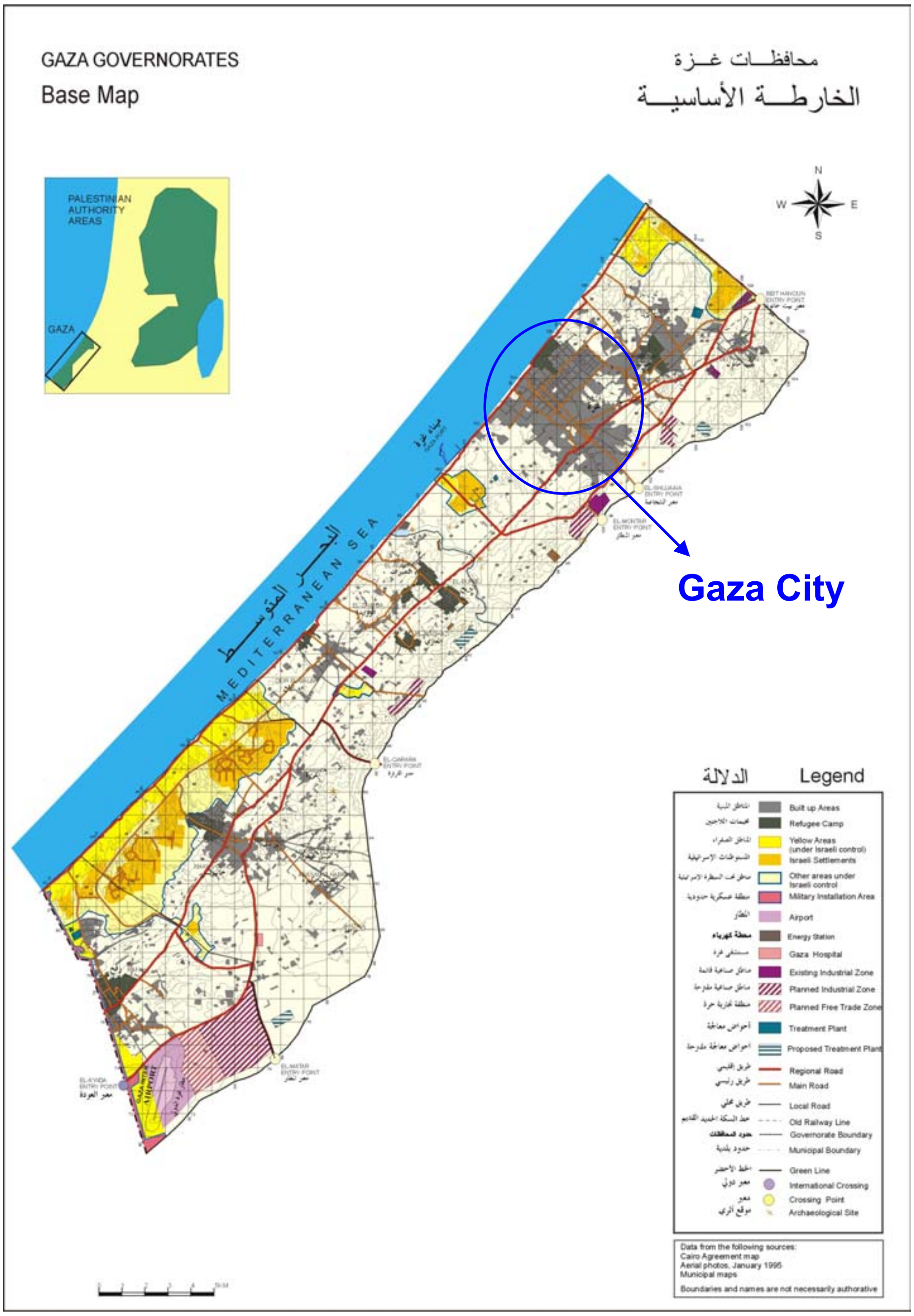
In April, 2005 a Management Contract was signed between CMWU and a consortium of an Austrian company and a Saudi company (joint venture) to operate the Gaza Emergency Water Project (GEWP) financed by the World Bank which will lead to the activation of the CMWU on the ground as a responsible body for the water services in all Gaza strip. Today, the Operator (Inframan) is responsible for the maintenance and operation of GWWTP (CMWU Quarter Report, 2005).

2.1.3 Climate

The climate in the Governorate of Gaza is typical Eastern Mediterranean with hot dry summers and mild winters. Rainfall average about 425 mm annually based on 35 years record (USAID, 1997). The average daily mean temperature is 25 degrees centigrade in the summer and 13 degrees centigrade in the winter (MOPIC, 1998). During the hot summer season, the daily maximum temperature generally exceeds 30 degrees centigrade and the maximum relative humidity exceeds 90%. Winds prevail from the northwest in the summer, with velocities up to 3.9 m/s (USAID, 1997). During the winter, the most frequent wind direction is southwest and average velocity is about 4.2 m/s **(USAID, 1997)**.

2.1.4 Land Ownership and Land Use

Historically Gazans have generally had the freedom and opportunity to own and develop their own lands. This trend continues today and land is one of the most important commodities in Gaza with values rating from $200 to more than $1000 per square meter within the city limits. These prices are high, even by industrialized country standard, and will influence the land use in the MOG area. The development in Gaza increased after the Oslo accord by both private owners and donors. Returning residents and refugees have added to this rapid development rate. Even during the *Intifada*, investment in land remained high. Today growth is occurring at a rapid and uncontrolled pace. There are no planning controls in place that effectively direct growth or control the type of use. The Ministry of Planning (MOP) has developed general plans for the Gaza strip defining target uses such as agriculture, industry, and public facilities including locations of a future harbor and the regional wastewater treatment sites. This general plan assumes that growth within the city limits will continue.



Map (1): Location of Gaza city and existing treatment facilities.

2.2 Sewerage system and coverage

The sewer system in Gaza City is reported to date back to a Roman drainage system; however, the extent and location of this early history are unknown. Such a system was probably used for the removal of both storm flows and sewage **(USAID, MOG, 1996).** Over the last 30 years, the sewage network was constructed primarily by the MOG while under various authorities and controls. Records, such as as-built drawings were not usually developed or kept, nor was maintenance undertaken except for emergencies. UNRWA addressed the need for cleaning and repairs of a key portion of the system. A critical part of this work was a detailed inspection of the network and recording of the physical as-built data of the network. These as-built conditions were combined with available city records and survey data collected for the Master Plan to create a detailed as-built record of the network. The sewer network covers around 75% of the total area of the MOG and around 90% of the population with a total number of sewage subscribers around 26,000 subscriptions in the Gaza Governorate **(PWA, Fiscal Report, 2004)**.

2.3 The Gaza Wastewater Treatment Plant

This section describes the Gaza Wastewater Treatment Plant demonstrating the development of the original design, the existing facilities of GWWTP and the prevailing conditions of the plant. The ongoing and planned activities of GWWTP are then produced along with the environmental effects of plant on the aquifer and habitat. This section concludes with a description of the treatment process scheme.

- ***Hydrogeology of Plant Location***

The Gaza Strip is essentially a foreshore plain gradually sloping westwards, and underlain by a series of geological formations. The area within MOG consists mainly of sands dunes in undulating formations, interspersed with clay lenses. Some areas have relatively deep layers of clay soils **(USAID, 2001)**. These are experienced mostly in Al-Zeiton and Al-Tofah catchments while the coastal zone is primarily sand. Groundwater levels in the city ranges from 1.0m to 2.0m relative to mean sea level and the fresh water aquifer under the city is reported to have thickness of up to about 90m (**USAID, 1996)** The GWWTP is situated on a hill with elevation of 44.2masl in Sheikh Ejleen sand dune area with a percolation rates in the range of 8.6m/day **(USAID, 1997).**

- **Existing GWWTP Facilities**

The existing GWWTP includes on-site treatment facilities, as well as off-site infiltration basins. All flow is pumped to the site through three force mains. An effluent pump station and pipeline are used to transport effluent from the main plant to the infiltration basins or to the Wadi Gaza. An existing force main/gravity line is also used to discharge effluent to the sea.

The main plant facilities consist of three anaerobic ponds in series followed by an aerated lagoon and two bio-towers. The anaerobic lagoons are heavily loaded compared to typical design recommendations. Limited dredging is practiced in the first of the three anaerobic lagoons. The aerated lagoon is equipped with floating mechanical aerators and the bio-towers are filled with high-density plastic media. The bio-tower effluent is directed to an effluent polishing pond where solids sedimentation occurs and limited solids collection is possible. Solids are removed by a series of draft tubes. The sludge is directed to un-aerated solid holding ponds. Polishing pond effluent is pumped to the off-site infiltration basins, Wadi Gaza or to the sea. Three basins are used for infiltration. The plant has a sodium hypochlorite disinfection system used for the effluent which is directed to the sea. Solids dredged form anaerobic pond 1 and biological solids from the sludge holding pond are directed to the on-site sludge drying beds for dewatering. The following point gives short description to the main existing treatment facilities of GWWTP **(CAMP,2001)**

- ***Influent Structure***: The structure consists of a side inlet structure accepting the 900mm force main from Pumping Station No.7B, and a main box inlet structure which accepts the flow from the remaining two pumping stations: Pumping Station No.1 and Pumping Station No.6A.
- ***Anaerobic Ponds*****:** The anaerobic ponds include two initial anaerobic ponds (1 & 2) each with volume of 22,000m^3. The two ponds can be operated in series or parallel. The third anaerobic pond (3) has a volume of 32,000m^3.
- ***Aerated Pond*****:** The aerated pond includes ten 25hp floating surface aerators and six 50hp units. Each of the 16 aerators is fixed in the pond by cables and anchors. The pond has the total capacity of 45,000 m^3**.** The effect of the number of the aerators will be shown and discussed in chapter 4 (Results& Discussion).

- ***Bio-Tower Feed Pump Station*****:** A submersible pumping station pumps aerated pond effluent plus recycle to two high-rate bio-towers. Four 60hp pumps, each rated at 667m^3/hr, are available**.**
- ***Bio-towers***: The two high rate bio-towers are 27 m in diameter with 7.3 m of media depth. The units operate in parallel and are designed for 85% BOD_5 removal. Countercurrent natural ventilation openings are provided at the base of the units around the circumference on a 45-degree center. Bio-tower effluent drains to the downstream settling pond through a 1,000mm pipe.
- ***Bio-Tower effluent Distribution Chamber*****:** This structure divides effluent flow from the bio-towers proportionally between the settling pond and recycle back to the bio-towers. It also allows bypass of bio-tower effluent to the existing effluent pump station. Stop gates provide for six recycle rates ranging from 20%-67% of the bio-tower effluent.
- ***Effluent Polishing Pond*****:** The pond is divided by a concrete wall creating a settling pond and a chlorination contact zone. The settling pond is 13 m wide by 83 m long concrete hopper-bottom settling zone. Sludge is removed through 150 mm suction pipe draped along the existing 3:1 slope. The suction manifold is connected to a diesel pump. The pump discharges sludge into the sludge holding pond. The suction drop pipes are spaced on 4 m centers and each includes a plug valve located at the top of the beam so the suction pipes can open individually.
- ***Sludge Holding Pond*****:** This un-aerated pond is used as an anaerobic sludge holding pond with a total capacity of 10,700m^3.
- ***Effluent Pump Station***: A submersible pump station is constructed in the beam of the effluent polishing pond. It consists of two wet wells, each containing two 60hp and 1,000 m^3/hr submersible pumps.
- ***Chlorination Facility***: Sodium hypochlorite storage and dosing equipment are provided, but are not currently in use.
- ***Effluent Pipeline***. A 600mm pipeline can deliver plant effluent to the infiltration basins or to the Wadi Gaza. A separate pipeline can carry effluent to the sea.
- ***Infiltration Facilities***. Effluent reuse facilities consist of three infiltration ponds with a total area of 37,000 m^2, 5,000 m^3 storage tank and 2,000 m^3/hr booster pump station.

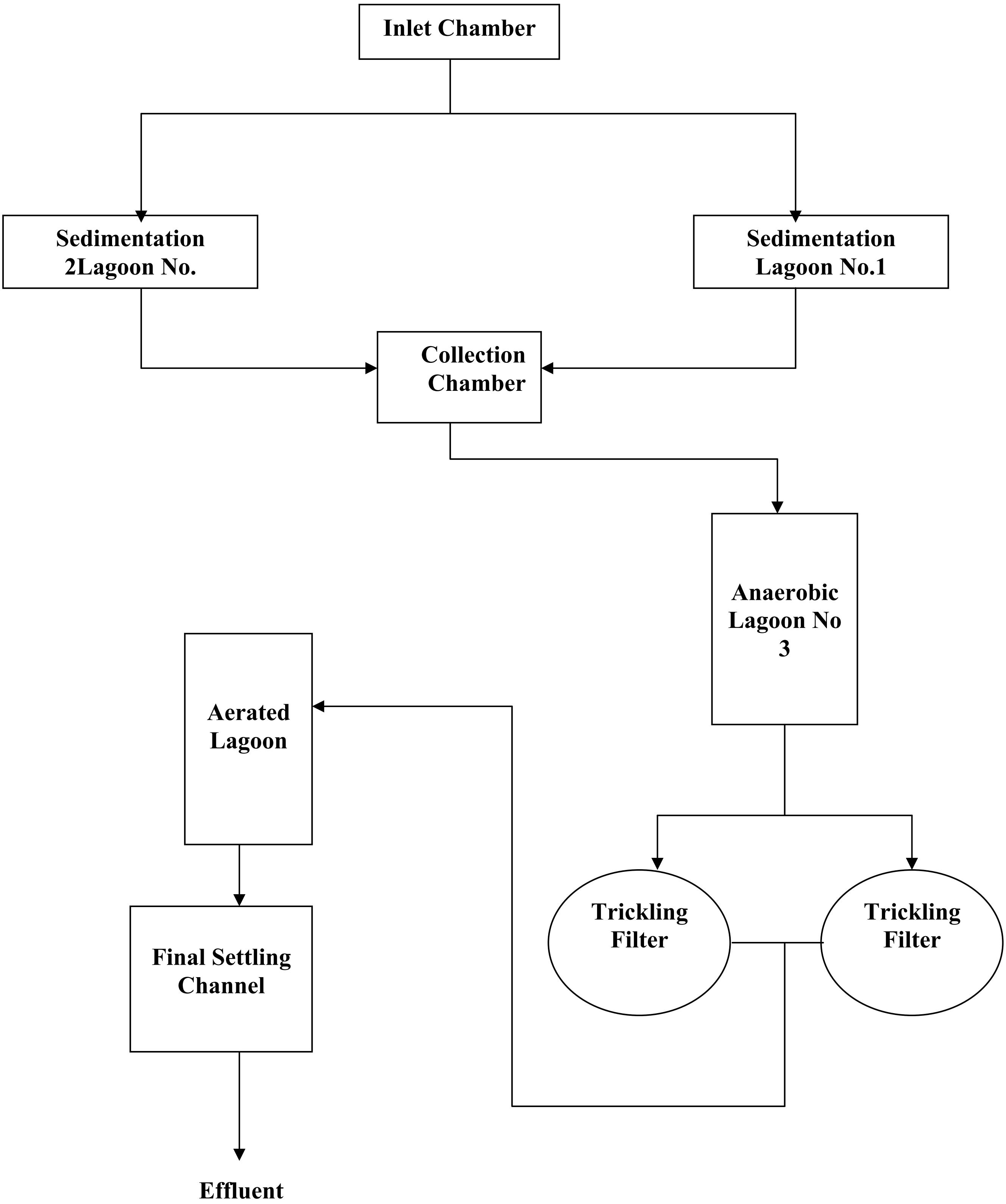


Figure 2.1: The Current Flow Scheme of GWWTP

2.3.4 Existing Operation Condition

The GWWTP was upgraded in 1997-1998 to receive and treat influent quantity up to 32,000 m^3 daily in 2005 from Gaza City. When it was put into operation, flows were found already to exceed 30,000m^3/day. The GWWTP regularly received over 50,000m^3/day up to summer2004, after this time the flow meter was out of order. Based on site visit and discussions with plant operators and engineers, the following points are noted:

- The GWWTP was already over loaded since it started operation in 1998 and today the plant receives more than 55,000m^3/day. The lagoons are almost full to the edges and the capacity to discharge effluent is limited to the capacity of the effluent pumping station which is less than 2000m^3/ HOUR. To reduce the flows received at the GWWTP, raw sewage is being discharged direct to the sea from two locations at Gaza beach.
- The anaerobic lagoons had not been desludged (cleaned) for more than three years. As a result, the settlement lagoons are now full of grit and sludge. Since anaerobic lagoon no.1 is filled with grit totally, this part of treatment plant is bypassed. The sludge layer of bond 2 is almost 20cm below the water surface. Less than one eighth of the tank volume is operating as a settling zone. Anaerobic bond 3 seems to be not significantly better than pond 2.
- The official Gaza Municipality landfill site is located on eastern Gaza near the Green Line with Israel. Access to the land fill had been frequently blocked during the past two years. During these periods, municipal garbage had been dumped in the GWWTP site as the only available alternative. It is estimated that more than 700,000$m^3$of solid waste had been dumped at the site. Wind blown plastic bags are causing frequent blockage of the aerators and bio-tower distributors.
- Although the Bio-towers are damaged by Israeli actions, they are operating well and the media is in good shape. However, the surface area of the trickling filter is partly covered with solid waste (plastic bags) which influences the hydraulic flow patterns negatively. Part of the openings of the trickling filter flow distributors seem to be clogged and the first layer of the media seems to be clogged and need to be cleaned.

- In the end of October 2004, the GWWTP received more aggressions from the Israeli Army, where the administration building had been damaged, the parking shelter had been completely destroyed and one of the bio-towers has been hit by a tank gun. Moreover, the lab of plant also received its share in the damage that led to complete suspension of the monitoring program of the quality of influent and effluent, moreover, the automatic sampler for the influent was also totally damaged.

2.3.6 Research Contribution to GWWTP

The research manipulates the above two assessments with the gathered historical record of GWWTP for the period from 2000 to the end of 2004 when the sampler and the flow meter of the plant were out of order. The historical records include organic effluent and influent characteristics such as the BOD and COD, the solids such as the TSS and TDS, nutrients such as the Total Kjeldhal Nitrogen (TKN) and the microbial content (Fecal Coli form).

For the design loads and flows, the projected data from the charts will be used instead of assumptions. And the removal efficiency for each component of the plant was measured practically which will make process calculation more accurate. Finally, this research will come with recommendations for solving the problems of hydraulic overload, solids, and bacteria removal with the lowest cost possible and will propose the best scheme for the flow process to get the best effluent quality especially if discharged to the sea. The recommendations will also include the Best Management Practice to operate the GWWTP.

CHAPTER (3): LITERATURE REVIEW

3.1. Historical development of original design and related studies

The existing treatment plant was constructed in 1977 by the Israeli administration authority with initially two lagoons. In 1986, the UNDP funded treatment plant upgrading program to cope with increasing wastewater load and expanding the initial plant capacity by more than three times. The UNDP upgrade added two more lagoons and planned that all four lagoons would be aerated. The technology of the plant was based on using a high level of mixing and aeration in the first stage (2 lagoons in parallel) and a lower level in the second stage (also two lagoons in parallel) and the system was called Dual Power Multicellular Aerated Ponds **(USAID, 1996)** The reuse facilities were constructed which included two large recharge basins totaling 40 dunums, two pumping stations, a 5000 m^3 storage tank and an overflow line terminating in Wadi Gaza. The treatment process could never meet acceptable standards required for reuse. Therefore, the reuse facility has remained unused.

The effluent from the treatment was pumped to Wadi Gaza via the UNDP reuse distribution system. The reuse pumping station 1 was located at the treatment plant while pump station 2 was located to the east in the vicinity of the farms that expected to receive the treated effluent. As the level of treatment continued to deteriorate, the pollution in Wadi Gaza became a problem. This was worsened by wastewater flows from the near by middle area and the dumping of seepage into the Wadi. To mitigate the problem, a new effluent line was constructed between the GWWTP and the beach in 1994. In that year, the MOG and UNRWA upgraded the treatment plant by; removing sand and sludge from the lagoons, upgrading their physical condition and altering the treatment process to adequately treat an estimated flow of 12,000m^3/day **(USAID,1996)** . In 1993 the Dutch firm DHV contracted by UNRWA, conducted assessment and redesign study which was meant to serve as an immediate urgent program to put the treatment facility back into working condition basically at its original design capacity. The DHV study reviewed six treatment options. Three of these included an Up flow Anaerobic Sludge Blanket (UASB) system in combination with other less sophisticated processes **(Qasim,1999).** The UASB technology consists of a digester in which the wastewater is introduced at the bottom. Organic waste is converted into biogas. As the gas bubbles escape by rising, mixing is provided. The upper part of the digester contains devices to separate the biogas, the sludge and the treated water. The system or reactor is commonly used for industrial applications and has no mechanical parts and the BOD5 removal efficiency is estimated to be up to 60 to 80% **(Qasim, 1999).**

Another alternative reviewed by DHV study in 1993, was the use of an intermittent sand filter, a low cost system that is reported to achieve a very high degree of removal of both organic and microbial pollution. The system consists of a thick layer of sand that is batch loaded. A lack of maintenance of the system would lead to flooding of the treatment site. The DHV analyses targeted the design needs for the year 2010 with an estimated population served of 435,000 and an average daily flow of 36,540m^3. Loading in kg/day was projected to be 17,400 for BOD_5, 43,500 for COD, 17,400 for TSS and 3,480 for N **(USAID, 1996).** The construction cost, area requirement and the treatment process for the six options reviewed are summarized in table 2.1.

Treatment Option	Construction cost US $	Site requirements (ha)
1. UASB+ Trickling Filter	10.4 million	9.9
2. UASB+ Aerated facultative lagoons	7.9 million	29.9
3.Activated Sludge	7.6 million	26.6
4. Anaerobic + Aerated + Facultative Lagoons	5.8 million	30.9
5. UASB + Intermittent Sand Filter	5.8 million	15
6. Anaerobic Lagoons+ Intermittent Sand Filter	2.2 million	18

Table 2.1: Treatment process, construction cost and area requirement for the six options in DHV study **(UNRWA/DHV, 1993).**

Based on DHV study in 1993, operating costs vary considerably between the options and are primarily a function of power consumption and chemical demand. Options 2, 3 and 4 require high levels of power for the aeration and activated sludge processes while options 5 and 6 would minimize chemical requirements because of the effectiveness of the intermittent sand filter. Option 3 would also have much higher costs for sludge disposal than the other options. Disregarding the cost of land, option 6 clearly has the lowest investment cost. Both options 5 and 6 would have low operating costs.

In 1997 the USAID appointed Metcalf and Eddy as its consultant in Gaza to conduct the necessary studies, assessments and design an upgrading of GWWTP. The proposed design criteria for the upgrading of GWWTP were to achieve a level of treatment of 35/30 mg/l for BOD_5 and TSS and some nitrogen removal. The use of the existing facilities and converting them to cope with the treatment process proposed was one of the terms of reference in the conceptual design. The Bio-tower technology was adopted for the treatment process based on its lower construction and annual cost and power demand as compared to other treatment techniques. The plant was designed for an average flow of 32,000m^3 and peak flow of 48,000 m^3 and proposed six schemes for the treatment process **(USAID, 1997)**

3.1.1. Studies Conducted on GWWTP for Upgrading

There are two comprehensive studies performed by international consultants for upgrading GWWTP along with cost estimates:" **the feasibility study and conceptual design for GWWTP**" performed by **METCALF&EDDY** for the **USAID** in March,2001 and "**Assessment of Existing Gaza WWTP**" performed by **DORSCH CONSULT for the KfW** in July, 2005**.**

- **Feasibility Study and Conceptual Design for GWWTP(March, 2001)**

This study was proposed on March, 2001 and was performed by Metcalf and Eddy on the behalf of the United States Agency for International Development (USAID) after two years of the upgrading it carried out for GWWTP.

The purpose of this study is to define the facilities needed to provide additional treatment capacity for the near-term. The scope of the project consisted of providing additional wastewater treatment and sludge treatment facilities needed to expand wastewater treatment capacity. These facilities include on-site wastewater treatment plant facilities and off-site sludge drying facilities, an effluent pipeline and infiltration basins.

