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Process Monitoring and Performance Evaluation of Existing Wastewater Treatment Plants in Palestinian Rural Areas / West Bank

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December, 2012

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The findings, interpretations and the conclusions expressed in this study don't express the views of Birzeit University, the views of the individual members of the MSc committee or the views of their respective.

TO MY COUNTRY ,,,
, Palestine

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Abstract

In Rural Palestine, the implementation of onsite wastewater treatment plants has focused during the last decade. A fairly wide range of technologies suitable for onsite wastewater treatment plants have been developed. This thesis aimed to evaluate and monitoring the technical issues of different technologies of onsite wastewater treatment plants. A questionnaire has been designed and distributed into different Palestinian rural areas in order to provide a specific technical data about existing onsite wastewater treatment plants.

Fourteen onsite wastewater treatment plants at different levels consist of household, collective, and community were evaluated over a period of 6 months distributed in different Palestinian rural areas in West Bank. At household level, four plants used Activated sludge process and six plants used Up-flow gravel filters. At collective level, one plant used extended aeration process; one plant used Duckweed pond and another used aerobic and anaerobic gravel filters followed by polishing sand filters. At community level, one plant used constructed wetland process. The study compares the observed effluent quality and the removal efficiencies in terms of BOD, COD, TSS, TKN, and TC/FC with typical values reported in the technical literature. In view of the large performance inconsistent observed, the existence of a relationship between design/operational parameters and treatment performance was verified.

The highest values of general efficiency (EG) were found in $UFGF._{Sr}$ plant using technology of aerobic and anaerobic gravel filters followed by polishing sand filter at collective level with efficiency indicator value of 74.2%. On the other hand, the plants using Activated sludge systems at household level had values of EG in a range of 32.5–70.03%, while the plants using up-flow gravel filter technology at household level had a values of EG in a range of -10.08-59.07%. The plants which have a values of EG in a range of 50-60% are $AS._{H}$, $AS._{B.O.}$, $UFGF._{B.L.}$, $UFGF._{B.A}$ and $UFGF._{B.S}$. The plant $EAP._{N}$ using Extended Aeration Process at collective level had values of 63% EG. While the Duckweed-based pond systems and up-flow Anaerobic Sludge Blanket following by Horizontal Flow Constructed Wetlands at community level were found with general efficiency indicator values less than 40%. The differences of values of EG among the different technologies reflect the status of environmental and the operational conditions for each plant.

Turning to the operational performance evaluation in case of activated sludge systems at household level, the different F/M ratios and HRT values did not influence substantially the performance of the aeration zone for $AS._{B.O.}$, $AS._{H}$ and $AS._{N.}$, but observed a clear decline in the performance of the aerobic zone at $AS._{B.}$. This is a result of the operating at underloading conditions with high BOD_5 effluent concentration. The difference shown between the influent flows of $AS._{B.O.}$, $AS._{H}$ and $AS._{N.}$ did not influence significantly the plants' performance, considering the effluent quality. No plants operating at overloading due to lack of water consumption, while $AS._{H}$ and $AS._{N}$ operating at critical loading, as for $AS._{B.O}$ and $AS._{B}$ are operating at underloading conditions. In case of septic tank – up flow gravel filters systems at household level, the performance of the septic tanks need to regular desludging which estimated every 36 months which is never happened for any plant, and the performance of filters are expected to operate without maintenance for 18-24 months, then the filter medium needs to washed out by fresh water which also did not happened for any plant, which is affect on the voids space of filter medium leading to clogging it preventing to provide sufficient HRT like in case $UFGF._{S.}$ plant which is consider completely destroy inside because of its long life cycle period without maintenance. $CW._{N.}$ at community level was found its theoretical design data which calculating depending on the actual design

capacity as reported and the reported design data was not similar with the origin one. Moreover, the sewage that reaches the constructed wetlands infiltrates into the surrounding layers and does not reach the effluent storage tank because of the enormous pressure of overloading of wastewater on the wetlands lagoons leading to destroying its surrounding wall. From the results obtained from all systems and levels, no stationary relationship between loading rates and effluent quality was found. The influence of loading rates differed from plant to plant and from technology to other, and the effluent quality was indicated by several factors related to design and operations parameters. Only a sharp significantly downloading or overloading influenced the effluent quality like in case $AS._{B,}$ $DWP._{Ar,}$ and $CW._{N.}$

The good point that found through the analysis of the questionnaire that, 13% of the existing onsite wastewater treatment plants in Palestinian rural areas which are working well, while 39% working with moderate efficiency, the plants which work with less efficiency estimated as much as 15%, whilst the rest of the plants had been stopped. Where, the plants which were working on bad situation affected by the periodic follow up of operation which is the main factor that affecting on the failure of these plants. The most important result was deduced by this studying is that the availability of experienced engineering designer, skilled personnel, spare parts for repair, and effective operation, maintenance and monitoring are more crucial than the type of technology.

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List of Abbreviations

A: Surface Area
ABPs: Algae based ponds
ABR: Anaerobic Baffled Reactor
ACH: Action Against Hunger
AGFs: Anaerobic Gravel Filters
AL: Aerated Lagoon
AnP: Anaerobic Pond
ARIJ: Applied Research Institute -Jerusalem
AS: Activated Sludge
ASP: Activated sludge process
AT: Aeration tank
ATF: Aerobic Trickling Filter
AUFGF: Anaerobic Up-flow Gravel Filter
BF: Bio-filter
BOD: Biochemical Oxygen Demand
BS: Biochemical System
BZUTL: Birzeit University Testing Laboratories
CD: Chlorine Disinfection
CFU: Colony Forming Unit
CH_4 Methane
CM: Cubic Meter
CO_2: carbon dioxide
CBOD: carbonaceous oxygen demand
COD: chemical oxygen demand
CS: Chlorine Disinfection
CSP: Contact Stabilization Pond
CW: Constructed Wetland
d: day
DL: Duckweed lagoon
DW: Duckweed
DWBP: Duckweed-based pond system
DO: Dissolved Oxygen
EAP: Extended Aeration Process
EAS: Extended Aeration System
EC: Electrical conductivity
eff: effluent
ELV: Environmental Limit Values
EST: Effluent Storage Tank
EU: European Union
FC: Fecal Coliform
FOG: Fat, Oil and Grease
FP: Facultative Pond
g: gram
GLS: Gas-Liquid-Solid
GW: gray wastewater
H_2S hydrogen sulfide gas

HDPE: high density polyethylene
HFCW: Horizontal Flow Constructed Wetlands
HRT: hydraulic retention time
HWE: House of Water and Environment
HWTU: Household Wastewater Treatment Unit
m: meter
MBAS: methylene Blue Active Substance
MCRT: Mean Cell Residence Time
mg: milligram
MGF: Multimedia Granule Filtration
ml: milliliter
MLSS: mixed liquor suspended solids
MLVSS: mixed liquor volatile suspended solids
MOU: Memorandum of Understanding
MPN: most-probable number
MTF: Multilayer Trickling Filter
N_2 nitrogen gas
N_2O: nitrous oxide
NGO: non-governmental organizations
NH_3: Ammonia
NH_4^+ Ammonium
NKj-N: Kjeldhal Nitrogen
OLR: Organic Loading Rate
ortho-PO_4^3 orthophosphate
P: Phosphorous
PARC: Palestinian Agricultural Relief Committees
PCBS: Palestinian Central Bureau of Statistics
PCU: Platinum Cobalt Unit
PE: People Equivalent
PH: Acidity
PHG: Palestinian Hydrology Group
PNA: Palestinian National Authority
PP: Polishing Pond
ppm: Parts per million
PSF: Polishing Sand Filter
PSI: Palestinian Standards Institute
PVC: Polyvinyl Chloride
PWA: Palestinian water authority
RGR: Relative growth rate
RS: raw sewage
SBRs: Sequencing Batch Reactors
SC: Secondary Clarifer
SCF: Save the Children Foundation
SDB: Sludge Drying Bed.
SDT: Subsurface Drainage technique
SF: Sand Filtration
SO_4^{-2}: Sulfate
SP: Stabilization Pond

SS: Suspended Solids
ST: Septic Tank
STD: standard deviation
S.V.I: Sludge Volume Index
T: temperature
TC: Total Coliforms
TDS: Total Dissolved Solids
TF: Trickling Filter
TMTC: Too Many To Count
TS: Total Solids
TSS: Total Suspended Solids
UASB: Upflow anaerobic sludge blanket
UFGF: Upflow Gravel filter
Uv: ultraviolet
UvD: Ultraviolet Disinfection
UWAC: Union of Agricultural Work Committees
VFCW: Vertical Flow Constructed wetlands
WaSH MP: Water, Sanitation and Hygiene Monitoring Programme
WBG: West Bank and Gaza
WD: Water depth
WSPs: Waste stabilization Ponds
WWTP: wastewater treatment plant

CHAPTER ONE
INTRODCTION

1.1 GENERAL BACKGROUND

Palestinian occupied territories consist of two geographical entities – the West Bank and Gaza Strip – with mid-year 2010 estimated population was of 4.05 million (Figure 1.1). About 2.51 million live in the West Bank, and the rest in Gaza Strip. Approximately 52% of the West Bank population lives in 12 urban areas, 42% in over 500 villages and around 6% in 19 refugee camps. In Gaza Strip approximately 64% of the population lives in the five main urban areas, 5% in rural areas and the remaining 32% in eight refugee camps (PCBS, 2010).

Figure 1.1 Governorates of Palestine

In comparison with other countries in the Middle East and North Africa, water supply and sanitation in the Palestinian territories has serious problems with water shortage, because of Israeli occupation. According to the Water, Sanitation and Hygiene (WaSH) Monitoring Programme, Israelis use 85% of the water available from the mountain aquifer in the West Bank, and 82% of the water from the coastal aquifer under Gaza (WaSH MP, 2006).

Generally, the availability and quality of water and wastewater services are considerably worse in the Gaza strip when compared to the West Bank. Moreover, the results indicated that 123 localities (22.9%) in the Palestinian Territory, with 177,275 residents, have no public water network, 251 localities in the Palestinian Territory have old networks, 247 localities suffer from the problem of interruption of water supply, and 192 localities suffer from the problem of non-served areas, all in the West Bank. The results reveal also that 64 localities were connected to public water network after 1998, of which 58 in the West Bank. Regarding the main source of water, data show that 110 of the connected localities to public water networks in the West Bank in 2008 obtain water through Israeli Mekorot Company, while 112 of the connected localities to the public network in the West Bank obtain water through the West Bank Water Department. However in Gaza Strip, 17 of the connected localities to the public network obtain water through Wells, and only 6 localities obtain water through Israeli Mekorot Company. While, there are 157 localities in the Palestinian Territory depend on groundwater wells as an alternative to the public water network. And 421 localities in the Palestinian Territory depend on rainwater collecting wells as an alternative to the public water network.

The wastewater sector in the West Bank and Gaza is characterized by poor sanitation, insufficient treatment of wastewater, unsafe disposal of untreated or partially treated water and the use of untreated wastewater to irrigate edible crops (World Bank, 2004). 85 localities (16%) are connected to public sewage system, of which 64 are in the West Bank (PCBS, 2008). In addition 511 localities use cesspits for wastewater disposal, 478 in the West Bank and 33 in Gaza Strip. In the main cities, the percent of connection to wastewater network ranges from 55% to 86%, while in the refugee camps, the percent of connection ranges from 90% to 98% (PWA, 2009). On the other hand, the main methods of disposing wastewater in the rural areas are cesspits and sewage tanks, with a capacity of 15 to 25CM.These sewage tanks are built close to the house by digging a hole in the ground. They can have concrete walls (septic tanks), or just be earth pools (cesspit) to allow wastewater to infiltrate in the ground. In most cases, cesspits like septic tanks become after few years "waterproof". The sewage tanks must be normally emptied once a month. This is done by private tank trucks with a capacity of 5 CM. The evacuation of one sewage tank is a rather heavy operation: the cost per 5CM truck is in the 50 NIS range (10€). So the monthly cost of sanitation is in the 200 NIS range (40€ for a typical housing in the West Bank) (EU, 2009), however some of the rural areas that are adjacent to main cities, are connected to wastewater network with a percentage that does not exceed 20% of the total houses in those villages (HWE, 2009).

It is worth mentioning that most localities in the Palestinian Territory require appropriate management for establishing sanitation infrastructures in rural communities, effective wastewater treatment plants, and for the promotion of the sustainable practices to protect the environment and public health (EMWATER, 2004). There are 307 localities in the Palestinian Territory need construction of sewage network, 136 need treatment plants, 34 need to cover the open channels of wastewater and finally there are 42 localities that need to develop the sewage network (PCBS, 2008). However there are no plans and budgets to fulfill these requirements.

1.1.1 Existing Technologies of Wastewater Treatment Plants at Urban Areas (Centralized Systems)

In the progress of the wastewater treatment, very little has been made through the past 15 years. Only Eight wastewater treatment plants exist in the urban Palestinian territories. Five in the West Bank with only one of them currently operating in Al-Birah, It has been designed for 50,000 capita capacity and is extendable to 100,000 capita capacity (PWA, 2009). The currently implemented technologies which have been used in the urban areas of West Bank and Gaza Strip are as following:

- Aerated Lagoon (AL)
- Stabilization Pond (SP)
- Extended Aeration System (EAS)
- Anaerobic Pond (AnP)
- Polishing Pond (PP)
- Facultative Pond (FP)

1.1.2 Existing Technologies of Onsite Wastewater Treatment Plants at Rural Areas (Decentralized System)

Several attempts were made to install low cost treatment facilities in the West Bank villages by Palestinian NGOs with international funds. There are three levels of onsite wastewater treatment plants which are at community, collective and household distributed in different Palestinian rural areas. Each of these levels contains different type of technologies arranged in several systems. The below points summarizes the implemented systems and technologies of onsite wastewater treatment plants at rural areas.

At Community Level:

- Up-flow anaerobic sludge blanket (UASB) - Septic Tank (ST)
- Contact Stabilization Pond (CSP)
- Up-flow Anaerobic Sludge Blanket (UASB)-Horizontal Flow Constructed Wetlands (HFCW)
- Extended Aeration Process (EAP) - Chlorine Disinfection (CD) and Sand Filtration (SF)
- Anaerobic Pond (AnP) - Facultative Pond (FP) - Polishing Pond (PP)
- Waste Stabilization Ponds (WSPs)

At Collective Level:

- Septic Tank (ST) - Anaerobic Up-flow Gravel Filter (AUFGF) - Aerobic Trickling Filter (ATF) followed by Polishing Sand Filter (PSF)
- Anaerobic Gravel Filters (AGFs) followed by Polishing Sand Filters (PSFs)
- Small Scale Activated Sludge (Extended Aeration Process (EAP) - Chlorine Disinfection (CD) and Sand Filtration (SF))
- Septic Tank (ST) - Constructed Wetland (CW)
- Septic Tank (ST) - Horizontal Flow Constructed wetlands (HFCW)
- Up-flow Anaerobic Sludge Blanket (UASB) - Vertical Flow Constructed wetlands (VFCW)
- Anaerobic Baffled Reactor (ABR) – Activated Sludge process (AS) – Multimedia Granule Filtration (MGF) – Ultraviolet Disinfection (UvD)
- Septic tank (ST) and Bio-filter (BF) Anaerobic Up-flow Gravel Filter (AUFGF)
- Septic Tank (ST) followed by Trickling Filter (TF)
- Septic Tank (ST) - Multilayer Trickling Filter (TF) - Polishing Pond (PP)
- Duckweed-based pond system (DWBP) - Small-scale biochemical system (BS) - Aeration tank (AT)
- Duckweed and Algae based ponds (DW & ABPs)
- Sequencing Batch Reactors (SBRs)

At Household Level:

- Septic tank (ST) - Up-Flow Gravel filter (UFGF) – Sand Filtration (SF)
- Activated Sludge (AS)
- Constructed Wetland (CW)
- Subsurface Drainage technique (SDT)
- Septic Tank (ST) – Trickling Filter (TF) – Sand Filter (SF)

The preliminary results of testing these onsite systems recommended two; one for the municipal (Total) wastewater and another for gray wastewater. Based on World Bank report (2004), it is recommended as option for wastewater treatment systems in Palestinian rural areas to use partial up flow anaerobic sludge blanket (UASB) reactor followed by facultative ponds for the treatment of municipal wastewater. The treated effluent then can be used for irrigation. Using this type of reactors encourages the use of low-cost and reduces the organic load dramatically. So that post treatment phase can focus on reducing the faecal and nutrient loads. The gray wastewater treatment is based on collecting this type of water in septic tanks followed by an up-flow gravel filter. It has been found that in case of Palestinian rural areas they used in the collective onsite gray wastewater treatment plants system consist of anaerobic pond, gravel filter, sand filter and polishing pond. This kind of system is suitable for a small number of houses (20 -30). A third recommended alternative is the trickling filter. This can be used for schools, hospitals or for a small village. This is a relevant option for rural areas in the West Bank because it is cost effective, easy to operate and has low operation and maintenance costs.

Therefore, low cost technology and small scale treatment plants remain appropriate for rural areas in the West Bank and to a lesser degree in the Gaza Strip. Most of these existing onsite wastewater treatment plants in Rural Areas require upgrading and improvement. There is evidence of poor technology selection and lack of support for operation and maintenance. For the most part, they are undersized, poorly planned, and inadequately designed. However, each technology needs to be carefully assessed on a case by case basis and more information on their efficiency is also required.

1.2 The Main Objective of this Thesis

The main objective of this Thesis is to assess the best ways of developing sustainable wastewater treatment plants in Palestinian rural areas focusing on the technical aspect.

<u>Specific Objectives:</u>

1- Assess the current status of existing onsite wastewater treatment plants and evaluate the design of all different system plants.
2- Monitor the technical performance of existing wastewater treatment plants and comparing their performance with their reference value as reported in the literature and with their either calculated or reported design/operational parameters.
3- Compare the efficiency of the monitored treatment systems in various rural areas among them with other systems elsewhere.

1.3 Significance of this Thesis

This Thesis is expected to provide insight on making the WWTPs in rural areas to be more effective to monitoring the performance of the various systems in order to identify priorities for improving the current status. So, this Thesis will make the following contributions to wastewater treatment in rural Palestine:

- There is no study had been done in the past about the comparisons between the different systems of onsite wastewater treatment plants in Palestine rural areas, thus, this study will contributes to provide a comprehensive review of existing treatment plants and comparing the efficiency among them.

- On the other hand, this Thesis will contribute to provide scientific information on each plant in terms of operating performance for all of their unit operations.

- This study will contribute to supply an overview with detailed information on the status of the onsite wastewater treatment plants in Palestinian rural areas. In addition, it will show us the true relationship between the technical/analytical and operational performance evaluation of existing wastewater treatment systems and the comparison between them.

1.4 Scope of this Thesis

The subject matter of Thesis is presented in five chapters. This first chapter outlines the general background, project area, problem definition, the main objectives, and significance. The second chapter represents the literature review including centralized and decentralized treatment system, wastewater treatment stages, wastewater characterization, wastewater treatment processes, components of wastewater treatment systems, and existing onsite wastewater treatment systems in Palestinian rural areas. The third chapter reviews the materials and methods of field data collection, questionnaire design, operational methods, sampling and analytical methods, and calculations. The fourth chapter discusses the results of questionnaire, the technical/analytical and operational performance evaluation of existing wastewater treatment systems, and evaluation of different treatment systems performance. The overall conclusions and recommendations are provided in chapter five.

CHAPTER TWO

LITERATURE REVIEW

2.1 Introduction

It may be surprised to learn that the treatment of wastewater is a relatively modern practice. Although sewers to remove foul-smelling water were common in ancient Rome, it was not until the 19th century that large cities began to understand the necessity of reducing the amount of pollutants in the used water they were discharging to the environment.

All persons generate wastewater, also known as sewage, as they go about their daily activities of washing dishes and clothes, showering and bathing, and using the toilet. To protect public health and environmental quality, wastewater must be cleaned (treated) before it is returned to the environment for further use. What happens in a wastewater treatment plant is essentially the same as what occurs naturally in an ocean, lake, river or stream. The function of a wastewater treatment plant is to speed up this natural cleansing process. The practice of wastewater collection and treatment has been developed and perfected, using some of the most technically sound biological, physical, chemical and mechanical techniques available. As a result, public health and water quality are protected better today than ever before.

Two types of treatment facilities are used in the occupied Palestinian territory: centralized (off-site) and decentralized (on-site). Both systems treat wastewater by separating solids from the water then biologically degrading the remaining organic materials. It will be mentioned and explain later the on-site systems that were used in the Rural Palestinian areas. (See section 2.5)

2.1.1 Centralized and Decentralized Treatment System

2.1.1.1 Centralized (off-site) Wastewater Treatment System

Centralized treatment system is also called off-site system. This type of system used to treat wastewater for large residential area as a city. The centralized treatment has been applied very successfully in industrialized countries (Winderer and Schereff, 2000). This approach is only suitable when there are a financial ability for high cost investment for construction of sewer systems. Centralized system not only requires so much money for operation, maintenance, and collection wastewater from generate point to treatment place. This system also needs very good infrastructure support for its operation such as pipeline system, pump stations and electricity system. In case of Palestine, review section 1.1.1 for the technologies that were used as centralized systems.

2.1.1.2 Decentralized (onsite) Wastewater Treatment System

The term “decentralized wastewater treatment” is defined as “An onsite or cluster wastewater system that is used to treat and dispose of relatively small volumes of wastewater, generally originating from individual or groups of dwellings and businesses that are located relatively close together”. Decentralized treatment involves using a combination of treatment technology options, both traditional and innovative (National Small Flows Clearinghouse, 2000). It consists of wastewater collection, wastewater treatment, reuse and disposal of municipal wastewater. Not every Decentralized wastewater treatment system have all of component as above, but it can be applied difference technology in order to get effective treatment same as centralized system. Decentralized system is used in rural and urban for long time in both developed and developing countries. In urban areas, it seemed as pretreatment of wastewater and in rural areas this system used as the best solution for treating of wastewater. In case of

Rural Palestine, review section 1.1.2 for the technologies that were used as decentralized systems and their classifications.

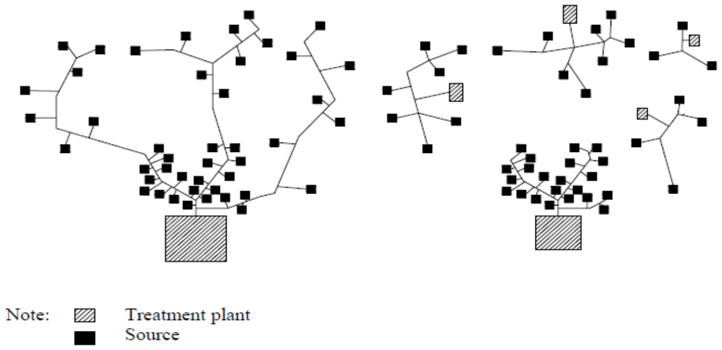


Figure 2.1 Centralized and Decentralized Approaches

According to the National Small Flows Clearinghouse (2000), decentralized approach to wastewater treatment is beneficial for a number of reasons.

- It help to save money by deciding on a preventive strategy such as assessing needs and conditions of community to manage waste before a crisis occurs, thereby avoiding unnecessary cost for treat and re-create environment.
- Allow homeowners to continue use their onsite systems with properly functioning.
- Eliminating the large transfers of water from one watershed to another that happens with centralized treatment.
- Strategy may be the most cost-effective for treatment in rural communities with sparse populations.
- It is appropriate for varying site conditions including ecologically sensitive areas. The treatment methods can be tailored to suit different site conditions.