The study came up with two alternatives:

- Alternative 1- Bio-towers: This consists of converting Anaerobic Pond no.3 to an aerated lagoon and adding an additional bio-tower. The aerated lagoon and bio-tower group will be configured in series as they currently are. The existing bio-tower feed pumping stations will be sized for sustained peak loading from 64,000 m^3/day to 144,000 m^3/day. The facilities require two 40 m diameter clarifiers sized at an overflow rate of 57m^3/day/m^2.
- Alternative 2-Activated Sludge: This consists of converting Anaerobic Pond No.3 to an activated sludge aeration basin and converting the existing aerated lagoon also to an activated sludge aeration basin. The activated sludge facility would run in parallel with the existing two bio-tower group.

  A flow splitter structure would be constructed to divert flow to two existing bio-towers.

  Three final clarifiers will be provided, sized at an overflow rate of 32 m^3/day/m^2 at sustained peak flows. The facilities required are three 32 m diameter clarifiers.

The two defined alternatives were compared to each other. The evaluation criteria were process performance, power requirements, construction cost, annual cost, operability, and effect on the local environment.

The conclusion was that Alternative 1- Bio-towers is the most favorable alternative in terms of power requirements, construction cost, and annual cost and is equivalent in terms of process performance. The feasibility study recommended the implementation of alternative 1- Bio-towers.

- **Gaza Central Wastewater Project, Assessment of Existing Gaza WWTP (July,2005)**

This assessment was conducted by Dorsch Consult for the German Bank for Development (KfW) in July, 2005. Gaza Municipality and Palestinian Water Authority requested urgent assistance from the KfW to restore the existing wastewater treatment plant on December 2004.The objective of the assessment was to identify the essential works needed to restore GWWTP to reasonable operating conditions the 2010 when the Central Wastewater Treatment Plant in Buriej is assumed to start operation, which means to optimize use of the existing facilities, and not to provide new facilities.

The objectives of the assessment were:

- Restore the Operation of GWWTP to an acceptable and sustainable standard.
- Increase the hydraulic capacity of the GWWTP and effluent discharge so that all sewage from Gaza City receives at least partial treatment and stop the direct sea discharges.
- Review and optimize the GWWTP treatment process.

The assessment came up with the following:

1 A list of materials and works required to improve the standard of operation and maintenance of GWWTP.

2 Review of the hydraulic capacity for the internal piping of GWWTP along with identification of critical pipe segments in the plant.

3 Process Options for improving the effluent quality with two alternatives:

   - Alternative 1: to follow the same flow scheme of the existing treatment process but to split the flow after the anaerobic pond no.3 into two branches; one to the aerated pond and the other to the bio-towers and then the two branches meet at the maturation pond.
   - Alternative 2: the same as alternative 1 above but the anaerobic pond NO3 is partially aerated by installing aerators for oxygen supply.

4 Other potential measures for Process Optimization.

3.1.2 Deficiencies of the previous studies and assessments of GWWTP

As mentioned above, there are two assessments conducted on GWWTP: one by the USAID and the other by KfW. The common factor between the two studies is that they are dealing with GWWTP with temporary and interim solutions, and the proposed actions are emergency actions for operation continuity. The prevailing unstable situation in Gaza has led to postponement of many strategic projects, and by these postponements many circumstances change on the ground such as the land availability, the served population and the service area. This has led to the fact that temporary solutions become permanent status with all environmental problems associated with it, which is reflected negatively on the Palestinian people.

- The feasibility study and conceptual design performed by Metcalf and Eddy proposed construction of an extra bio-tower with a construction cost of US $3,191,000, which is considered to be very high if not financed by the USAID, especially it was proposed for PWA to be totally financed by the USAID and was rejected. Moreover, the study proposes construction of bio-tower pumping station along with addition of more aerators which will double the electricity bill which is around one million dollars annually. The bio-towers have in general the potential to generate odors which will be reflected on the local environment.

The assessment performed by DORSCH CONSULT for the KfW proposed priorities to implemented starting with increasing the hydraulic capacity of piping, and proposes parallel operation of bio-towers and the aeration pond and then to implement additional aerators and start sludge recycling. The cost estimate for these works was from 275,000 EUR to 335,000 EUR depending on the required performance. The DORSCH assessment assumed design loads and flows; for example the assumed BOD used in the model HYDKA was 500 mg/l while the average value for the BOD for the five years was 533 mg/l. also in the process calculation there was assumed performance for the plant components (anaerobic pond, aerated pond and the trickling filter).

3.2 Treatment Process Evaluation

The final engineering design report of the GWWTP was revised to evaluate the treatment process and the criteria set for the modification of the plant along with assumptions made in considering the upgrading of the plant.

There are six options to operate the treatment system proposed and only two options were practiced by allowing the flow after the anaerobic lagoon either to pass through the trickling filters then to the aerated lagoon or to let the flow pass through the aerated lagoon then to the trickling filters. The first option seemed to be less power consumption than the second option and was the approved scheme.

The lagoons were almost full to the edges indicating a hydraulic overload and hence a low treatment efficiency. This problem arises the need to propose solutions for solving the problem of over loading, treatment process management and improvement of the effluent quality.

3.3 Industrial Wastewater Generation

The final disposal of the industrial effluent has to be defined to avoid the damage of the treatment process and to protect public health and ecological system. Furthermore, it is important for the possible reuse of treated effluent in agriculture or recharge to the groundwater aquifer **(Afifi, 1998).** Industrial wastewater loading in the Gaza Governorate is insignificant compared to the domestic loading. Limited quantities of wastewater with varying characteristics are produced by small industries distributed in the City **(Afifi, 2003)**. The quantities and characteristics of the generated wastewater depend on the kind and size of industries. The tile, laundry services, and juice industries represent the major wastewater producing industries. Flows coming from food processing industries are mainly generated from washing fruits and soft drink bottles **(CAMP, 2001).** The wastewater generated by laundries is the disposed wash water. In the slaughter house there is a pre-treatment system which is also connected to the municipal system.

As mentioned earlier, the characteristics of industrial wastewater are of more concern than expected flow levels. While flows remain relatively small, the characteristics can range from relatively high BOD_5 concentration in the slaughter house and food processing industries, to relatively high concentration of nitrates and phosphate in wastewater discharged by laundry services from the use of detergents (**InfraMan, 2006)** Most industries are involved with washing activities and will have some combination of washing compounds and BOD_5 loading. Future trends in industrial development in Gaza City involve some uncertainty resulting both directly and indirectly from the political instability in the region.

3.4 Trickling Filter-Aerated Lagoon Combined System

In this section, the function theoretical background of the trickling filter and the aerated lagoons is described with a highlight on the advantages and disadvantages of each system. The various types and functions of the trickling filters are also discussed, along with the removal efficiency and mechanisms of each the trickling filters and the aerated lagoons systems.

Many unit operations and processes can be combined to develop a process diagram to achieve a desired level of treatment. The level of treatment may range for removal of BOD and TSS, nitrogen, and phosphorus, to complete demineralization (**Qasim, 1999).**

To develop the best possible process diagram, designers must evaluate many factors that are related to operation and maintenance, process efficiency under variable flow conditions and, and environmental constrain**s (Qasim, 1999).** The factors that are considered important in the selection of process diagram are:

1. Land requirements.
2. Adverse climatic conditions.
3. Ability to handle flow variations.
4. Ability to handle Influent Quality variations.
5. Industrial pollutants affecting process.
6. Reliability of the process.
7. Ease of operation and maintenance.
8. Occupational hazards.
9. Air pollution.
10. Waste products.

In upgrading GWWTP the flexibility of the process was taken into consideration **(USAID, MOG, 1997**). Seven process options were proposed for the operation of GWWTP.

3.4.1 Advantages and disadvantages of Trickling Filters

Using the trickling filters system has many advantages especially in some developing countries for power consumption reasons. On the other hand the system could have some disadvantages where land resources are scarce. In the following points, the advantages and disadvantages of the system will be short listed **(Horan, 1991)**

1- Major advantage of trickling filters

- They are comparatively simple to operate**.**
- Have very low running costs.
- They are able to tolerate shock and toxic loads owing to the short contact time of the wastewater with the slime layer.

2- Major disadvantages of the trickling filters,

- Their land requirements are high

- They can only provide limited treatment efficiency (although a 20/30 effluent is readily attainable).
- In hot countries they are associated with odors and fly nuisance.
- Although it is a relatively simple technique, the degree of skill required for operation and maintenance is frequently not available

3.4.2 Function of Trickling Filters

A trickling filter (also referred to as a bio-filter, bacteria bed or percolating filter) is a reactor of rectangular or circular plan which is filled with permeable media. Wastewater is distributed mechanically over the media and percolates down the filter to collect in an under drain system at its base. A microbial film develops over the surface of the media and this is responsible for removal of BOD during passage of sewage through the bed. Filtration doesn't occur and the process is solely a biological one **(Horan, 1991).** The efficiency of the system depends upon an even distribution of settled sewage over the whole surface of the filter and also upon the circulation of air throughout the filter media.

On circular filters, the distribution and circulation is achieved by influent entering at the center of the bed and passing into radial distributor arms above the bed surface. The distribution arms are fitted at intervals with spurge holes such that discharge of sewage through them provides the necessary force to drive the arms around the central column. In case of rectangular filter, the distributor is driven forward and backwards with the liquid being siphoned from a channel running along the length of the bed. This wetting followed by a rest period is an essential requirement for successful filter operation **(Horan, 1992).**

3.4.3 Removal Efficiency and Mechanisms in T.F.

The organic material and nutrient removal efficiency of the trickling filters is reviewed in following section from theoretical point of view:

1- BOD_5 Removal

For low loaded Trickling Filter systems the daily volumetric organic loading rate (Lv) is in the range of 0.1- 0.2 kg BOD_5/ m^3 **(Qasim,1999)** The normal ranges of hydraulic surface loading rate are 0.1 to 0.2 $m^3/m^2.h$ **(Horan,1991)** In the low loading rate TF system, the sludge retention is rather long and this will ensure a high degree of Bio-film stabilization within the filter and BOD_5 removal efficiency greater than 85% **(Horan,1991)** The high rate trickling filter has been developed to further reduce the area requirements and the construction cost of low-rate filters by permitting a higher organic load **(Horan, 1991).** The high BOD_5 load results in rapid Bio-film growth and a comparatively high production of biological sludge **(Horan, 1991)**. The hydraulic load on the filter must therefore be sufficiently high to produce flushing effect. The volume load (Lv) is chosen between 0.2-1.5 kg BOD_5 / m^3.day depending on the required BOD_5 removal efficiency. The hydraulic surface load should be between 0.3 and 1.2 m/h. **(Qasim,1999)** assumed that BOD_5 removal by trickling filters was proportional to the contact time of the wastewater with the biological slime layer and also to the total active microbial mass in the slime layer and came up with following equation:

$$L_e/L_o = \exp(-kD/Q^n)$$

Where:

L_e/L_o is the removal efficiency of the BOD_5,

K is the BOD_5 removal rate constant,

D is the filter depth (m) and

Q is the surface hydraulic loading (m^3/d) .

2- Nutrient Removal

For low loaded trickling filters systems, a high degree of nitrification can be realized (more than 75%) at the bottom part of the filter where autotrophic nitrifies will accumulate **(Qasim, 1999)**. De nitrification can only be achieved if the nitrified effluent is recycled to the filter. A maximum of 20-35% of N can be denitrified if anaerobic conditions prevail in the inner Bio-film **(Horan, 1990)**. This is however in practice very difficult to control. For high rate Trickling Filters, Nitrification will hardly take place. Only at low (Lv) loading less than 0.3 kg BOD_5/m^3.day partial nitrification (less than 50%) can be observed. At higher temperature this may increase to some extent **(Horan, 1990).**

3.4. 4 Aerated Lagoons System

An aerated lagoon resembles a waste stabilization pond in that it is a shallow basin between 2 and 5m deep, with a large surface area, which receives a continuous flow of wastewater. It differs in that the bio-oxidation of organic material is forced by mechanical aeration and not naturally by algal photosynthesis. In ponds where mechanical aerators have been installed, there is a great change in the ecology of the lagoon, leading to a complete disappearance of the algae and their replacement by a mixed heterotrophic bacterial community, which grows in the form of flocs. Thus, ecologically, aerated lagoons most resemble an activated sludge process operated without cell recycle (**Metcalf & Eddy, 1992)**. Aerated lagoons generally employ surface aerators, which is either floating or fixed. In deeper lagoons, submerged turbines must be provided in order to provide adequate mixing. Concrete pads are usually placed under the mixer in order to prevent scour of the lagoon bottom. These are not required when the lagoon has been completely lined. Aerated lagoons typically are classified by the amount of mixing provided. A partial mix system provides only enough aeration to satisfy the oxygen requirements of the system and does not provide energy to keep all total suspended solids (TSS) in suspension (**EPA, 2002**). In some cases, the initial cell in a system might be a complete mix unit followed by partial mix and settling cells.

The power level required for mixing is a function of the lagoon size, geometry and the concentration of suspended solids **(Metcalf Eddy, 1992)**. If the BOD or solids loading to the lagoon increases, then more aerators can be added as required, but a minimum spacing is required to prevent interference .

The effluent from an aerated lagoon requires some form of settlement stage before it is fit to discharge to a watercourse. Although this can be a conventional sedimentation tank, it is more usual to discharge into one or more ponds, depending upon the final effluent quality required. Aerated lagoons can reliably produce an effluent with both biochemical oxygen demand (BOD_5) and TSS less than 30 mg/l if provisions for settling are included at the end of the system **(Horan, 1991)**. Significant nitrification will occur during the summer months if adequate dissolved oxygen is applied **(Metcalf Eddy, 1992).** Many systems designed only for BOD_5 removal fail to meet discharge standards during the summer because of shortage of dissolved oxygen. The suspended solids content of aerated lagoon effluent is high and thus the first pond in the series acts as a settlement tank. A retention time of up to 10 days is required and depths are typically 2m. The remaining ponds in the series behave as maturation ponds. The sedimented solids from aerated lagoons should ideally be subject to anaerobic digestion or placed on sludge-drying beds until it has stabilized. The factors that must be considered in the process design of aerated lagoons include **(Horan, 1991):**

1. BOD removal.
2. Effluent characteristics.
3. Oxygen requirements.
4. Temperature effects
5. Energy required for mixing
6. Solids separation.

Depending on the detention time, the effluent from an aerated lagoon contains about one-third to more than one half the value of the incoming BOD_5 in the form of cell tissue. Most of these solids must be removed by settling prior to discharge (a settling tank or basin is a normal component of most lagoon systems). If the solids are returned to the lagoon, there is no difference between this process and a modified activate sludge process **(EPA, 1999)**

3.4.4.1 BOD Removal

Depending on the detention time, the effluent from an aerated lagoon contains about one-third to more than one half the value of the incoming BOD in the form of cell tissue. Most of these solids must be removed by settling prior to discharge (a settling tank or basin is a normal component of most lagoon systems). If the solids are returned to the lagoon, there is no difference between this process and a modified activate sludge process.

3.5 Alternative Treatment Systems

A review of the individual unit processes that could be implemented in GWWTP with their drawbacks is highlighted in the following sections:

3.5.1 Anaerobic Ponds

Anaerobic ponds are simple and effective for small plants but the quantity of BOD to be removed by GWWTP combined with land limitation make additional anaerobic ponds not practical for the expansion.

3.5.2 Aerobic Lagoons

Converting anaerobic lagoons to aerobic lagoons makes efficient use of existing facilities and conserves land. The BOD removal efficiency per land area required is significantly higher than anaerobic lagoons but less than a bio-tower. Furthermore, the operating cost of aerators is considerably higher than that of a bio-tower.

3.5.3 Natural Systems

Natural treatment systems consist of man – made wetlands or reed bed cultivation type systems. These facilities are effective for small systems but require a large amount of land for facilities of moderate capacity. The design parameters for such systems are not well established and these facilities normally develop from small pilot scale systems which are demonstrated and then scaled up. This requires time, which is not available to the Gaza WWTP facility. In addition to difficulties with land acquisition, environmental documentation and permitting would be time consuming due to the lack of a local precedent.

3.5.4 Rotating Biological Contactors (RBC)

Rotating Biological Contactors (RBCs) are a fixed film system similar to bio-towers. RBCs are often cost effective when they can be added to existing tankage. In the case of the GWWTP tankage would have to be constructed to house the RBCs, thus negating that potential advantage. RBCs would have higher construction costs, similar to bio-towers, and they complicate plant operation by adding another different biological process. No advantage over expansion of the existing fixed film process could be identified.

3.5.5 Oxidation Ditch

An oxidation ditch is a specific configuration of the activated sludge process. The activated sludge process is recognized as a viable alternative, but the utilization of the existing lagoons for basins would be more cost-effective than constructing additional basins in that specific configuration.

3.5.6 Sequencing Batch Reactor (SBR)

A sequencing Batch Reactor (SBR) can be a cost-effective process for a plant of GWWTP size**.** The SBR has the construction cost advantage of achieving oxidation and sedimentation in the same basin, thus eliminating the need for separate sedimentation basins. Constructing SBRs to reduce the remaining BOD may require parallel flow trains and therefore separate clarification behind the existing biological treatment systems. The advantage of not constructing separate clarifiers is then lost.

2.5.7 Lagoon Clarification

Using the existing lagoon capacity for clarification was considered. This has the advantage of not having to construct clarifiers. Disadvantages include the loss of lagoon capacity for biological reduction and the expected reduced efficiency and reduced operability of a lagoon clarification system. A lagoon clarification system would be oversized compared to a circular clarifier tank to account for short circuiting and non-uniform solids distribution. Solids removal would also be a more difficult operation since it would be likely require some type of dredge equipment. The operators find the existing suction tube system relatively difficult to operate. The final clarification is important to comply with effluent requirements and it is beneficial to rely on proven unit processes for this key facility.

Chapter 4: RESULTS AND DISCUSSION

This chapter demonstrates and discusses the results of the collected data and their analysis of GWWTP. The performance of system -as a total – is presented by comprising the level of different physical, chemical and biological parameters in influent and the effluent. In addition, results showed the performance of different system components. Analysis of the performance aspects of GWWTP and factors leading to inadequate performance of the system is discussed. Moreover, modifications on the treatment system that can enhance the system performance have been demonstrated.

In general, these results showed a decline of treatment capacity and decrease of removal efficiency of the system over the study period.

Correlation between Data and Research Aims

One of the research aims is to evaluate the behavior of GWWT and the performance of the plant through analyzing the biological, chemical, solids, nutrients and bacteriological removal efficiency of the plant. The BOD, COD, TSS, TDS, TKN and Faecal Coliform were chosen as parameters to reflect the removal efficiency of the plant. The data collected from the GWWTP lab were recorded daily and the tests were performed weekly, bi-weekly or monthly. The data collected were filtered from the odds and were averaged as monthly readings for each month to indicate the values of the selected parameters for each month and then to indicate the removal efficiency of the plant for the whole year as a performance indicator. The tables shown in annex A show an average value for each parameter for each year from 1998 to 2002.

4.1 Total System Performance of GWWTP.

The analysis of the total system performance through collected data is demonstrated in the following sections. The main parameters included in the analysis were the removal efficiency of organic materials, nutrients (Nitrogen), microbiological indicator (FC) and solids. These parameters were compared with the hydraulic flow over the study period. These parameters are:

- Biochemical Oxygen Demand (BOD)

- Chemical Oxygen Demand (COD)
- Total Suspended Solids (TSS)
- Total Dissolved Solids (TDS)
- Total Kjeldhal Nitrogen (TKN)
- Fecal Coli form (F.C)

Annex (1) presents the Results of the GWWTP lab monitoring program data for period from the year 2000 to 2004 based on daily readings and averaged for each month of the year.

Table 4.1 shows the maximum, minimum and the average monthly values for the flow quantities, BOD, COD, TSS and TKN for the period from January 2000 to December 2004 of both influent and effluent of the total system.

Parameters	Average monthly value		Max monthly value		Min monthly value	
	Influent	Effluent	Influent	Effluent	Influent	Effluent
Flow (m^3/d)	43786	-	59419	-	34508	-
BOD (mg/l)	533	31	1300	54	297	17
COD (mg/l)	1046	106	2180	380	752	65
TSS (mg/l)	661	40	1589	107	318	18
TKN (mg/l)	80.5	61	154	91	35	37.5

Table 4.1: Maximum, Minimum and the Average monthly values of different studied parameters in GWWTP for both influent and effluent of the total system for the study period.

The results show a decline of treatment capacity and decrease of removal efficiency of the system over the study period. This problem was basically due to the underestimates in the design criteria for the projected flow for the planed operation period from 1998 to 2005.

4.1.1 Biochemical Oxygen Demand (BOD_5)

Figures 4.1 to 4.3 show the results of the average monthly effluent BOD_5 level and removal efficiency versus flow of GWWTP during the study period. The results show the dramatic increase of the average flow rate from 2000 to 2004 from 42,185 to47, 916 m^3/daily. In the same time, the average BOD_5 effluent increased from 26.8 mg/l to 35.2 mg/l for the same period (Figure 4.1). A considerable variation between summer and winter values can be observed.

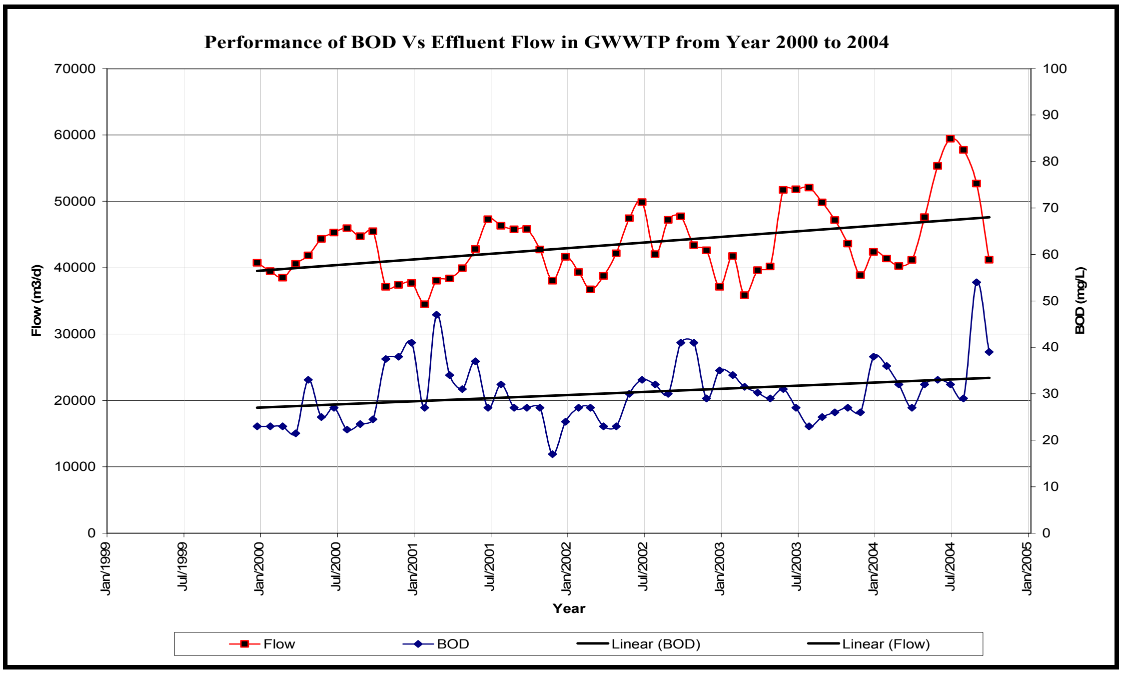


Figure 4.1: Average Effluent BOD_5 variations and average monthly flow rate during the study period in GWWTP.

Figure 4.2 presents the percentage of BOD_5 removal in the effluent of the treatment system. The removal efficiency was decreased from 94.9% to 92.2% in the study period. Figure 4.3 showed the variation in the percentage BOD_5 removal efficiency and BOD_5 concentration in the GWWTP effluent in the period from 2000 to 2004.

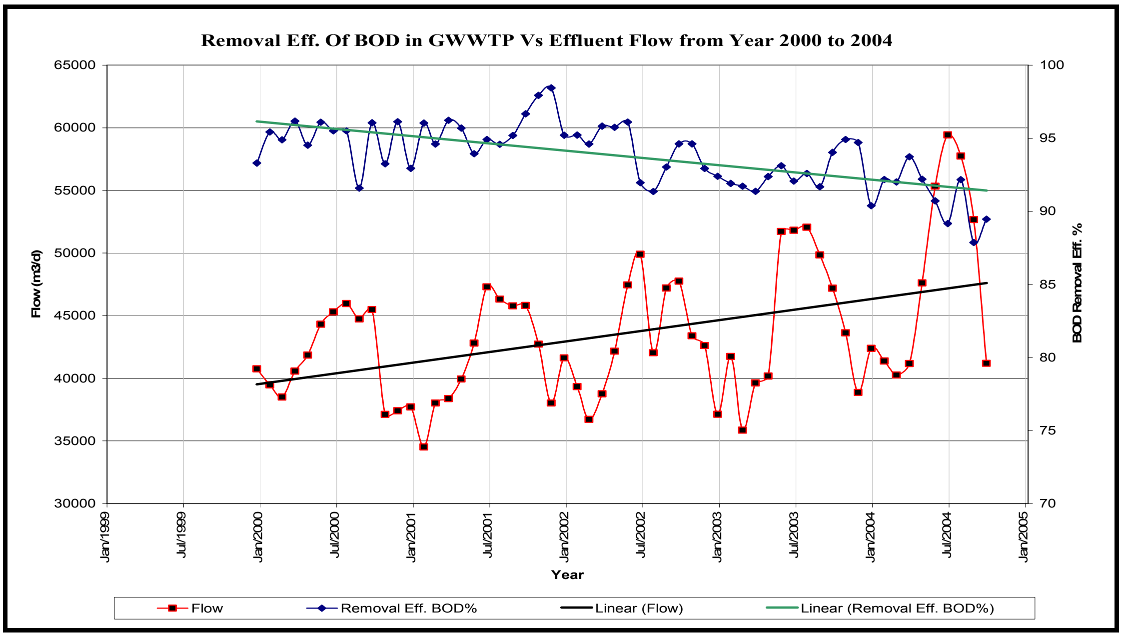


Figure 4.2: Relation between percentage of the BOD_5 effluent removal and average monthly flow rate from 2000 to 2004 for GWWTP.

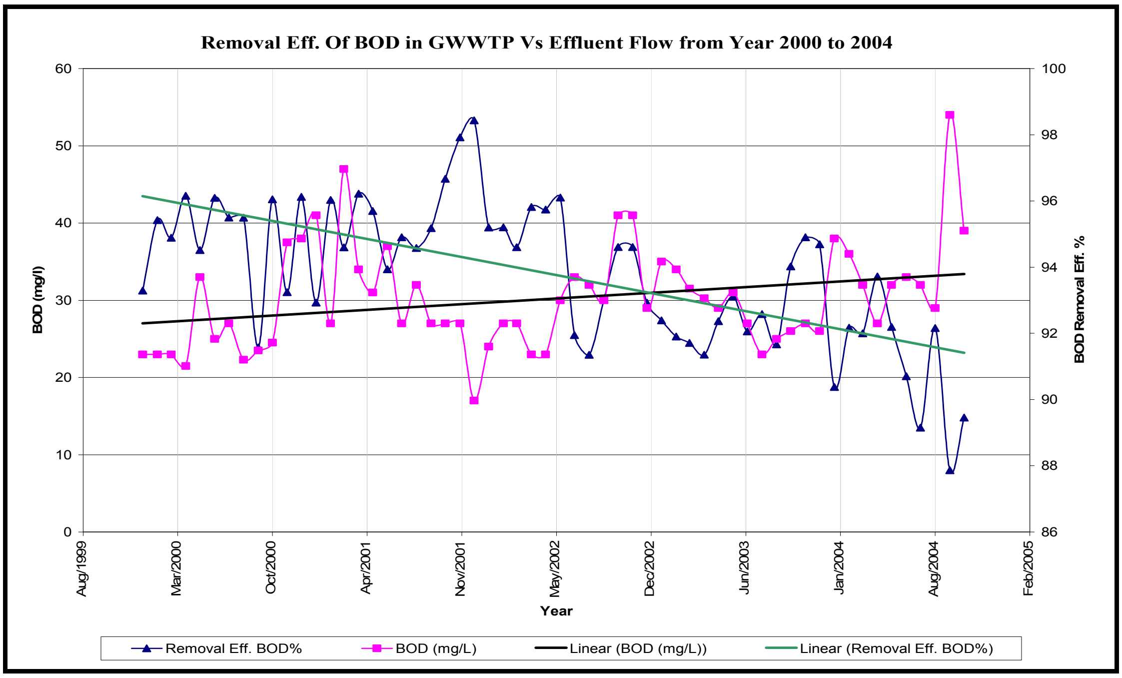


Figure 4.3 Variation in the percentage BOD_5 removal efficiency and average BOD_5 concentration in the GWWTP effluent in the period from 2000 to 2004.

The Biochemical Oxygen Demand (BOD_5) is the most important parameter in the treatment process design and effluent discharge or reuse (**Qasim, 1999**). Lowering of BOD_5 is both a physical process by way of settling of organic particles and a biochemical process through decomposition and mineralization of organic and inorganic compounds **(Reed, 1995).** In a good designed system, aerated lagoons can reliably produce an effluent with both biological oxygen demand (BOD_5) and TSS less than 30mg/l if settling is provided at the end of the system (**EPA, 2002**). A combined system in which the process includes a fixed film reactor followed by a contact channel 10-15% of the size of an aeration basin of a conventional activated sludge plant will have a removal efficiency of BOD_5 up to 85% (**Qasim, 1999**). The BOD_5 values for influent and effluent of GWWTP generally increase in the winter months than in the summer months. This is due to the increase in the water consumption in the summer season than in the winter while the biological load is constant, which leads to the dilution in the influent BOD_5. The average value for February influent BOD_5 value is 525 mg/l, while the average value of influent BOD for September is 403 mg/l (see appendix 1).

The effluent BOD_5 value is increasing with the annual increase in the influent quantity and the most recent tests taken in October 2006 indicates a value of 45-50 mg/l, as the quantity of effluent reached more than 55,000m^3/d as shown in the extrapolation of Figure 4.1. The explanation for this increase in the value of effluent BOD_5 is that the GWWTP is already hydraulically over loaded of more than 70% of the designed hydraulic load and an estimated quantity of 15000m^3 is being diverted to the sea from sewage pumping stations 1&2, which leads to decrease in the retention time of the sewage water when passing through the plants components, which reflects the need to find a solution to retain the original designed retention time for the treatment process and hence obtain the required values of the BOD_5. The GWWTP BOD_5 removal efficiency has decreased from around 95% in the year 2000 when the average flow was around 42,000m^3; to around 91% in the year 2004 when the average flow was around 48,000m^3 and recent tests made on October 2006 show that it is dropped to less than 90% (45-50 mg/l) where the influent quantity reached to more than 55,000m^3 (see figure 4.2).

Taking into consideration that the effluent is discharged to the sea and the increasing quantity of wastewater received by GWWTP, the environmental impacts of GWWTP is considered to be negative. The trend of BOD_5 removal efficiency and the effluent values of the BOD_5 are demonstrated in figure 4.3 which showed linear like decrease in the removal efficiency and an almost linear increase in the effluent BOD_5.

4.1.2 Chemical Oxygen Demand (COD)

The Chemical Oxygen Demand (COD) is another important factor highlighted in this research for the performance of GWWTP. The average values of the effluent COD and removal efficiency against flow rates of GWWTP and the trend of the removal efficiency during the study period are demonstrated in figures 4.4 to 4.6. The results show the same trend as in case of the BOS_5 parameter. Increasing the hydraulic load of the plant was reflected negatively on removal efficiency and increases the average effluent COD concentration of the plant.

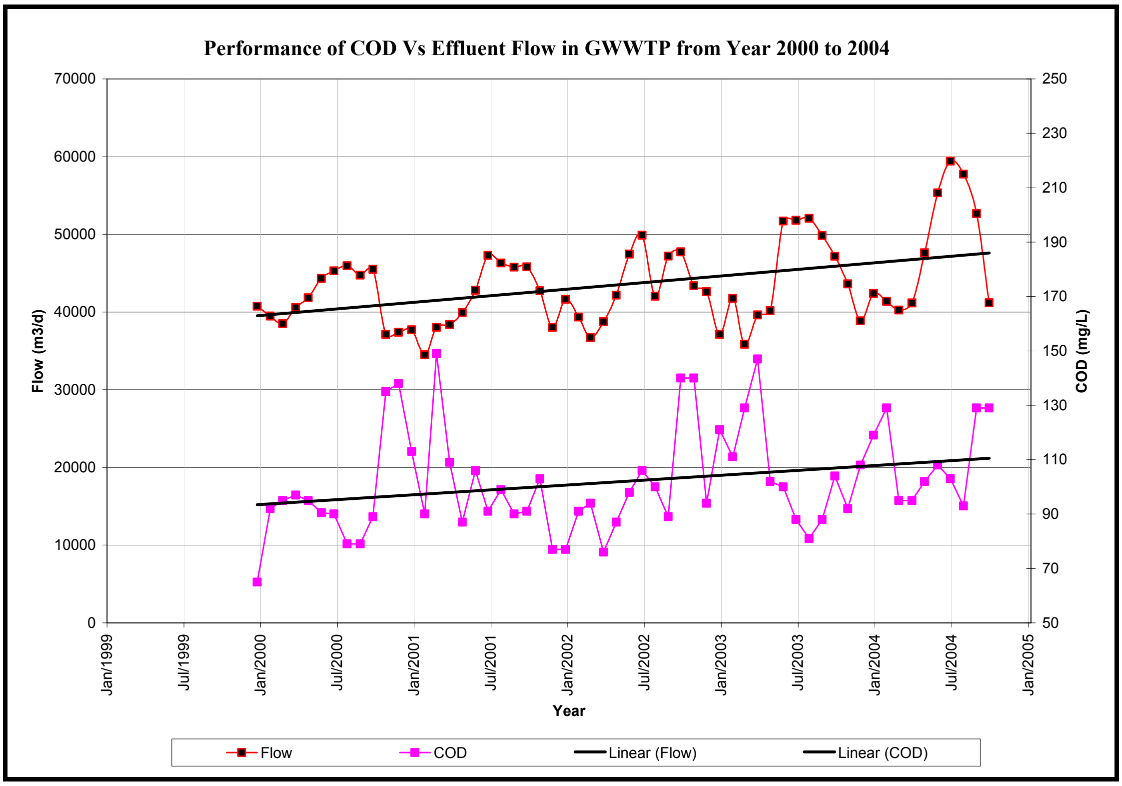


Figure 4.4 The average Effluent COD variations and average monthly flow rate during the study period in GWWTP.

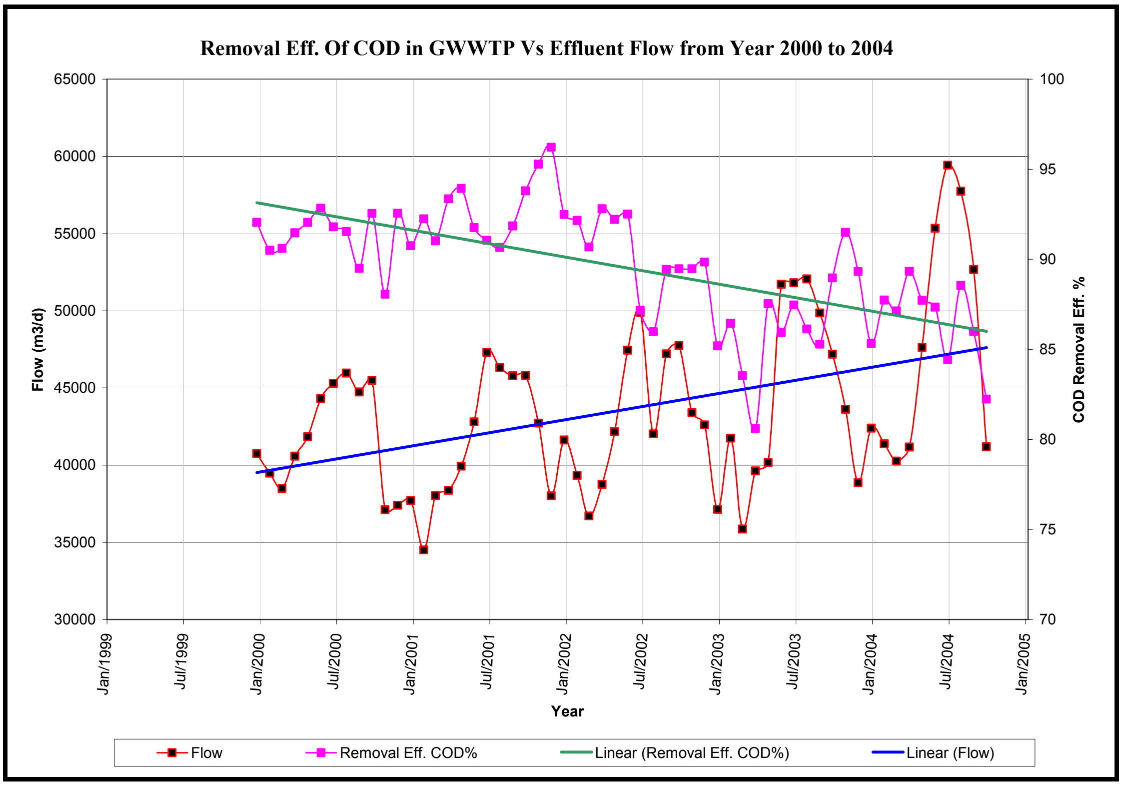


Figure 4.5: Relation between percentage of the COD effluent removal and average monthly flow rate from 2000 to 2004 for GWWTP.

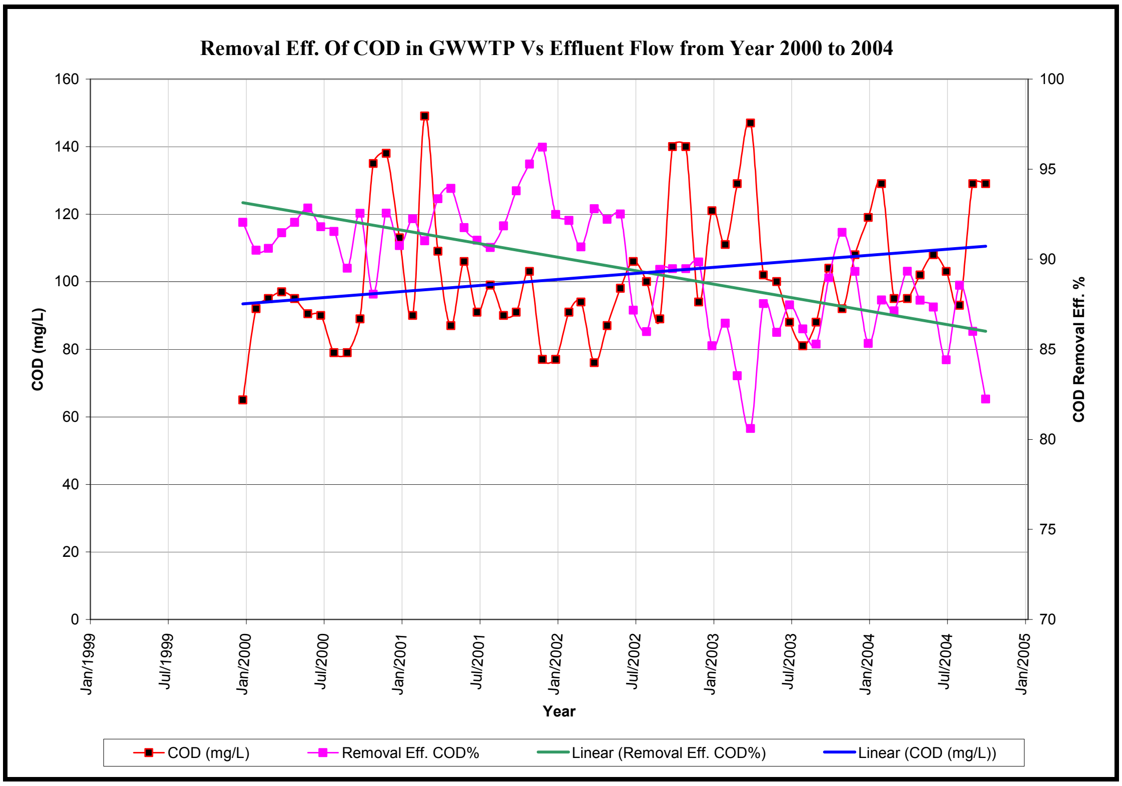


Figure 4.6: Variation in the percentage COD removal efficiency and COD concentration in the GWWTP effluent in the period from 2000 to 2004.

The COD test is rarely used in effluent discharge control, but primarily in assessing the strength of trade or industrial effluents (**Tchobanoglous and Schroeder, 1999**). For the reason that the COD test is a simple chemical assay, it is easy to point out its drawbacks and limitations. The results showed that the average COD values for effluent increased from the year 2000 of an average of 95.4 mg/l to an average of 110 mg/l in the year 2004 and the COD removal efficiency decreased from 91% in the year 2000 to 86% in the year 2004, which indicates a poor performance of GWWTP due to the increasing hydraulic load. The most recent tests of GWWTP lab show that the effluent values of the COD ranges between 120mg/l to 130mg/l indicating that the removal efficiency of COD was less than 83% by October 2006. The general trend of the COD removal of GWWTP along with the values of effluent COD is shown in figure 4.6. It is worth mentioning here that the numeric value of the COD removal efficiency is less than the BOD because of the non removal of the non-degradable fraction of the COD.

In a combined aeration lagoons –trickling filter system, the COD removal could reach up to 90% (**Qasim, 1999**). Values obtained from the tables in annex 1 for influent COD test simply showed a relation of **2:1** of **COD: BOD** which is complying with theoretical values of COD: BOD ratio of municipal settled sewage (**Horan, 1990**). Values obtained for effluent for **COD: BOD** ratios show a relation of **3:1**. This is owing to the fact that biodegradable fraction of the waste decreases through the biological treatment whereas the non-biodegradable fraction remains unchanged (**Horan, 1990**). In view of its simplicity and rapidity, the COD test is the most suitable assay for the determination of the strength of both raw and treated wastewater.

4.1.3 Total Suspended Solids (TSS)

The results of effluent TSS average values and the trend of the TSS removal efficiency compared with the hydraulic loading of the GWWTP during the study period are shown in figures 4.7 to 4.9. Figure 4.7 showed that the average effluent TSS doubled and increased from an average of about 25 mg/l to 50 mg/l from the year 2000 to 2004 and the GWWTP lab tests on October 2006 showed that the TSS values in the effluent reached to more than 65mg/l.

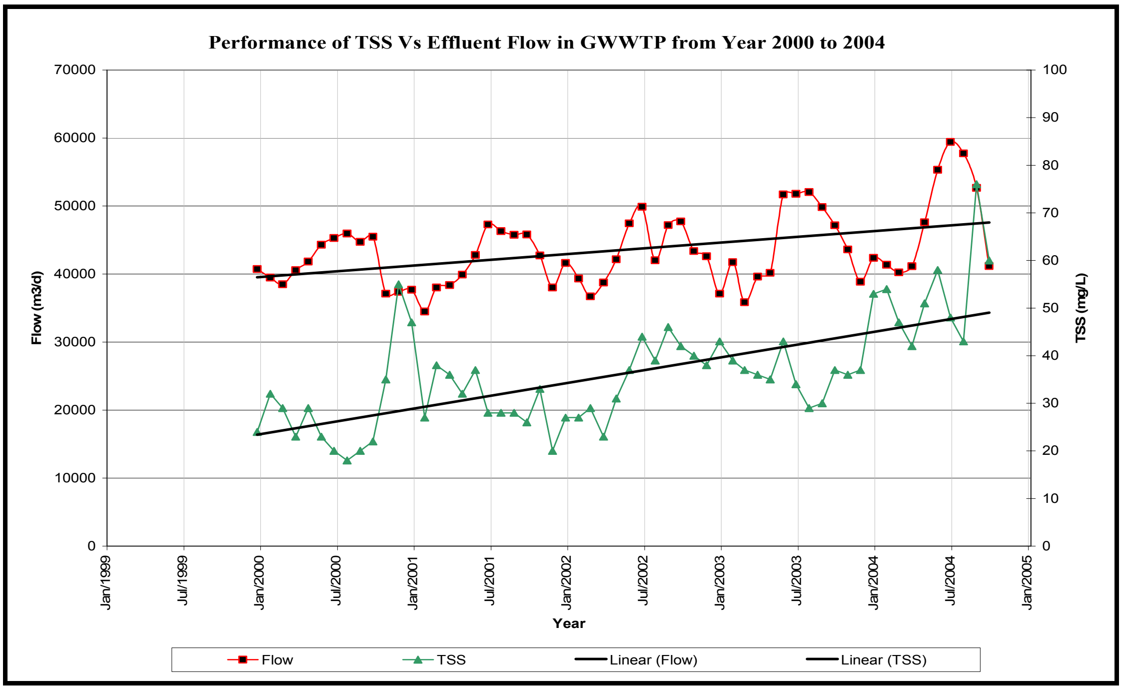


Figure 4.7: The average effluent TSS variations and average monthly flow rate during the study period in GWWTP.

The percentage of TSS removal efficiency of the system was decreased from about 97% in 2000 to less than 90% in 2004 (figure 4.9) against increasing of hydraulic load for the same period.

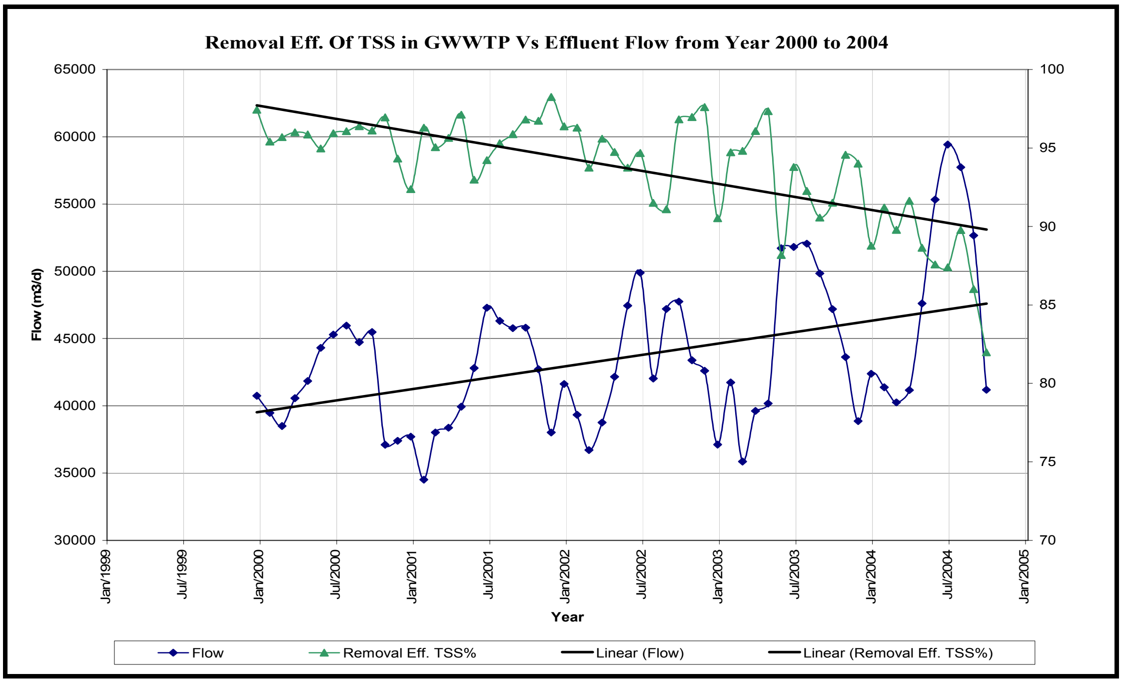


Figure 4.8: Relation between monthly flow rate and percentage of TSS removal from the GWWTP effluent in the period from 2000 to 2004.