In order to meet public health and environmental protection goals using decentralized systems, a combination of process to treat and disposal of wastewater is the best way to achieve treatment goals. The combination consists of selection of technology, management, monitoring, operation, and maintenance. The selection of technology is first part and very importance. At present, there are many options existing for wastewater treatment that can be applied for onsite process such as septic tank, constructed wetland…etc. Each of options has advantages and disadvantages. In order to get the best effect on treatment objective the selection must be careful carried out by technician. The second part is management and it is a key that keeps decentralized treatment system operating effectively. The management consists of installation, operation, maintenance, and monitoring. The options of onsite wastewater treatment in rural areas are discussed below:

2.2 Onsite Wastewater Treatment Options in Rural Communities

Wastewater treatment systems for small communities in rural areas are a matter of concern to every country. They represent the majority of the existing treatment plants subjected to high seasonal and even daily variations in wastewater flow and load on the one hand and on the other need to be easy to manage and to operate (Kramer et al., 2007). In Palestine, There are numerous technologies to deal with the treatment of wastewater in rural areas. Many of these technologies have been used in the Pacific however, for many reasons have failed. These reasons include inappropriate technology, insufficient operation and maintenance practices, lack of funding and lack of skilled personnel to name a few

(UNEP, 2002). The best choice of system depends on a number of factors including whether a new system is being installed or a disused wastewater system is being converted because the households have been connected to sewer or not. The wastewater treatment options as shown in Figure 2.2 include aerobic and anaerobic reactors, septic tanks, oxidation ponds etc.

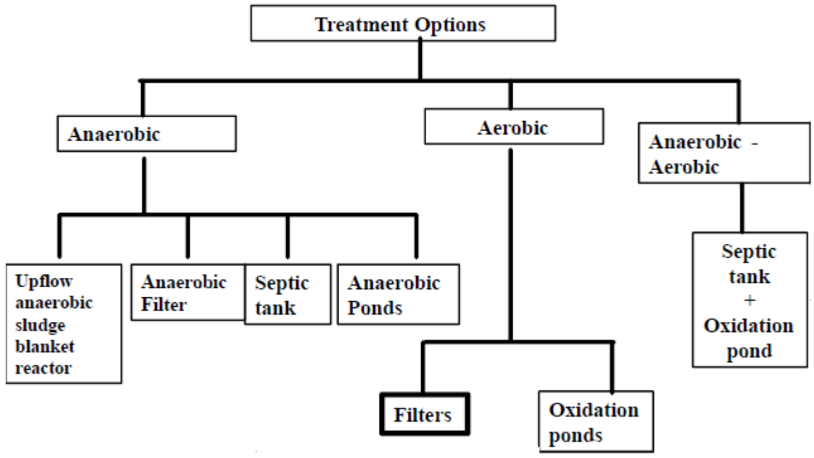


Figure 2.2 Wastewater Treatment Options

2.3 Wastewater treatment stages

Wastewater treatment processes are classified into the following stages, related generally to the quality of effluent produced by the process.

- **Preliminary Treatment:** The aim of preliminary treatment is to protect the principal treatment processes that follow by the removal of plastic, grease. Scum, solids and grit which can block and wear pipe work, valves, pumps and treatment equipment. Methods and equipment used to remove these materials may include physical, chemical addition, preaeration, bar racks, screens and shredding devices, and grit chambers. Preliminary treatment may also consist of a single process or a combination of processes, such as coagulation, flocculation, and flotation.

- **Primary Treatment:** In primary treatment system, a sedimentation tank is used to coarsely screen out oils/greases and solids prior to reuse. Sedimentation can remove all the readily settleable matter from the wastewater, giving a corresponding reduction in Suspended Solids (SS) and Biochemical Oxygen Demand (BOD) concentrations. Grease and fatty materials float to the surface to form a scum which can be removed. A number of different types of sedimentation tanks or clarifiers are used for primary sedimentation including septic tanks, Imhoff tanks, clarigestors, rectangular, and circular tanks. This system is recognized as an economically attractive option for Wastewater reuse because it requires minimal maintenance.

- **Secondary Treatment:** In secondary treatment system basically consists of some form of biological process. The main objective of secondary treatment is to remove most of the fine suspended and dissolved degradable organic matter which remains after primary treatment, so that the effluent may be rendered suitable for discharge. Most of the any biological treatment processes for secondary treatment can be classified as attached growth or suspended growth systems. Each system relies on an established mixed population of microorganisms in the presence of oxygen and trace amounts of nutrients. The microorganisms consume organic material in the waste to sustain their life processes and to produce new microorganisms. In attached growth systems, the mass of microorganisms affecting treatment are attached to supporting media. Examples of attached growth systems include trickling filters and rotating biological contactors. Suspended growth systems have reactors containing microorganisms suspended in the wastewater. These systems include lagoons and the many variations of activated sludge process. This reduces health risk at end use with human contact and provides additional safety for reuse.

- **Tertiary Treatment:** Tertiary treatment is carried out where the effluent must be of a higher quality than that obtainable by secondary treatment. The main objective is usually effluent polishing (the removal of fine suspended solids). Because these are mostly organic, their removal will result in a reduction in the effluent BOD. Effluent polishing can be carried out using physical separation of suspended solids from the effluent or by more complex processes which involve biological as well as physical action. Physical separation processes include microstrainers and various types of filter ranging from slow sand filters to rapid sand, dual media and mixed media filters. Processes involving biological action include tertiary ponds, grass filtration, land filtration and wetlands. Fixed film biological rotating drums, membrane bioreactors, biologically aerated filters, activated sludge and membrane treatment systems are all included in this category. Other processes, which are gaining greater use in tertiary treatment, include ozonation and ultraviolet (UV) radiation, which act to reduce levels of pathogens in the effluent. Most "package" plants available provide secondary treatment. However when used in conjunctions with another secondary treatment process may provide tertiary treatment.

- **Advanced treatment:** Advanced wastewater treatment may be used to reduce the concentrations of nutrients, nitrogen or phosphorous and soluble organic substances to levels below those normally attained through tertiary or secondary treatment. The Advanced wastewater treatment process may include physical, chemical, or biological processes, or combination of these. The treatment requirements for compliance with the effluent limitations in the permit usually influence the types of advanced wastewater treatment processes, if any, selected for a specific plant.

- **Disinfection:** Disinfection of wastewater treatment plant effluent inactivates or destroys pathogenic bacteria, viruses, and amoebic cysts commonly found in wastewater. Pathogens may cause outbreaks of waterborne diseases such as typhoid, cholera, paratyphoid, bacillary dysentery, poliomyelitis, and infectious hepatitis. Generally, disinfection processes may be classified as natural, chemical, physical or radiation. Historically, chemical treatment using halogens, particularly chlorine, has dominated wastewater disinfection practices.

- **Sludge Treatment:** Sludge, the settled solids accumulated and separated from liquid treatment train, must be treated prior to disposal because raw sludge is unstable, and contains pathogenic organisms. Generally, the three types of wastewater residues primary, biological, and chemical sludge can be characterized by their source in the liquid treatment train. Wastewater treatment produces other residuals including screenings, grit, and scum. Primary sludge, which usually ranges from 40% to 60% of the influent suspended solids, generally has a concentration of 2 to 6% solids when removed from the primary clarifiers. Biological sludge, such as waste activated sludge; attached growth sludge, such as trickling filter sludge; and rotating biological contactor sludge. Chemical sludge characteristics depend on the type of chemical that is, alum, ferric salts, or lime used in wastewater treatment processes (WEF, 2007). Sludge treatment processes typically consist of the following:

- Thickening: This follows separation of sludge from wastewater and involves sludge volume reduction by removal of waste. Thickening technologies include gravity thickening, dissolved air flotation thickening, centrifugation, and rotating drum thickening.
- Chemical stabilization: generally consists of raising the pH of the sludge above 11.0 to reduce pathogens and odors. Lime is the most commonly used chemical for sludge stabilization.
- Digestion: a means of stabilizing sludge, reduces the volatile content and pathogen count, thereby producing a less odorous and putrescible material. Typical technologies include anaerobic and aerobic digestion.
- Dewatering: dewatering further reducing sludge volume and weight. Dewatering equipment includes belt filter presses, sand drying beds, vacuum-assisted drying beds, centrifuges, plate and frame filter presses, and vacuum filters.
- Composting: composting reduces both volume and odors and destroys pathogens. Its use is increasing because the compost material is an excellent soil conditioner. Three types of composting processes include window, aerated static pile, and in-vessel.
- Heat drying: heat drying processes, also used to reduce sludge volume and destroy pathogens, include flash-drying, rotary, toroidal, or spray drying systems. Digestion is generally not required for heat drying of secondary sludge.
- Incineration: incineration maximizes sludge volume reduction and destroys pathogens and many organic toxic substances. Incineration processes include multiple hearth, fluidized bed, electric, and cyclone furnaces.

- **Sludge Disposal:** After sludge treatment has been treated, it must used or disposed of in an environmentally acceptable manner. Sludge use or disposal alternative include the following:

    - Landfilling,
    - Land application of a liquid or dewatered sludge,
    - As a soil conditioner, and
    - Incineration and ash disposal to a landfill.

Treatment options described above involve various processes predominantly physical, chemical and biological processes for treatment of various parameters of wastewater. Physical processes for removing solids include screening, sedimentation, and filtration. Chemical processes used as an aid to sedimentation include chemical coagulation and precipitation. Activated carbon adsorption is a physical-chemical process for removing organic pollutants. Chemical processes for removing nutrients include breakpoint chlorination for nitrogen reduction and lime addition for phosphorous reduction. These processes are not necessarily put in sequence and do not form part of treatment systems.

2.4 Wastewater Treatment processes
2.4.1 Aerobic Treatment Process

The goal of aerobic treatment is the degradation of organic and inorganic compound in the presence of oxygen as an electron accepter in the redox reaction which is used as secondary treatment process in the treatment of wastewater, often measured in mg/l of O_2 consumed over a 5-day incubation period which is referred to as BOD. The following equation 2.1 shows the process of aerobic degradation

$CH_2O + O_2 \Longrightarrow CO_2 + H_2O$ + new biomass ……………………………………….... 2.1

The basic aerobic treatment process involves providing a suitable oxygen rich environment for organisms that can reduce the organic portion of the waste into carbon dioxide and water in the presence of oxygen. The common options for secondary aerobic wastewater treatment illustrated as shown in table 2.1

Table 2.1 Common options for secondary aerobic wastewater treatment (Parr et al., 2000)

Treatment process	Description	Key features
Activated sludge process (ASP)	Oxygen is mechanically supplied to bacteria which feed on organic material and provide treatment	Sophisticated process with many mechanical and electrical parts, which also needs careful operator control. Produces large quantities of sludge for disposal, but provides high degree of treatment (when working well).
Aerated lagoons	Like Waste Stabilization Ponds (WSPs) but with mechanical aeration	Not very common; oxygen requirements mostly from aeration and hence more complicated and higher operation and maintenance costs.
* Land treatment	Wastewater is supplied in controlled conditions to the soil.	Soil matrix has quite a high capacity for treatment of normal domestic sewage, as long as capacity is not exceeded. Some pollutants, such as phosphorus, are not easily removed.
* Reed (or constructed wetlands) beds	Swage flow through an area of reeds	Treatment by action of soil matrix and, particularly, the soil/root interface of the plants. Requires significant land area, but no oxygenation requirement.
Rotating biological contractor (or biodisk)	Series of thin vertical plates which provide surface area for bacteria to grow	Plates are exposed to air and then the sewage by rotating with about 40 per cent immersion in sewage. Treatment by conventional aerobic process. Used in small-scale applications in Europe.
Trickling (or 'percolating') filters	Sewage passes down through a loose bed of stones, and the bacteria on the surface of the stones treats the sewage	An aerobic process in which bacteria take oxygen from the atmosphere (no external mechanical aeration). Has moving parts, which often break down in developing county locations.
* Waste-stabilization ponds (WSPs) ('lagoons' or 'oxidation ponds')	Large surface-area ponds	Treatment is essentially by sun light, encouraging algal growth which provides the oxygen requirement for bacteria to oxidize the organic waste. Requires significant land area, but one of the few processes which are effective at treating pathogenic material. Natural process with no power/ oxygen requirement. Often used to provide water of sufficient quality for irrigation, and very suited to hot, sunny climates.
Oxidation ditch	Oval-shaped channel with aeration provided	Requires more power than WSPs but less land, and easier to control than processes such as ASP.

* Indicates processes more suitable for developing countries.

2.4.2 Anaerobic Treatment Process

Anaerobic treatment is the use of biological processes in the absence of oxygen to stabilize organic (carbonaceous) material by conversion to methane (CH_4) and inorganic products, including orthophosphate (ortho-PO_4^3), carbon dioxide (CO_2), hydrogen sulfide gas (H_2S), nitrogen gas (N_2) and ammonia (NH_3), and in additional of anaerobic biomass. Treatment in an anaerobic reactor removes the major part of the carbonaceous oxygen demand (CBOD) from raw wastewater, but substantial nitrogenous oxygen demand remains (McCarty, 1986). The anaerobic process is done by putrefactive bacteria, which break down the organic material under airless conditions. The conversion processes in the anaerobic degradation are done by five major groups of bacteria:

1. Fermentative bacteria;
2. Hydrogen – producing acetogenic bacteria;

3. Hydrogen consuming acetogenic bacteria;
4. Carbon dioxide – reducing methanogens;
5. Aceticlastic methanogens

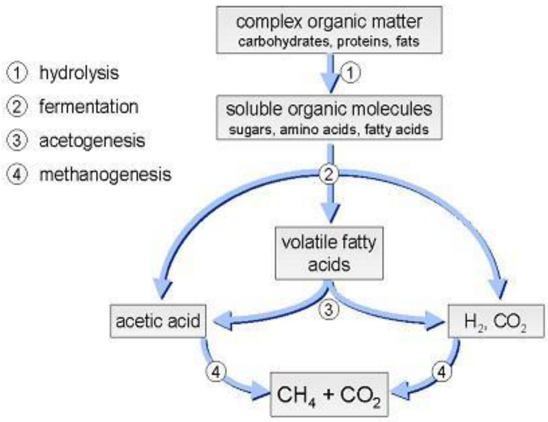


Figure 2.3 Anaerobic degradation Processes

Four different phases can be distinguished in the overall conversion process; these are Hydrolysis, Acidogenesis, Acetogenesis and Methanogenesis. The mechanisms of these different processes are as follows:

- Hydrolysis: First, complex polymeric materials such as polysaccharides, proteins and lipids (fats and grease) are hydrolysed by extracellular enzymes to soluble products of a size small enough to allow their transport across cell membrane.
- Acidogenesis: These relatively simple, soluble compounds are fermented or anaerobically oxidised to short – chain fatty acids, alcohol, carbon dioxide, hydrogen and ammonia.
- Acetogenesis: The short –chain fatty acids (other than acetate) are converted to acetate, hydrogen gas and carbon dioxide.
- Methanogenesis: Methanogenesis occurs from carbon dioxide reduction by hydrogen and from acetate to produce methane.

2.4.3 Differences between Aerobic and Anaerobic Treatment Processes

In the wastewater engineering field organic pollution is measured by the weight of oxygen it takes to oxidize it chemically, referred to as the "chemical oxygen demand" (COD). The best way to appreciate anaerobic wastewater treatment is to compare its COD balance with that of aerobic wastewater treatment, as shown in figure 2.4 the aerobic digestion transforms oxygen consuming substances in the wastewater into a residual sludge (McCarty, 1986). The resulting sludge contains large amounts of volatile solids, mostly in the form of bacterial biomass, that require further stabilization smaller amounts of oxygen consuming substances and solid material remain in the effluent, but the large amounts of unstable sludge mean an additional disposal problem, which costs lots of money to get rid of in developed countries with less area, but can be of interest as low-cost fertilizer in developing countries if the sludge is not contaminated. Elemental oxygen has to be continuously supplied by aerating the wastewater. The COD in wastewater during anaerobic treatment is highly converted to methane, which is a valuable fuel. Very little COD is converted to sludge. No major inputs are required to operate the system. Nevertheless it depends on stable preconditions as i.e. temperature to make the process stable. The residual sludge does not require additional treatment because it is more stable, i.e. it is more thoroughly biodegraded than an aerobic sludge. Anaerobic sludge has better settling properties than an aerobic sludge and is easier to dewater. Where a secondary quality treated effluent is required, additional treatment is needed to remove the residual oxygen demand and suspended solids from the anaerobic enhanced primary treated effluent.

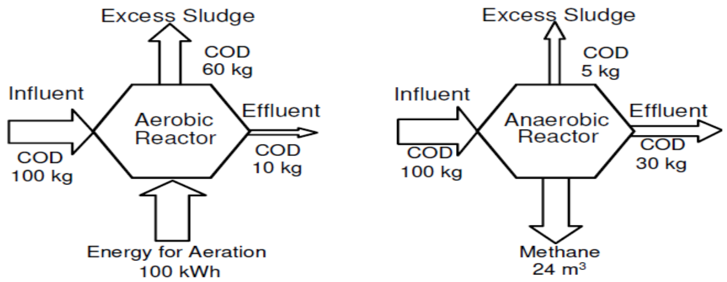


Figure 2.4 COD Balance and Energy Comparison between Aerobic and Anaerobic Treatment Processes

Treatment processes for nitrogen removal are generally premised on what is termed “sequential nitrification/denitrification”. These sequential reactions require different environments and are often carried out in separate areas in the wastewater treatment system.

2.4.4 Ammonification Process

The first step in the removal of Total-N during biological treatment is conversion of organic N to ammonia/ammonium. For domestic sewage, where organic N consists of urea and faecal material, this already takes place to a certain extent while travelling through sewer pipes.

- The ratio of ammonia (NH_3) versus ammonium (NH_4^+) is affected by pH and temperature. At conditions typical for most onsite wastewater treatment plants (pH of 6 to 8.5, temperatures of 10 to 40 ºC), far more ammonium than ammonia is produced.

2.4.5 Nitrification/ Denitrification Process

The first step in the sequence uses aerobic processes to transform the organic nitrogen and ammonium products to nitrate. A variety of treatment systems can be used to accomplish this aerobic process in any aerobic treatment units. For example, when septic tank effluent is applied at a low organic loading rate to deep, well aerated media, such as gravel or sand filter, nitrification has been effectively accomplished.

The second step requires shifting the process from an aerobic environment to an environment without dissolved oxygen (referred to as an anoxic process) where different species of bacteria can grow. These bacteria utilize the nitrate-bound oxygen formed in the first step to oxidize organic matter and in the process transform the nitrogen to gas. These bacteria also need organic carbon during the process in order to form new cell tissue. Inadequate supplies of organic carbon will limit the denitrification process. Conceptually, the two-step process would be as illustrate in figure 2.5:

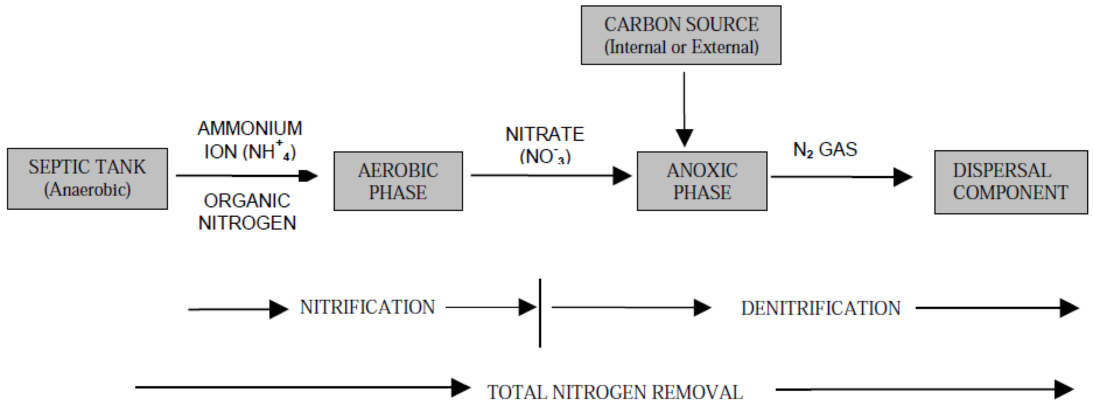


Figure 2.5 Biological Nitrification/Denitrification in Onsite Wastewater Systems

Biological nitrification occurs under optimal conditions for growth and sustenance of the aerobic autotrophic nitrifying bacteria. These conditions include the following:

- Fluctuating Flow Rates: The bacteria involved in both the aerobic and anoxic sequences can be adversely affected by either diminished flows (such as when the homeowners are on vacation or suffer from constant interruption of water), or by surge flows (such as large gatherings that cause peak flows).

- Fluctuating Waste Strengths: Similar to waste flow impacts, varying waste strength can have an adverse impact on the bacterial colonies that keep the biological processes working (in terms of onsite mixed wastewater treatment systems).

- Carbon Source: Too much BOD will result in competition with heterotrophic bacteria.

- Dissolved Oxygen (DO): Rate of nitrification is reduced at DO concentrations below 2 mg/l (Hammer et al., 1994) and the conversion of nitrite to nitrate is greatly inhibited at DO concentrations below 0.5 mg/l (Tchobanoglous et al., 2003).

- pH and Alkalinity: The pH and alkalinity of the source water will have a dramatic effect on the rate of nitrification. The optimum pH range for nitrification is 6.5-8.0 (Tchobanoglous et al., 2003). The biochemical process involved in nitrification consumes alkalinity and in areas with water sources that are low in alkalinity, nitrification will lower the pH to inhibitory levels for the nitrifying bacteria.

- Temperature: Temperature variations can significantly affect the various bacteria involved in the nitrification. Optimum temperature is 30 - 35ºC with little nitrification occurring below 5ºC or above 40ºC (Hammer et al., 1994). Microbes will work twice as fast at 24ºC compared to 12ºC. In order to compensate for this factor, longer detention times may be necessary in colder climates.

- Inhibitory Compounds: Because nitrogen transformation relies on bacterial processes, some chemicals can have immediate and serious impacts on the bacterial colonies living within the system. Nitrifying bacteria, in particular, are very susceptible to organic and inorganic inhibitors. Very small amounts of an inhibitor can kill these bacterial colonies and upset the nitrification process (WSDH, 2005).

Biological Denitrification occurs under optimal conditions for growth and sustenance of the anaerobic heterotrophic denitrifying bacteria. These conditions include the following:

- Dissolved Oxygen (DO): Denitrification occurs when oxygen levels are depleted and nitrate becomes the primary oxygen source for microorganisms. The process is performed under anoxic conditions, when the dissolved oxygen concentration is less than 0.5 mg/l, ideally less than 0.2. When bacteria break apart nitrate to gain the oxygen, the nitrate is reduced to nitrous oxide (N2O), and, in turn, nitrogen gas (N2). Since nitrogen gas has low water solubility, it escapes into the atmosphere as gas bubbles. Free nitrogen is the major component of air, thus its release does not cause any environmental concern.

- pH and Alkalinity : Optimum pH values for denitrification are between 7.0 and 8.5. Denitrification is an alkalinity producing process. Approximately 3.0 to 3.6 mg/l of alkalinity (as CaCO3) is produced per mg/l of nitrate, thus partially mitigating the lowering of pH caused by nitrification in the mixed liquor in case of activated sludge systems.

- Fluctuating Waste Strengths: Since denitrifying bacteria are facultative organisms, they can use either dissolved oxygen or nitrate as an oxygen source for metabolism and oxidation of organic matter. If dissolved oxygen and nitrate are present, bacteria will use the dissolved oxygen first. That is, the bacteria will not lower the nitrate concentration. Denitrification occurs only under anaerobic or anoxic conditions.

- Carbon Source : Another important aspect of denitrification is the requirement for carbon; that is, the presence of sufficient organic matter to drive the denitrification reaction. Organic matter may be in the form of raw wastewater, or supplemental carbon.

- Temperature: Temperature affects the growth rate of denitrifying organisms, with greater growth rate at higher temperatures. Denitrification can occur between 5 and 40ºC, and these rates increase with temperature and type of organic source present. The highest growth rate can be found when using methanol or acetic acid. A slightly lower rate using raw wastewater will occur, and the lowest growth rates are found when relying on endogenous carbon sources at low water temperatures.

- Inhibitory Compounds: Denitrifying organisms are generally less sensitive to toxic chemicals than nitrifiers, and recover from toxic shock loads quicker than nitrifiers.

2.5 Existing Onsite Wastewater Treatment Systems in Palestinian Rural Areas:

Onsite (or decentralized) wastewater treatment systems are used to treat wastewater normally within the boundaries of individual household, collective or community properties. In Palestine, There are numerous technologies to deal with the treatment of wastewater in rural areas. Many of these technologies have been used in the Pacific however, for many reasons have failed. These reasons include inappropriate technology, insufficient operation and maintenance practices, lack of funding and lack of skilled personnel to name a few (UNEP, 2002). Initial results of these onsite wastewater treatment plants were found to be of the same magnitude as those for large conventional secondary treatment systems. The reported elimination rates were for COD 90 – 95 %; BOD 90 – 95 %; and for TSS 90 – 99%. The treatment efficiency has to be shown mainly affected by the electromechanical parts used in the systems, provided proper operation and maintenance. Most failure, so far, concerns pumps, where blockage through fouling may cause breakdown of the system. Beside process efficiency and reliability, sludge disposal, land requirement, environmental impact, capital and operational expenditure, sustainability and process simplicity are considered as critical items in selecting a treatment option for rural areas in Palestine. This section will focus on the most treatment technologies that are currently used in the West Bank rural areas, grouped under the following headings:

2.5.1 Community Onsite Wastewater Treatment Systems
2.5.1.1 Contact Stabilization System:

A small-scale Contact Stabilization System is used in Birzeit University campus as an institutional wastewater treatment plant. Domestic wastewater from all buildings including the main restaurants and cafeterias as well as various laboratories of Birzeit University campus is collected with a central sewerage network. The amount of sewage collected and treated is about 60-80 m^3/d. Pre-aeration of wastewater influent is accomplished in the holding tank to freshen the sewage and control odor problems. The treatment plant consists of a communitor with a bar screen, surge tank, tertiary tank which consists of three parts: a sand filter well A, well B, and the chlorination unit in well C, a sludge basin, main treatment unit (circular part) which consists of the core of the clarifier and three chambers surrounding the clarifier: contact zone, digester zone, and re-aeration zone). The core of the clarifier is made of a circular steel chamber with a concrete fill provided at the bottom. A schematic flow diagram of the contact stabilization units is illustrated in Figure 2.6. The process of de-sludging is performed each 6 months. The dimensions of the units of the treatment plant as mentioned in the design report are: radius of clarifier is 280cm, depth of clarifier is 400 cm, volume of clarifier is 100.99m^3, surface rate of effluent is 20m/d, volume of contact zone is 70.6m^3, detention time of contact zone is 3hrs, volume of re-aeration zone is 188.6m^3, retention time of re-aeration zone is 8hrs, volume of digester zone is 126.4m^3, organic load on sand filter is 20.4Kg/d, suspended solids on sand filter is 9.7m^2, rate of filtration is 0.0403m/min, and particles used in sand filter are no more than 0.8mm.

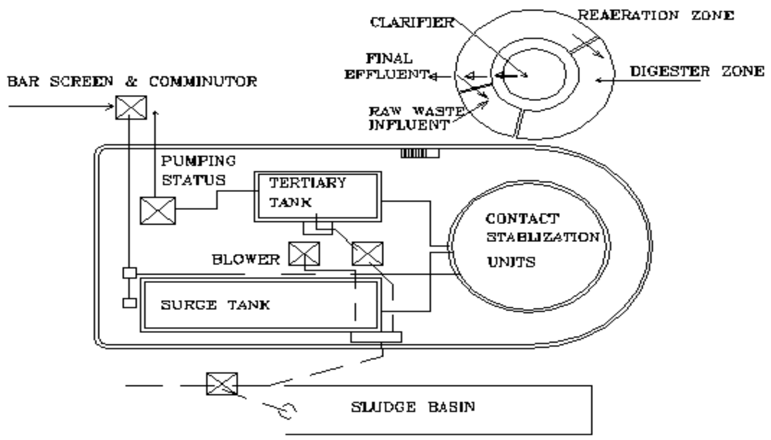


Figure 2.6 Contact Stabilization Systems

The purification capacity of the treatment system was studied over the last few years. COD removal was 85% and the effluent COD concentration was less than 110 mg/L (average value was 88 mg/l). Suspended solids were removed with equal efficiency. Aerobic stabilization of organic solids was efficient and excess sludge had to be removed on a sludge drying bed once to twice a year. High nitrification (70% of the influent nitrogen were nitrified) could be maintained at 15 °C, and 42% of the oxidized nitrogen was denitrified. The specific oxygenation capacity of the treatment system is relatively high and reached about 5 kWh/kg COD (Al-Sa`ed and Zimmo, 2004). The actual specific wastewater treatment cost is about 0.52 US$/m$^3$ or about 58 US$ per population equivalent per year. According to Al-Sa`ed and Zimmo (2004), these high specific costs are not technology specific, but rather operational mode related, which can be reduced through regulation of the aeration process and installment of pre-denitrification stage.

2.5.1.2 Talitha Kumi Waste Stabilization Ponds (WSPs)

Waste stabilization Ponds (WSPs) were build at campus of Talitha Kumi School in the year 2001 by Palestinian Hydrology Group (PHG). WSPs technology is consider as one of the most important natural methods for wastewater treatment. WSPs are mainly shallow man-made basins comprising a single or several series of anaerobic, facultative or maturation/Polishing ponds. The primary treatment takes place in the anaerobic pond, which is mainly designed for removing suspended solids, and some of the soluble element of organic matter BOD. During the secondary stage in the facultative pond most of the remaining BOD is removed through the coordinated activity of algae and heterotrophic bacteria. The main function of the tertiary treatment in the maturation pond is the removal of pathogens and nutrients (especially nitrogen). WSPs technology is the most cost-effective wastewater treatment technology for the removal of pathogenic micro-organisms. It is particularly well suited for tropical and subtropical countries because the intensity of the sunlight and temperature are key factors for the efficiency of the removal processes (Mara, D. et al. 1992).

Talitha Kumi community WWTP was studied by a master thesis research. Theodory (2002) found that the original design of the unit operations of the WSPs was not based on real data of wastewater analysis but assumed values. The actual design data obtained after commissioning the treatment facility revealed that the anaerobic, facultative and polishing ponds were overloaded. Based on the results obtained from the monitoring phase of the WSPs, Table 2.2 illustrates the design and actual design data for an adequately treated effluent.

Table 2.2 Theoretical and actual design data for Talitha Kumi WSP (Theodory, 2002)

Design Parameter	Unit	Anaerobic Pond (AnP)		Facultative Ponds (FP)	
		Design	Actual	Design	Actual
Dry weather flow rate	(m^3/d)	38			
Surface area (A)	(m^2)	22.5		157.5	
Water depth (WD)	(m)	2		1.1	
Hydraulic retention time (HRT)	(d)	2	1.2	11	4.5
Volumetric organic loading rate	(g BOD/m^3.d)	186	307		
Surface organic loading rate	(kg BOD/ha.d)			167	540

As the polishing ponds would have no role in the overall treatment efficiency of WSPs system, the design of these units were not rechecked. From Table 2.2, it is obvious that the WSPs would not properly function as designed for. The researcher was aware of the fact that the design of the unit operations of the WSPs was made by a non-experienced engineering office and the implementing agency did not make an accurate budget for a well-engineered design. During the monitoring period, the anaerobic pond with a short hydraulic retention time of 1.2 days was able to achieve a reduction of 38% and 45% in both total BOD and filtered BOD respectively. A removal rate of 41% was also recorded for both total and filtered COD. In the same range, removal rates of nutrients were noticed. For ammonium, nitrate and total phosphate, reduction rate of 42%, 46% and 9% were achieved respectively, whereas only 39% of the TSS was achieved in the anaerobic pond. The effluent quality of the facultative and maturation ponds was very poor. There was an increase in total BOD, total COD and filtered COD concentrations in the effluent. The removal rate for phosphorus and ammonium were 30% and 12% respectively. It was noticed that there was a slight increase in the nitrate concentration. Finally, an increase in the TSS concentration was also noticed. The obtained results from this research study showed that the performances of the anaerobic pond, facultative and maturation ponds were not satisfactory. An upgrading scheme for the WSPs suggested, where a fixed film technology in some facultative ponds applied to create additional surface area without extra civil works or further financial investments, this can be achieved via installment of plastic boxes filled with coarse and medium size gravels and stones ($4m^3$). The specific surface area of such fixed bed material is between 80 and 100 m^2/m^3. This will make about 6-8 boxes, which can be distributed within the first two facultative ponds. The total hydraulic retention time (HRT) in these ponds will be about 3 days. See Photo B.1 shows the current status of waste stabilization Ponds "Appendix B".

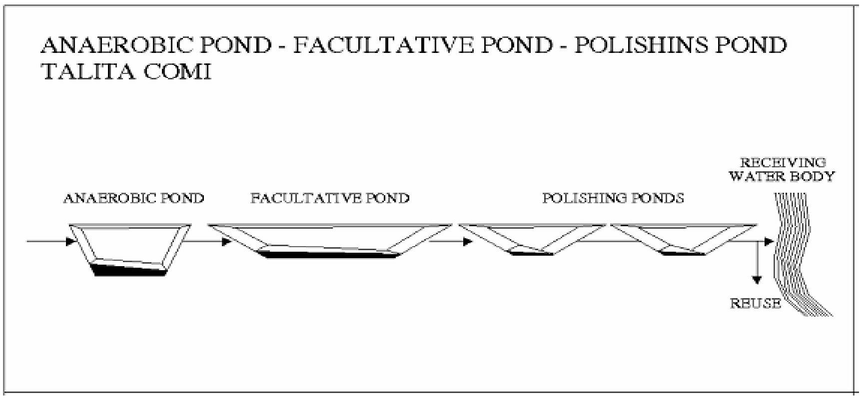


Figure 2.7 Waste stabilization Ponds located in Talitha Kumi School

WSPs technology is not widely spreading in the West Bank rural areas. The large area needed for WSPs is mainly considered the main factor limiting the use of this system in the Palestinian rural areas.

2.5.1.3 Up-flow Anaerobic Sludge Blanket (UASB) - Horizontal Flow Constructed Wetlands

In the year 2002, PHG designed and implemented a project of wastewater collection and treatment Plant in Kharas and Nuba villages in Hebron Governorate. The design capacity of treatment plant was $120m^3/d$ that is equivalent to 200-300 houses. The fenced treatment plant site is 2000 m^2 area of which the treatment plant itself occupies an area of 1063 m^2.

The UASB is a tank of 5 m depth and has a square surface area (4m*4m) which uses an anaerobic process. The sewage inters the tank bottom through 4 vertical 4" PVC pipes equipped with flow splitter. The water leaving the tank is draining through the V-notch channel at the water level meeting point. The actual flow rate during the beginning operation varies from 25 to $50m^3/d$. The reactor is equipped with Gas-Liquid-Solid (GLS) separator with a deflector. A gas collection system, which allows collection and treatment of all the gas produced from the reactor, is available. The wetlands (WL), which are selected, are subsurface flow wetlands and are planted with reed plants. This stage contains lagoons lined in base and sides with high density polyethylene (HDPE) that prevents any expected underground leakage. The wetlands include different sizes of gravel; the smallest are placed on the surface while the largest at the bottom, with reed plants planted at the surface. These plants make aeration in the upper half-meter of the water column through developing some 60 cm root zone. This enables the treatment to be aerobic. The basic biochemical reaction is the nitrification. Once the ammonia-rich UASB effluent water enters the wetlands, nitrification-denitrification is expected to take place in the wetlands. The subsurface flow pattern suppresses the possibility of insects breeding at the water-air interface. The hydraulic retention time was at the first flow conditions of two to three years of operation about 14 d while it predicts to be 7 d under maximum design flow conditions. The surface area for the wetland is approximately 1000 m^2 while the water column depth is about 1 m. There is also a separate sludge drying bed which contains gravel with size decrease from bottom to top. It drains the water from its bottom and it can daily treat 2 m^3 of watered sludge. The drained water that results from this sludge dewatering process receives treatment in the wetland through conveyance pipes that carry it to the wetlands. The scraped sludge is disposed of in the area's landfill.

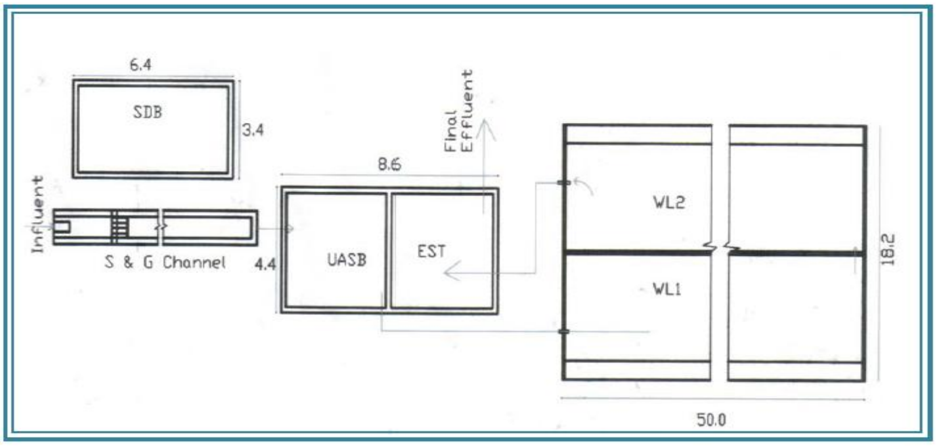


Figure 2.8 Schematic Diagrams of the Kharas and Nuba WWTP. S: Screen, G: Grit, UASB: Up-flow Anaerobic Sludge Blanket, WL: Wetland, EST: Effluent Storage Tank, and SDB: Sludge Drying Bed.

The performance of Community Nuba and Kharas Onsite WWTPs, at the first three years of operation the system was successful in treating sewage to more than 90% in terms of COD. The COD varies from 1501.7 mg/l as raw sewage to 609.31mg/l as UASB effluent to 109 mg/l as final effluent by wetland and after the wastewater enters the storage tank the value of COD is 98.16 mg/l. They observed that about 893 mg/l have been removed in UASB reactor, while 500.3 mg/l have been removed by wetland, and small removal from storage tank was 10.84 mg/l.

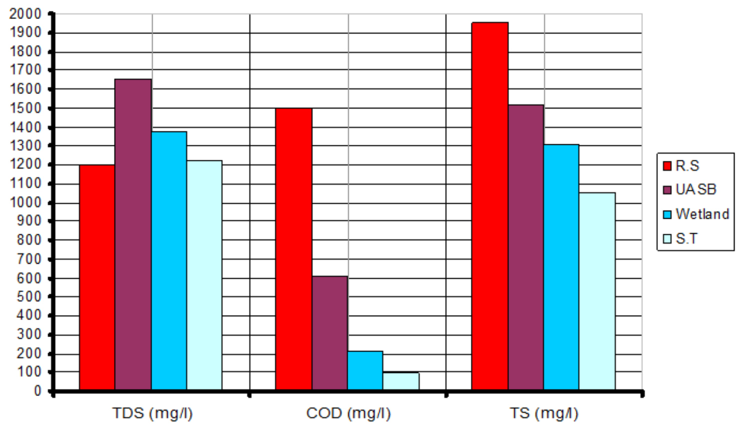


Figure 2.9 Removal Effecincy of Kharas WWTP. RS: raw sewage, UASB: effluent of UASB, Wetland: effluent of wetland, ST: effluent storage tank

In recent times, Community Kharas Onsite WWTP has been stopped due to lack of the constructed wetlands maintenance related to the utilized gravel media. In addition it was found that there is a flooding of wastewater from wetlands causing a blocking in wastewater flow paths which prevented the microorganisms to living in the filtering media and on the roots of the reed plants which reduces the breaking down and utilizing the organic material in wastewater effectively. Currently, the wastewater is collected and bypass the Kharas plant where is discharged into an open area called Thera' Al-Masalha Wadi at a distance of 900 m from the plant.

On the other hand, the Community Nuba Onsite WWTP is overloaded. The actual daily of wastewater flow to Nuba plant was estimated at 200 m^3. The wastewater consists from domestic, commercial and industrial sources. The industrial wastewater sources were originating from a mineral water bottling factory and a plastic factory. Moreover, the sewage that reaches the constructed wetlands infiltrates into the surrounding layers and does not reach the effluent storage tank. The explanation of that is due to the design failure of constructed wetlands was observed at the beginning of operation of the plant.

2.5.2 Collective onsite wastewater treatment systems

A collective (cluster) onsite system is a wastewater collection and treatment system that serves two or more dwellings, but less than an entire community.

2.5.2.1 Algae-Based and Duckweed-Based Waste Stabilization Ponds

A pilot scale of Algae-Based and Duckweed-Based Waste Stabilization Ponds were carried out at the campus of Birzeit University. The pilot plant was built with reinforced concrete walls to ensure water tightness. It consists of a holding tank (2.2 m length, 1.3 m width and 1.9 m depth) followed by two parallel systems: algae-based ponds (ABP) and duckweed-based (Lemna gibba) ponds (DBP). Each system consisted of a sequence of 4 equal ponds (3 m length, 1m width and 0.9 m depth) in series (Figure 2.10). Baffles at the outlet of each pond were constructed to avoid short-circuiting and transfer of floating materials to the consecutive ponds. The pilot plant was operated under two different conditions: From December 1998 till the middle of July 2000 wastewater from Birzeit University was used. From the middle of July 2000 - February 2001 wastewater from Al-Bireh city was used. Approximately 0.9 m^3 (0.38m^3/d to each system) of sewage was pumped daily to the ABP and DBP. Duckweed-based ponds were started with Lemna gibba species at a density of 600 g fresh weight/m^2. The final effluent of each system flows into a collection box and is channeled to adjacent BZU activated sludge plant (section 2.5.1.1). A regular monitoring schedule was started 5 months after the pilot plant start-up (Zimmo et al., 2002).

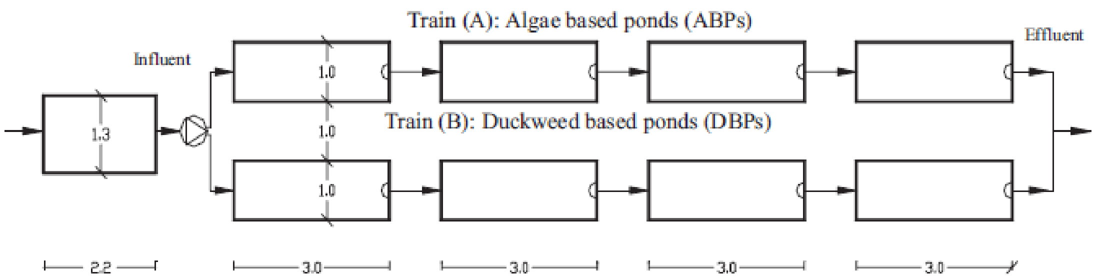


Figure 2.10 Schematic presentation of the treatment pond systems consisting of 4 algae and 4 duckweed-based stabilisation ponds (HRT=7d each), preceded by a holding pond (HRT=1d).

Physical and chemical parameters and the removal of organic matter, nutrients and faecal coliforms (FC) were monitored within each treatment system over a period of 12 months. The results show clear differences in the environmental conditions. In ABPs, significantly (P>0.05) higher pH and DO values were observed than in DBPs. DBPs were more efficient in removal of organic matter (BOD and TSS) than ABPs. The Fecal Coliform reduction was higher in ABPs. However, the quality of the effluent from the third and fourth duckweed pond (total retention time of 21 and 28 days) did not exceed the WHO criteria for unrestricted irrigation during both the summer and winter period, respectively (Zimmo et al., 2002). During the summer period, the average total nitrogen was reduced more in ABPs (80%) than in DBPs (55%). Lower values were measured during the winter period. Seasonal nitrogen reductions of the two systems were significantly different (P>0.05). In DBPs, 33% and 15% of the total nitrogen was recovered into biomass and removed from the system via duckweed harvesting during the summer and winter period, respectively (Zimmo et al, 2002).

2.5.2.2 Septic Tank and Trickling Filter

The Palestinian Hydrologic Group (PHG) had implemented a small-scale project for wastewater collection, treatment and reuse in Abassan region in Gaza Strip (PHG, 1999). This project aimed at reusing the treated wastewater as alternative water resource for irrigation purposes and to minimize the cesspit uses in the project area. The project was implemented in the year 2001 to treat 12 m^3/day of black wastewater. In the first stage, thirteen houses were connected by sewerage network and the collected black wastewater had been treated using septic tank and trickling filter.

The tank dimensions are consider as the total length of the compartment is about 10 m, length of the first compartment is about 6.4 m, length of the second compartment is about 3.6 m, the tank width is about 3 m and the tank height is about 2.5 m.

The septic tank with 50 m^3 volume primarily acts as settling pond. The hydraulic retention time of 2 days was suggested because the area is considered as hot area. Trickling filter has a volume capacity of about 4.6 m^3. Rock media ranges from 2-6 cm are used. The media is placed in pre-cast concrete ring where the height of the ring is 1.5 m. An adequate air flow was taken into account in the design of the trickling filter. The treated wastewater was used by subsurface drainage for irrigation purposes. The system consists of a series of narrow and relatively shallow (0.6-1.0 m) trenches filled with a porous medium (gravel). In the middle of the gravel, plastic tube was placed. The tube has three holes of one centimeter of diameter in every meter length of the tubes. The gravel is covered by small layer of sand. The total cost of the wastewater treatment plant was 18,836 $. The monitoring and evaluation was carried upon a routine program. The quality of the treated wastewater was examined four times a year. Wastewater samples were analyzed in each treatment BOD and COD samples were taken from septic tank and the trickling filter. Figure 2.11 & Figure 2.12 illustrate the analysis results of BOD and COD.