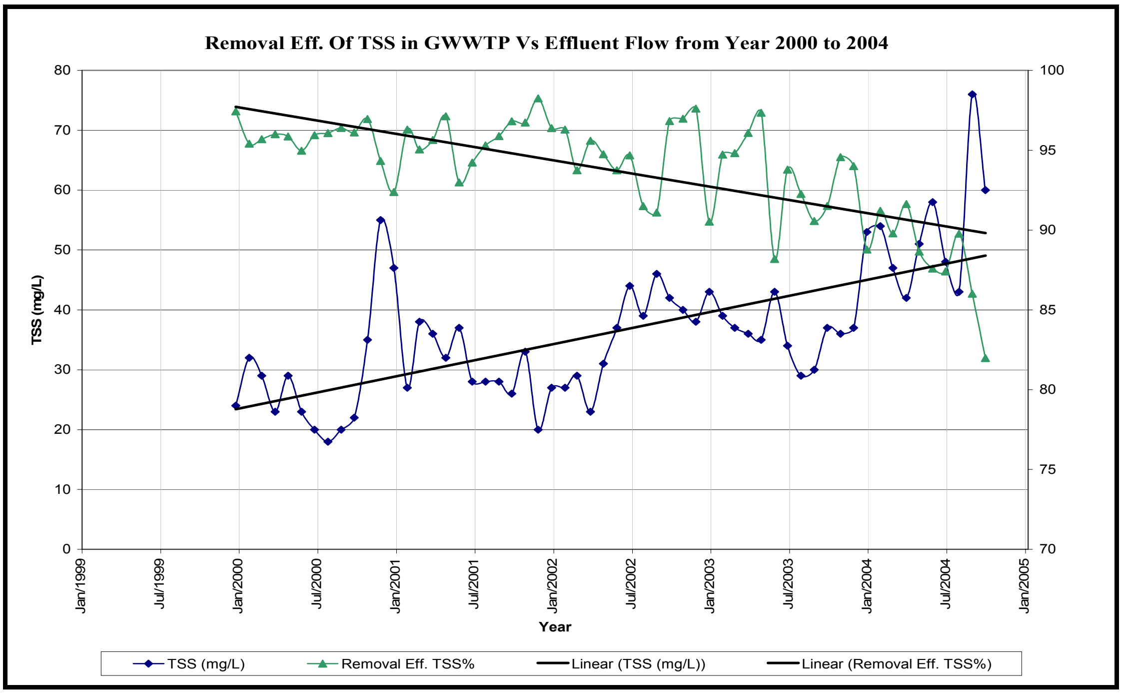


Figure 4.9 Variation of TSS removal efficiency and TSS level in the GWWTP effluent from 2000 to 2004.

The results in annex (1) show the ratios of TSS to BOD for both influent and effluent for the wastewater received by GWWTP. Theoretically the value of TSS should not exceed 80% of the value of BOD in the wastewater (**Horan, 1990**). The ratios of TSS to BOD in the effluent increased from 105% in the year 2000 to 150% in the year 2004. This indicates a decrease in the ability of GWWTP to remove suspended solids. This can be referred to their larger size and more rapid settling velocities and form a sludge blanket in the settling lagoons (anaerobic) (**Crites, Tchobanoglous, 1998**) which could be seen as small sludge islands in the two initial sedimentation lagoons. Consequently, this will reduce the hydraulic capacity of GWWTP (retention time) and the removal efficiency of the TSS.

The organic material deposited on the bed of the treatment process is subject to biodegradation, which results in a depletion of the oxygen source at a faster rate than it is supplied with. This material is therefore subject to anaerobic breakdown, which results in the production of methane and toxic hydrogen sulfide gases. It is worth noting that the values of TSS in the influent in the last five years, except 2001, had exceeded the normal values which indicate no grit removal in the sewage pumping stations that delivers wastewater to GWWTP. It is worth mentioning here that the two anaerobic sedimentation lagoons were cleaned (desludged) in April 2003 which explains the behavior of the curve of TSS which reached a maximum value in March 2003 then started to drop linearly after. The general trend of the removal efficiency of the TSS and the effluent values is shown in figure 4.9 which indicates a decrease in the removal efficiency and increase in the effluent TSS values.

4.1.4 Total Dissolved Solids (TDS)

The results of effluent Total Dissolved Solids (TDS) values and the trend of the TDS of the influent compared with the hydraulic loading of the GWWTP during the study period are shown in figures 4.10 and 4.11. The results show an increase in the average effluent TDS from about 1300 mg/l to about 1650 in the study period. The same tendency was observed for the level of TDS of the influent (figure 4.11).

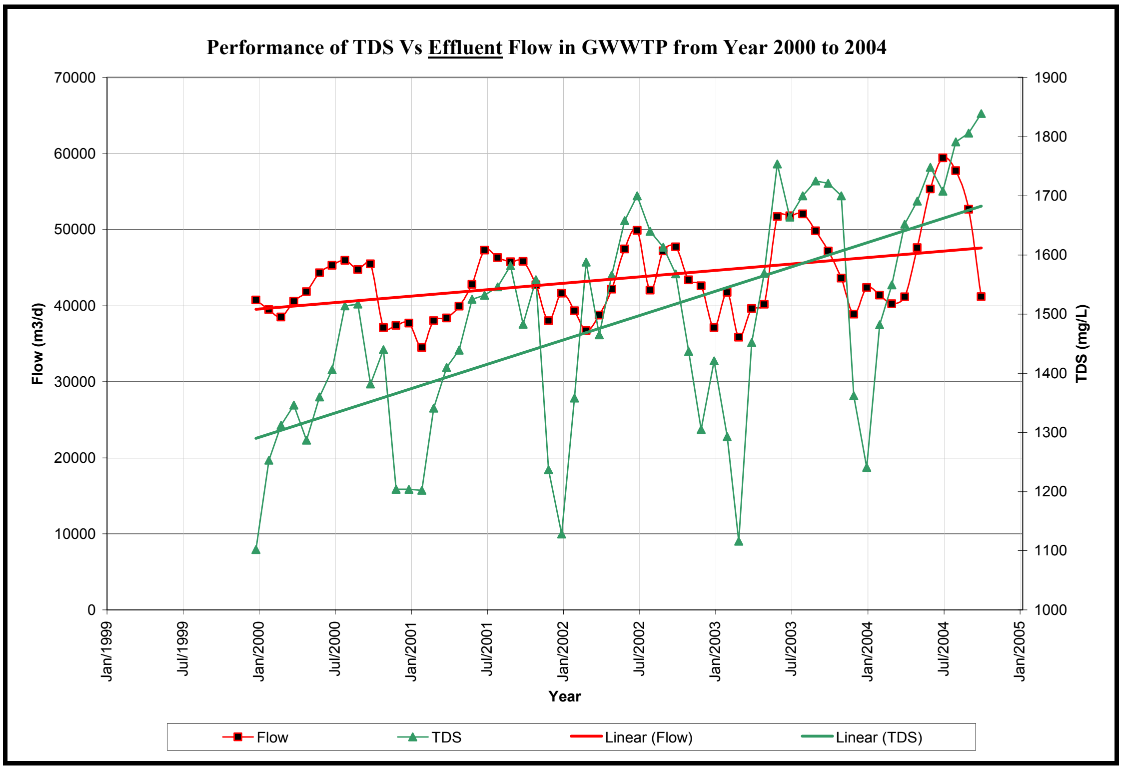


Figure 4.10: The variation of average effluent TDS and average flow rate of GWWTP for the period from 2000 to 2001.

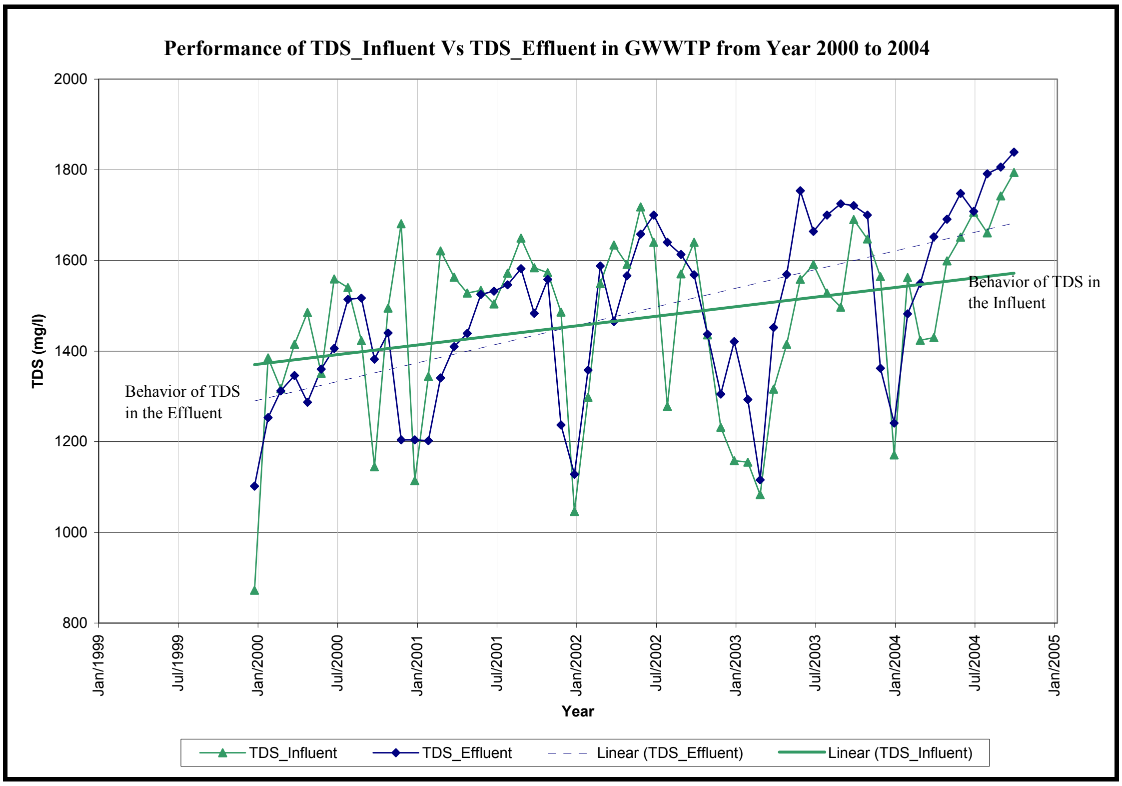


Figure 4.11: The variation of both influent and effluent TDS of GWWTP in the period from 2000 to 2004.

Dissolved solids are a portion of organic and inorganic solids that is not filterable and consists of two main categories: Fixed (noncombustible or mineral components of total dissolved solids) and Volatile (combustible or organic components of total dissolved solids). Normally, solids smaller than one mill micron are fall in this category and in a concentration range of 250-800 mg/l (**Qasim, 1999**). Considering the period from the year 2000 to 2004, the values of the effluent TDS are increasing with time and this is mainly due to the deterioration of water quality of the aquifer, which is reflected on the incoming influent to GWWTP as figure 4.11 showed the trends of both effluent and influent. The values of TDS increased a little when passing through the treatment process due to evaporation which is clear in summer months. This was reflected clearly after the year 2002 as the effluent TDS average values were higher than the influent values as given in figure 4.11.

4.1.5 Total Kjeldhal Nitrogen (TKN)

Figures 4.12 and 4.13 present the results the level of effluent TKN and the general trend between the removal efficiency and the effluent values of the TKN in the study period from the yeas 2000 to 2004. The average effluent level of TKN values were increased from about 50 mg/l to about 70 mg/l in the study period (figure 4.12).

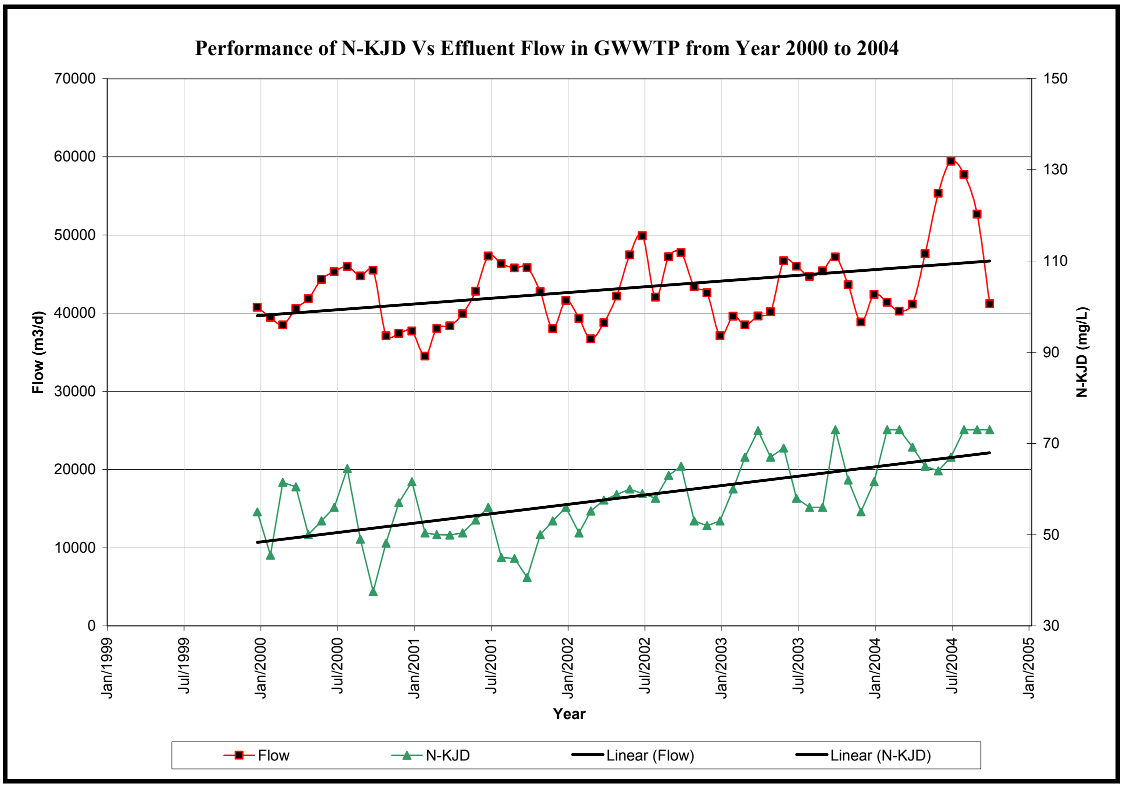


Figure 4.12: Average effluent TKN and flow rate of GWWTP during the study period.

The removal efficiency average was nearly constant and less than 25% in the same period with high variation. The maximum removal efficiency was 55.2% in February 2003 which indicates nitrification-denitrification processes occurred for the ammonia at that time as shown in the table in annex (1).

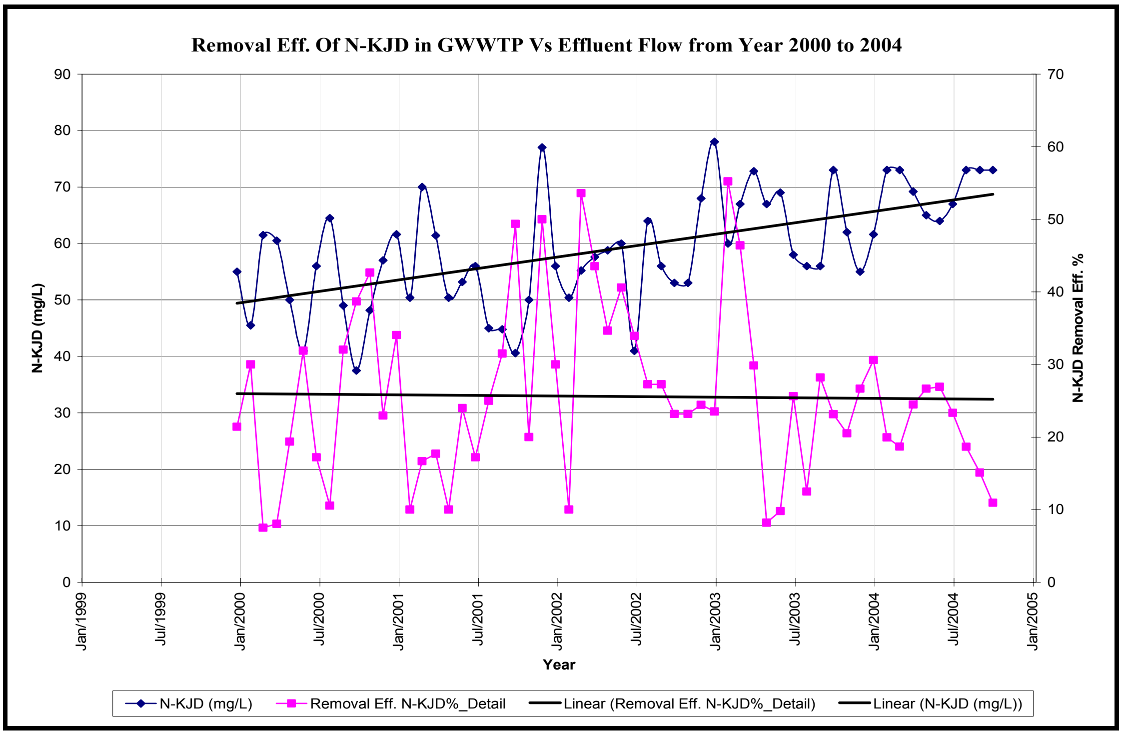


Figure 4.13: Variation of the average removal efficiency and effluent average values of the TKN of GWWTP for the period 2000 to 2004.

The major problems likely to arise from sewage effluent discharges are nutrient enrichment (Eutrophication) with its associated algal blooms and deaeration of the watercourse resulting from oxidation of ammonia to nitrate by nitrification. Where nitrification does occur, and water is abstracted as a potable source, there is the associated problem of nitrate toxicity. Although sewage effluents are not the only source of nitrogen pollution, they are major one, and also the one which is most amenable to control.

The reduced form of total nitrogen content of a wastewater is often referred to as the Total Kjeldal Nitrogen (TKN). During any biological treatment process, from 20-30% of the total nitrogen is removed in cell synthesis by ammonification, in addition a small fraction of the influent nitrogen will be removed during the sedimentation process (**Crites, Tchobanoglous, 1998)** In order to remove the remaining ammonia, a nitrification followed by denitrification processes should be undertaken. The average removal efficiency of TKN for the last five years was about 23% in average, which is in the range of the theoretical value as shown in figure 4.13. The removal efficiency of GWWTP for the TKN is not related to temperature or to hydraulic load of the plant. There is no clear trend for the removal efficiency in winter when the temperature and flow decrease, or in summer when the flow and temperature increase. The GWWTP was not upgraded to nutrients removal, this is explained as follows; a soluble BOD_5 as low as 20 mg/l is required before sufficient oxygen is available to permit nitrification **(Horan, 1991**) and this value of the BOD_5 can never be achieved by the existing trickling filters according to the upgrading design. The nutrient removal is highlighted in this research as part of the evaluation of GWWTP for this aspect.

4.1.6 Faecal Coli form (FC)

Faecal Coli form (FC) is used as an indicator parameter for the potential removal of biological contamination. Figure 4.14 presented the average values of influent and effluent Faecal coli form of GWWTP in the study period from 2000 to 2004. The average values of FC in influent were $5x10^8$ cfu/100cm while the average values of FC in the effluent is $6x10^6$ cfu/100cm. The difference between the influent and effluent FC is two logs of cfu/100cm, which indicates a very low microbiological removal efficiency of GWWTP. One log is removed in the anaerobic pond and one is removed in the facultative pond (final settling channel).

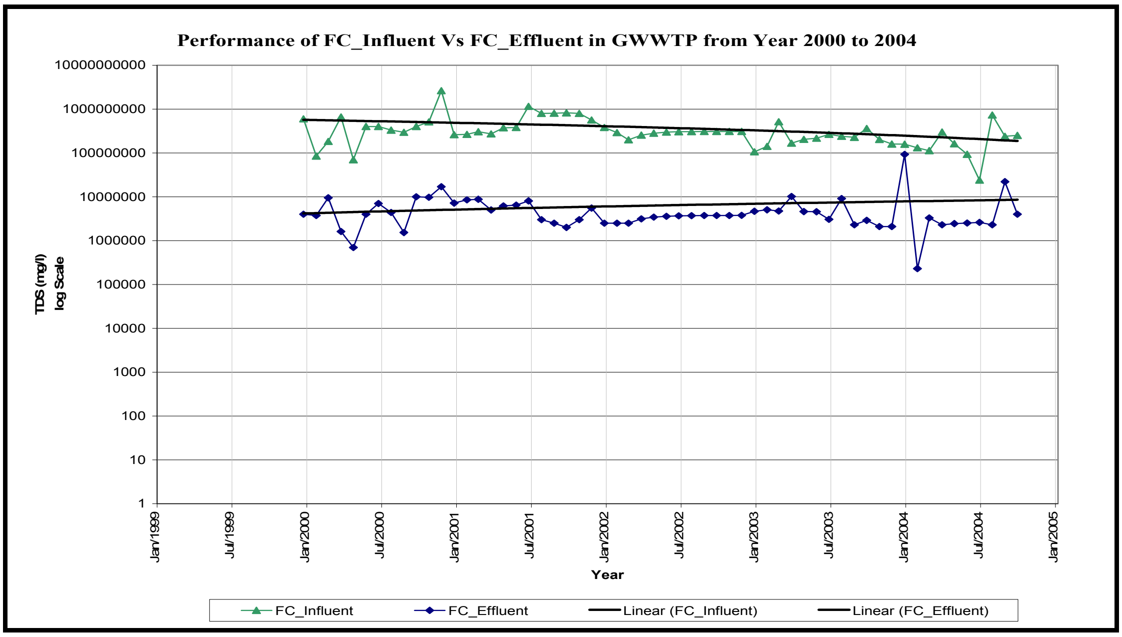


Figure 4.14: Average values of influent and effluent Faecal coli of GWWTP in study period from 2000 to 2004.

The plant is equipped with a chlorination station with dosing pumps and the sodium hypochlorite should be used for disinfection. Due to lack of financial resources, the chlorination unit had been stopped since the summer of 1999 and this is the main reason of having the high values of the Faecal Coliform in the effluent. The discharged effluent to the sea or to the infiltration basins will have a bad environmental impact on the on the aquifer and bathing seawater quality, and it is worth mentioning here that the beach was closed for bathing for three consecutive summers(2004,2005,2006).

4.2 Performance of each of the GWWTP Components

The existing GWWTP plant facilities consist of three anaerobic ponds in series followed by an aerated lagoon and two bio-towers (see figure 2.1). The aerated lagoon is equipped with floating mechanical aerators and the bio-towers are filled with high-density plastic media. The bio-tower effluent is directed to an effluent polishing pond where solids sedimentation occurs and limited solids collection is possible. The assessment of the performance of each component of the plant is important to identify whether each component is performing according to the proposed performance and highlight the deficiencies in the system.

Contrary to the data available for the whole system, the data for each component performance is little and rare. The tests for the BOD_5, COD, and TSS were only performed upon request when there is a problem in the system results and a need for maintenance is required. Another factor for the limited number of the tests for each component of the system is the cost of such tests and the efforts required to be performed by the lab and operation teams. The available data analyzed in this research are scattered for short intervals of time but almost covers the performance of each system component along the study period which gives a good indicator for their performance.

In the last quarter of the year 1999 and upon a request from MOG Mayor, Dr. Afifi conducted a COD test for each of GWWTP components with the operation of 7 aerators and 14 aerators in the period from 27/9/1999 to 3/10/1999 to determine whether it is economic to operate the 14 aerators. The COD test was chosen due its short time results. The results of the removal efficiency are demonstrated in the following two graphs (4.15a, 4.5b) which represent the cumulative removal efficiency of the combined components for the whole system (Annex c).

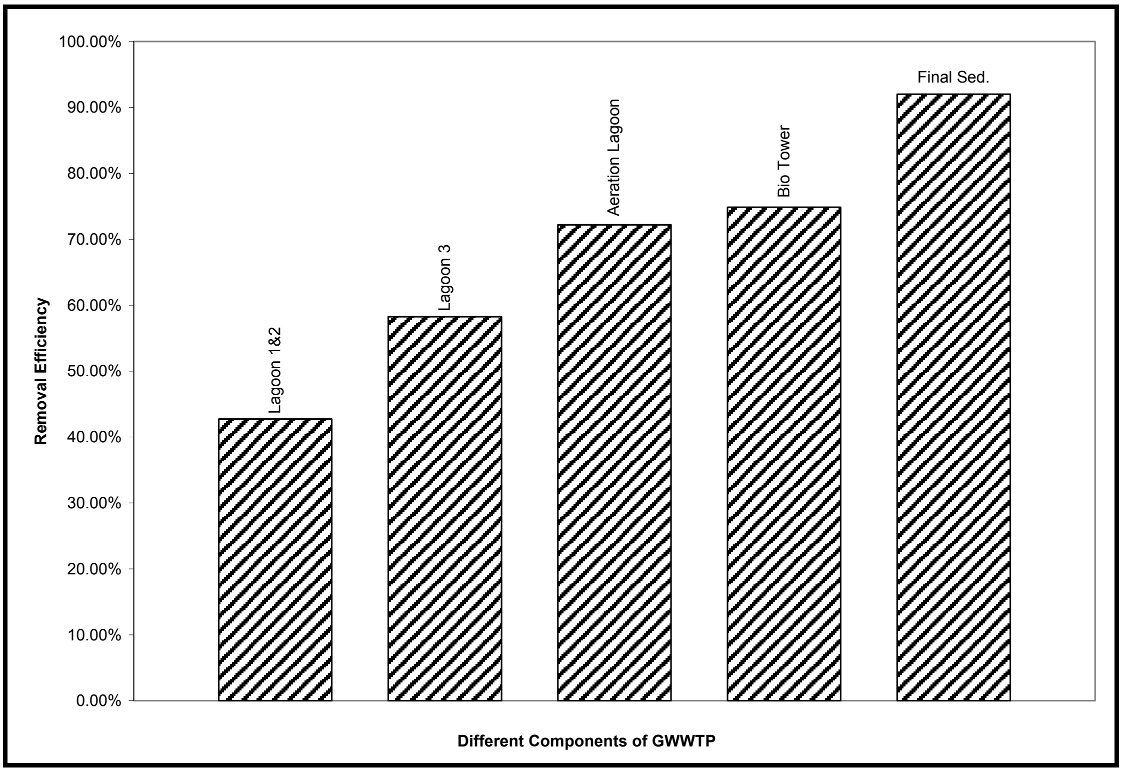


Figure 4.15a: The cumulative removal efficiency of each component of GWWTP for the period from 27/9/1999 to 3/10/1999.

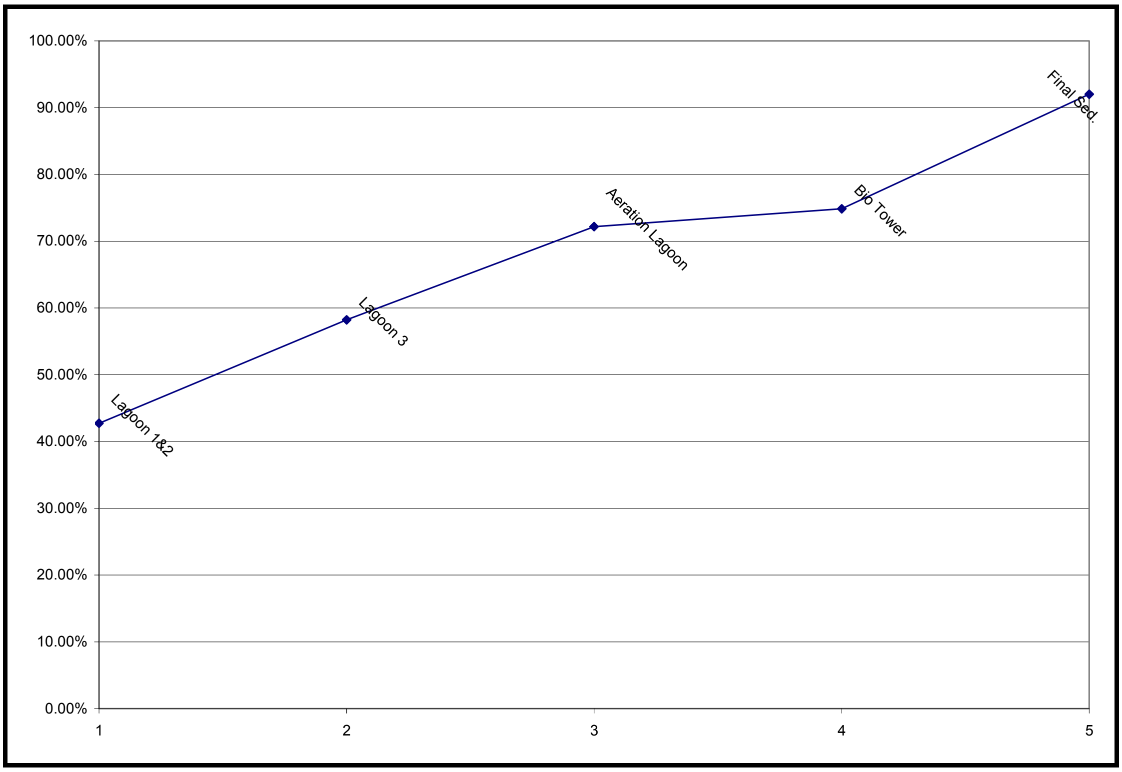


Figure 4.15b: The removal efficiency of each component of GWWTP for the period from 27/9/1999 to 3/10/1999.

In the year 2003, GWWTP was under the direct supervision of the operator Lions Des Eau & Khatib and Alami(LEKA) of the World Bank project and the operator(LEKA) made contracts for cleaning the two initial sedimentation lagoons and replaced the arms of the Bio-Tower, and to evaluate the results of his work, BOD_5 tests were made for each components of GWWTP and the results are given in annex C. Figure 4.16 demonstrate the cumulative BOD_5 removal efficiency of different treatment system components which is most reliable of the results.

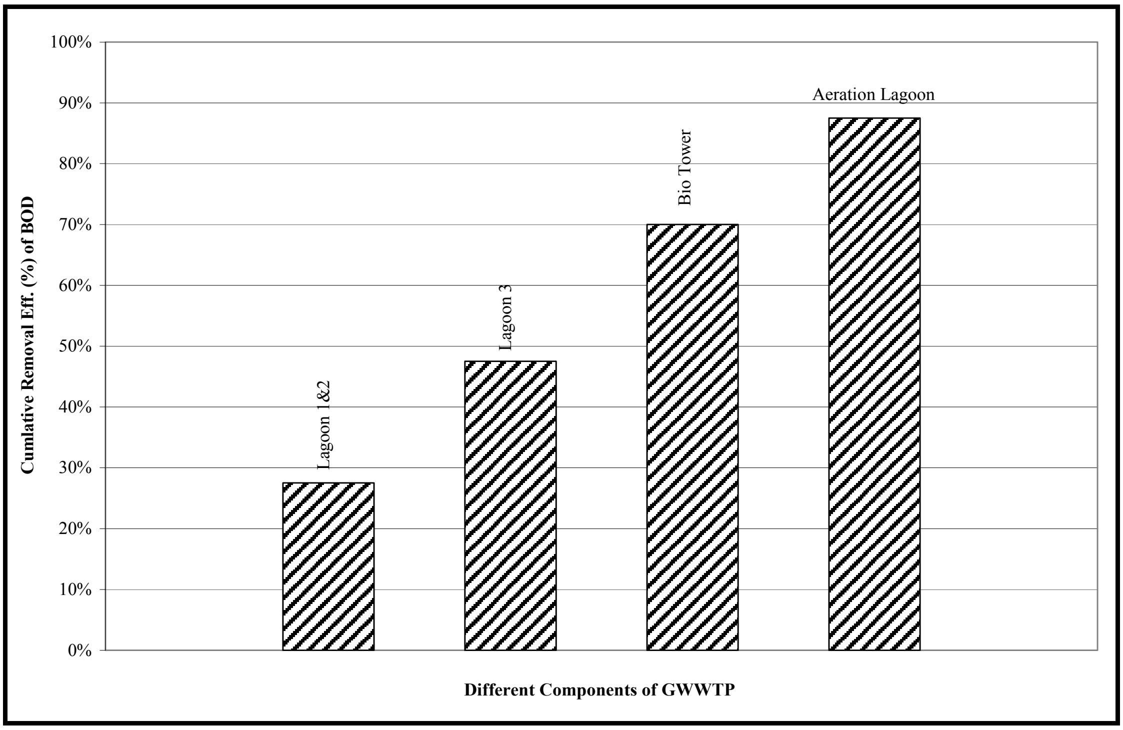


Figure 4.16: Cumulative BOD_5 removal efficiency of GWWTP components for the period of February and June 2003.

4.2.1 Initial Sedimentation Lagoons 1&2

The objective of treatment by sedimentation is to remove readily settleable solids and thus reduce the suspended solids content. Primary sedimentation is used as a preliminary treatment step in the further processing of the wastewater. Efficiently designed and operated primary sedimentation tanks should remove from 50 to 70 percent of the suspended solids and from 25 to 40 percent of BOD_5 (**Crites and Tchobanoglous, 1998**).

Comparing the available data for measuring the removal efficiency of the two initial sedimentation lagoons, it is clear that at the beginning of the operation of GWWTP in 1999 the BOD_5 removal efficiency was more than 40% (figure 4.15). A well designed sedimentation tank can remove 40% of the BOD_5 in the form of settle able solids **(Horan, 1991**). In the year 2003, the BOD_5 removal efficiency dropped to almost 25%. The plant has no grit removal chamber and hence all grit and sand coming from the sewage pumping stations was settled in the two initial sedimentation lagoons and compiling, leading to decrease in the volume and retention time in the two lagoons. Since the plant started operation in late 1998 till 2003 without sludge cleaning of the sedimentation lagoons which represent additional reason behind the drop in the removal efficiency of this unit. Further to that, the increase of the hydraulic load was significant in this period which decreased the retention time.

4.2.2 Anaerobic Lagoon 3

In the year 2003, the BOD_5 removal efficiency was about 27% which means that the removal efficiency of the anaerobic lagoons was not altered with time and the increase of average flow. The average daily flow for the year 2003 was about 44, 000 m^3/day and a maximum flow of about 52, 000 m^3/day. The explanation for this is that the total volume of the three anaerobic lagoons together is approximately 80,000 m^3 which gives a retention time for the influent of 44, 000 m^3/day for two days which is enough for the anaerobic lagoon to remove the designated value of the BOD.

The total BOD removal for the three anaerobic lagoons was 50% as shown in figure 4.16, which is according to the theoretical values of 40-60% BOD_5 removal **(Horan, 1991).**

4.2.3 Bio-Tower

In the upgrading design of GWWTP performed by the USAID considered the bio-towers as the main organic treatment component in the GWWTP and the calculation of the removal efficiency was about 85% given that the average temperature in summer is 30 degrees and the average in winter is 20 degrees (**CAMP, 1997).** The records of GWWTP show that the average summer temperature is 27 degrees and winter average temperature is 18 degrees. The normal BOD_5 removal efficiency ranges from 80 to 85 percent for high rate trickling filters (**Qasim, 1998)** with little nitrification.

The results of the data collected within the years 1999, 2002 and 2003 showed that the removal efficiency of the Bio-tower was almost constant and was around 75% of the filtered BOD_5 (after settling) as shown in figure 4.17. This is clear when taking the average BOD_5 removal of the Bio-Tower for the three years as shown in Annex F. According to the formula **(Metcalf, Eddy, 1991**) of calculating the removal efficiency of the trickling filters, the efficiency drop could appear significantly if the change in the loads is significant.

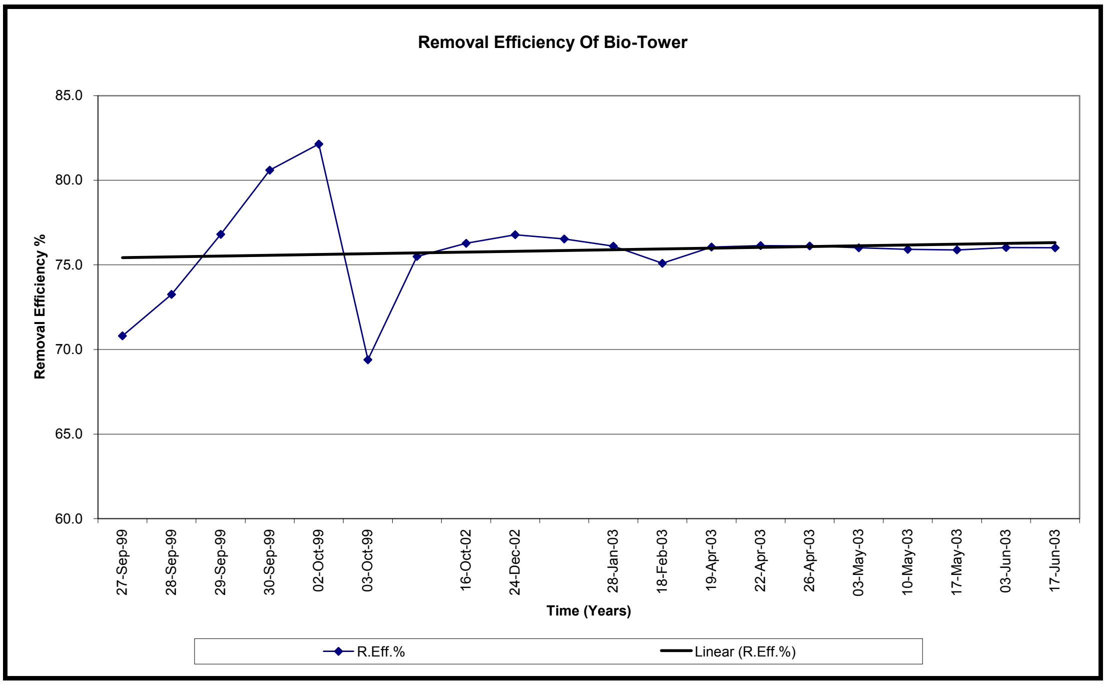


Figure 4.17: The measured removal efficiency of GWWTP Bio-Tower throughout the years: 1999, 2002, and 2003.

The annual increase in flow of GWWTP in the period from the year 2000 to the year 2003 was around 1000 m^3 each year and the only a jump in the flow was in 2004 (around 4000 m^3/year). Unfortunately there are no records for the year 2004 to measure the removal efficiency of each component of GWWTP although it is expected according to theoretical calculations for a flow of 55000 m^3/day to drop to 63% removal of BOD and this is close to realistic since the recorded values for the years 1999, 2002 and 2003 are almost typical to the calculated values.

The filters are not designed for Nitrogen removal as mention before and no data is available to measure the performance removal of it. According to (**Qasim, 1998**), the nutrient removal efficiency ranges from 8-15% for the high rate trickling filters.

4.2.4 Aerated Lagoon

Aerated treatment systems are those in which microorganisms remain in suspension, and aerobic conditions prevail. For Modified aeration, the process is used as an intermediate treatment to reduce the organic loading in subsequent process. It is similar to conventional treatment with short aeration period (2-3 hours) (**Qasim, 1998**). In general, the overall BOD_5 removal efficiency of the aerated lagoons ranges from 40-80% and the soluble BOD_5 removal ranges from 90 to 97 % **(Qasim, 1998**). The only available data found for the removal efficiency of the aerated lagoons was on September and October 1999 and only for one week time. The COD removal efficiency is shown in figure 4.18.

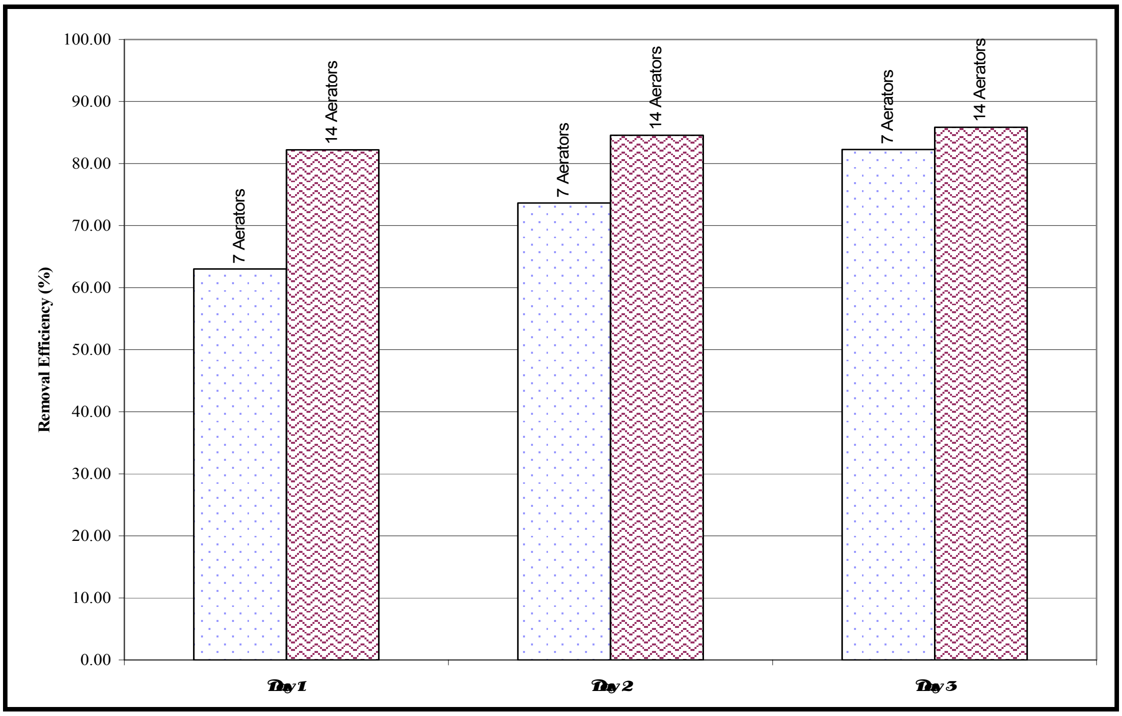


Figure 4.18: COD removal efficiency of Aerated Lagoons of GWWTP for the period from 25 September to 5 October 1999 for 7 and 14 aerators operation.

The average removal efficiency for the aerated lagoons was around 15% according to the operation scheme of GWWTP. The removal efficiency of the aerated lagoons using 14 surface aerators exceeds the removal efficiency using 7 aerators by an average of 5-10% as can be shown in figure 4.18 which is considered very low and negligible . This is related to the scheme of the flow of wastewater through the treatment plant since the water passes through the Bio-Tower before passing the aeration lagoons. The logic for this is based on the nature of the organic material in wastewater. A portion of the BOD is readily biodegradable and is removed rapidly by the filters, the remainder is less readily degradable and in overloaded filters this fraction is discharged to the next treatment component which in GWWTP case is the aerated lagoon. For the reason that the remained organic substance is biodegraded more slowly, it requires more retention time in the aerated lagoons to be removed completely which is not the case on GWWTP and hence the removal efficiency is considered to be low in the aerated lagoons.

4.2.5 Final Sedimentation Tank

The removal efficiency of the final sedimentation tank (clarifier) could reach up to 40% of the BOD **(Qasim, 1998).** As fig.4.15a show that removal efficiency of the final sedimentation tank reached to around 15%. The role of the final clarifier is important since after the biological treatment process either by aeration or by the trickling filter, the TSS need to be settled after the turbulence caused by the biological treatment process. It is clear that the removal efficiency of the final sedimentation is much below the theoretical proposed values due to the over loading of the plant.

4.3 Proposed System Modifications

There are three main problems facing GWWTP that are hindering the goal of the treatment process. The hydraulic load is the most urgent problem that needs to be solved. The high values of the TSS, which also settles in the lagoons leading also to a decrease in the hydraulic capacity of the plant, need also to be solved. A treatment management process needs to be recognized that will attain the effluent quality.

4.3.1 Hydraulic Capacity Proposed Modification

This research reviewed individual unit processes and evaluated many treatment alternatives. Due to the unique conditions and constrains of the Gaza strip, the following parameters were set as an evaluation criteria for the selection of any new treatment process for the GWWTP:

1. Land Availability
2. Construction Cost
3. Running Cost (Operation and Maintenance)
4. Local Operating Experience
5. Process Performance

Three options seem to be the most practical for the expansion and upgrading of GWWTP:

- Construction of a new anaerobic lagoon.
- Construction of a new trickling filter.
- Modification of the process to activated sludge

4.3.1.1 Alternative 1: Addition of a new Anaerobic Lagoon

Anaerobic ponds are simple and effective for small plants if combined with an aerated pond and a clarifier. An effective anaerobic lagoon of a capacity of 50,000 m^3 (24000 m^2) would increase the retention time of the current flow into more than two days. According to the estimated construction cost **(CMWU, 2005)**, the required budget will be around U.S $ 300,000, and the running cost is considered to be very low.

4.3.1.2 Alternative 2: Bio-tower

The biotower alternative consists of converting the Anaerobic Pond No.3 to an aerated lagoon and adding an additional biotower **(CAMP, 2001**). The aerated lagoons will run in series with the biotower group, and the biotowers will run in parallel. The power consumption for the proposed aerators and the biotower pumps is considered to be very high, and putting into consideration the required maintenance for the machinery and biotower arms, this option's running cost is considered to be very high. The construction cost of this alternative is estimated to be U.S. $ 2.5 million **(PWA, 2001).** Around 5000 m^2 area is required for this alternative, and since the existing of these biotowers in GWWTP, the local experience with operation and maintenance for this option is considered to be good.