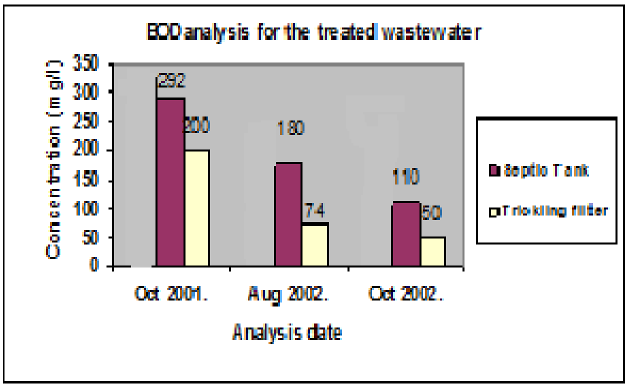


Figure 2.11 BOD concentration for the treated wastewater in different treatment steps

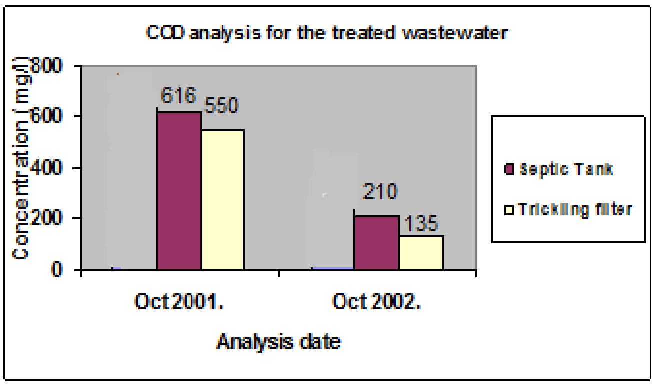


Figure 2.12 COD concentration for the treated wastewater in different treatment steps

2.5.2.3 Anaerobic pond- Up-flow Anaerobic Filter- Sand filter – Polishing Pond System

This treatment system is not widely used in the West Bank rural areas. A sample below is one of these plants conducted in Biet-Diko village in the West Bank. The plant started the operation under anaerobic conditions and it was connected to around 20 houses with about 180 inhabitants. The site was located south of Biet-Diko village with an area of 150 m^2. This site was sufficient for construction of the treatment plant facilities of up to a capacity of 15m^3/d. The treatment plant is designed to serve about 300 persons with gray wastewater production of 50 l/c/d. The topography of the site has natural slope, it was adapted for the treatment units.

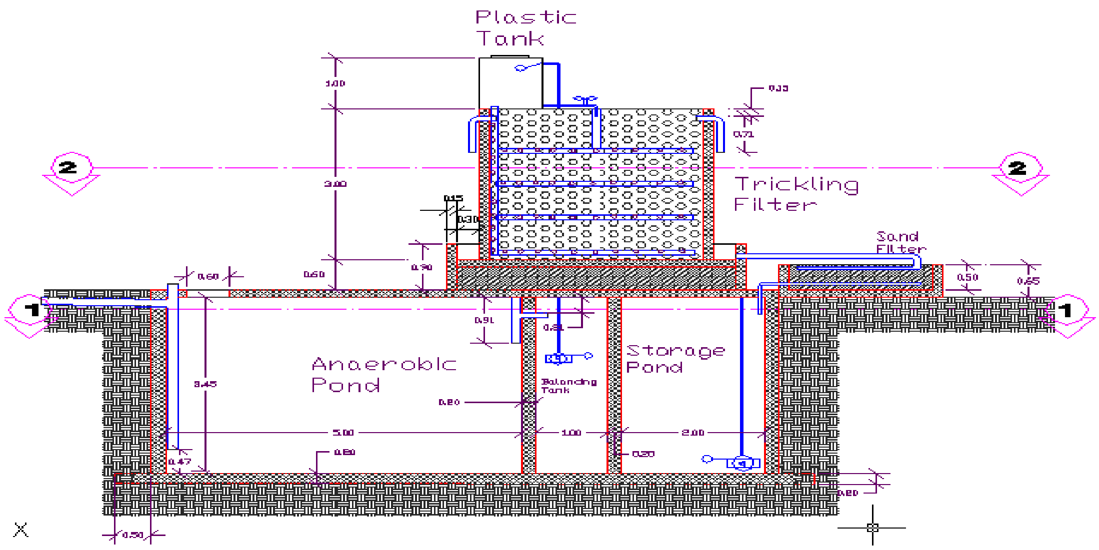


Figure 2.13 Sketch of Biet-Diko collective gray wastewater treatment plant (Mustafa, 2000).

The Wastewater flows by gravity into the treatment plant through bar screens that manually cleaned. Then the Flow passing through the manual screen to the anaerobic pond, where the solids settle down, the grease and foam float on the surface. Total retention time in this pond was designed to be at least 2 days, and the settled solids to be removed every two years (it was noticed that the level of accumulated solids was low). The water from this pond flows to a balancing pond where a submerged pump was installed. Pretreated wastewater was pumped from the balancing pond to a tank where the wastewater was controlled over the bottom of the filter bed media to act as up-flow anaerobic filter. The water from the gravel filter drops from the top of the filter touching the ambient air, going through a collecting basin to the sand filter which act as an intermittent fine sand filter, it receives water from the gravel filter basin removing the suspended solid like sloughed bacteria. The sand filter surface area is two square meters with 0.6 meter depth; it was divided into four compartments. The water flows into a polishing pond of three days storage capacity and a depth of 1.5 meter; the pond surface was subjected to ambient air. The purpose of this pond was to eliminate pathogens by sun rays and to act as three days storage tank for recirculation and irrigation. A Recirculation submerged pump was installed on the polishing pond, its purpose was to keep a certain level of water in the balancing pond in order to provide a minimum organic load for the bacteria in the gravel filter when the sewer system goes dry at night. Another pump for irrigation was installed on this polishing pond. Electrical floats were used to control the pumps. The treatment plants were operated and monitored for almost two years (2000-2002). More than 90 samples were taken from the effluent of the treatment plants. Average data records of testing results from four household wastewater treatment plants, in Biet-Diko village are shown in table 2.3 and figure 2.13 show the viewable treatment efficiency for the plant.

Table 2.3 Average results of wastewater samples / Biet diko (Mustafa, 2000).

Parameter	Unit	Drinking water	Settling Pond	Balancing Pond	Gravel Filter	Sand Filter	Polishing Pond
pH		7.37	6.65	7.06	7.43	7.25	7.61
Conductivity (EC)	µs / cm	1118	1041	1378	1210	1200	1190
TDS	mg/l	543.3	531.2	703	601	620	620
COD	mg/l		847	302	329	95	97
BOD	mg/l		383	138	149	26	32
TS	mg/l		1046	686.7	853.3	686.7	866.4
Chloride (Cl^-)	mg/l	173	102	155	155	161	152
Bicarbonate	mg/l as $CaCO_3^-$	230	308	380	304	292	297
Nitrate (NO_3^-)	mg/l as NO_3^-	1.7	50.45	14	11.39	9.6	10.76
Sulfate (SO_4^{2-})	mg/l as SO_4^{2-}	11	28	27	21	18	21
Phosphate (PO_4^{3-})	mg/l as PO_4	0.2	7.1	16.8	6	8.6	4.4
Calcium (Ca^{2+})	mg/l	69	52.4	60.2	31.3	35.2	42.5
Magnesium (Mg)	mg/l	32	4	14	3	11	8
Sodium (Na^+)	mg/l	90	146.8	187.5	112.4	133.3	153.3
Potassium (K)	mg/l	3.6	12.76	24.11	18.3	19.96	25.31
Total Coliforms	CFU/100 ml	**	TMTC	1250	3050	1300	2500
Feacal Coliforms	CFU/100 ml	**	**	**	**	**	**

** Not detectable.

2.5.2.4 Al-Aroub College Duckweed Based Pond System

Duckweed Based Pond System is used in Al-Aroub College in Hebron Area. Environment Quality Authority started the plant construction in year 1997. A proper infrastructure was constructed (sewer line, manholes and three small ponds, two of them with a volume of 2000 liters and the third with a volume of 3750 liters. Plastic sheets were installed at the bottom of the ponds to prevent seepage. Duckweed (Lemna gibba) was found at kheirbat Addair in a rocky pool located in Hebron desert at the end of the dry season. Wastewater was collected in the first pond (SettlingTank) for about 3 hours from which it was discharged to the second and third ponds. The system was operated as a semi-continuous system by removing and adding different amount of effluent and influent to keep the retention times around 3,6,10 days. The same was done for the third pond but with amount of effluents and influents matched with its volume. Duckweed-based pond system treats 8 m^3/d of wastewater from the agricultural school that consists of Al–Aroub College and the adjacent stable of the cows. The effluent water is used for producing seedling in a forest-tree nursery constructed for reuse in irrigation or groundwater recharge.

The laboratory analysis was conducted to evaluate the performance of DWBP (Lemna gibba) as a tool to reduce the amount Phosphorous (P) pollutants in the wastewater and to reuse the effluent in irrigation. Under the meteorological conditions of the central highlands of Palestine, Lemna gibba grows successfully in wastewater. The removal efficiency of duckweed ponds for BOD and COD was found in the range of 85-90% and for NH_4^+-N 78%. Under adequate operational conditions, duckweed systems can match the quality characteristics of secondary effluents reused to grow a range of crops. In addition, another study was conducted by W. M. Awadallah (2005) examined the performance of duckweeds at full-scale for long term period. Awadallah monitored the seasonal variations of parameters such as N-content, Production and effluent NH_4^+-N and Relative growth rate (RGR) and their effects on the system performance. It was found that there was a strong correlation between plant production and NH_4^+-N removal. Also, Awadallah indicated that the major mechanism for N removal, in addition to plant uptake, in such lagoons is the combined effect of nitrification-denitrification rather than ammonia volatization. Table 2.4 shows the treatment efficiency of the wastewater treatment plant units. Nowadays, the plant is managed by Al-Aroub College and they are facing financial obstacles to continue for successful treatment process.

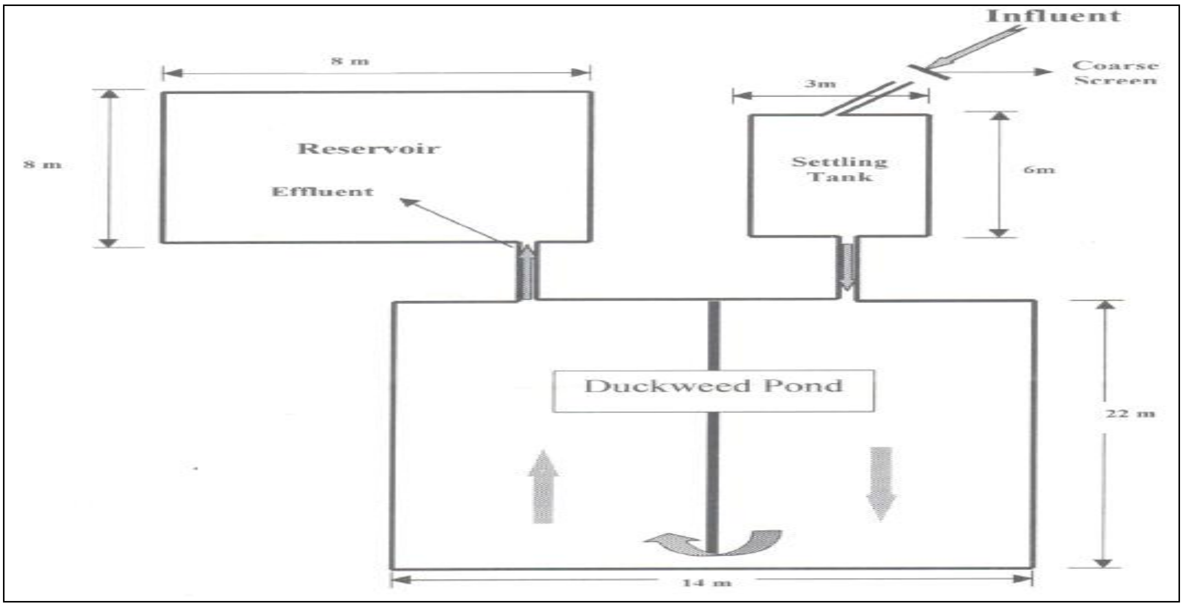


Figure 2.14 Pilot scale Collective WWTP using duckweed at Al-Aroub Collage (EQA, 1999)

Table 2.4Treatment efficiency of duckweed wastewater treatment plant units (Awadallah, 2005)

Parameter	**Fresh wastewater**	**Septic tanks effluent**	**Effluent DW1**	**Effluent DW2**
pH	7.16 ± 0.23	7.57 ± 0.37	7.65 ± 0.05	8.11 ± 0.5
HRT(d)			11	10
DO(mg/l)	0.62 ± 0.20	1.53 ± 1.22	2.07 ± 0.7	4.49 ± 3.02
EC(µs/cm)	1934 ± 280	2282 ± 359	1776 ± 145	1664 ± 351
TDS (mg/l)	nm*	nm*	nm*	803.6
COD(mg/l)	245.9 ± 66.1	148.3 ± 56.3	67.8 ± 39.6	67.1 ± 34.0
BOD(mg/l)	113.5 ± 27.2	77.9 ± 19.5	15.1 ± 6.4	13.0 ± 6.9
NH_4^+-N(mg/l)	51.9 ± 4.0	63.9 ± 29.6	31.3 ± 14.2	20.6 ± 10.5
TKN(mg/l)	nm*	66.5 ± 15.9	nm*	nm*
NO_3^--N(mg/l)	0.0	0.0	1.8 ± 0.2	2.15 ± 0.09
NO_2^--N(mg/l)	nm*	0.0	0.25 ± 0.01	0.3 ± 0.04
Transparency	nm*	8.00 ± 1.5	27.3 ± 2.8	24.0 ± 6.9

nm* : Not measured

2.5.2.5 Septic tank (ST) – up-flow anaerobic biofilter (BF) hybrid system

As a part of Deir Samet sanitation project, a treatment plant was constructed inside the campus of school in Deir Samet in Hebron district in year 2001 under the supervision of PHG foundation. The effluent was designed to be used in the irrigation of olive trees in fields found close to the treatment plant. The treatment plant capacity is 20m^3/d including the wastewater discharged from 40 houses and a school. The treatment plant covers an area of 125m^2 and located at a distance of 200-300 m away from the built up area and at a distance of 20-200 m close to the olive trees fields.

The system consists of two in series septic tanks, four-upflow anaerobic biofilters and a collecting tank. The excess sludge is dried on gravel beds so as to be used later as soil fertilizers. The total volume of the septic tank is 60 m^3 with a depth of 3.5 m and a 4 days retention time. Each one of the used filters has a volume of 15 m^3 with a depth of 3m and contains 3 layers of crushed wadi stones with a thickness of 1m, 1m, 0.5m respectively from down to up with a 3 days retention time. The filters use physical mechanisms, including flocculation, sedimentation and absorption for the removal of organic matter. The collecting volume has a total volume of 50 m^3 and the treated wastewater is taken from the surface. COD is reduced from 1200mg/l to 420 mg/l when it treated through the septic tank. It reaches a concentration of 84 mg/l after passing the 4 filter compartments which means an 80% reduction in COD concentration.

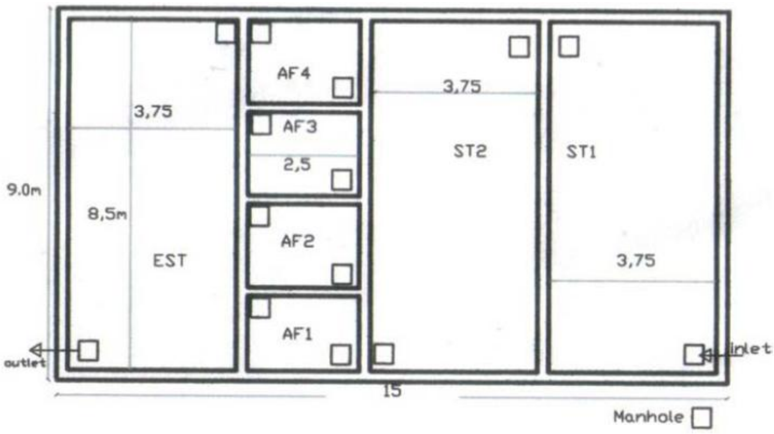


Figure 2.15 Schematic Diagram of Collective WWTP in Deir Samit. ST: septic tank, AF: anaerobic filter and EST: effluent storage tank.

The irrigation system used is even drip irrigation or submerged irrigation. The produced amounts of reclaimed wastewater are sold with a price of 0.45 $ \m^3 to the farmers. These amounts of money are collected by the village council and are used in the costs of operation and maintenance. In the times where the effluent is not used in the irrigation process, the effluent is allowed to pass through a 6" perforated pipes under a depth of 1m and for a length of 20m of crushed stones and so that it infiltrates finally to the ground water. Samples for testing are to be taken by the PHG employees monthly at first, then seasonally, and then yearly so related to PHG results the system was working efficiently. On the other hand, the treatment plant is considers as a primary treatment system which provides only partial treatment of wastewater due to low reduction of non-organic matter (nutrients) or pathogens. Therefore, the effluent must be further treated before it released into the surrounding environment or even used for irrigation.

2.5.1.6 Upflow Anaerobic Sludge Blanket (UASB) septic tank - Horizontal Flow Constructed Wetlands (WL) system

Upflow Anaerobic Sludge Blanket (UASB) septic tank followed by Wetland (WL) wastewater treatment plant is located in the town of Western Bani Zaid (Beit Reema and Deir Ghassaneh) 27 km North –West from Ramallah city. This system was implemented by PHG since 2005 and designed to treat the wastewater from 100 houses.

A septic tank of three basins was constructed. It was well plastered to avoid any water leakage. A screen is placed before the UASB septic tank for solids sedimentation. The septic tank is about 300 m^3 (3.5*12.25*7.15) m^3, where the basins are: for first basin (3.5*9*7.15) m^3, second basin (4.2*3*3.5) m^3 and third basin (2.2*3*3.5) m^3. Reinforced concrete cover, through which steel gates were installed as required, closed the entire tank surface. The first chamber was converted into UASB. The work included shaping, painting (by coal tar epoxy) and fixing the inclined submerged steel plates with effluent gutters, weirs, deflectors and effluent collection V-notch channels along this compartment.

Openings in the roof were created and covered by movable steel gates in order to make access for maintenance and daily operation works. The influent was split into 7 portions by a distribution box connected to 7 HDPE pipes which end at the reactor base at equal division distances. The reactor was equipped with a sludge conveyance 6" pipe with gate valve to use gravitational flow, which is allowed by the topography of the site, to dispose excess sludge to the sludge drying bed at the downstream of the WWTP. A gas collection system was installed over the top of the UASB and included 4 bar safety valve and gas storage facility. Gas tightness was ensured in all of the process. The design was based on influent COD of 1500 mg/l and 300 mg/l as effluent with not less than 80% removal. The second chamber and the third chamber were converted into sedimentation tanks with openings in the roofs covered by steel gates. These two chambers could easily be converted, one or both, into another UASB tank(s) in the future (i.e. after 15 years) as required. All of the internal concrete and steel work was painted by the coal tar epoxy. Internal piping connecting the three compartments were modified and higher water level was allowed to take place. At the head of the sewage works a sand and grit removal channel was installed and covered by painted steel gates. A bitumen layer also painted all of the external walls and the roof. Wetland is consists of four basins of plastic, sand and gravel which were planned to be planted with roots of reeds. The basins are about 1800-squared meter with an area of $450m^2$ for each. Storage Tank is a concrete tank with the capacity of 70 m^3 (4*5*3.5) m^3 and it collects water going out of the wetland before using it for agriculture.

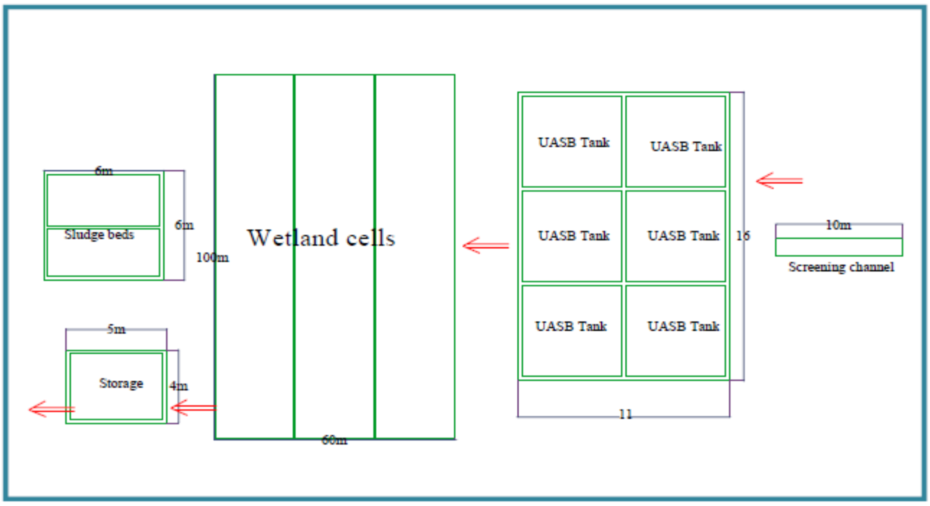


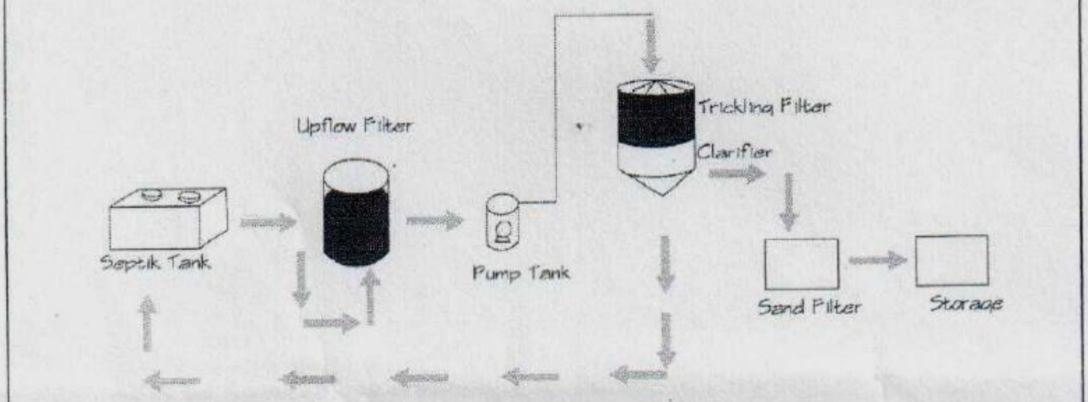
Figure 2.16 Schematic Diagrams of the Community Bani Zaid Onsite WWTP

The actual daily flow to Bani Zaid WWTP is estimated at 20 m^3 of wastewater originating from domestic and commercial sources, serving about 31 housing units and a school. The effluent from the plant flows freely in Wadi Kufr ‘Eim. According to PHG the plant is operating with low efficiency. However there is no precise data that reveals the performance of the plant as there is no regular monitoring of the treated effluent quality. Some of the results data was provided by PHG show that the influent and effluent of UASB COD was 4,580.8 mg/l and 1,030 mg/l, respectively. This demonstrates, the COD removal efficiency at the first treatment stage (i.e. after the UASB) was 77.5%. Moreover, it should be mentioned that the treatment plant is not working in full capacity. In addition, the municipality is currently not properly operating and maintaining the plant due to technical problems related to the fact that the diameters of the main pipelines to the plant are small which causes clogging in addition to that lack of financial resources.

2.5.2.7 Septic Tank - Anaerobic Upflow Gravel Filter - Aerobic Trickling Filter followed by Polishing Sand Filter

The Palestinian Agricultural Relief Committees (PARC) designed and implemented many of collective wastewater treatment systems that utilize combined anaerobic-aerobic processes for treating domestic wastewater. This system is found in ‘Attil and Zeita localities in Tulkarem Governorate and Sir Locality in Qalqiliya Governorate. This hyprid system consists of a septic tank followed by an anaerobic upflow gravel filter; an aerobic trickling filter and a polishing sand filter. These collective treatment plants were designed to treat 14 m^3 of domestic wastewater per day.

The septic tank is used to pre-treat the wastewater prior to applying the effluent to the anaerobic upflow gravel filter that uses physical mechanisms and anaerobic digestion for the removal of organic matter. The anaerobic filter provides further BOD and TSS reduction, thus it is used prior to the trickling filter to improve nitrification process. The trickling filter is used to further remove organic matter from wastewater by utilizing microorganisms attached to a medium (i.e. an attached-growth process). Sludge from the trickling clarifier is returned to the septic tank. The effluent from the trickling filter is fed into a sand filter which is used as a final polishing stage. In this filter the sand traps residual suspended material and bacteria and provides a physical matrix for bacterial decomposition of nitrogenous material, including ammonia and nitrates, into nitrogen gas. Excess sludge is removed from the system once per year and is placed in an open area for 6 months until it stabilized. It is worth mentioning that the effluent from the trickling filter could be recirulated either to the septic tank or to the influent line of the anaerobic gravel filter in certain (PARC, 2010). Most of the treated effluents for these existing technologies are currently reused by local farmers for restricted irrigation of agricultural land cultivated with fruit trees using aboveground drip irrigation.