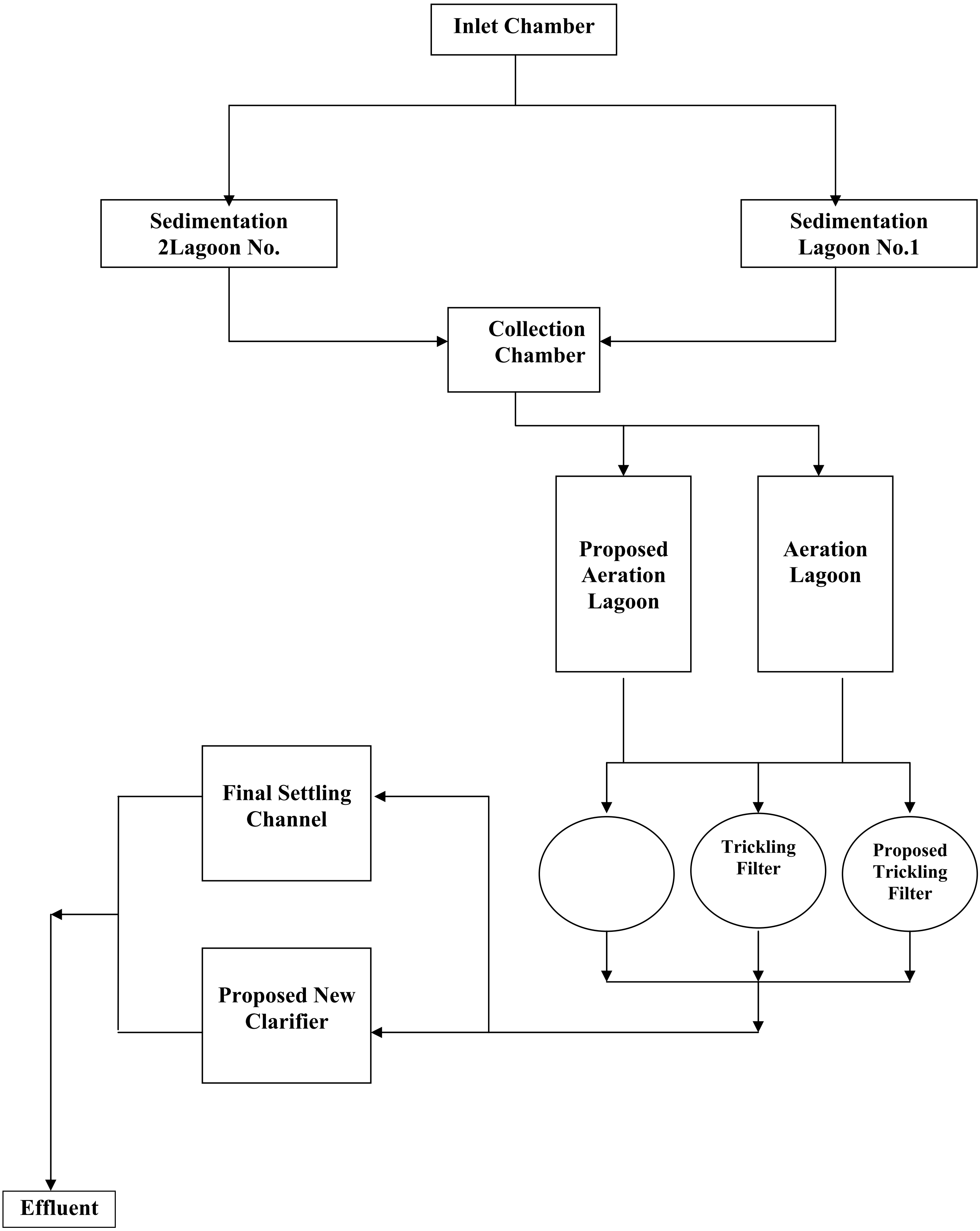


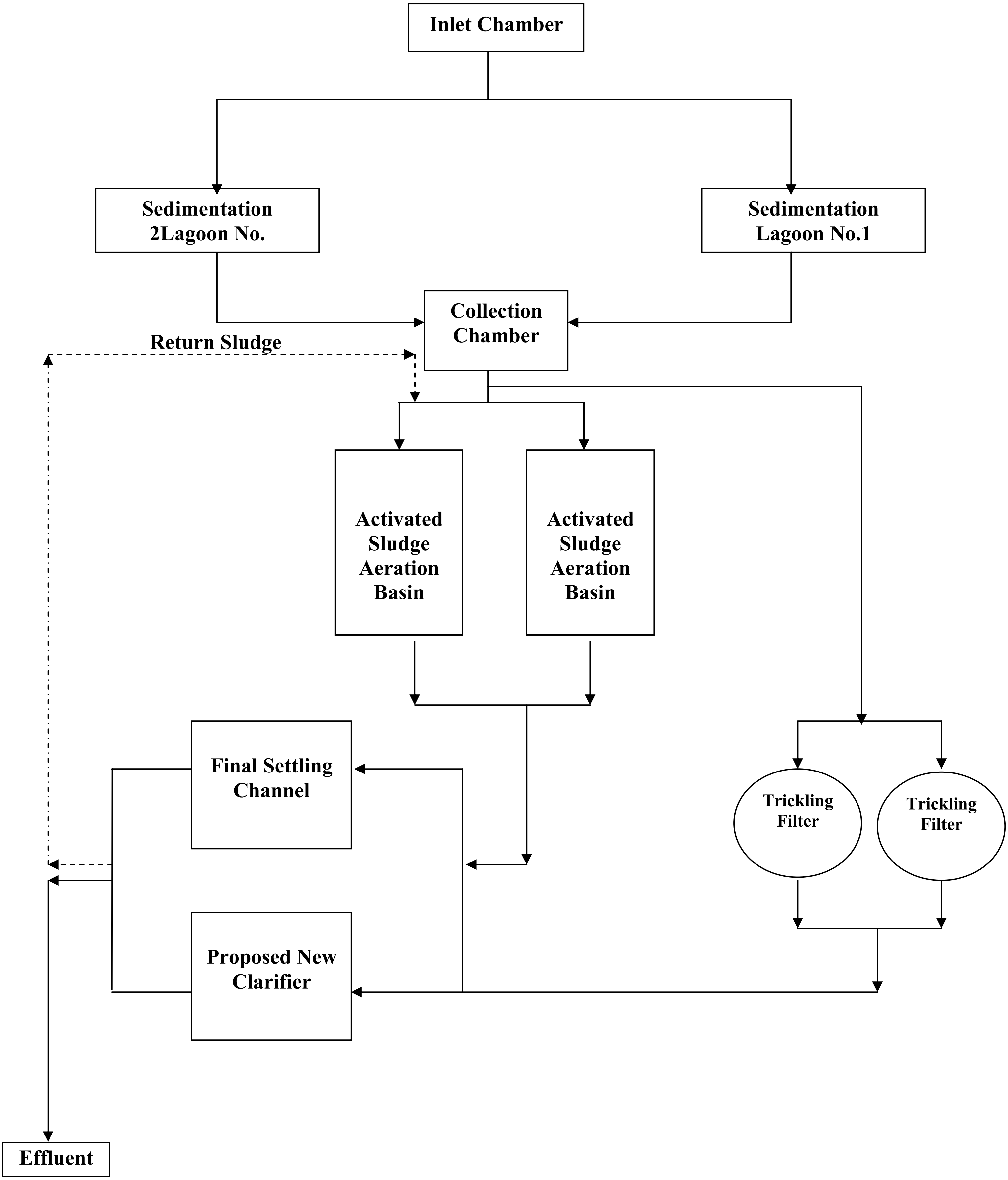
Figure 4.19: flow diagram for Alternative 2: Addition of a new trickling filter .

4.3.3.3 Alternative 3: Activated Sludge

The activated sludge alternative consists of converting Anaerobic Pond No. 3 to an activated sludge aeration basin and converting the existing aerated lagoon also to an activated sludge aeration basin. The activated sludge facility would run in parallel with the existing biotower group. This system is never practiced before in Gaza strip and no local experience is available to run such a system. Land is considered to be minor for this option, but the construction cost is estimated to be U.S. $ 3.0 Million **(CAMP, 2001).** The running cost for blowers, aeration system, return sludge pumps and thickener pumps is considered to be very high. The quality of the effluent of the activated sludge is considered to be the best among all the available alternatives.

The comparison between the three options is shown in table 4.2 below were values of Minimum (min), Moderate (Mod) and Maximum are given for the evaluation criteria.

Figure 4.20: flow diagram for Alternative 3: Activated Sludge option



To evaluate the most appropriate treatment process for the proposed for GWWTP, seven criteria were evaluated and weight by academics, municipal staff working in the GWWTP, consultants and Palestinian Water Authority wastewater project managers and the result of the survey is shown in Fig.4.2 and the questionnaire is demonstrated in Annex H.

The Running (O&M) is the most important factor in the evaluation criteria due to the economic situation in Gaza and the sewage treatment cost is not incurred in the municipal billing system , so it was given a higher weight in the evaluation process. The Process Performance was given a lower weight in the evaluation (the second rank). The three remaining criteria (Land, Sludge Handling, Land Availability and Local experience) are given almost the same weight in the evaluation due to the equity between them in the importance according to the survey. Table 4.2 shows the proposed weights for each criterion.

Criteria	**Weight %**	**Comments**
Land Availability	15	Area Required in m^2
Construction Cost	13	Amount in U.S.$
Running Cost	26	Amount in U.S.$/ year
Process Performance	19	The BOD/TSS in influent
Sludge Handling	14	Sludge degradation
Local Operating Experience	13	Available or not Available

Table 4.2: The evaluation criteria and their weights according to Questionnaire.

The land availability is calculated in Annex E for the proposed Anaerobic lagoon and the land required for the trickling filter option is measured according to the existing ones in GWWTP. The third option is estimated from the USAID study in 2001, the Feasibility Study and Conceptual Design for Gaza Wastewater Treatment Plant.

The running cost is calculated for the third treatment option (activated Sludge) from the cost equations given in Annex C in **(Qasim.1999)** which is derived from the EPA's *Area wide Assessment Procedure Manual: Performance and Cost*, and the the running cost for the first alternative is calculated from the data available for GWWTP at an average daily flow of 52000m^3 /day and the value of the second option is also calculated from the available data for the power consumption of the trickling filters and the aerators. While the construction cost is mentioned in the CMWU report for the first option and in the USAID Feasibility Study for the other two options. The sludge handling will be given three values (Excellent, Good and Poor) depending on the process retention time and sludge degradation, while the process performance will be given the values of BOD theoretically expected from the proposed process configuration, and the local operating experience will be given one of the three values(highly available, available and little available).

Treatment process	**Land (m^2) Availability (requirement**	**Construction Cost (US$)**	**Running Cost (US$/ year)**	**Process Performance**	**Sludge Handling**	**Local Operating Experience**
Anaerobic lagoon (Alt.1)	15,000	300,000	153,500	35	Excellent	Highly Available
Trickling filter (Alt.2)	4,000	2,500,000	209000	30	Good	Available
Activated sludge (Alt.3)	4,000	3,000,000	274,500	20	Poor	Little Available

Table 4.3: comparison between the proposed three alternatives of treatment for the GWWTP modification.

The evaluation of the best option for the treatment process is shown in table 4.4. which shows that Alternative (construction of an anaerobic pond and splitting the flow) is the best option according to the proposed criteria?

Criteria	Max Wt.	Alternative(1)	Alternative(2)	Alternative(3)
Land (requirement)	15	4	15	15
Construction Cost.	13	13	1.6	1.3
Running Cost.	26	26	19	14.5
Process Performance	19	10.9	12.7	19
Sludge Handling	14	14	10.5	7
Local Operating Experience	13	13	13	6.5
Total	***100***	***80.9***	***71.8***	***63.3***

Table 4.4: The weighted evaluation of the three criteria according to the proposed Criteria.

By the year 2011, the peak influent flow is expected to reach 70,000m^3/day **(CAMP, 1999.).** The year 2011 was the proposed date for the operation of the Central Wastewater Treatment Plant in Bureij, which was proposed and financed by the KfW. An anaerobic lagoon with a capacity of 78,600 m^3 and a clear depth of 4 m will increase the retention time of the influent for more than two days (see Annex E) which will produce the best removal efficiency for the anaerobic treatment which is important in decreasing the BOD and settleable solids and decreasing the load and improving the removal efficiency for the next treatment processes. The choice of the construction of anaerobic lagoon seams to be suitable since the construction cost of such a lagoon will not exceed US$300,000 and the removal efficiency of the anaerobic lagoons could reach up to 60% of BOD and up to 70% of the TSS. The proposed space for this lagoon is available within the land area of GWWTP between the two initial sedimentation lagoons and the second anaerobic lagoon (see figure 4.19). The required surface area of the proposed lagoon is 28,000 m^2.

The piping system between the newly proposed lagoon and the other anaerobic lagoons needs to be scaled according to the assumed influent (70,000 m^3). Also the rest of the piping system within the treatment plant needs to be redesigned according to the expected peak flow of 70,000m^3. The most dominant part of the GWWTP for the hydraulic capacity is the effluent pumping station which is limited to maximum flow of 48,000 m^3/ day.

4.3.2 Proposed Modification for TSS Removal Efficiency

The analysis of the results shows increased values of TSS than normal conditions. The high TSS content of the effluent will affect the reuse purpose of the effluent specially when used in the percolation lagoons to recharge the aquifer. It also affects the media of the trickling filters by clogging and thus affects the performance of the trickling filters. Moreover, the existence of sand in the water will cause wear problems for the pumping stations of the bio- tower and effluent pumps.

The high sand and grit content in the influent led to the settling of the sludge and grit in the two sedimentation lagoons at the inlet of the treatment system. That almost filled them completely, reducing the hydraulic capacity of the plant and hence decreasing the treatment efficiency of GWWTP. A grit removal channel along with a final sedimentation tank (final clarifier) is proposed to be added to the plant. The proposed grit removal channel could be constructed to the southern part of the inlet flow chamber of the GWWTP parallel to the two initial sedimentation lagoons (figure 4.19) with a total length of 31 m and a depth of 2.5 m (see Annex G) with an option to divert the flow from the channel to the two initial sedimentation lagoons in case of cleaning the channel. The land is available within the area of GWWTP for the proposed grit removal channel and the estimated cost for construction is US$40,000**(CMWU, 2005**). The removal of grit could be done using a normal wheeled excavator on a weekly basis.

Since the settling channel in the final settling lagoon will be hydraulically overloaded, it will be working only as a distribution and flocculation unit. Settling will take place mainly in the main part of the settling lagoon. In order to improve the flocculation of the sludge and hence the settlement efficiency, a baffle could be constructed as a floating curtain wall along the overflow weir. By including a bottom-up flow, using the effect of a "floc-filter", sedimentation could possibly be improved and thus effluent TSS is minimized. Land is available for the construction of the final sedimentation tank in the southern area of the final settling pond (figure 4.19). The estimated construction cost for the tank will be around US$180,000.

4.3.3 Proposed Flow Scheme

To have the optimal performance of the biological treatment units, the ***Bio-Towers*** with the highest removal efficiency in the GWWTP SYSTEM and the ***Aerated Lagoon***, the flow after the anaerobic lagoons should be split into two flows: one for the bio-towers and the other to the aeration lagoon (figure 4.19). The maximum flow to the bio-towers should not exceed 34,000m^3 to attain an 80% removal efficiency of the bio-towers according to the design criteria **(USAID, 1996**). Parallel operation of trickling filter and aerated lagoon reduces the load to the aerated lagoon (limited aeration capacity) and uses the BOD removal capacity of the trickling filters to a higher extent. The hydraulic head losses can be reduced as well.

This measure should be implemented directly after cleaning the anaerobic lagoons, since this can be done without additional equipment and cost.

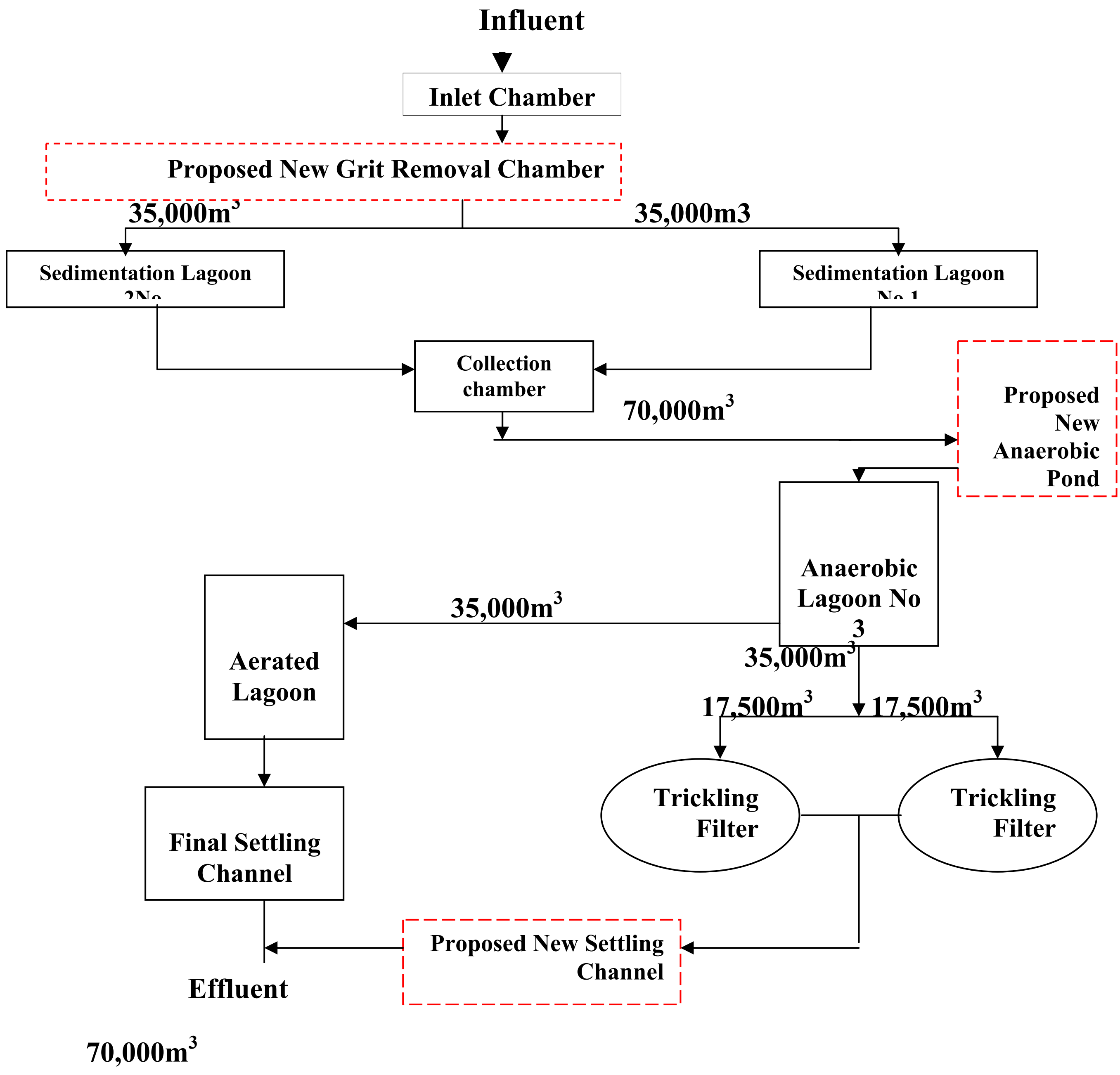


Figure 4.21: Proposed Flow Scheme and Proposed System Modifications of GWWTP.

4.3.4 Plant Management

Since October 2004 the GWWTP lab was out of work due to the Israeli aggression against the plant. The automatic sampler was also stopped since then and up to this date the sampler and the lab are not in performance. Moreover, the flow meter of the plant is also out of service since February 2004. Before the withdrawal of the Israeli Army from Gaza Strip, it was dangerous and some times forbidden to go to GWWTP. The MOG, even with the bad financial situation, could have made the flow-meter and the sampler repaired or replaced because of the importance of these two instruments for the monitoring program of the effluent of GWWTP. The lab should be independent from the plant operation department from the administration point; the lab should be reporting any abnormal result in the various plant components immediately and daily to the water and wastewater department head who takes the necessary actions in case of any operational deficiency.

There should be a clear daily, weekly and monthly check list for the operational personnel along with a clear job description for each member and a clear plan for the preventive maintenance on a weekly, monthly and yearly basis to keep the plant in good shape. Moreover, a quality assurance and instruments calibration plan for the lab with the periodic maintenance plan for the instruments. Removal efficiency of each plant components should be measured at least once monthly; in the summer and winter to have a more specific indicator about the performance of the plant.

Finally, a detailed training program along with the human resources management plan should prepared to highlight the needs of the operation team and lab staff.

CHAPTER 5: CONCLUSIONS AND RECOMMENDS

5.1 Conclusions

The parameters which led to the bad performance of GWWTP were highlighted by analyzing the historical records of GWWTP in the period from the year 2000 to the year 2004 when the GWWTP lab was destroyed by Israeli army and the sampler was completely stopped and the flow meter was out of service. The research came with the following conclusions:

1. The GWWTP was upgraded to receive 32,000m^3/ day influent, a value which was attained in the first year the plant was in operation. Due to the natural increase in population and new area connection to the sewage network, the influent to GWWTP has continued to increase dramatically and today it is estimated to be around 55,000m^3/ day. So, the plant is hydraulically over loaded and thus the water retention time in the plant is cut into half the designed period leading to poor plant performance in the treatment process.
2. The TSS is higher than usual in both influent and effluent. For influent the reason behind this high value is the malfunctioning of the sand traps and grit removal in the sewage pumping stations and this value continues to be high throughout the plant treatment stages leading to high maintenance cost and bad effluent quality for reuse purposes especially for infiltration.
3. The removal efficiency of GWWTP has decreased throughout the years from 2000 to 2004 from 95% to 91% for the BOD and from 96% to 88% for the TSS and from 91% to 87% for COD. The main components of the plant which is the bio-tower and the anaerobic lagoons are performing with less than there design removal efficiency, which reflected in general as deterioration in the treatment process for GWWTP. This problem was basically due to the underestimates in the design criteria for the projected flow for the period in which the plant should be operating and the proposed design temperatures for summer and winter.
4. The removal efficiency of the Faecal Coli form is very minor limited only to two logs and the effluent content of the coli form is considered to be high putting in mind that this effluent will be discharged to the sea or sometimes used for agriculture which will cause bad environmental impact on man. In addition, the plant is not designed for nutrient removal.

5. The absence of financial resources from the MOG to be allocated for the GWWTP had led to significant operation and maintenance problems such as the compilation of sludge in the anaerobic and settling tank which decreased the treatment process dramatically and the disability of the MOG to remove the garbage packed at the eastern side of GWWTP which has led to many problems due to flying plastic bags into machinery and treatment equipments.
6. The absence of a grit removal channel had led to the accumulation of sand which was combined with sludge in the two initial sedimentation lagoons and the anaerobic lagoon leading to significant decrease in the volumes of these anaerobic lagoons and hence decreasing the retention time significantly which was finally reflected on the removal efficiency of the plant.
7. The study proposed system modifications includes hydraulic capacity modification, modification for TSS removal efficiency, proposed flow scheme and plant management system.
8. There is no difference between operating 7 aerators and 14 in the removal of organic matter from the wastewater influent (BOD &COD) in the GWWTP in the current operation scheme.

5.2 Recommendations

- The political situation in Gaza is unstable and consequently affects the donors contribution towards developing the water sector in general and temporarily solutions becomes permanent solution. So GWWTP should be part of the development plant of the Palestinian Authority until a real sensible alternative is existed on the ground.
- Due to the population growth and sewer connections to GWWTP, it is recommended to allocate some land in GWWTP neighborhood for any future expansions of the plant.
- There should be a clear Operation and Maintenance plan for the plant a long with a proposed budget to be allocated either from the CMWU or others.
- To implement personnel safety program including the necessary training, proper safety clothes and the necessary premises for cleaning and taking food. A first aid material should always be available for any accidents at site.

- To construct the proposed new anaerobic lagoon and the grit removal channel and the final sedimentation tank with an estimated cost of US$ 520,000 which will sustain the plant till the year 2015.
- The lab instruments are out of date and a complete lab instruments should be supplied along with an automatic refrigerated sampler for day and night flows along with portable instruments for site inspections.
- To adopt flow- proportional composite sample technique in taking the samples for more accurate and reflecting results.
- The solid waste dumping site should be removed as soon as possible and should be in the first priorities of MOG for its effects on the environment, aquifer and the operation and maintenance cost of GWWTP. Moreover, the removal of this site could make at lest 20 donems of land available expansion of GWWTP.
- The sand and grit removal screens in sewage pumping stations to GWWTP should be maintained and operated properly to reduce the quantities of sand entering into GWWTP and thus reduce the desludging time and cost of initial sedimentation lagoons and enhance the removal efficiency of the plant.

Glossary

Aerated lagoon: A holding and /or treatment pond that speeds up the natural process of biological decomposition of organic waste by stimulating the growth and activity of bacteria that degrade organic waste.

Aerator (mechanical): A means of aerating wastewater to increase dissolved oxygen, to enhance aerobic treatment and reduce offensive odors.

Aerobic Treatment: Process by which microbes decompose complex organic compounds in the presence of oxygen and use the liberated energy for reproduction and growth. (Such process include extended aeration, trickling filtration, and rotating biological contactors).

Algae: Simple rootless plants that grow in sunlit waters in proportion to the amount of available nutrients. They can affect water quality adversely by lowering the oxygen content of water.

Anaerobic: A biological environment that is deficient in all forms of oxygen, especially molecular oxygen.

Biochemical Oxygen Demand (BOD): A measure of the amount of oxygen consumed in the biological process that breaks down organic matter in water. The BOD is an indicator for pollution.

Bio-filter: Known also as trickling filter, bacteria bed, or percolating filter. It is a reactor of rectangular or circular plan which is filled with a permeable media. Wastewater is distributed mechanically over the media and percolates down the filter to collect in an under drain system at its base. A microbial film develops over the surface of the media and this is responsible for removal of BOD during passage of sewage through the bed.

Chemical Oxygen Demand (COD): A quick chemical test to measure the oxygen equivalent of the organic matter content of wastewater that is susceptible to oxidation by a strong chemical reagent.

Detention (Retention) time: The theoretical length of time for water to pass through a basin or tank, if all the water moves with the same velocity.

Dissolved Oxygen (DO): The oxygen dissolved in water, wastewater, or other liquids; usually expressed in milligrams per liter, parts per million, or percent of saturation.

Domestic Wastewater: Wastewater that comes primarily from dwelling, business buildings, institutions, and does not generally include industrial or agricultural wastewater.

Effluent: Treated wastewater discharging from a water or wastewater treatment plant.

Grab Sample: A single sample collected at a particular time and place that represents the composition of the water, air, or soil only at that time and place.