Reference: (Palestinian Agricultural Relief Committees, 2010)

Figure 2.17 Schematic flow Diagram of the collective WWTP in Attil Locality

The laboratory analysis was conducted by Water and Environmental Studies Institute (WESI) At An-Najah National University to evaluate the performance of these types of technologies as a tool to reduce the amount of pollutants in the wastewater and to reuse the effluent in irrigation. Table 2.5 shows the results of collected wastewater samples from the wastewater treatment units in ‘Attil, Zeita, and Seir during August 2008 – September 2009. The COD Removal efficiency was 78.72%, 80% and 80% in ‘Attil, Zeita, and Seir, respectively. Moreover, at Zeita Plant the sewage network was expanded to connect other 80 housing units to the plant resulting in exceeds of the design capacity to be 35 m^3 per day which in turn could effects on the efficiency of the treated effluent.

Table 2.5 The results of ‘Attil, Zeita, Biddya and Seir WWTPs.

	Ser No.	Location	Exact location of sampling	Date of sampling	F.Coliform cfu/100ml	BOD mg/l	COD mg/l	TSS mg/l	TDS mg/l	Nitrogen mg/l
August 2009	1	ATTIL	Influent	17/8/2009	70.0E6	1020	1734	620	1420	191
	2		Effluent	17/8/2009	1.3E6	194	369	80	1395	72
	3	ZEITA	Influent	17/8/2009	120.0E6	1200	1920	2155	1280	148
	4		Effluent	17/8/2009	10.0E6	286	384	68	1230	134
	5	BIDYA	Influent	19/8/2009	20.0E6	1215	1850	624	1920	234
	6		Effluent	19/8/2009	0.2E3	208	345	34	1856	76
	7	SEIR	Influent	19/8/2009	120.0E6	1085	1600	472	1152	144
	8		Effluent	19/8/2009	10.0E6	237	320	78	1216	120
September 2009	1	ATTIL	Influent	9/9/2009	70.0E6	550	1000	262	1203	182
	2		Effluent	9/9/2009	3.5E6	167	320	30	1185	172
	3	ZEITA	Influent	9/9/2009	50.0E6	1638	2400	6610	1075	258
	4		Effluent	9/9/2009	3.5E6	227	400	62	1062	148
	5	BIDYA	Influent	10/9/2009	90.0E6	1570	2400	948	2016	182
	6		Effluent	10/9/2009	8.0E6	373	800	178	1920	158
	7	SEIR	Influent	10/9/2009	150.0E6	1176	1600	196	1472	302
	8		Effluent	10/9/2009	40.0E6	240	400	40	1452	96

2.5.2.8 Septic Tank - Horizontal Flow Constructed wetlands

Septic tank followed by horizontal flow constructed wetlands contain a reed bed aerobic filtration system collective onsite wastewater treatment technology was designed by PARC and implemented in Biddya locality in Salfit Governorate in the year 2007. The capacity of this plant is 11.2 m^3 per day from 42 housing units. According to Biddya Municipality, 2010 the treatment system currently is receive the wastewater generated from 38 housing units, one clinic, one mosey and one building complex. Some of influent and effluent quality analysis results are provided in table 2.5 that reflect the performance of the system. However, the early average removal efficiency at the beginnings of operation of the system of BOD and TSS was 77% and 81% respectively. Actually, Bidyya treatment plant is currently malfunction with low efficiency as well as overloaded. It should be mentioned that there is no regular evaluation of the treated effluent. The treated effluent is currently discharged to an open area called Wadi Abu Helayem. According to PARC and as reported in the design report the excess sludge should be removed once per year from the treatment system and should be dried in a solar sludge drying bed but this has never done and the sludge is not removed from the system. In addition, the treated effluent should be reused for restricted irrigation of agricultural lands cultivated with fruit trees but this was not achieved because the required pump did not install by PARC.

Another system utilizes a sedimentation tank followed by tow horizontal flow constructed wetlands was designed and implemented by PHG in Hajja Locality in Qalqiliya Governorate since 2004. The domestic wastewater collected by the sewage network flows to the 120 m^3 sedimentation tank from which the primary wastewater is transferred to the two horizontal constructed wetlands; each has an area of 500 m^2, via a 1 km main pipeline. The treated effluent is then collected in a 30 m^3 storage tank. The general sludge is removed once a year from the sedimentation tank and disposed of in nearby lands (PHG, 2010). The treatment plant was designed to treat 40 m^3 per day. As PHG mentioned the Treatment system is operating well with moderate efficiency. The average removal efficiency of BOD and TSS was 80% and 85%, respectively. The village council is currently reusing the treated wastewater for restricted irrigation with almonds and olive trees, while there was no wastewater reuse scheme associated with the design report project that implemented by PHG in the year 2004. Moreover, the current status of the plant needs to rehabilitate.

2.5.2.9 Anaerobic Baffled Reactor – Activated Sludge process – Multimedia Granule Filtration – Ultraviolet Disinfection

A pilot collective Onsite Wastewater Treatment Plant located in 'Ein Siniya in Ramallah Governorate was designed and implemented in the year 2007 by Birzeit University (BZU) utilizing a rotary screen followed by an Anaerobic Baffled Reactor (ABR) and Activated Sludge (AS) Process as a secondary treatment followed by a multi-media granule filter and disinfection through ultraviolet (UV) unit as an advanced tertiary treatment. The pilot plant is designed to receive an average flow of $10m^3/d$.

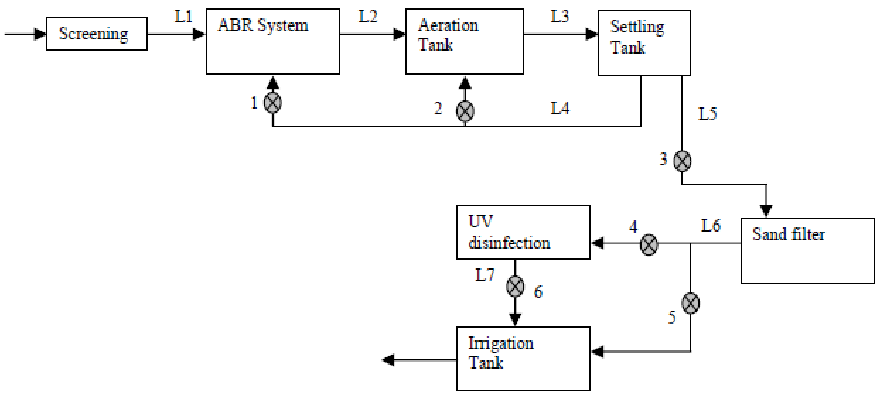


Figure2.18 Schematic diagram of the operation of collective onsite 'Ein Siniya WWTP

The plant receives wastewater from the existing sewage collection network of the Jalazun Camp, Jifna, Dura El Qare', and 'Ein Siniya through a main trunk pipeline. Wastewater diverted from these existing closed channels sewer system flows through a bar screen so that large solid particles are trapped and kept from flowing into inlet station pit. The inlet station pit collects the wastewater, and acts as a buffer zone in order to balance inlet flow during peak periods and interruptions. Two grinder submersible pumps (one duty/one stand by) are used to transfer wastewater from the inlet station pit. Wastewater from inlet pit first passes through a rotary screen for preliminary treatment. The influent flows through a cylindrical surface where solid particles are retained on the outside screen surface. The outlet flow from the rotary screen is then stored in the header tanks. Header tanks act as a buffer zone to balance outlet flow from the rotary screen. The anaerobic baffled reactor (ABR) is the first stage in secondary treatment of wastewater. Wastewater from header tank flows and distributed uniformly over the bottom of the first part of the ABR. The effluent then flows to the second part of the ABR via another distribution system. Piping systems are constructed and installed in the ABR to ensure uniform distribution of wastewater. Denitrification takes place in the third part (Anoxic Zone) of the ABR, by circulating a portion of the clarified effluent containing nitrates from the setting tank. Two dedicated pumps (one duty/one stand by) are used for circulation. The fourth and last part of the ABR is a sludge trap. The second stage of secondary treatment is an activated sludge process, where aeration tank is the main chamber where biological aerobic treatment takes place. Fine bubbles of air are diffused into liquor by means of two air blowers (one duty/one stand by.) Oxygen transferred to sewage water to provide the bacteria with suitable environment for reproduction. To save operational costs, air blowers operated for six hours / day; i.e. activated sludge system was operated as intermittent aeration. The settling tank, which is a part of the aeration tank, serves as clarifier and sludge circulation source. The aerated sewage flows to the inclined part of the settling tank, where the effluent faces a sudden drop in kinetic energy allowing enough time for the suspended particles to settle down to the bottom of the tank. Clear effluent continues and flows to storage tank. Part of the settled matter is circulated back to aeration tank for continuous feed of activated sludge to maintain the volatile microorganism's concentration. Excess sludge in settling tank is transferred to sludge holding tank for storage and truck disposal. The tertiary stage of treatment consists of filtration via a multi-media granule filter and disinfection through ultraviolet (UV) unit (UV unit is out of order). Clarified effluent from storage tank is pumped by two filter feed pumps (one duty/one stand by) through the filter. Filtered effluent is then directed to the UV system for disinfection. Then the filter was backwashed everyday by two backwash pumps (one duty/one stand by). Disinfected water is then transferred to irrigation tank, where treated water is stored. Water distribution for restricted irrigational purposes is achieved by two submersible irrigation pumps (one duty/one stand by) (M. Adas, 2010).

The results of influent and effluent of the WWTP in the early beginnings of operation in the period between April 2008 and January 2009 indicating that the WWTP was operating with a high COD, BOD, and TSS removal efficiencies of 90.4%, 90.3% and 99.5% respectively. However the treatment plant is currently stopped due to lack of financial resources for the operation and maintenance.

2.5.2.10 Extended Aeration Process – Chlorine Disinfection and Sand Filtration

The Applied Research Institute – Jerusalem (ARIJ) has implemented a collective wastewater treatment plant in Nahhalin Village in Bethlehem Governorate. The treatment plant is designed to treat 50 m^3 of domestic wastewater per day collected by a 7 m^3 vacuum truck collection system that emptied from cesspits. The plant is composed from extended aeration treatment process contains the following treatment steps: primary treatment (equalization); secondary treatment (aeration and separation); tertiary treatment (disinfection and filtration); and sludge collection system. The equalization/settling tank receives the incoming raw wastewater and acts as a buffer zone to absorb peak flow rates at peak times so as not to overload the other treatment process. The raw wastewater is then transferred to the aeration tank on patches by grinder, submersible type batch pumps. The pumped raw wastewater enters the first part of the aeration tank, passing through a manual basket screen to remove any large particles. A submersible mixer installed in the first part of the tank is used to create homogeneous liquor and prevent settlement. Mixed raw wastewater then flows to the second part of the tank (biological treatment). Aeration is introduced by means of fine bubble diffusers via two air blowers to provide the bacteria with suitable living environment for reproduction, and to minimize odor. The wastewater then flows to the separation tank, where the effluent faces a sudden drop in kinetic energy allowing enough time for the suspended particles to settle to the bottom while clear effluent continues to the chlorination tank by gravity. Settled matter is recycled back to the aeration tank for continuous feed of activated sludge by means of air lift pumps. Excess sludge is then transferred to the sludge holding tank for storage and disposal. The chlorination tank is designed and sized to allow enough contact time between added chlorine and treated effluent for disinfection purposes. Chemical dosing pump is used to precisely inject calculated amounts of chlorine in the form of liquid Sodium Hypochlorite. From the chlorination tank, the disinfected water is pumped by two centrifugal pumps through a sand filter. The filters effluent is stored in a storage tank/treated wastewater tank where it can be reused for restricted irrigation (ARIJ, 2007). Moreover, this system is currently not reused the treated effluent and it flows freely into the closed mountain area from the plant. In addition, this plant is consider as an economic burden on the Nahhalin village council, where they believed that the plant does not work technically, and the operating and maintenance expenditure are higher than the expected efficiency of the treated effluent that released to the environment.

2.5.2.11 Septic Tank - Subsurface treatment system

ANERA foundation has implemented many of onsite subsurface techniques of wastewater treatment in few schools in Hebron villages. These systems were simply made of a collecting manhole connected to a three-compartment septic tank. The effluent was allowed to pass out of the septic tank through a perforated PVC 4" pipe under a 20cm gravel layer. The collecting manhole had a volume of $0.25m^3$. The septic tank had a total volume of 17 m^3 and was made of reinforced concrete. However, there was no enough information about these treatment plants. The plants were not followed up by ANERA after the finish of the project and they were left neglected. No samples were taken to check the plants efficiency and to control the plants operation. Moreover, no design report is found (ANERA, 2005).

2.5.3 Individual (household) onsite wastewater treatment systems

Onsite system is conceder as a mechanical device used to collect, treat, and discharge or reclaim wastewater from an individual dwelling. A conventional household onsite system includes a septic tank. Other types of alternative household onsite systems include media filters, small aerobic units, or pressure distribution systems.

2.5.3.1 Septic Tank - Upflow Gravel Filter System

The upflow gravel filter is designed as gravity loaded system; maximum flow at day hours and Zero flow at night hours. The main treatment part is anaerobic process followed by aerobic multi-layer filter (sand, coal, gravel).The unit requires that the household plumbing separates Gray wastewater (GW) from toilet wastewater. Toilet wastewater is discharged to the existing or modified cesspit, while Gray wastewater is directed to the treatment plant.

The gravel filter media were mainly hard crushed stones or washed wadi gravel of hard limestone of 0.7 to 3 cm in size. The pilot plants are made of concrete or/bricks. Each unit divided into four compartments, where the first compartment is used as septic tank and grease trap and receives the gray wastewater – from the shower, kitchen, sinks and washing machine – through a 5 or 7.5 cm diameter PVC pipe, via a screened manhole, by means of a T-shaped outlet. One end of this outlet is directed upward and open to atmospheric pressure and the other is at a level of about 30 cm from the bottom of the tank. The second and the third are used as upflow graduated gravel filter, the fourth compartment is act as a balancing tank for treated gray wastewater where a submersible pump is installed. The pump lifts the water to a multi-layer aerobic filter, the water pass through the layers (sand, coal, gravel) to a storage tank from where it used for irrigation. Part of the balancing tank is used as an aerobic filter in some plants. Any accumulated grease is prevented from continuing through the system by the T-shape pipe, with gray wastewater taken from a depth far enough below the surface (to avoid taking in accumulated grease) and above the base of the tank (to avoid settled solids from being taken in) (J. Burnat el al, 2004).

Upflow Gravel filter design is based on the following parameters:
- Void space is 40% in the first compartment of the gravel filter.
- Void space is 50% in the second compartment of the gravel filter.
- Organic loading of 0.388 kg BOD/day for 10 persons household.
- The hydraulic retention time is 2 days for the septic tank and 1.8 days for the up-flow gravel filter.

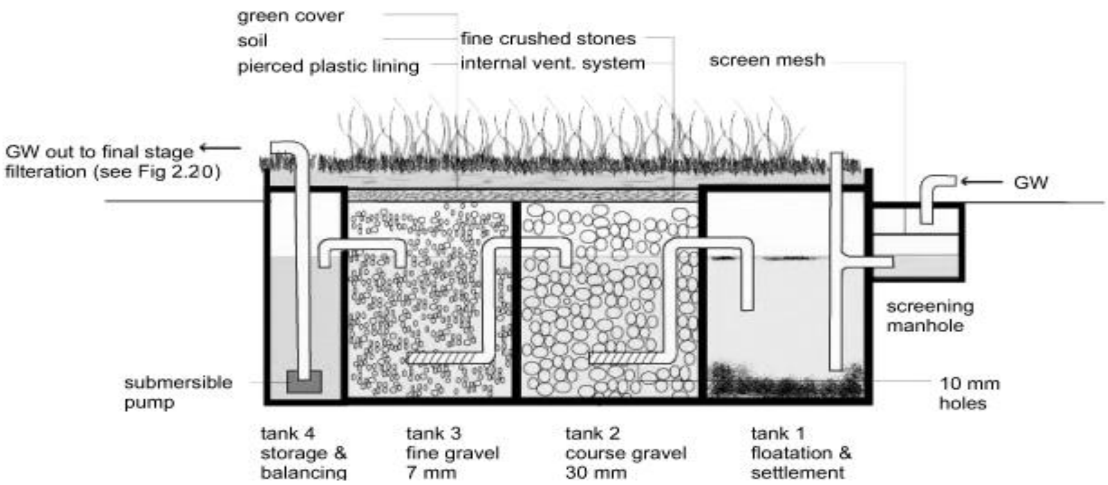


Figure 2.19 Septic tank up-flow gravel filter treatment unit

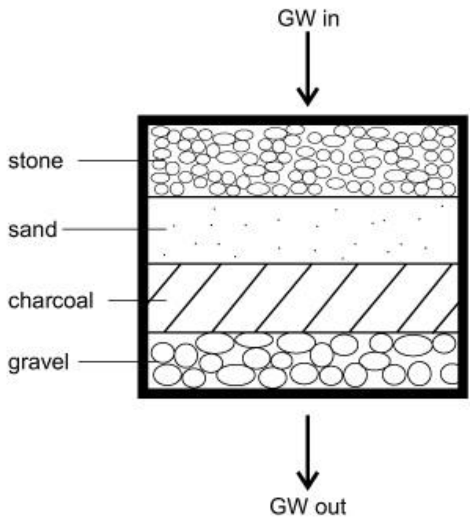


Figure 2.20 Final stage gray wastewater filtration

The system was operated and monitored for almost two years (2000-2002) in Bilien village in the West Bank. Four Septic Tank - Upflow Gravel Filter (ST - UFGF) plants were installed at four different houses. More than 90 samples were taken from the effluent of the treatment plants. Not much accumulated settled solids were noticed after one year of operation. That might be because garbage is not allowed to enter the sewer pipe, accumulation of grease occurred at the top of the surface of the first tank, and the process is totally anaerobic. Average data records of testing results from four household wastewater treatment plants, in Bilien village are shown in Table 2.6

Table 2.6 The performance results of four on-site Septic Tank-Upflow Gravel Filter household gray wastewater treatment plants/ Bilien village/ Palestine (Mustafa, 2000)

Sample Information		**TP1 (10 persons)**	**TP2 (6 persons)**	**TP3 (7 persons)**	**TP4 (14 persons)**
Sample Location		Pump Compartment	Pump Compartment	Pump Compartment	Pump Compartment
Parameter	Unit				
PH	***	7.22	7.23	7.28	6.99
Conductivity (EC)	µs / cm	2710	2220	1850	1980
TDS	mg/l	1472	1238	1053	1073
COD	mg/l	145	80	85	284
BOD	mg/l	65	27	28	129.6

Settable Solids	ml/l	0.1	0	1.6	0.1
TS	mg/l	1546.6	1300	1133.3	1233
TSS	mg/l	70	54	78	97
Chloride (Cl^-)	mg/l	330	295	268	286
Bicarbonate as CaCO3-	mg/l	818	661	500	474
Nitrate (NO_3^-)	mg/l	23	16	10	22
Sulfate (SO_4^{2-})	mg/l	23	4	11	22
Phosphate (PO4)	mg/l	47	13	27.4	47.9
Calcium (Ca)	mg/l	86	112	72	70.6
Magnesium (mg)	mg/l	34	34	38	31
Sodium (Na)	mg/l	248.3	192.2	174.5	191
Potassium (K)	mg/l	23.8	18.5	9.81	8.4
Total Coliforms	CFU/100 ml	447	TMTC	364	TMTC
Feacal Coliforms	CFU/100 ml	0	0	0	0

TMTC: Too Many To Count.

A modified design approved by the PWA for the previous gray wastewater treatment plant that designed by PARC is adopted. This has been decided after studying two pilot plants, one designed by the Action Against Hunger (ACH) and the other was designed by PARC. Three types of different capacities were used. The first type is for family member of 8 and below. The second was for 8-16 consumers and the third was for more than 16 up to 24 consumers (EQA, 2006).

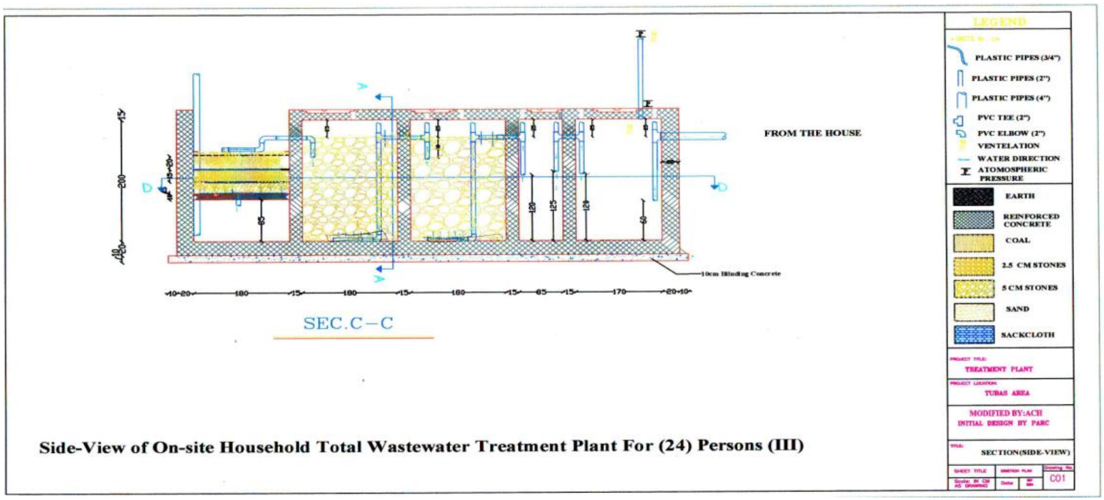


Figure 2.21 Side-View of on-site household wastewater treatment plant for 24 persons (PARC, 2005)

The septic tanks of the Household Wastewater Treatment Unit (HWTU) were designed for a retention time of 5 days and the filters were designed for a retention time of 2 days. These have been estimated based on a water consumption of 40 liters per capita per day (l/c/d) and 80% wastewater conversion factor. These get the sizes of the septic tank for the 8 family member type HWTU at 1.28 m^3 and that of the rock filter with 30% - 40% void ratio at 1.28 - 1.72 m^3. The constructed size of the first compartment septic tank is 100x100x175 cm with 25 cm clearness. This totals 1.5 m^3 with about 20% more than the above. The size of the second compartment is half of the first providing additional size for septic treatment.

2.5.3.2 Individual Household Activated Sludge system

Small scale of Individual Household Activated sludge treatment system is widely used in the Southern part of West Bank rural areas was applying by ARIJ. The capital cost of the treatment plant package is about 2000$ for 5 PE to 6,200$ for 30 PE. This type of technology includes a cylindrical bioreactor having as its main body a plastic tank with a total volume of 2500 liter, partitioned into four hydraulically independent zones. These are the mechanical debris collection basket, the de-nitrification zone, the activation / nitrification zone and the separation zone. The Treatment of wastewater in this system is based on biological process using single heterogeneous activated sludge kept in suspension. A plastic cylindrical water tank was used to form the main body of the wastewater treatment plant. This tank was attached to a metal frame that was painted with a corrosion protective coating and consisted of three rings linked together by three parallel profiles that were perpendicular to the three rings and fixed with a displacement between each other of 120 degrees from the center of the rings. Later on, Polyethylene rigid high density polyethylene-HDPE sheets were added to form the divisions between the different zones of the wastewater treatment reactor. The effective volume for the treatment process was approximately 85% of the total tank volume. The zones volume is summarized in Table 2.7.

Table 2.7 Zones volume of the treatment system

Zone	Liter	Percent (%)
Screen	85	3.4
Activation/nitrification zone	1062.5	42.5
De-nitrification / anoxic zone	523	20.92
Separation	450.5	18.02
Empty part at top of WWTP	301.35	12.05
Interior walls and others	77.65	3.11
Total volume	2500	100

The treatment process starts with the screening of certain suspended solids present in the wastewater; these suspended solids are filtered by the use of a removable screen basket with filtering slots 5-8 mm. The recycled activated sludge is brought just underneath the basket from the separation zone, and where is mixed with the incoming wastewater. After the screening the de-nitrification and Activation/Nitrification process take place. In the de-nitrification zone, oxygen is removed from nitrate and nitrite to form nitrogen gas and water. From the de-nitrification zone, wastewater overflows into the aeration (nitrification/activation) zone, which is the largest zone and provides a space where the bacterial mass is aerated and maintained for the longest period of time. This allows for the maximum utilization of nutrients and conversion of the contaminants in the raw sewage into less harmful compounds; carbon dioxide and water in the process of oxidation, and nitrite and nitrate in the process of nitrification. The aeration system goal was to maintain the dissolved oxygen at 2-3 mg/l, maintain of solids in suspension and ensure proper recirculation of the activated sludge. Air was diffused from the bottom of the aeration zone. It is important to mention that the typical wastewater does not contain nitrate that means that no de-nitrification can occur unless a nitrification was preceded and due to that, the de-nitrification of the treatment plant was accomplished through the use of a circulating pipe that returns the flow to the screen and therefore to the de-nitrification compartment assuring by that the de-nitrification process. Then the half conical shape of the separation zone ensured that the upward velocity of the sludge flocks decreases as the flocks rise until they form a stationary sludge blanket when gravitational and uplift forces reach equilibrium. Wastewater passes through the sludge blanket, fine suspended solids are retained and the filtered effluent rises above it. The effluent is then discharged out of the system. The growing flocks of the sludge at the bottom of the separation zone are recycled by means of an air lift pump back to the screen and de-nitrification zone of the bioreactor. Also located in the separation zone is a device to skim and remove flocks of sludge occasionally breaking away and floating on the surface of the separation zone by means of an air lift pump. The average volume of air injected to the system during aeration was approximately 1.1 Lit / Sec. The running of the treatment facility was totally automatically controlled, including: steering, recirculation of activated sludge, injected oxygen volumes and treated wastewater pumping. Figures 2.22 and 2.23 illustrate the Wastewater Treatment Plant Process (ARIJ, 2007).

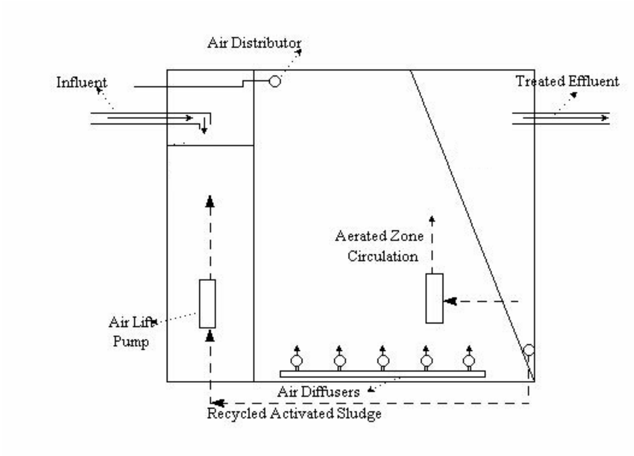


Figure 2.22 schematic diagram of individual onsite Activated Sludge wastewater treatment plant

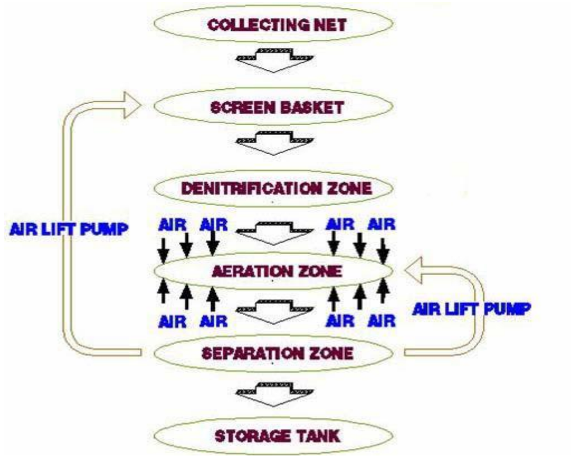


Figure 2.23 Wastewater Treatment Plant Process Diagram

This obtained effluent can be further treated by a normal or slow sand filter, and / or disinfected to be reused for irrigation purposes after taking into consideration the local reuse standards and recommendations. According to lab analysis results obtained by ARIJ (2005) reported that the treated effluent could be reused for irrigation after disinfection.

Table 2.8 Effluent quality of individual onsite Activated Sludge wastewater treatment system

Effluent sample (mg/l)	pH	TS	TSS	COD	BOD_5	TP	NH_4
Sample 1	7.1	653	173	27	<15	5.08	4.6
Sample 2	7.4	651	171	17	<15	5.23	4.1

2.5.3.3 Trickling Filter system

A trickling filter for the treatment of gray wastewater from one house with 13 persons has been built (Mustafa, 1996) in many rural areas. The effluent from the plant is used in the garden. It was found that gray wastewater has COD and nitrogen concentrations, which were sufficient for biological growth in the trickling filter. In places where this system was constructed, the house installation was changed to separate the gray wastewater from the black wastewater. The black wastewater was discharged into the existing cesspit, while the gray wastewater was treated in the pilot plant.

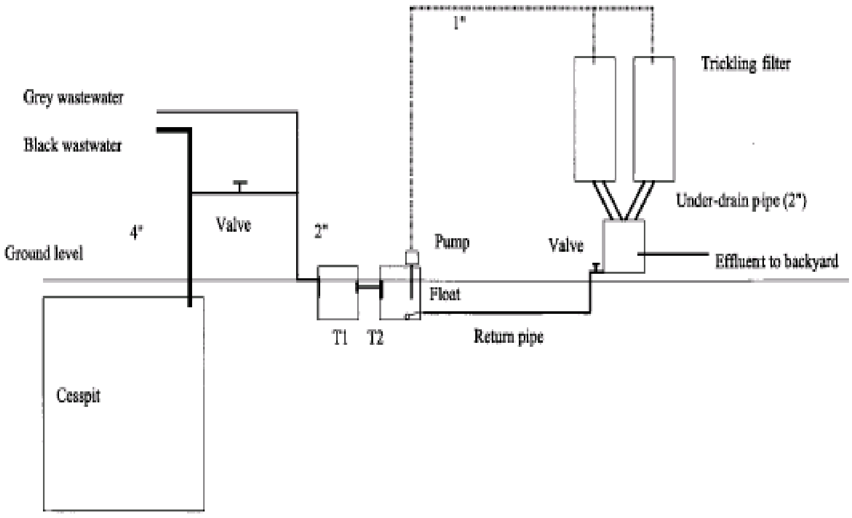


Figure 2.24 Onsite trickling filter plant (Mustafa, 1996)

The trickling filter has been designed as a low loaded system with recirculation in order to wet the media at night, to minimize odor and to prevent breading of filter files. The filter media were washed wadi stones of 2 to 6 cm in size. The media were placed in iron barrels of 4 mm wall thickness, 91 cm high and 61 cm in diameter. The volume of the filter was 1.0 m^3. Septic tank (T1) made of plastic received the gray wastewater. The retention time of the wastewater in the septic tank is 1.5 to 2 days. A storage tank (T2) with a variable water level had been placed to receive the recirculated effluent from the trickling filter. Effluent from the under drain of the filter is collected in a balance tank. A recirculation pipe of 1" diameter was placed 35 cm above the bottom and overflow pipe of 1" diameter was installed, overflow effluent is used to the backyard plants. A float fixed at a level of 30 cm from the bottom of T2 controlled recirculation. Water was pumped from T2 at a level of 15 cm from the bottom every half hour to the trickling filter, a pumped, controlled by a timer doses were used for that purpose. The pumped water falls in a bucket above the filter media and spreads over the surface of the filter through 2 mm holes at distance of 5 cm. The trickling filter design was based on the following parameters:

- Maximum flow 0.02 l/s (23 litters is pumped every 30 minutes).
- Organic loading rate =0.14 kg BOD/m^3
- Average hydraulic loading rate of 0.12 m/h

The septic tank-trickling filter plant was operated and monitored from 3 January 1997 to 30 May 1997. Monitoring results show that the efficiency of the system improved with time and increased with (ambient air) temperature. The efficiency also improved when the hydraulic flow rate was increased from 0.065 to 0.12 m/h, the effluent COD decrease from about 600 mg/l to about 60 mg/l. the COD/BOD ratio in the influent is generally 2.4. More than 80 % BOD removal was achieved. Not much accumulated settled solids were noticed in T1 after three months of operation. Garbage was not allowed to enter the sewer pipe and the accumulated grease that occurred at the top of the surface of septic tank was not covered by the whole surface. The scum layer has a thickness of about 5mm, and gets thicker near the tank wall. Rising gases and smell of H_2S were clearly noted in tank T1, the color was black and no strong H_2S smell was noted in tank 2 even though the wastewater still had a black color (EQA, 2006).

2.5.3.4 Subsurface Drainage Technique

Typical units of Subsurface Drainage Technique (SDT) treatment system were designed, supported and constructed in different rural areas by Save the Children Foundation (SCF) between 1989 and 1998. It was implemented in Tamoun, Oareen, Aldowareh, Sair, Bani Naim and Alwalajeh towns in the West Bank rural areas. The reclaimed wastewater was used for agricultural basis. The SDT unit consists of three main parts: (1) the sewer line from the house (2) the sedimentation tank (3) and the biological filter. The units are supposed to be cleaned every three years. The sewer line carries the sewage from the house to the septic tank, which forms the main part of the SDT unit. The septic tank is made of concrete or lined blocks and hermetically sealed. It consists of two compartments with total volume of $13m^3$ with 2m depths. A distribution box outside the septic tank connects it with the penetration field. It consists of perforated pipes laid 60cm under the surface of the ground with a maximum slope of 1 percent. The pipes are perforated with holes at 15cm intervals and separated from each other by 2m. They are buried in gravel with a minimum thickness of 15cm and a 5cm layer of ash laid over the gravel.

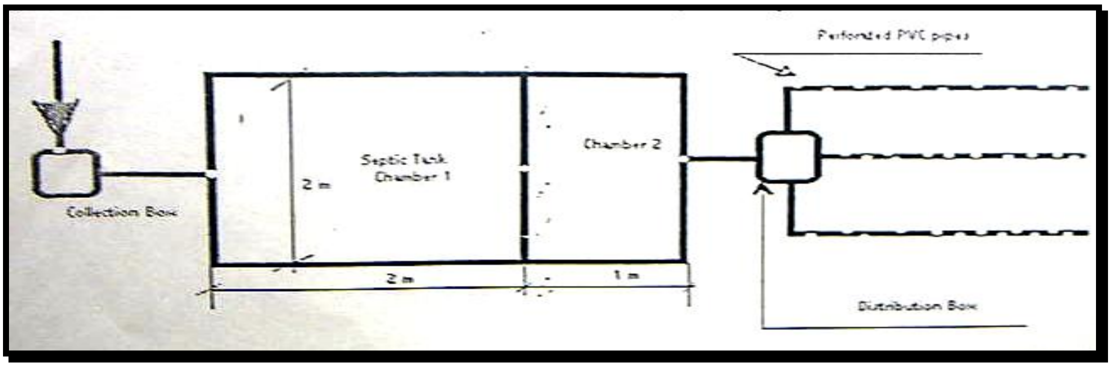


Figure 2.25 Plan view of the subsurface drainage technique (SCF, 1998)

In most of the SDT units no odor problems have been witnessed. The technology is considered cheap and affordable, reduced the cesspools evacuation cost if technical good built and connected, the SDT does not require a special operation skill. However, they were poor followed by SCF after completion of the project. The SCF did not assure the good operation of the SDT units after the end of the project, neither the commitment of the beneficiaries to the SCF technical specifications while constructing the units. No selection criteria were adapted to select treatment plants sites and beneficiaries. Moreover, the SCF did not take any samples for quality test to prove success and efficiency of the system. Some of the SDT units suffered from poor drainage fields, where the SDTs have been executed in rocky areas, where no garden or field was suitable for disposal. Likewise, most of the SDTs systems effluent were not used for reuse of disposal wastewater as well as the most important of all is that the effluent concentrations did not meet any of the standards set for any kind of irrigation. The EC value ranges between 0.33-6.15 ms/cm. The influent TSS ranged from 60-1010 mg/l, while it ranged from 180-33mg/l in the effluent of the SDT units. The samples also showed high values of organic nitrogen (156mg/l). The effluent BOD and COD concentrations were higher than 400mg/l and 1000mg/l respectively in most of the samples analyzed by An-Najah National University. The efficiency of treatment achieves removal percentages of 17-52% BOD, 23-61% COD and 60-90% SS, this removal efficiencies may be changed from location to another mainly due to the influent wastewater characteristics and operation and maintenance procedure. (SCF, 1998; PHG, 1998; AN-Najah National University, 1998; Bethlehem university, 1998).

2.6 Palestinian Standards

For a long time, Palestine did not have any specific wastewater regulations, references were usually made to the WHO recommendations or to the neighbored country's standard (ex. Egypt, Jordan). Recently, the Environment Quality Authority with coordination of Palestinian ministries and universities has established specific wastewater reuse regulations. The draft of Palestinian legislation for reuse of treated wastewater is still under study in the Palestinian Standard institute. On the other hand, PWA recognizes the importance of establishing proper Environmental Limit Values (standards and guidelines) for effluent from domestic wastewater treatment plants as well as the industrial standards for wastewater to be discharged on the sewage systems (EMWATER-Project, 2005). (Appendix A)

Table 2.9 Reclaimed wastewater classification

Class		Water Quality Parameters		
		BOD_5	TSS	Fecal coliforms
Class A	High quality	20 mg/l,	30 mg/l,	200 MPN/100 ml.
Class B	Good quality	20 mg/l,	30 mg/l,	1000 MPN/100 ml
Class C	Medium quality	40 mg/l,	50 mg/l,	1000 MPN/100 ml
Class D	Low quality	60 mg/l,	90 mg/l,	1000 MPN/100 ml

Despite meeting the regulation and guidelines, the reuse of wastewater is not entirely a risk-free. Continued research will result in developing new technologies or improving the existent methodologies used for assessment of health risk associated with trace contaminants, evaluation of microbial quality, treatment systems, and evaluation of the fate of microbial, chemical and organic contaminants (MEDAWARE, 2005).

2.7 Wastewater Characterization

The wastewater from toilet is called black water. Amount of this water is very small but contain high in solid, COD and significant nutrients (as nitrogen and phosphorous). Other wastewater that generated and discharged from living activities of human such as cooking, bath, washing are called greywater. The greywater is high volume and contain high amount of organic matter but low in nutrients. The black water can be separated into two types that are faeces and urine before it is mixed in the toilet. Faeces are known as brown water and urine is called yellow water. The various type of human waste in household is generated and discharged as illustrated in the below sketch:

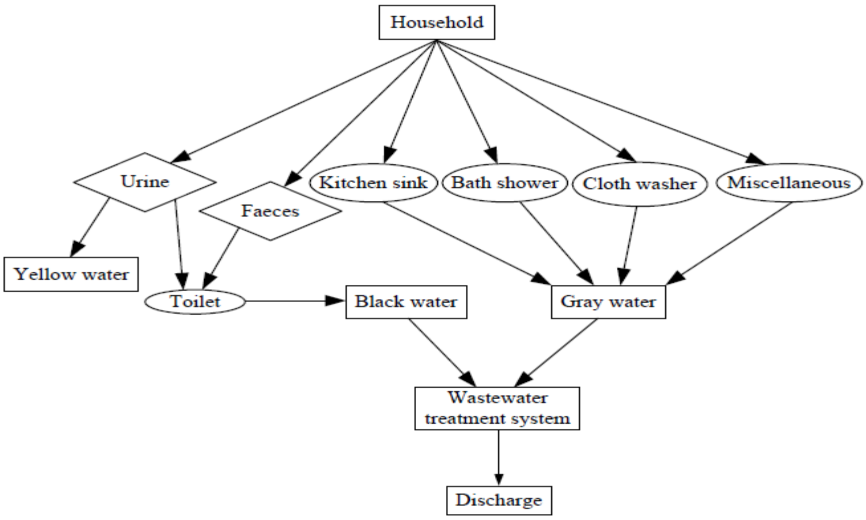


Figure 2.26 Various types of human waste

2.7.1 Wastewater Characteristics in Palestinian Rural Areas

Some few analyses were conducted to measure the gray and black wastewater characteristics in the West Bank rural areas. Few institutions and researchers measured wastewater characteristics before conducting the proposed wastewater treatment plant. In addition, these analyses were conducted in specific locations and periods; hence, they do not represent the actual wastewater characteristics of the rural areas. Table 2.10 presents an example for the characteristics of the wastewater in the West Bank rural areas.

Table 2.10 Characteristics of gray and black wastewaters from one house of 13 persons in Bilien village/ Palestine (Mustafa, 2000)

Parameters	**Gray Wastewater***		**Black Wastewater*****	
	Range	Median	Range	Median
BOD (mg/l)	222 – 375	286	255 – 322	282
COD** (mg/l)	600 – 850	630	566 – 643	560
BOD:COD	1.6 – 2.58	2.25	2.1 – 2.7	2.26
Dissolved Oxygen (DO in mg/l)	5.24 – 6.5	5.9	5.5 – 7.0	6.25
Temperature ºC	18.5 – 25.4	22	15 – 16	15.7
NH_4^+-N (mg/l)	7 - 12	10	371	
Kj-N (mg/l)	16 – 17	16.7	292 – 381	358
Phosphate total (mg P/l)	15 – 17	16	34	34
PO_4^- (mg PO_4^-/l)	45 – 52	49		
Sulfate SO_4^- (mg/l)	52-54	53	46	
NO_3^- (mg/l)	0 – 1.3	1		
Total Suspended Solids (TSS) (mg/l)	94 – 181	125		
Settling Solids (ml/l)	0.3 – 4.5	1.7		
Total Dissolved Solids (TDS) (mg/l)	628 – 1212		2540	
Chloride (mg/l)	180 – 220	200	773	
pH	6.6 – 7.4	7	8 – 8.5	8.2

* Thirty samples were collected in triplicate and analyzed from the first compartment of the septic tank, where the retention time of the wastewater is one to one and half day; hence some treatment might take place there.

** The COD values for fresh gray wastewater samples are attained from samples collected before the first compartment of the septic tank, the dissolved Oxygen (DO) measured for a sample is 5.24 ppm at temperature 18.5 ºC while the fresh water DO was 5.44 ppm at 16 ºC.

*** The samples were taken from the top part of the cesspit from a place next to the outlet of the toilet pipe to the cesspit.

2.8 Reuse in Agriculture

The Palestinian experience in the reuse (especially in the agricultural sector) is very immature and poor. Currently the supply of water through irrigation in the West Bank is estimated at 89 MCM/yr and the total irrigated area is around 136,866 dunums which only represent 6% of the total cultivated area in the West Bank (Jayyousi & Srouji, 2009).That implies that Wastewater reuse is a very important option in the agricultural sector which could enhance the water sector in the West Bank.

2.9 Impacts of Treated Wastewater Reuse

There are major real potential health, environmental and economic impacts as a result of poor sanitation, improper disposal of treated and untreated wastewater, and use of raw or partially treated wastewater to irrigate edible crops. Most of the wastewater treatment plants are not working efficiently and the effluent's characteristics are not meeting the Palestinians standards and guidelines which have been formulated by the Palestinian standard Institute (PSI) (HWE, 2009). Therefore the effluent of the treated wastewater may cause a serious hazard not only on the surrounding environment but also on the public health. These impacts are described below.

2.9.1 Environmental Impact

The impact of wastewater irrigation on the environment can be positive and negative. The positive impacts concern water conservation and avoidance of pollution effects. However, while some of the effluent's substances (nitrogen, phosphorus, and potassium) in the wastewater are beneficial, others (nitrate, heavy metals, and salinity) can have a negative impact on soil and groundwater. A significant problem for determining the net environmental impact is the lack of consensus on the methodologies for the quantification of these impacts. Therefore, qualitative ranking is often used because even when the physicochemical composition of the wastewater is known, the long-term impacts they will have on the environment are still uncertain (HWE & FEW, 2007).

2.9.2 Health Impacts

Irrigation with raw wastewater in the rural areas presents a major health hazard to consumers of vegetables, farm workers and farm workers families. Where, to minimize cesspits flooding and emptying cost, some people in the rural areas are disconnecting the gray wastewater from the interior sewer network and use it without treatment to irrigate some plants existing in the home gardens, this practice leads to increase mosquitoes and potential of negative health impact in case it used for raw eaten vegetables or unrestricted irrigation (EQA, 2006). In addition, undersized, poorly planned designed and poorly maintained wastewater treatment plants present major health hazards in the rural areas of overflow and system surcharging.

2.9.3 Economic Impacts

The economic assessment of the reuse of wastewater should be comprehensive and should take into consideration many factors including the benefits and the costs of the reuse of wastewater. The economic assessment must contain the financial costs, the costs associated with health risks and the environmental degradation .In addition the benefits must be considered such as the agricultural added value, the avoided costs of developing new potable water resources. The type of irrigated crops is also of great importance since it determines the degree of treatment, factors such as socio economy, climate, soil and topography also strict the crops available for wastewater irrigation. The financial costs of the reuse of wastewater in the agriculture are high because of the costs related to the operation and maintenance of the collection and conveyance system. These costs are variable depending on the type and technology of treatment and conveyance (HWE, 2009).

A detailed cost benefit analysis for the reuse of wastewater in the west bank must be conducted and studied in order to evaluate the real economic impact of this new water option and its efficiency. However the study on the treatment and reuse for irrigation costs and benefits in Wadi Al -Nar had highlighted several economical benefits of the Wastewater reuse in irrigation in the West Bank and calculated the economical benefit for the farmers of the study area. The economical benefits that the study focused on are:

- Agricultural Added value: this takes into consideration the lower costs of fertilizer application (since treated wastewater contain the essential nutrients for the crops), and the increased crop productivity. The economic added value for farmers was estimated to be 0.29 US$/$m^3$ of treated wastewater used in irrigation in 2015.

- Health - related economic impact: the health benefit of the reuse of treated wastewater originates from the better sanitation techniques and irrigation systems compared to the no treated systems. Where the farmers will not be directly exposed to the non treated wastewater and get diseases. In addition the crops will be of better quality. Moreover, the public health will status will be improved by the sanitation function of removing the wastewater from the urban area and the environment at large. If such a collection system were not installed, a much larger population would likely suffer from exposure to wastewater. Additionally, when a sewerage system without treatment exists and discharges to surface water, usually a much larger downstream population would be subject to negative impacts.