Grit chamber: A chamber or tank that is used in primary treatment where wastewater slows down and the heavy, large solids(grit) settle out and are removed.

Influent: The flow of raw sewage entering the plant.

Kjeldahl Nitrogen: The combined amount of organic and ammonia nitrogen. Also known as Total Kjeldahl Nitrogen (TKN).

Milligram per liter (mg/l): The weight of a substance measured in milligram contained in one liter. It is equivalent to 1 part per million in water measures.

Monitoring: Periodic or continuous surveillance or testing to determine the level of compliance with statutory requirements and /or pollutant levels in various media or in human, plants, and animals.

Municipal sewage: Wastewater originated from a community and is composed of domestic wastewater and/or industrial discharge.

Nitrate: A form of nitrogen found in oxygenated wastewater. Nitrate is a nutrient for plants so it can contribute to prolific weed growth in waterways.

Nutrients: Key nutrients associated with wastewater are nitrogen and phosphorus. Nutrients are an important contaminant in wastewater as they cause prolific weed growth in waterways, adversely affecting ecology.

Parts per million (PPM): A measure of concentration of one unit of material dispersed in one million units of another.

PH: An expression of the intensity of the basic or acid of a liquid; ranges from 0 to 14, where 0 is the most acid and 7 is neutral. Natural water usually have a PH value between 6.5 and 8.5.

Phosphorus: Phosphorus is an essential element in the metabolism of biological organisms. A minimal concentration is necessary to achieve optimum operation of biological treatment systems. Because it has been implicated as a contributing factor in the development of noxious algal blooms, more emphasis is being placed on controlling the amount of phosphorus discharged in the treatment plant effluent. Phosphorus may exist in many different forms in aqueous solutions. These forms maybe classified as (a) orthophosphorus,(b) pyro-, poly-, and metaphophates, and (c) organic phosphorus. The orthophosphates are of most concern because they are freely available for biological metabolism.

Primary treatment: The first stage of wastewater treatment that removes settle able or floating solids only; generally removes 40% of the suspended solids and 30-40% of the BOD in the wastewater.

Pump: Mechanical device that allows water to be lifted or raised.

Sampler: A device used with or without flow measurement to obtain an aliquot portion of water or wastewater for analysis purposes. May be designed for taking single sample (grab), composite sample, continuous sample, or periodic sample.

Sampling frequency: The interval between the collection of successive samples.

Screen: A device to remove large suspended or floating debris from wastewater.

Screening: The removal of relatively coarse floating and suspended solids by straining through racks or screens.

Secondary treatment: The wastewater process where bacteria are used to digest organic matter in the wastewater.

Settle able Solids: Material heavy enough to sink to the bottom of a wastewater treatment tank.

Settling Pond: a lagoon in which settle able solids removed by gravity.

Settling Tank(sedimentation tank or clarifier): A vessel in which solids settle out of water by gravity during wastewater or potable water treatment process.

Sewage: the spend water of a community. This term is known as wastewater.

Sewerage System: The complete sewage collection, treatment and disposal system.

Solids: The determinations of various forms of residue are useful in controlling wastewater treatment plant. Total Solids (TS), Suspended Solids (SS) and Dissolved Solids (DS), and their volatile and fixed fractions, may be used to assess wastewater strength, process efficiency, and unit loadings. Measurements of the various residue concentrations are necessary to establish and assure satisfactory operational control. It is important that the operator develop sufficient knowledge of these measurements and their interpretation so that they become routine daily process.

Stabilization: Conversion of the active organic matter in sludge into inert, harmless material.

Standards: Norms that impose limits on the amount of pollutants or emissions produced.

Suspended Solids: Solids in suspension in a water or wastewater which can be removed by filtration.

Total Dissolved Solids (TDS): The sum of the inorganic and organic materials dissolved in water.

Total Solids (TS): TS, is a term applied to the weight of material per unit volume of sample remaining in a previously weighed crucible after evaporation of the sample at a temperature of 103 to 105 degrees Celsius.

Total Suspended Solids (TSS): A laboratory measurement of the quantity of suspended solids present in wastewater that is one of the main indicators of the quantity of pollutants present.

Treated Wastewater: Wastewater that has been subjected to one or more physical, chemical or biological processes to reduce its potential of being a health hazard.

Wastewater Treatment Plant: A facility containing a series of tanks, screens, filters and other processes by which pollutants are removed from water.

Wastewater: The spent or used water from a house, community, farm or industry that contains dissolved or suspended matter.

Water-born disease: a disease spread by contaminated water.

References

Abu-Jalalah, S. (1999). Wastewater Treatment in Gaza.
http:// www.mofa.go.jp/region/middle_e/peaceprocess/ewg/mission9903/gaza2.htm

Afifi, s. (1998). Potential of wastewater reuse in Gaza-Palestine. Advantages and disadvantages, Third International Water Technology Conference IWTC.

Afifi ,S and Nassar, A(1999). Training Course in Wastewater Treatment Plant Management,April.

Afifi, S,(2003). Septage Treatment in A Vertical Flow Reed Bed System, International Conference on Engineering and city Development, IU-Gaza-Palestine, September, 2003.

Al-Jamal, K and Yaquobi, A, 2004). Water Resources Management in Gaza Strip

Al-Khaldi ,R (2006). Performance Evaluation of Beit-Lahia Wastewater Treatment Plant in Northern Gaza Strip,March.

Austin Water. Septic Tank to Anaerobic filter to Trickling Filter with Recirculation to Anaerobic Filter.
http://www.ci.austin tx.us/wri/treat12.htm

CAMP(Coastal Aquifer Management Program),(1999).Feasibility Study and Conceptual Design For Gaza Wastewater Treatment Plant, P.W.A.

CAMP(Coastal Aquifer Management Program),(2001). Standard Laboratory Procedures, Municipality Of Gaza

Coastal Municipalities Water Utility, (2005). First Quarter Report for the Gaza Emergency Water Project(October,2005).

Crites . Tchobanoglous, (1991). Small and Decentralized Wastewater Management Systems, McGraw Hill.

DORSCH CONSULT.(2005).Assessment Existing Gaza WWTP. July.

Daigger, Glen T. and Buttz, Johm A.(1992). Upgrading Wastewater Treatment Plants.

El-Hawi,M. and Hamilton,A. 2001. Public Participation in Solid Waste Management Strategy for Essex, UL.

Elmanama, Abdelraouf,2004. the Use of Sand and Seawater in the Assessment of Microbial Quality of Gaza Beach and Public Health.

Horan , N.J, (1990). Biological Wastewater Treatment System, John Wilet & sons Ltd. England.

InfraMan,(2006). Water and Wastewater Sampling and monitoring program for the Gaza Emergency Project.

Karen Mancl,(2004). Wastewater Treatment Principles and Regulations.
http://ohioline.osu.edu/aex-fact/0768.html

Kelly & Dioxin, H (1995). Environmental scarcity and violent. The case of Gaza, American Association for Advancement of Science, University of Toronto, Canada

Malina,Joseph F. and Pohland , Frederick G.(1992). Design of Anaerobic Processes for the treatment of Industrial and Municipal Waste.

Logan, B.E (July,2002). Trickling Filters Design.
http://www.engr.psu.edu/ce/enve/tf.htm.

McGhee, Terence J.(1991). Water Supply and Sewerage, McGRAW- HILL International Editions.

Metcalf, Eddy (1991). Wastewater Engineering Treatment, Disposal and reuse, 3rd edition. McGraw Hill.

MOPIC (Ministry Of Planning and International Cooperation),(1998). Master Plan for Sewage and Storm Water Drainage in the Gaza Governorates-Final Master Plan Report, Main Report. Palestinian National Authority.

Municipality Of Gaza (1996). Master Plan – Analytical Studies. USAID, December.

Municipality Of Gaza (1997). Final Engineering Design Report. USAID, November.

PCBS (Palestinian Central Bureau of Statistics), (1997). Preliminary Results for the Census of Population and Establishments, Palestinian National Authority, Ramalla.

PCBS (Palestinian Central Bureau of Statistics), (2005). Small Area Population, Revised Estimates 2004-2005. http:/www.pcbs.org/Portals/_pcbs/population/

PNIC (Palestinian National Information Center), (1999). The Administrative Division of Governorates. http:/www.pnic.gov.ps/English/geography/Residential

P.W.A (Palestinian Water Authority), (2004). Hydraulical Impact Assessment of Infiltration Basins on Local Groundwater.

P.W.A (Palestinian Water Authority),(2000). Drinking Water Quality Standards (Draft) in Gaza Strip

P.W.A (Palestinian Water Authority), 2004. fiscal report for the Gaza strip municipalities.

P.W.A (Palestinian Water Authority),(2002). Water Resources and Management Issues (Gaza Strip/Palestine), July.

P.W.A(Palestinian Water Authority),(1999). Wastewater Treatment Plant and Sludge Disposal, Technical paper no.44,February.

P.W.A(Palestinian Water Authority),(1998). Wastewater Treatment Plant Operation Manual, Technical paper no.39, July.

P.W.A (Palestinian Water Authority),(1999). Wastewater Treatment Plant Operation Tests, Technical paper no.53, August.

P.W.A (Palestinian Water Authority), (2005). Agricultural And Municipal Water Demand in Gaza Governorates for 2004.

Qasim, Syed R.(1999). Wastewater Treatment Plants Planning, Design, and Operation, TECHNOMIC publishing co. Inc.

Reed, S.C, 1995. Nitrogen Removal in Wastewater Stabilization Ponds, Water Pollution Control Federation Journal. 57(1)39-45.

Statistical Abstract of Israel, (2002). Water consumption for Israeli citizens.

Tchobanoglous, George and Schroeder, Edward D (1999). Water Quality Characteristics, Modeling and Modification, ADDISON-WESLEY Publishing Company.

Thomas E.Schultz (October,2005).Biological Wastewater Treatment.
http://www.usfilter.com/nr/rdonlyres/biological tr .pdf

UNEP(United Nations Environmental Program), (2003). Desk study on the Environment in the Occupied Palestinian Territories, Switzerland.

Varon, M,(2002). Advanced Primary Treatment of Domestic Wastewater in Tropical Countries: Development of High –Rate Anaerobic Ponds. Doctorate Thesis; School of Civil Engineering, Leeds University; UK.

Victoria Mitchell, October.1994. Trickling filter solution, Water and Waste Treatment, Vol.37, No.10, pp125-126.

Wolfe, M.D.Tremblay,A.M. (2001). Partial Aerated Lagoons: an Optimal Choice for Small Community Wastewater Treatment, McGill University.

العدوى، محمد صادق(1990). هندسة الصرف الصحى، دار صادق للنشر. الاسكندرية.

Annex A

Results of the GWWTP lab monitoring program data for the period from the year 2000 to 2004

Results of the Year 2000 of GWWTP

Date	Flow	Effluent						Influent					
		BOD	COD	TDS	TSS	F.C	N-KJD	BOD	COD	TDS	TSS	F.C	N-KJD
	m^3/day	mg/l	mg/l	mg/l	mg/l	cfu/100cm	mg/l	mg/l	mg/l	mg/l	mg/l	cfu/100cm	mg/l
Jan-00	40743	23.0	65	1102	24	4.00E+06	55.0	343	817	872	936.5	6.00E+08	70
Feb-00	39472	23.0	92	1253	32	3.74E+06	45.5	502.5	968	1385	696.75	8.50E+07	65
Mar-00	38500	23.0	95	1312	29	9.50E+06	61.5	450	1010	1317	673	1.83E+08	66.5
Apr-00	40576	21.5	97	1346	23	1.62E+06	60.5	560	1136	1415	575	6.60E+08	65.8
May-00	41841	33.0	95	1287	29	7.00E+05	50.0	602	1193	1485	700	7.00E+07	62
Jun-00	44315	25.0	90.5	1360	23	4.00E+06	41.0	640	1264	1351	457	4.00E+08	60.2
Jul-00	45297	27.0	90	1406	20	7.00E+06	56.0	600	1098	1559	493	4.00E+08	67.65
Aug-00	45957	22.3	79	1514	18	4.40E+06	64.5	495	933	1540	458	3.30E+08	72.1
Sep-00	44739	23.5	79	1517	20	1.53E+06	49.0	279	752	1423	555	2.95E+08	72.1
Oct-00	45489	24.5	89	1382	22	1.00E+07	37.5	620	1194	1145	566	4.00E+08	61.15
Nov-00	37109	37.5	135	1440	35	9.80E+06	48.2	555	1130	1495	1147	5.10E+08	84
Dec-00	42693	38.0	138	1204	55	1.70E+07	57.0	980	1854	1681	969	2.63E+09	74
Average	**42185**	**26.8**	**95.4**	**1344**	**28**	**6E+06**	**52.1**	**552**	**1112**	**1389**	**685.5**	**5E+8**	**68.38**

Results of the Year 2001 for the GWWTP

Date	Flow	Effluent						Influent					
		BOD	COD	TDS	TSS	F.C	N-KJD	BOD	COD	TDS	TSS	F.C	N-KJD
	m^3/day	mg/l	mg/l	mg/l	mg/l	cfu/100cm	mg/l	mg/l	mg/l	mg/l	mg/l	cfu/100cm	mg/l
Jan-01	37704	41	113	1204	47	7.20E+06	61.6	580	1222	1114	617	2.60E+08	93.4
Feb-01	34508.36	27	90	1202	27	8.50E+06	50.4	680	1160	1344	729	2.65E+08	56
Mar-01	38021.68	47	149	1341	38	8.75E+06	70.0	870	1660	1621	767	3.05E+08	84
Apr-01	38369.33	34	109	1410	36	5.00E+06	61.4	900	1639	1563	825	2.70E+08	74.6
May-01	39934.71	31	87	1439	32	6.20E+06	50.4	720	1433	1528	1113	3.75E+08	56
Jun-01	42806.73	37	106	1525	37	6.45E+06	53.2	610	1285	1534	527	3.80E+08	70
Jul-01	47294.84	27	91	1532	28	8.05E+06	56.0	530	1017	1504	485	1.15E+09	67.65
Aug-01	46315.84	32	99	1546	28	3.00E+06	45.0	590	1059	1572	595	7.95E+08	60
Sep-01	45779.77	27	90	1582	28	2.50E+06	44.8	560	1104	1649	679	8.00E+08	65.4
Oct-01	45808.68	27	91	1483	26	2.00E+06	40.6	810	1467	1584	815	8.20E+08	80.2
Nov-01	42721.73	27	103	1558	33	3.00E+06	50.0	1000	2180	1573	1011	7.95E+08	
Dec-01	38019.13	17	77	1237	20	5.51E+06	77.0	1085	2033	1486	1137	5.65E+08	154
Average	**41440**	**31.2**	**100**	**1422**	**32**	**5.51E+06**	**55.0**	**770**	**1438**	**1506**	**775**	**5.65E+08**	**154**

Results of the Year 2002 for the GWWTP

Date	Flow	Effluent						Influent					
		BOD	COD	TDS	TSS	F.C	N-KJD	BOD	COD	TDS	TSS	F.C	N-KJD
	m^3/day	mg/l	mg/l	mg/l	mg/l	cfu/100cm	mg/l	mg/l	mg/l	mg/l	mg/l	cfu/100cm	mg/l
Jan-02	41628.97	24	77	1128	27	2.50E+06	56.0	500	1025	1046	746	3.80E+08	39.2
Feb-02	39334.82	27	91	1358	27		50.4	563	1160	1298	729		56.0
Mar-02	36708.48	27	94	1588	29	2.50E+06	87.0	500	1009	1549	464	2.00E+08	119.0
Apr-02	38759.23	23	76	1465	23		91.0	550	1055	1634	522		102.0
May-02	42164.77	23	87	1566	31		82.0	540	1117	1591	590		90.0
Jun-02	47442.37	30	98	1658	37		60.0	770	1307	1718	592		101.0
Jul-02	49894.42	33	106	1700	44		41.0	410	826	1640	826		35.0
Aug-02	42029	32	100	1640	39		64.0	370	713	1278	459		88.0
Sep-02	47200.7	30	89	1613	46		56.0	430	842	1570	517		77.0
Oct-02	47745.71	41	140	1568	70		53.0	760	1329	1640	1320		69.0
Nov-02	43392	41	140	1437	70		53.0	760	1329	1436	1320		69.0
Dec-02	42603.9	29	94	1305	38	3.75E+06	68.0	410	925	1232	1589	3.10E+08	90.0
Average	**43242**	**30**	**99.3**	**1502**	**40**	**2.92E+06**	**63.5**	**547**	**1053**	**1469**	**806**	**2.97E+08**	**77.9**

Results of the Year 2003 of the GWWTP

Date	Flow	Effluent						Influent					
		BOD	COD	TDS	TSS	F.C	N-KJD	BOD	COD	TDS	TSS	F.C	N-KJD
	m^3/day	mg/l	mg/l	mg/l	mg/l	cfu/100cm	mg/l	mg/l	mg/l	mg/l	mg/l	cfu/100cm	mg/l
Jan-03	37126	35	121	1421	43	4.65E+06	78	460	817	1158	454	1.06E+08	102.0
Feb-03	41744	34	111	1293	39	5.05E+06	60	420	819	1155	740	1.41E+08	134.0
Mar-03	35860	50	380	1116	107	4.75E+06	67	380	765	1083	714	5.15E+08	125.0
Apr-03	39629	43	147	1452	89	1.02E+07	72.8	350	758	1316	349	1.67E+08	103.8
May-03	40173	29	102	1569	35	4.61E+06	67	380	818	1415	1318	2.04E+08	73.0
Jun-03	51718	31	100	1754	43	4.55E+06	69	450	711	1558	364	2.16E+08	76.5
Jul-03	51804	27	88	1664	34	3.05E+06	58	340	702	1591	547	2.65E+08	78.0
Aug-03	52050	23	81	1700	29	9.10E+06	56	310	584	1528	375	2.41E+08	64.0
Sep-03	49850	25	88	1725	30	2.30E+06	56	300	598	1497	318	2.28E+08	78.0
Oct-03	47185	26	104	1721	37	2.90E+06	73	435	942	1690	436	3.60E+08	95.0
Nov-03	43620	27	92	1700	36	2.10E+06	62	530	1080	1647	663	2.03E+08	78.0
Dec-03	38867	26	108	1362	37	2.10E+06	55	490	1011	1564	617	1.60E+08	75.0
Average	**44136**	**31.3**	**127**	**1540**	**46.6**	**4.61E+06**	**64.48**	**404**	**800**	**1434**	**574.6**	**2E+08**	**90.2**

Results of the Year 2004 of the GWWTP

Date	Flow	Effluent						Influent					
		BOD	COD	TDS	TSS	F.C	N-KJD	BOD	COD	TDS	TSS	F.C	N-KJD
	m^3/day	mg/l	mg/l	mg/l	mg/l	cfu/100cm	mg/l	mg/l	mg/l	mg/l	mg/l	cfu/100cm	mg/l
Jan-04	42387	38	119	1241	53	9.20E+07	61.6	395	811	1171	472	1.58E+08	77.4
Feb-04	41379	36	129	1482	54	2.30E+05	73.0	460	1052	1562	614	1.31E+08	93.0
Mar-04	40258	32	95	1549.5	47	3.30E+06	73.0	400	738	1424	460	1.13E+08	81.0
Apr-04	41167	27	95	1652	42	2.30E+06	69.2	430	890	1430	502	3.00E+08	87.0
May-04	47613	32	102	1691	51		65.0	410	831	1599	449		84.0
Jun-04	55333	33	108	1748	58		64.0	355	853	1651	467		76.0
Jul-04	59419	32	103	1708	48	2.60E+06	67.0	295	661	1706	381	2.40E+07	73.0
Aug-04	57742	29	93	1791	43	2.30E+06	73.0	370	812	1661	420	7.30E+08	86.0
Sep-04	52667	54	129	1806	76	2.20E+07	73.0	445	921	1742	543	2.40E+08	100.0
Oct-04	41194	39	129	1839	60	4.00E+06	73.0	370	726	1794	333	2.50E+08	118.0
Average	**47916**	**35.2**	**110**	**1650.8**	**53**	**7.73E+06**	**69.2**	**393**	**830**	**1574**	**464**	**3.11E+08**	**87.5**

Annex B

GWWTP Removal Efficiency of Organic material, Solids, Nutrients and microbiological materials for the period from 2000 to 2004

Month	Removal efficiency of year 2000					Removal efficiency of year 2001					Removal efficiency of year 2002				
	BOD	**COD**	**TDS**	**TSS**	**N-kjd**	**BOD**	**COD**	**TSS**	**TDS**	**N-kjd**	**BOD**	**COD**	**TSS**	**TDS**	**N-kjd**
Jan	93.3	92.0	-26.4	97.4	21.4	92.931	90.753	92.4	-8.1	34.0	95.2	92.488	96.38	-7.839	42.9
Feb	95.4	90.5	9.5	95.4	30.0	96.029	92.241	96.3	10.6	10.0	95.2	92.155	96.3	-4.622	10.0
Mar	94.9	90.6	0.4	95.7	7.5	94.598	91.024	95.0	17.3	16.7	94.6	90.684	93.75	-2.518	26.9
Apr	96.2	91.5	4.9	96.0	8.1	96.222	93.35	95.6	9.8	17.7	95.82	92.796	95.59	10.34	10.8
May	94.5	92.0	13.3	95.9	19.4	95.694	93.929	97.1	5.8	10.0	95.74	92.211	94.75	1.571	8.9
Jun	96.1	92.8	-0.7	95.0	31.9	93.934	91.751	93.0	0.6	24.0	96.1	92.502	93.75	3.492	40.6
Jul	95.5	91.8	9.8	95.9	17.2	94.906	91.052	94.2	-1.9	17.2	91.95	87.167	94.67	-3.659	17.1
Aug	95.5	91.5	1.7	96.1	10.5	94.576	90.652	95.3	1.7	25.0	91.35	85.975	91.5	-28.33	27.3
Sep	91.6	89.5	-6.6	96.4	32.0	95.179	91.848	95.9	4.1	31.5	93.02	89.43	91.1	-2.739	27.3
Oct	96.0	92.5	-20.7	96.1	38.7	96.667	93.797	96.8	6.4	49.4	94.61	89.466	94.7	4.39	23.2
Nov	93.2	88.1	3.7	96.9	42.7	97.923	95.275	96.7	1.0	20.0	94.61	89.466	94.7	-0.07	23.2
Dec	96.1	92.6	28.4	94.3	23.0	98.433	96.212	98.2	16.8	50.0	92.93	89.838	97.61	-5.925	24.4
Average	**94.9**	**91.3**	**1.4**	**95.9**	**23.5**	**95.6**	**92.7**	**95.6**	**5.3**	**25.5**	**94.3**	**90.3**	**94.6**	**-3**	**13.5**

Month	Removal efficiency of year 2003					Removal efficiency of year 2004					
	BOD	COD	TDS	TSS	N-kjd	BOD	COD	TDS	TSS	F.C	N-kjd
Jan	92.4	85.19	-22.71	90.5	23.5294	90.4	85.3	-6.0	88.8	41.8	30.6
Feb	91.9	86.447	-11.95	94.7	55.2239	92.2	87.7	5.1	91.2	99.8	20.0
Mar	86.8	50.327	-3.047	85.0	46.4	92.0	87.1	-8.8	89.8	97.1	18.7
Apr	87.7	80.607	-10.33	74.5	29.8651	93.7	89.3	-15.5	91.6	99.2	24.5
May	92.4	87.531	-10.88	97.3	8.21918	92.2	87.7	-5.8	88.6		26.7
Jun	93.1	85.935	-12.58	88.2	9.80392	90.7	87.3	-5.9	87.6		26.9
Jul	92.1	87.464	-4.588	93.8	25.641	89.2	84.4	-0.1	87.4	89.2	23.3
Aug	92.6	86.13	-11.26	92.3	12.5	92.2	88.5	-7.8	89.8	99.7	18.7
Sep	91.7	85.284	-15.23	90.6	28.2051	87.9	86.0	-3.7	86.0	90.8	15.1
Oct	94.0	88.96	-1.834	91.5	23.1579	89.5	82.2	-2.5	82.0	98.4	11.0
Nov	94.9	91.481	-3.218	94.6	20.5128						
Dec	94.7	89.318	12.916	94.0	26.6667						
Average	**92.2**	**84.2**	**-7.41**	**91.9**	**28.5**	**91.0**	**86.6**	**-4.9**	**88.3**	**97.5**	**21.5**

Annex c

GWWTP lab data for the removal efficiency of each treatment stage for the period from 2000 to 2004

The Removal Efficiency of each stage of GWWTP measured in 2003.