2.10 Socio-Economic and Cultural Assessment of Applied Wastewater Treatment Technologies

The development of sustainable and affordable wastewater treatment systems will have a positive impact on the Palestinian economy through direct positive impact on poverty alleviation in addition to environmental protection. Research study conducted through Corotech Project aimed at examination of onsite sanitation systems from the perspective of the community with special emphasis on social and economical aspects were conducted in 2002 in the three Palestinian rural areas located in Ramallah-Al-Bireh district. These areas were Birzeit, Jifna, Ein Sinya and Jalazoun camp. Except the latter, all other towns do have septic tanks but no sewerage system. In this study, the evaluation of the existing sanitation systems, installment alternative at low cost, decentralized treatment technology, willingness to participate, pay and utilization of the treated effluent in agriculture are evaluated in questionnaire. The basic information obtained from the questionnaire includes the following (Al-Saed, 2003):

- People do not have money for the construction equipment and those who have are not ready to pay.
- Social and cultural traditions do not allow or accept persons who work on monitoring reactors to enter their homes.
- Some people (85%) accepted the idea of having a decentralized sanitation system but they wanted technical and financial support from the local community.
- People who have special cesspits think that they do not need to participate in new on-site sanitation facilities.

- The majority people (90%) use the treated wastewater in irrigating indoor plants and some people also refuse to buy any vegetable or fruits that were irrigated with treated wastewater.
- A few people (20%) want to pay only for the construction part but refused to pay for the ongoing monitoring and maintenance costs.
- The on-site area is an unpleasant view for people. In addition, houses are not designed to consider on-site sanitation systems; especially source separation of wastewater.
- During the questionnaire, it was noticed that the majority of people (80%) prefer to construct central sewerage networks and construct off-site treatment facility rather than on-site sanitation systems.
- Many people believe in a safe wastewater disposal with less pollutant to valleys instead of discharging sewage without treatment.
- All people did not accept separation of black and domestic wastewater. They prefer collecting the wastewater from kitchen and toilet together.
- Some people (40%) accepted the onsite sanitation system with reservation; unless they are sure it will not cause waterborne diseases or harbor/transmit harmful insects.

2.11 Institutional and Political Assessment of Applied Wastewater Treatment Technologies

According to the major findings of the 10 national stakeholder workshops for "building a participatory national consensus on wastewater reclamation and reuse in Palestine" were reported (Abu-Madi, et al, 2009):

- Wastewater reuse is recognized by all stakeholders as a valuable non-conventional water resource that needs better utilization.
- Institutional conflicts among the water institutions in the country are a major constrain for development of reclaimed wastewater as a non-conventional resource. The role of all institutions should be based on the principle "complete not compete" to avoid institution conflicts.
- There is poor utilization of the existing knowledge and experiences due to poor dissemination of research and projects implemented by various institutions. There is a need to initiate a national open-access data bank of all documents related to the subject.
- There exists a controversy on the appropriateness of centralized and decentralized wastewater treatment systems. However, there was a consensus on the application of both systems depending upon the number of targeted population. Onsite collection and treatment seems to be more appropriate in the Palestinian rural and peri-urban areas due to large landscape and availability of land. Offsite collection and treatment seems to be appropriate for large urban communities. Onsite systems such as grey water systems do not require permission from the Israelis. Therefore, it was recommended to encourage their application in the Palestinian rural and peri-urban communities.
- The technical and economic feasibility of different treatment systems is driven by many factors. It was agreed that technologies could function properly under a sound enabling environment. This means that availability of skilled personnel, spare parts, and effective monitoring is more crucial than the type of technology. The Palestinian expertise in the field of wastewater treatment is well established. However, adoption of certain treatment systems in the country is influenced by donors and foreign consultancy firms.

- The importance of offering capacity building programs to technicians and managers working at treatment plants to help in cost reduction and more efficiency in operating and maintaining these plants. Selection of appropriate technology for wastewater treatment should focus on suitability of effluent for irrigation and low treatment costs.
- Secondary treated effluent is potentially appropriate for agricultural irrigation. Therefore, there is a consensus that reuse of reclaimed wastewater should be permitted only for restricted irrigation (irrigation of fodder crops and landscape) in order to minimize health risks. This might be developed in the future to allow unrestricted irrigation such as vegetables and crops eaten raw or uncooked.
- Wastewater treatment plants should be located close to agricultural lands in order to reduce transport and conveyance costs.
- The increasing fear of applying wastewater reclamation and reuse projects is attributed to many social, farming, marketing and particularly health considerations – which represents obstacles in applying the use of treated wastewater in agriculture. There is an ultimate need for extensive awareness campaigns in order to increase community acceptance of wastewater projects.
- The current Palestinian Standards for reuse of treated wastewater in irrigated agriculture were concluded from various international guidelines such as WHO, Jordanian, and Israeli guidelines. There is a need for consistent evaluation of the current Palestinian Standard to make it suitable for the Palestinian needs. This requires involving the major public bodies especially the PWA, the Institute of Standards, Environmental Quality Authority, Ministry of Health, and other stakeholders.
- Palestinian farmers in general have access to freshwater at very low price. Therefore, pricing of freshwater for irrigation should be recalculated and carefully adjusted in order to make reuse of treated wastewater attractive.
- There seems to be evidence that some Palestinian farmers use raw sewage illegally for agricultural irrigation, which might have serious health risks. There is a lack of applicable strategies, efficient monitoring systems, and emergency plans. There is a need to activate the mechanisms of applying rules and legislations including penalty on illegal practices.

CHAPTER THREE

METHEDOLOGY

3. Materials and methods

To achieve the main objectives of this thesis the following overall research methodology is adopted.

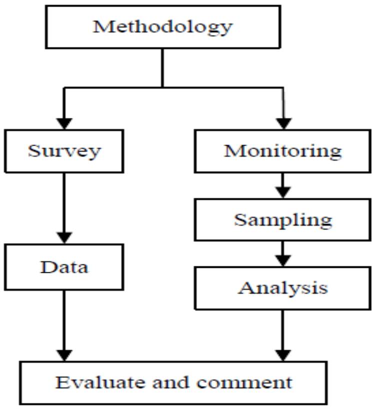


Figure 3.1 Overall of methodology

3.1. Field Data Collection

Site inventory evaluation began on February, 2010 to gather background data that would reveal the status of the sanitation in the Palestinian rural areas in terms of counting the number of onsite wastewater treatment plants that have been constructed by non-governmental organizations (NGOs), identify the existing types of technologies or systems, the status of each existing onsite treatment plant, level of onsite wastewater treatment plant, design capacity, year of construction, and selected location. Also, interviews have been conducted with researchers and relevant NGOs whom designed and implemented these onsite wastewater treatment plants and other interviews were conducted with the village councils where these onsite wastewater treatment plants were carried out inside (Appendix C).

3.2 Questionnaire Design

In order to make a preliminary study to assess the main issues related to the situation of onsite wastewater treatment plants in Palestinian rural areas, a questionnaire has been designed to provide a specific data about each existing onsite wastewater treatment plant which has been selected by Excel Selector Program within the stratified sample as will be explained later and carried out in the summer of 2010. The questionnaire assisted in collecting relative information towards monitoring of onsite wastewater treatment plants in the study area.

3.2.1 The methodology of the questionnaire:

The questionnaire is divided into three sections. The first one consists of 31 questions. Ten out of these questions are of multiple choices while the remainder requires a short answer. These questions are designed to gather the basic information about the wastewater treatment plants included the source of raw wastewater, current treatment technologies, operation issues, in addition to what mentioned before this section introduce the technical history for each plant.

In the second section, there are 3 tables. These tables are addressed the issues that related to the main operational and technical problems for each process (primary, secondary and tertiary). While the last section is identify the most critical process parameters that may affect the efficiency of the Wastewater treatment plant.

3.2.2 Stratified Sampling Design

In stratified sampling, the total collected onsite wastewater treatment plants are divided into separate groups (strata) which differ along selected characteristics in terms of type of technology used, capacity of the plant, number of wastewater treatment plants of each system, Year of construction, the status of the plants still working or not, Owner, Implemented Agency, Site Location, Village and Governorate. The stratified sample has been designed by the assistance of the Palestinian Central Bureau of Statistics via using the Excel-Selector program.

There are two main benefits of a stratified sample:

1. Stratified sampling ensures that an adequate number of wastewater treatment plants are gained for each subgroup of interest. This also helps to ensure that a representative sample is achieved.
2. For the same size sample, a superior estimate at the overall level and also at the subgroup level can be obtained by allocating a higher proportion of the sample to the groups with higher variability.

3.2.3 Sample Size (The study responders):

The total number of onsite community, collective and household wastewater treatment plants which have been accounted during the field data collection was 1137 plants and thus the number of responders for the questionnaire was 168. This number was computed using the following equation:

$$n=\frac{t^2 * N * P * (1-P)}{N * \frac{B^2}{4} + t^2 * P * (1-P)} \qquad \text{…….. (3.1)}$$

$$n=\frac{1.96^2 * 1137 * 0.5 * (1-0.5)}{1137 * \frac{0.14^2}{4} + 1.96^2 * 0.5 * (1-0.5)}$$

n=167

Where,

Indicators use	***Value use***
Main indicators	Treatment type
Type of estimate	Percentage
Percent of main value (P)	50%
Bound of error (B)	7%
Interval confidence (t)	1.96
Population (N)	1137
Sample (n)	167

It should be mentioned that it was took P = 0.5 (maximum variability) to produce a more conservative sample size.

Types of responders are classified into three categories depending on what is the required of each level of onsite wastewater treatment plant; the first one is to the implemented Agency to be asked to complete some of basic data and the general historical situation of their constructed plants. The second one is for the people who are beneficiaries from the treated plant. The last one is answered by me to make a self-technical check for each monitored process plant.

In addition to what mentioned above and because of the expensive of the laboratory tests where, it cannot analyze the laboratory samples including of 168 wastewater treatment plants. Therefore, it has been created a mini-stratified random sample which consists of all what has been previously reported in various locations in Palestinian rural areas of west bank as showing in the following tables.

Table 3.1 contains a mini stratified sample of onsite Household Level wastewater treatment plants which have been chosen to be monitoring in terms of analytical\operational method.

Type of Treatment	Agency implementation	Location	Design capacity	Year of construction	Type of Raw Wastewater
Septic Tank followed by Upflow Gravel filter - sand filter. (ST-UFGF-SF)	FAO	Beit Leed – Tulkarm	1 m^3 per day	2009	Gray Wastewater
	PARC	Beit Sira – Ramallah	0.7 m^3 per day	2006	Gray Wastewater
	FAO + QWC	Qebia- Ramallah	0.5 m^3per day	2006	Gray Wastewater
	PWEG	Beit Anan – Jerusalem	0.5 m^3 per day	2005	Gray Wastewater
	PARC	Sanur – Jenin	0.5 m^3 per day	2002	Gray Wastewater
Activated Sludge followed by sand filter. (AS-SF)	ARIJ	Battir- Bethlehem	1 m^3 per day	2007	Mixed wastewater
	ARIJ	Halhul – Hebron	1 m^3 per day	2007	Mixed wastewater
	ARIJ	Beit Ummar – Hebron	1 m^3 per day	2007	Gray Wastewater
	ARIJ	Nahhalin- Bethlehem	1 m^3 per day	2007	Mixed wastewater

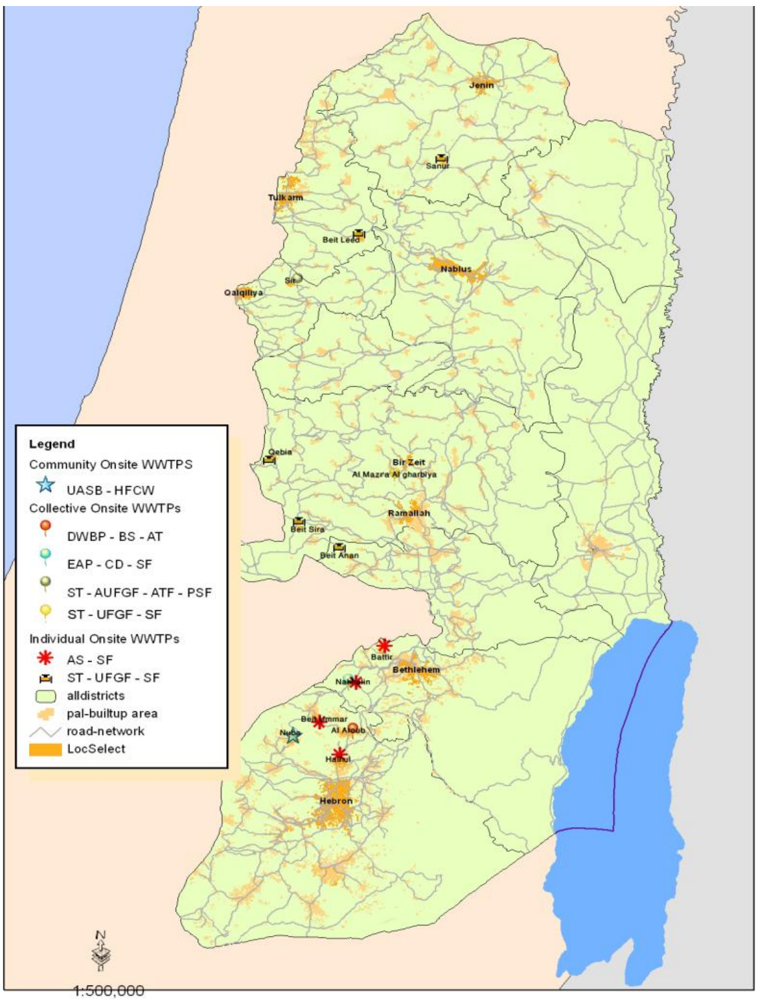
Table 3.2 contains a mini stratified sample of onsite Community Level of wastewater treatment plants which have been chosen to be monitoring in terms of analytical\operational method.

Type of Treatment	Agency implementation	Location	Design capacity	Year of construction	Type of Raw Wastewater
Up-flow Anaerobic Sludge Blanket - Horizontal Flow Constructed Wetlands. (UASB-HFCW)	PHG	Nuba-Hebron	120 m^3 per day	2003	Mixed wastewater

Table 3.3 contains a mini stratified sample of onsite Collective Level of wastewater treatment plants which have been chosen to be monitoring in terms of analytical\operational method.

Type of Treatment	Agency implementation	Location	Design capacity	Year of construction	Type of Raw Wastewater
Duckweed-based pond system - Small-scale biochemical system - Aeration tank. (DWBP_BS_AT)	EQA	Al Aroub agriculture school-Hebron	8 m^3/day	1997	Mixed wastewater
Septic Tank - Anaerobic Upflow Gravel Filter - Aerobic Trickling Filter followed by Polishing Sand Filter. (SF_AUFGF_ATF_PSF)	PARC	Qalqilya Sir City	14 m^3 per day	2006	Mixed wastewater
Extended Aeration Process – Chlorine Disinfection and Sand Filtration. (EAP_CD_SP)	ARIJ	Nahhalin - Bethlehem	50 m^3 per day	2006	Mixed wastewater
Septic Tank followed by Up-flow Gravel filter - sand filter. (ST_UFGF_SF)	PWEG	Al Mazr’a Al gharbiya - Ramallah	60 PE	2005	Mixed wastewater

The following map 3.1 indicates the geographical location for the mini stratified samples of onsite community, collective and household levels of wastewater treatment plants which were selected to be used for evaluation in terms of analytical\operational method.



Map 3.1 The geographical location for selected onsite WWTPs to be used for evaluation.

3.3 Sampling and Analytical Methods

Sampling and laboratory analyze are the first step required to monitor and control the wastewater treatment plants effectively. Grab sample from each selected plant were taken one time a month. Sample was kept at 4ºC until they were analyzed. So, It has been analyzed and measured at Birzeit University Testing Laboratories (BZUTL) the different physical, chemical and biological parameters which consist of Total Dissolved Solids (TDS), Total Suspended Solids (TSS), Acidity (pH), Temperature, Ammonia, Nitrite Nitrate, Dissolved Oxygen (DO), Phosphates, Sulfate, BOD, COD, Kjeldahl Nitrogen, Total Coliforms (TC), Fecal Coliforms (FC) (including E-coli). These parameters are considered as the key parameters for describing the wastewater characteristics and their corresponding ratio in the influent to effluent wastewater as a measure of the overall performance for each wastewater treatment plant. Household onsite systems considered is a "gate to gate" analysis including all the processes from the entrance of the influent of the wastewater treatment plant until its exit as an effluent. Except collective and community onsite systems just influent and effluent was included.

- <u>**Chemical Analysis:**</u>

  - **Chemical Oxygen Demand (COD):** Samples were used for measuring total COD, Where COD test done by using reflux method (acid destruction at 150 C^0 for 120 minutes where the absorbance was then measure by spectrophotometer at 600 nm wavelengths according to Standard Methods (APHA, 1995).

  - **Biological Oxygen Demand (BOD_5):** Samples from the influent to effluent of each system were used to determine DOD_5 at 20ºC. This test is done according to Standard Methods (APHA, 1995).

  - **Kjeldhal Nitrogen (NKj-N):** The Kjeldhal method (digestion, distillation and titration) according to Standard Methods (APHA, 1995) was used to determine the amount of the organic and ammonium nitrogen.

  - **Ammonia (NH_4^+-N):** Nesslerization method using spectrophotometer at absorbance of 425 nm wavelength used to determine the Amount of Ammonia (NH4-N) from paper-filtered samples and this regarding to Standard Methods (APHA, 1995).

  - **Sulfate (SO_4^{-2}):** Spectrophotometer at absorbance at 420 nm wavelengths was used to measure the amount of sulfate from paper-filtered sample and this was regarding to Standard Methods (APHA, 1995).

  - **Phosphates:** Spectrophotometer absorbance at 880 nm wavelengths was used to determine the amount of total phosphate regarding to Standard Methods (APHA, 1995).

- <u>**Physical Analysis:**</u>

  - **Total suspended Solids (TSS):** Total suspended solids were measured related to Standard Methods (APHA, 1995) by oven drying at 105 ºC this by using paper of glass microfiber filters (GF/C 125 mm f, CATNO 1822 122 Whatman)

.

- **pH:** pH was measured for total samples using pH meter (HACH).

- **Electrical conductivity (EC):** EC was measured for total samples using EC meter (HACH).

- **Temperature:** Wastewater temperature was measured in situ by alcohol thermometer.

- **Color:** Color was determined by visual appearance.

▪ **Biological Analysis:**

- **Total and Fecal Coliform (TC/FC)**: The TC/FC were measured using the membrane filter procedure.

The observed results of effluent concentrations and removal efficiencies of the constituents BOD, COD, TSS, TN (total nitrogen), TP (total phosphorus) and (TC/FC) (faecal or thermotolerant coliforms) are compared with the typical expected performance reported in the literature

Table 3.4 Typical mean influent, effluent concentrations and removal efficiencies, according to the literature review of the selected six treatment technologies. (PARC, ARIJ, PHG,2003-2010)

	Technologies	**ST-UFGF-SF***	**AS-SF***	**DWBP-BS-AT****	**SF-AUFGF-ATF-PSF****	**EAP-CD-SP****	**UASB-HFCW*****
Parameter	Number of WWTP evaluated	6	4	1	1	1	1
	Average flow (m^3/d)	0.5-1	1	8	14	50	120
BOD	Influent (raw) (mg/l)	222-997	282	495	1176	-	750
	Effluent (treated) (mg/l)	21-121	15	65	240	-	80
	Removal efficiency (%)	90.5-97.7	94.7	86.9	79.6	-	89.3
COD	Influent (raw) (mg/l)	600-2,405	560	789	1600	-	1501.7
	Effluent (treated) (mg/l)	58-266	17	102	400	-	89.16
	Removal efficiency (%)	90.3-88.9	96.9	87	75	-	90
TSS	Influent (raw) (mg/l)	36-396	-	1600	196	-	1900
	Effluent (treated) (mg/l)	4-24	171	450	40	-	1000
	Removal efficiency (%)	88.9-93.9	-	71.9	79.6	-	47
TKN	Influent (raw) (mg/l)	25-45	340	102	302	-	-
	Effluent (treated) (mg/l)	12-48	4.1	23	96	-	-
	Removal efficiency (%)	73	98.8	77.4	77.2	-	-
TP	Influent (raw) (mg/l)	16	34	-	-	-	-
	Effluent (treated) (mg/l)	13	5.23	-	-	-	-
	Removal efficiency (%)	18.8	84.6	-	-	-	-
FC	Influent (raw) (log)	4-5.5	-	-	-	-	-
	Effluent (treated) (log)	0-2	-	-	-	-	-
	Removal efficiency	0-1.99	-	-	-	-	-

*Household onsite wastewater treatment systems
** Collective onsite wastewater treatment systems
***Community onsite wastewater treatment systems

3.4 Operational Methods

It was too hard to obtain on some of data that related to typical design and operational parameters by implemented NGO's. So, the unknown operational parameters related to typical design was calculated depending on technical literature. The operational conditions were evaluated to verify the existence of a relationship between design and operational parameters and the performance of the treatment plants. Typical design and operational parameters recommended which calculated by the provided data in the technical literature are listed in Table 3.5.

Table 3.5 Typical design and operational parameters used to evaluate onsite WWTPs performance

Technologies		Parameter type	Value	unit
Household onsite wastewater treatment plants				
ST-UFGF-SF(10PE)	ST	HRT: hydraulic retention time	2	days
		OLR: Organic loading rate	0.388	Kg BOD/day
		Settleable SS/COD ratio	0.35-0.45	
		Desludging interval	36	months
	UFGF	HRT: hydraulic retention time	1.8	days
		Specific surface	120	m^2/m^3
		Void space\first compartment of the gravel filter	40 %	
		Void space\second compartment of the gravel filter	50%	
ST-UFGF-SF(60PE)	ST	HRT: hydraulic retention time	5	days
	UFGF	HRT: hydraulic retention time	2	days
AS	AT	F/M ratio	0.2-0.5	day^{-1}
		HRT: hydraulic retention time	6-8	hours
	SC	HLR: hydraulic loading rate	0.63	m/d
		SLR: sludge loading rate	1.9	$KgMLSSm^{-2}d^{-1}$
Collective onsite wastewater treatment plants				
DWBP-BS-AT	ST	HRT: hydraulic retention time	3-10	days
	DW1	HRT: hydraulic retention time	10	days
	DW2	HRT: hydraulic retention time	11	days
SF_AUFGF_ATF_PSF			n/a	
EAP_CD_SP			n/a	
Community onsite wastewater treatment plants				
UASB-HFCWs	UASB	HRT: hydraulic retention time	1.6	days
	HFCWs	HRT: hydraulic retention time	7 but 2.1	days

n/a: Not available information

3.5 Calculations
3.5.1 Flow Rate Measurements

The wastewater flow rate was measured by water bills which have been collected from the beneficiaries of the wastewater treatment plants by calculating the reading of water meters. Water meter which is designed to deal with clean water, which means it may not function properly if they are used to measure the flow of wastewater. Where, many water meters have small paddles or wheels that move around to measure flow. These moving parts can be easily plugged by solids in wastewater. One way to avoid this problem is to measure the flow of clean water before it is used in the house. These meters should measure the water used inside the house, but not the water used outside for watering gardens or washing cars, since this water does not enter the wastewater treatment plants. While it's difficult to install a water meter so that it doesn't include the water to be used outdoors, so, it should estimate the outside use, or try to data, when there is typically no outdoor use of water. The following equations were derived for this concern.