Date	lagoons1,2			Lagoon 3			Bio-tower			Aeration Lagoon		
	BOD in	BOD out	Removal eff.%	BOD in	BOD out	Removal eff.%	BOD in	BOD out	Removal eff.%	BOD in	BOD out	Removal eff.%
2/18/2003	400	290	**0.28**	290	210	**0.28**	210	120	**0.43**	120	50	**0.58**
6/03/2003	390	300	**0.23**	300	280	**0.07**	280	150	**0.46**	150	70	**0.53**
6/17/2003	300	330	**0.10**	330	240	**0.27**	240	110	**0.54**	110	60	**0.45**
7/13/2003	360	350	**0.03**	350	260	**0.26**	260	350	**0.35**	350	135	**0.61**

	7 AERATORS WORKING												**14 AERATORS WORKING**												**Average Removal Eff.**
Date	*Mon.27/9*				*Tue.28/9*				*Wed. 29/9*				*Thu. 30/9*				*Sat. 2/10*				*Sun. 3/10*				
Test	**EFFLUENT PONDs 1&2**																								
???	**8**	**10**	**12**	**14**	**8**	**10**	**12**	**14**	**8**	**10**	**12**	**14**	**8**	**10**	**12**	**14**	**8**	**10**	**12**	**14**	**8**	**10**	**12**	**14**	
???	**8**	**10**	**12**	**14**	**32**	**34**	**36**	**38**	**56**	**58**	**60**	**62**	**80**	**82**	**84**	**86**	**104**	**106**	**108**	**110**	**128**	**130**	**132**	**134**	
COD.B.(mg/l)	547	515	540	542	588	424	552	540	644	665	542	551	538	580	599	747	713	640	682	700	606	544	578	561	39.12%
IMHOF(ml/l)	2.9	3.0	2.7	2.5	3.5	3.3	2.9	2.9	3	3.1	2.9	2.5	2.5	2.6	2.1	3.1	3.5	2.9	2.9	2.8	2.9	2.5	2.9	2.8	
COD.A(mg/l).	337	320	357	343	454	412	416	381	393	535	378	439	385	496	617	465	455	492	530	568	376	409	420	400	
EFFLUENT POND 3																									
COD.B.(mg/l)	354	336	330	360	440	450	384	443	480	387	393	456	428	475	504	499	461	530	510	546	483	423	400	396	25.96%
IMHOF(ml/l)	0.5	0.5	0.4	0.3	0.5	0.4	0.4	0.3	0.5	0.5	0.4	0.4	0.5	0.4	0.2	0.3	0.5	0.4	0.4	0.5	0.5	0.3	0.4	0.4	
COD.A.(mg/l)	327	308	340	314	414	421	175	368	407	351	360	430	418	462	568	451	498	508	482	527	429	420	398	380	
EFFLUENT BIO-TOWER																									
COD.B.(mg/l)	215	197	248	283	362	183	267	308	416	267	403	356	297	453	410	477	337	334	372	277	338	279	311	244	77.62%
IMHOF(ml/l)	2.8	2.9	3.6	3.5	3	2.9	2.6	2.7	2.4	2.5	4	2.7	2	2.7	3.5	4	1.5	2.5	2	2.5	1.9	2	2.5	2.5	
COD.A.(mg/l)	108	90	82	94	97	88	87	94	83	93	90	92	95	93	89	88	96	86	90	88	87	93	88	92	
EFFLUENT AERATION																									
COD.B.(mg/l)	186	174	189	180	311	250	350	225	462	434	460	409	481	510	540	574	423	475	471	327	383	469	213	419	23.68%
IMHOF(ml/l)	2.5	1.9	2	1.4	3.6	3.7	3.2	2.9	7	7.8	7.2	7.5	8	7.5	7	7.5	6	7.3	6	6	7.5	8	6.5	7	
COD.A..(mg/l)	66	75	65	63	78	70	74	70	76	74	80	82	76	77	70	73	69	62	59	65	60	61	59	62	
EFFLUENT FROM THE PLANT																									
COD.B.(mg/l)	71	71	73	79	69	74	65	90	73	70	85	69	74	72	78	73	71	73	77	80	68	70	72	71	80.17%
IMHOF(ml/l)	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	
COD.A.(mg/l)	63	66	66	70	70	69	60	82	67	63	75	63	64	63	67	62	60	61	65	67	58	61	63	61	
INFLUENT																									
COD.B.(mg/l)	825	795	1088	870	1026	824	894	1095	1121	980	905	898	1213	990	935	895	1011	995	938	950	973	1050	990	960	92.39%
IMHOF(ml/l)	8.5	7	7.9	7.5	7.5	7	7.3	7.5	9.5	8	7.8	7.2	11	8.2	7.9	7	7.5	8	7.8	7.7	9	8.5	8.4	8	
COD.A.(mg/l)	436	401	507	448	432	418	435	470	560	468	459	453	534	504	465	430	512	465	460	463	498	502	495	485	

Annex D

Total Suspended Solids to Biochemical Oxygen Demand Ratio for both influent and effluent of GWWTP

		Effluent			influent	
Year	average BOD	average TSS	TSS/BOD%	average BOD	average TSS	TSS/BOD%
2000	26.78	28	**104.6**	552	685.5	**124.2**
2001	31.20	32	**102.6**	770	775	**100.6**
2002	30.00	40	**133.3**	547	806	**147.3**
2003	31.30	46.6	**148.9**	404	574	**142.1**
2004	35.20	53	**150.6**	393	464	**118.1**

TSS to BOD Ratios of influent and effluent of GWWTP

Annex E

The Calculation Sheet of the Newly Proposed Anaerobic Lagoon and the final Sedimentation Tank

Anaerobic Lagoon

Parameter	Value		
Winter temp. C	14	C°	
Summer temp. C	23.3	C°	
Q_{max} m^3/day	70800.0	m^3/day	
Inffluent BOD_5^{20}	440	mg/l	
water depth	3.2	m	

design temperature C°	>20		10 -- 20	11 -- 20	
volumetric loading gBOD/m^3/d	300	gBOD/m3/d	20T-100	180	gBOD/m3/d
BOD removal (%)	60%		2T+20	48	%

Retention time **2** days

BOD Removal 48-60 %

Lagoon Surface Area **A** = $L_iQ/\lambda D$

Where

Li Influent BOD mg/l

Q Daily flow

λ volumetric loading gBOD/m^3/d

D Depth of Lagoon

A Lagoon Surface Area

Lagoon Surface Area **A** = **54,083.33** m^2

	m^2	m^3
lagoon 1-1,	7200	16000
lagoon 1-2	7200	16000
lagoon 2	12000	31000
total	26400	63000
required lagoon 3	**27,683**	**78,600**

Final Sedimentation Tank

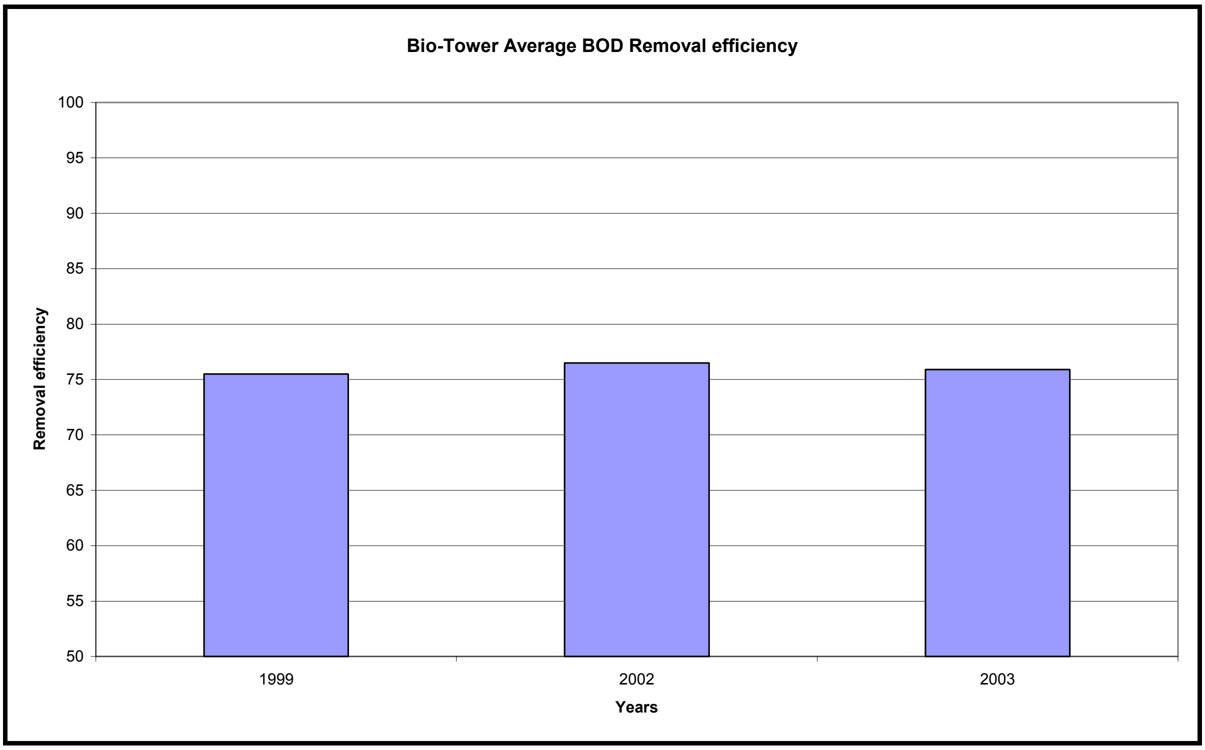
Design Wastewater Production	**3933.33**	m^3/h
Design Wastewater Production	**1.09**	m^3/s
Minimum Retintion Time	**1.25**	h
Min Sedimentation Tank Volume	**4916.67**	m^3
Max Surface Loading	**2**	$m^3/h/m^2$
Min Sedimentation Tanks Surface	**1966.67**	m^2
No. of sedimentation tanks	**2**	
Min Sedimentation EachTank Surface	**983.33**	
Chosen Effective Height of ST	**4.10**	m
length of channel	**80.00**	m
Chosen Mean Area of ST	**1118.00**	m^2
Existing ST mean Area	**599.00**	m^2
Total mean area S T provided	**1198**	m^2

Total surface area S T provided	**2236**	**m^2**
Total Tank volum provided provided	**5111.8**	**m^3**
surface loading provided	1.76	
Retintion Time provided	1.30	h

Annex F

Average BOD_5 DATA for the Trickling Filters

Bio-Tower						
	Filtered	Filtered		Unfiltered	Unfiltered	
Date	Influent BOD	Effluent BOD	Efficiency %	Influent BOD	Effluent BOD	Efficiency %
	mg/l	mg/l		mg/l	mg/l	
27-Sep-99	161.0	47	70.8	173	118	31.8
28-Sep-99	172.0	46	73.3	213	140	34.3
29-Sep-99	194.0	45	76.8	215	180	16.3
30-Sep-99	237.0	46	80.6	238	205	13.9
02-Oct-99	252.0	45	82.1	256	213	16.8
03-Oct-99	147.0	45	69.4	213	145	31.9
Average 1999	193.8	45.7	**75.5**	218	167	**24.2**
16-Oct-02	300.0	90	76.3	300	155	48.3
24-Dec-02	180.0	34	76.8	180	70	61.1
Average 2002	240.0	62.0	**76.5**	240.0	112.5	**54.7**
28-Jan-03	310.0	175	76.1	310.0	175	43.5
18-Feb-03	210.0	50	75.1	210.0	120	42.9
19-Apr-03	300.0	65	76.0	300.0	115	61.7
22-Apr-03	250.0	70	76.1	250.0	100	60.0
26-Apr-03	200.0	32	76.1	200.0	35	82.5
03-May-03	260.0	30	76.0	260.0	60	76.9
10-May-03	310.0	55	75.9	310.0	85	72.6
17-May-03	170.0	70	75.9	170.0	168	1.2
03-Jun-03	280.0	70	76.0	280.0	150	46.4
17-Jun-03	240.0	60	76.0	240.0	110	54.2
Average 2003	253.0	67.7	**75.9**	253.0	112	**54.2**



Annex G

Calculation Sheet for the Grit Removal Channel

Gaza waste water treatment plant

A. Inlet pipe

w.w. production	120	L/c.d
Q_p/Q_f	0.63	
h	0.70	
n	0.013	

Table(1)

Population	w.w prod.	w.w prod.	P factor	Peak flow	Q	Q_P	Slope	Profile	V_f	Q_f	Q_p/Q_f	V_p	h
	m^3/c.d	m^3/d		m^3/d	m^3/s	m^3/s	%	m	m/s	m^3/s		m/s	m
400,000	0.118	47200	1.5	70800.0	1.09	1.09	0.200	1.200	1.53	1.732	0.631	1.6	0.70

B. Bar Screen channel

The deph of flow in the bar screen channel (d_2) depends on the depth of flow in the inlet pipe (d_1)

d_{1}=	**0.70**	m
V_{1}=	**1.6**	m/s
Q_{max} =	**1.09**	m^3/s
V_{max} =	**1.0**	m/s
d_{2} =	**0.70**	m
A_{clear} =	**1.09**	m^2
W_{clear} =	**1.57**	m
S =	**0.025**	m
N_{open} =	**63.00**	No.
T =	**0.010**	m
W_{total} =	**2.20**	m

flow condition in the incoming conduit

diameter	1.20	m
slope %	0.20	
V_p	1.6	m/s
Depth h	0.70	m

clear area through the rack openings

$C_{lear} =$	Q_{max}/V_{max}	1.09	m^2

Clear width

$C_{lear} =$	$Aclear/d_2$	1.57	m

no. of spaces Wclear/spaceing 62.68

take 63

total No. OF bars 62

Clear width C_{lear} 1.575

width of the chamber

$T_{otal} =$	2.20	m

Actual depth in the approach channel using the energy equation:

$$\mathbf{Z_1+d_1+V_1^2/2g = Z_2+d_2+V_2^2/2g+0.3(V_1^2/2g-V_2^2/2g)}$$

Z1-Z2 = 0.1 m

$V_1^2/2g$ = 0.132

K1 = 0.889 $=(Z_1-Z_2+d_1+0.7V_1^2/2g)$

K2 = 0.009 $=.70(Q/W)^2*(1/2*9.81)$

The above equation is simplified in the following form to solve it for d2 by trail and error

$d_2^3-K_1*d_2^2+K_2=0=C$

d2	C
0.88	0.00

so the assumed d2 is the same as actual d2

The velocity between the bars of the rack:

d2 0.88 m

V Flow/net area at the rack

V 0.79 m/s o.k.

The velocity in the screen channel:

v flow/cross section area

v 0.57 m/s

head loss through the bar rack

H_L $(V^2-v^2/2g)*(1/0.7)$

H_L 0.022 m

the depth of the flow and velocity in the rack chamber blow the rack

$d_2+V_2^2/2g=d_3+V_3^2/2g+h_L$

The above equation is simplified in the following form to solve it for d2 by trail and error

K1 $hL-(d_2+(v_2^2/2g)$

K2 $(Q/W)^2/2g$

K1	-0.872
K2	0.013

$=(Q/W)^2*(1/2*9.81)$

d33+k1*d32+k2=0=C

d_3	C
0.85	0.00

d_3 0.85 m

V_3 0.58 m/s

head loss through the rack at 50% clogging

$d'2+V'^2_2/2g=d3+V_3^2/2g+h_{50}$

$$h_{50} = \frac{(\text{velocity through rack opening})^2 - v'^2_2}{2g} \times 1/0.7$$

velocity through rack opening at 50% clogging = $Q_P/A_{CLEAR} \times 0.50$

v_3 = 1.39 /d' m/s
70

v'_2 = Q_P/T_{otal}

v'_2 = 0.50 /d' m/s

K_1 $d_3+(V_3^2/2*g)$

K_2 $1/2*g(2.43(Q/T_{otal})^{2-}((Q/C_{lear})^2/0.7)$

K_2	-0.055
K_1	0.87

simplifying the equation

$d'^3_2 - K_1 d'^2_2 + K_2 = 0 = C$

d'_2	C
0.94	0.00

d'_2 0.94 m

V'_2 0.53 m

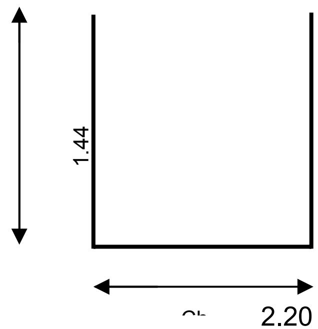
h_{50} 0.08 m

Summary of Depth of Flow, Velocity and Head Loss through Bar Rack at Design Peak Flow

	Up stream Channel			Downstream Channel		
Condition	**Depth of Flow (m)**	**Velocity (m/s)**	**Velocity through Rack(m/s)**	**Depth of Flow (m)**	**Velocity (m/s)**	**Head Loss (m)**
Clean Rack	0.88	0.57	0.79	0.85	0.58	0.016
50% Clogging	0.94	0.53	1.48	0.85	0.58	0.06

velocity V' Through rack opening 1.48 m/s

DESIGN DEPTH 0.94+0.50 **1.44** **m**
DESIGN WIDTH **2.20** **m**
Promotional Weir



$$Q = 1.57\, C_d*(2g)^{0.5} LH^{3/2}$$

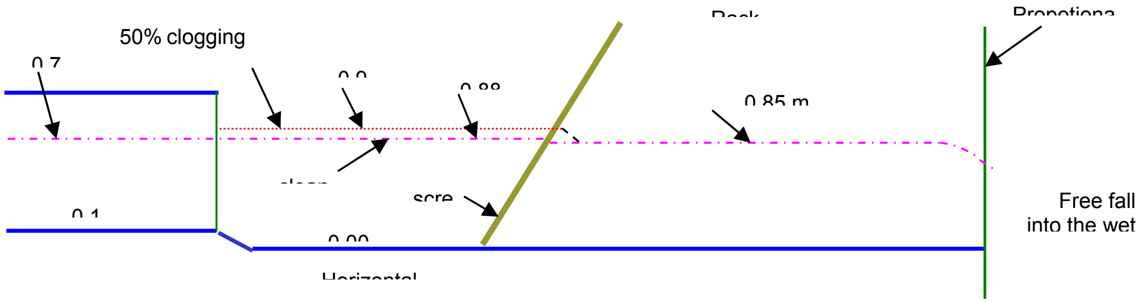
Q = 4.173 $LH^{3/2}$
L = $Q / 4.173\, H^{3/2}$
L = 0.44 m
$LH^{1/2}$ = 0.37 m

Propotional weir

Flow Q m^3/s	H m	L m	D m	V m
1.47	0.95	0.38	1.10	0.61
1.32	0.85	0.40	1.00	0.60
1.16	0.75	0.43	0.90	0.59
1.01	0.65	0.46	0.80	0.57
0.85	0.55	0.50	0.70	0.55
0.70	0.45	0.55	0.60	0.53
0.54	0.35	0.63	0.50	0.49
0.39	0.25	0.74	0.40	0.44
0.23	0.15	0.96	0.30	0.35

Quantity of screening at average flow

Maximum	2.5488	m^3/d
minimum	1.416	m^3/d



Grit Channel Design

0.15 mm sand particles and above = 89%
0.15 mm sand particles = 22%

Settling Velocity = V_s 0.004 < Vs < 0.018

Horizontal Velocity = V_h 0.05 < Vh < 0.2

Channel dimension

flow Q	1.09	m^3/s
Settling Vs	0.018	m/s
Horizontal V_h	0.3	m/s

Cross section area = Q_{max}/V_h = 3.64 m^2

Surface Area = Q_{max}/Vs = 60.70 m^2

Detention time T_d 45 - 90 Take 90 seconds

Td = D/Vs

Depth D 1.62 m

Td =L/Vh

Length L 27 m

Width = S_{urface}/Length 2.25 m

To avoid turbulence condition in the flow increase the length 15% of design length

Length L 31.05 m

Actual Depth 2.12

Length L	**31.05**	**m**
Width W	**2.25**	**m**
Depth D	**2.12**	**m**

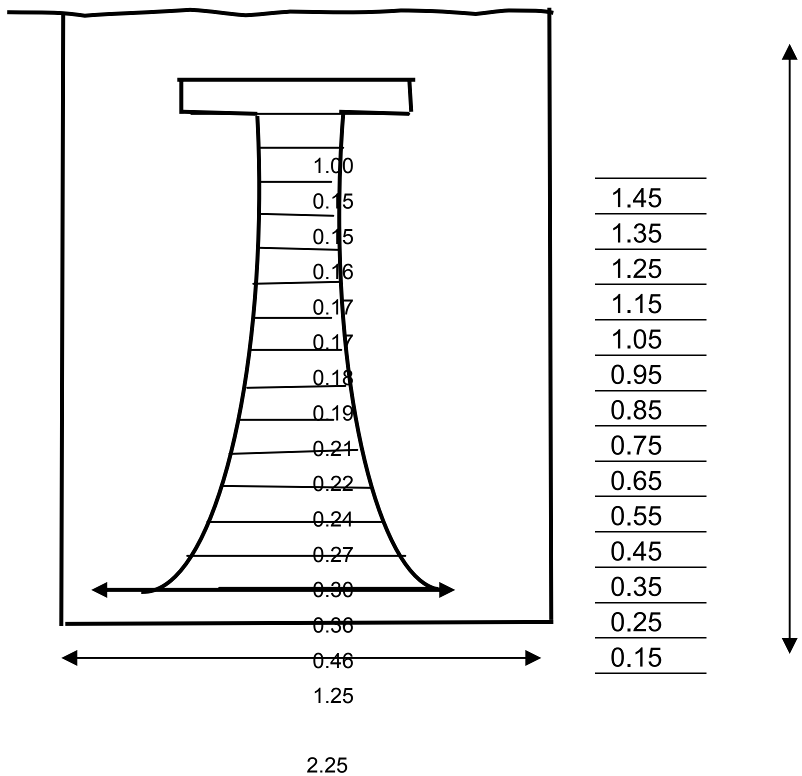
Promotional Weir

$$Q = 1.57\ C_d{*}(2g)^{0.5}LH^{3/2}$$

Q =	4.173	$LH^{3/2}$
L =	$Q\ /4.173\ H^{3/2}$	
L =	0.15	m
$LH^{1/2}$ =	0.18	m

Proportional weir

Flow Q m^3/s	H m	L m	D m	V m/s
1.23	1.65	0.14	1.80	0.30
1.15	1.55	0.14	1.70	0.30
1.08	1.45	0.15	1.60	0.30
1.00	1.35	0.15	1.50	0.30
0.93	1.25	0.16	1.40	0.30
1.09	1.15	0.17	1.30	0.37
0.78	1.05	0.17	1.20	0.29
0.71	0.95	0.18	1.10	0.29
0.63	0.85	0.19	1.00	0.28
0.56	0.75	0.21	0.90	0.28
0.48	0.65	0.22	0.80	0.27
0.41	0.55	0.24	0.70	0.26
0.33	0.45	0.27	0.60	0.25
0.26	0.35	0.30	0.50	0.23
0.19	0.25	0.36	0.40	0.21
0.11	0.15	0.46	0.30	0.17



Quantity of grit removed

3.54 m^3 sand daily

Annex H
Questionnaire of Wastewater Treatment Experts

Survey

Name (optional): Sc. Degree:

A. As part of my Masters degree "Performance Evaluation of Gaza Wastewater Treatment Plant". I proposed six criteria for the evaluation of any future construction of treatment plants or any modifications to the existing treatment plants. These criteria take into consideration the economic, environmental and land availability situation in Gaza. You are requested to support with your opinion in the evaluation of the following criteria weights to help in the Assessment of the options to any modification or construction.

No.	**Criteria**	**Weight**
1.	Annual (operation and maintenance) cost	
2.	Construction Cost.	
3.	Land availability (Requirements)	
4.	Local operating experience	
5.	Process performance.	
6.	Sludge handling	
	Total	**100%**