Q_{total} (m^3/day) = (2nd Consumption (m^3) – 1st Consumption (m^3)) / number of days

Q_{total} (m^3/day) = (2nd reading of water meter (m^3) – 1st reading of water meter (m^3)) / number of days

Q_{gray} (m^3/day) = 80% wastewater conversion factor of Q_{total}

= 80% (second reading of water meter – first reading of water meter) / number of days

Where, it consume that the Q_{gray} = Q_{total} – 20% of Q_{total} ………………………. (3.2)

3.5.2 Removal Efficiency

The removal efficiency of the different parameters has been calculated regarding to the following equation.

$$\text{Removal Efficiency \%} = \frac{(\text{Xinf} - \text{Xeff})}{\text{Xinf}} * 100\ \% \quad \text{……………………………………………… (3.3)}$$

Where:

% = Removal efficiency;
X_{inf} = Concentration of component in the influent (mg/L);
X_{eff} = Concentration of component in the effluent (mg/L).

➢ **<u>For Activated sludge systems:</u>**

3.5.3 Volumetric COD loading rate and Organic Loading Rate (OLR):

Organic loading rate (OLR) is presented as the weight of organic matter per day applied over a surface area, such as kg of BOD_5 per day per m^3. The BOD_5 is a measure of the oxygen needed to degrade organic matter dissolved in the wastewater over 5 days. It is reported as mg/l of oxygen consumed to degrade the wastewater in 5 days. BOD_5 is one way to measure the amount of easily degradable organic matter in sewage. To calculate organic loading the first step is to convert BOD_5 in mg/l to kg/m^3.

Where,

Organic Loading Rate (kg BOD_5/m^3.day) = (BOD_{5inf}. x Q) / (AT Volume of AS) ………… (3.4)

The volumetric COD loading rate is defined as the amount of COD applied in the aeration tank (AT) volume per day.

Where,

Vol. COD Loading Rate (kg COD/m^3.day) = (COD_{inf}. x Q)/ (AT Volume of AS)………… (3.5)

3.5.4 Sludge Loading Rate (SLR):

The mass loading rate in Kg/d of mixed liquor suspended solids (MLSS) per unit area of the secondary clarifer (SC).

SLR (kg MLSS/m^2.day) = (MLSS x Q) / (area of the secondary clarifer) …………………. (3.6)

3.5.5 Sludge Volume Index (S.V.I):

Sludge settleability is determined by measuring the sludge volume index (S.V.I), which is considered as the ratio of the volume in milliliters of mixed liquor activated sludge settled from a 1,000 ml sample in 30 minutes to the concentration of mixed liquor (in mg/l) multiplied by 1,000. SVI is a calculation which indicates how well aerated activated sludge solids thicken or become concentrated during the settling or clarification process; given by the following formula:

$$S.V.I = \frac{V \times 1.000}{MLSS} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3.7)$$

Where,

S.V.I (ml/gm): (settled sludge volume in milliliters after 30 minutes in a one liter cylinder or beaker divided by the MLSS concentration in mg/l) times 1,000 mg/gm

V (mL/L): Volume of settled sludge after 30 min.
MLSS (mg/L): mixed-liquor suspended solids

3.5.6 F/M ratio

One of the most fundamental control parameters for the activated sludge process is the relationship between the load (i.e. kg/day as opposed to mg/l) of BOD (or bacterial 'food') entering the aeration plant, and the 'mass' of bacteria in the aeration tank available to treat the incoming BOD. This is therefore known as the Food to Mass ratio (F:M ratio), also often referred to as the Sludge Loading Rate (SLR).

F\M Ratio = ($BOD_{5inf.}$ * $Q_{inf.}$) / (MLSS * V) ……………………………….. (3.8)

Where, $Q_{inf.}$ (m^3/day) : Influent flow

$BOD_{5inf.}$ (mg/L) : Influent BOD

V (m^3) : Aeration tank volume

3.5.7 Hydraulic retention time (HRT)

HRT can be calculated by equation 3.4

HRT = $COD_{inf.}$ / OLR ……………………………….………………………… (3.9)

Where:

HRT = Hydraulic retention time (d);

COD_{inf} = COD concentration in the influent (g COD/m^3);
OLR = Organic loading rate (g $COD/m^3.d$)

- **For Horizontal or vertical flow subsurface constructed Wetlands:**

Required surface areas;

$$A_s = Q\,(\ln BOD_{5inf.} - \ln BOD_{5eff.}) / K_T dn \quad \ldots\ldots (3.10)$$

Where,
$BOD_{5inf.}$ = influent BOD_5, mg/l
$BOD_{5eff.}$ = effluent BOD_5, mg/l
K_T = temperature -dependent first-order reaction rate constant, d^{-1}
Q = average flow rate through the system, m^3/d
d = depth of submergence, m = 0.6
n = porosity of the bed, as a fraction = 0.35
A_s = surface area of the system, m^2

The saturated cross-sectional area for flow through a horizontal flow subsurface constructed wetland can be calculated according to Darcy's law;

$$A_c = Q / K_s S \quad \ldots\ldots (3.11)$$

Where,
A_c = d*W, cross-sectional area of wetland bed, perpendicular to the flow direction, m^2
d = bed depth, m
W = bed width, m
K_s = hydraulic conductivity of the medium, m^3/m^2-d = 500
S = slope of the bed, or hydraulic gradient (as a decimal fraction) = 0.01

The unit flow velocity (Q/A_c which is equal K_sS) through a cross-section of the medium should not exceed 8.6 m/d.

The K_T (in d^{-1}) at water tempetature T (in ^oC) and is defined by:

$$K_T = K_{20}(1.1)^{(T-20)} \quad \ldots\ldots (3.12)$$

Where K_{20} is the rate constant at 20^oC = 0.86

The Hydraulic residence time;

$$t = LWdn/Q \quad \ldots\ldots (3.13)$$

Where,
L = bed length, m
W = bed width, m
d = depth of submergence, m = 0.6
Q = average flow rate through the system, m^3/d
n = porosity of the bed, as a fraction = 0.35

3.6 Data analysis

Statistical analyses for data were performed by using Microsoft Excel 2011.With this software most of data analyses (including averages, standard deviations, removal equations and correlations) and graphs were carried out. Furthermore, the questionnaire and the analysis of survey results were carried out using SPSS software for windows Release 12.0, SPSS© Inc. (2007).

CHAPTER FOUR
RESULTS AND DISCUSSION

The obtained results of this thesis provided several points of discussion, in the first place, from questionnaires which distributed as a statistical stratum samples and in the second place, from the technical laboratory analysis of the efficiency of the process of the treatment plants and their operational conditions.

4.1 Field Data Collection Analysis

The Preliminary results appear that there are three levels of community, collective and household onsite wastewater treatment plants which are estimated about 1137 plants distributed in different Palestinian rural areas. Each of these levels contains different type of technologies arranged in several systems. The results which have obtained through field data collection are shown in appendix “C”.

To discuss the general situation of onsite wastewater treatment plants by the obtained results of the questionnaires which have distributed in Palestinian rural areas in terms of assessing their process performance, the results of statistical survey was analyzed as following:

4.2 Analysis of Questionnaire Data

The sample size of designed questionnaire has 168 responders were distributed into different Palestinian rural areas of West Bank. The sample was selected as a stratified sample. The study results of the questionnaire are figured as following:

4.2.1 Basic Data

The following figures show the sample distribution due to its independent variables.

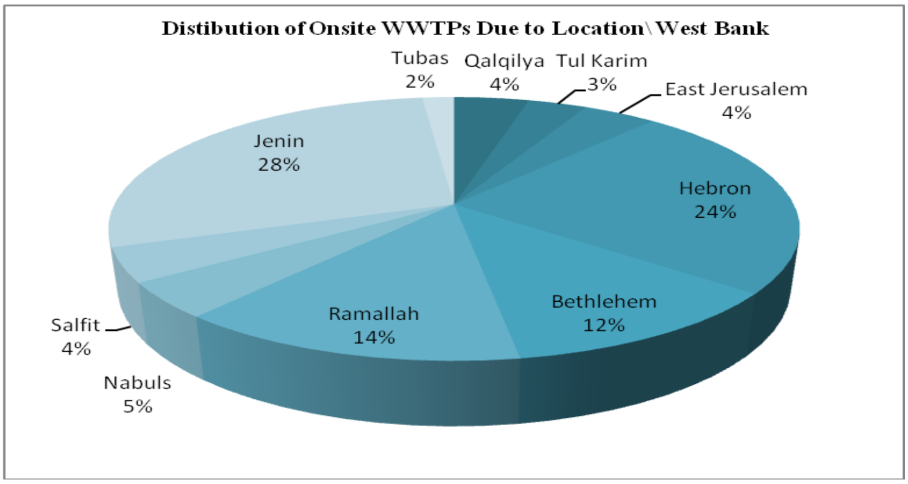
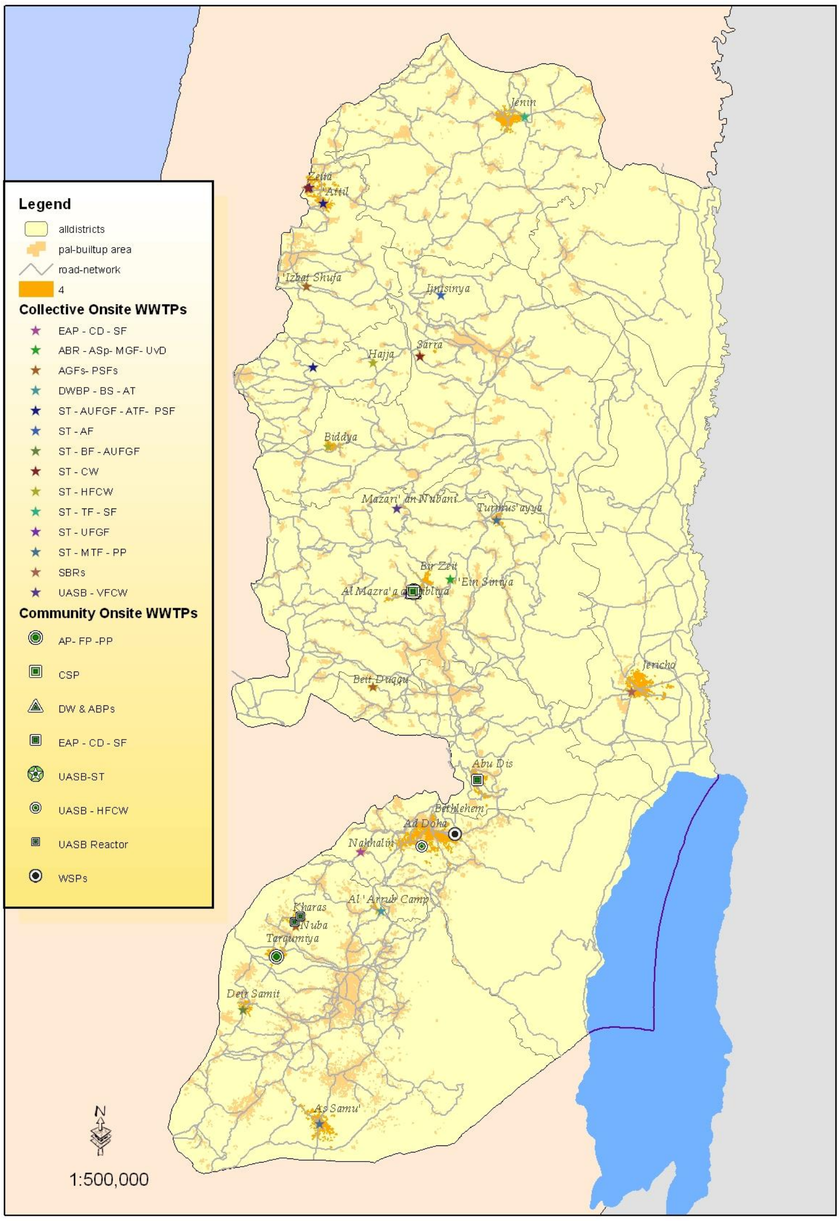


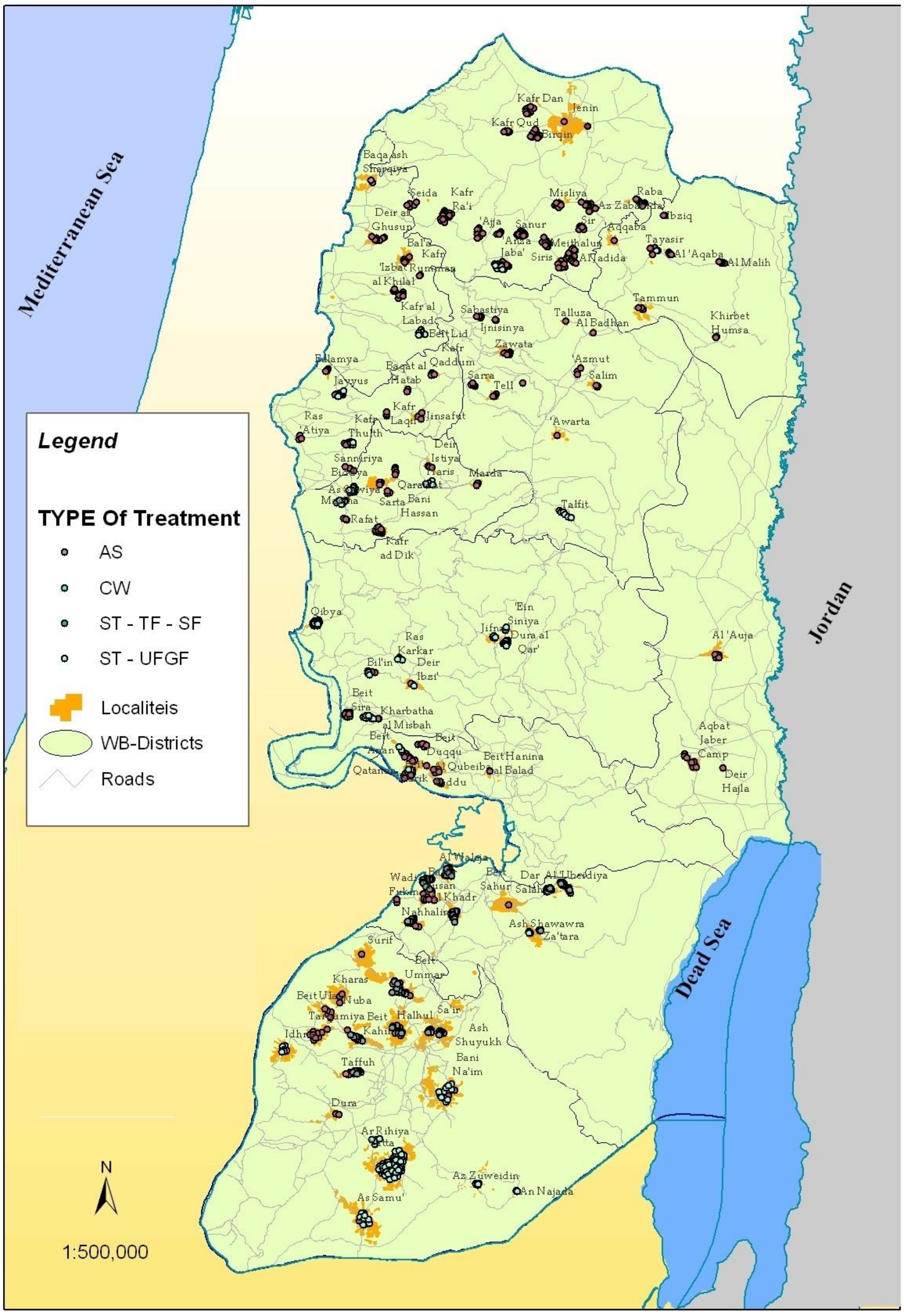
Figure 4.1: Sample distribution due to Location \ West Bank

The questionnaires were distributed within various Palestinian rural areas in West Bank as result of the Excel Selector program which has been designed as a stratified sample for the study area. Figure 4.1 indicates that the highest proportion of the questionnaires was distributed in the villages of Jenin Governorate estimated at 28% of the total questionnaires while the lowest proportion was distributed in Tubas to be estimated 2%, which means that more rate of implementing onsite wastewater treatment plants were in the villages of jenin Governorate.

The following Maps showing the location site of the existing onsite community, collective, and household levels of WWTPs which located in various Palestinian rural areas in West Bank.



Map 4.1 Implemented Technologies of onsite community and collective levels of wastewater treatment plants in Palestinian Rural Areas.



Map 4.2 Implemented Technologies of onsite household level of wastewater treatment plants in Palestinian Rural Areas

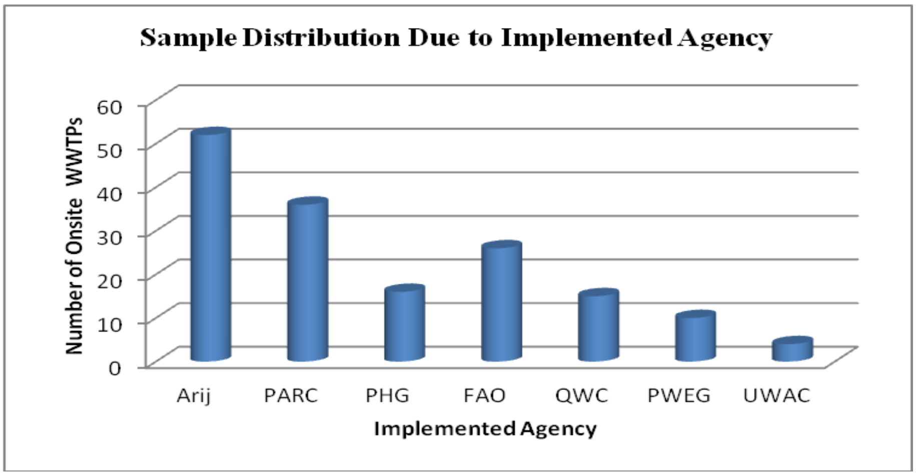


Figure 4.2: Sample Distribution due to Implemented Agency

The survey found that the existing of onsite wastewater treatment plants which have been designed and implemented by National NGOs in rural West Bank areas are varied, where as result of stratified sample by Excel selector indicates that the highest rate of frequency number of onsite wastewater treatment plants was implemented by Arij that estimated to 52 out of 168 plants. While less frequency number of onsite of wastewater treatment plants was implemented by UWAC that estimated to 5 out of 168 plants.

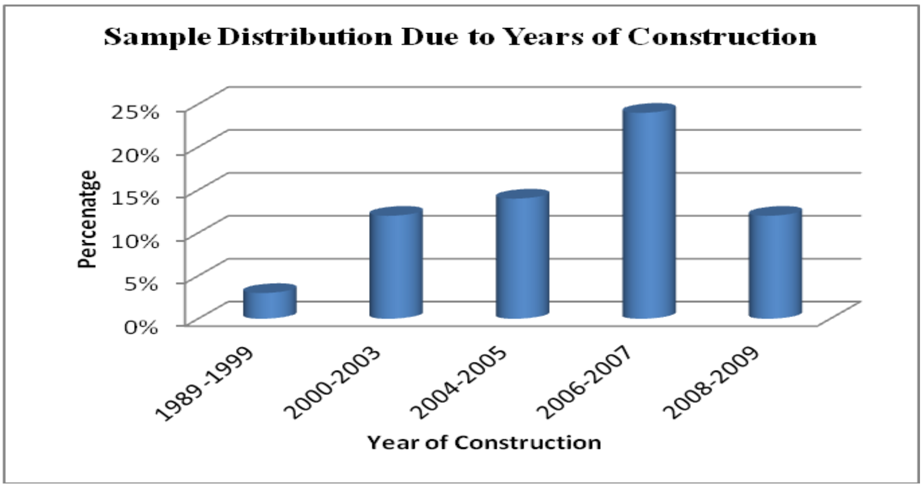


Figure 4.3: Sample distribution due to Year of Construction

The distribution of onsite wastewater treatment plants due to their year of construction are shown in figure 4.3. The results indicate that the highest percentage of distribution due to year of construction of plants were between the period of 2006-2007 which estimated at 24%.

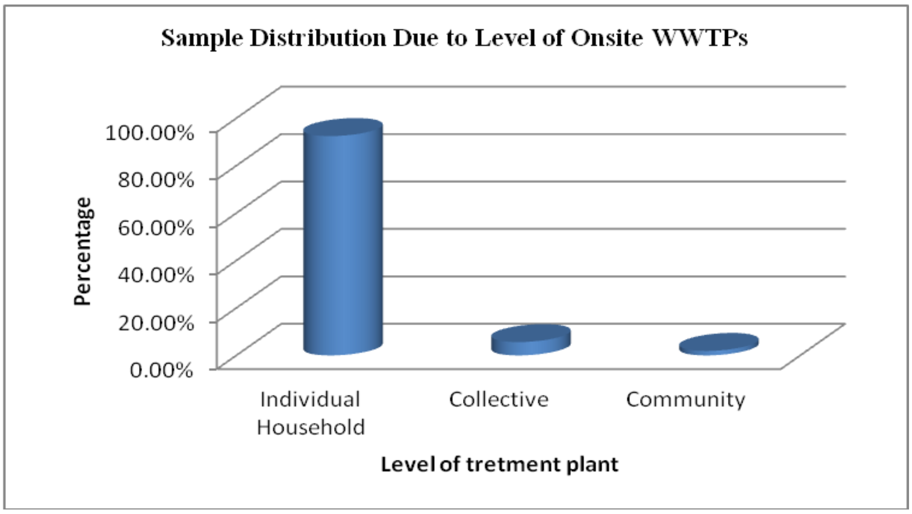


Figure 4.4: Sample Distribution due to Level of Onsite WWTPs

The survey reported that 92.5% of the onsite wastewater treatment plants which were built in Palestinian rural areas are at the household level, while 5.63% is at collective level and only 1.9% of existing onsite wastewater treatment plants is at the community level.

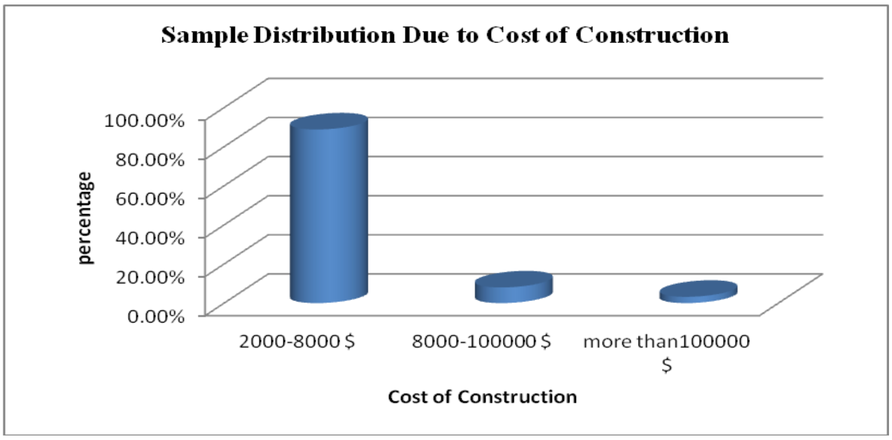


Figure 4.5: Sample Distribution due cost of construction

According to the survey, it could be inferred that the investment for the capital expenditure for the construction of the onsite collective and community systems are much higher than the onsite household systems due to vast scale and required devices. In case of Palestinian rural areas, the study indicates that the capital expenditure for onsite household plants ranging between 2000-8000$, while the largest level of onsite could ranging from 8000-100000$ and more than 100000$ is suitable investment in case of onsite community plants. The total cost of capital expenditure for construction the onsite wastewater treatment plants which have been covered during the questionnaire survey estimated at 1,075,800$, including 354,200$ have been exploited in the plants which have stopped working shortly after its construction and 163,000$ is the cost of the plants which still working well while the remaining amount is estimated 558,600$ is the cost of the plants which need to a real maintenance and rehabilitation.

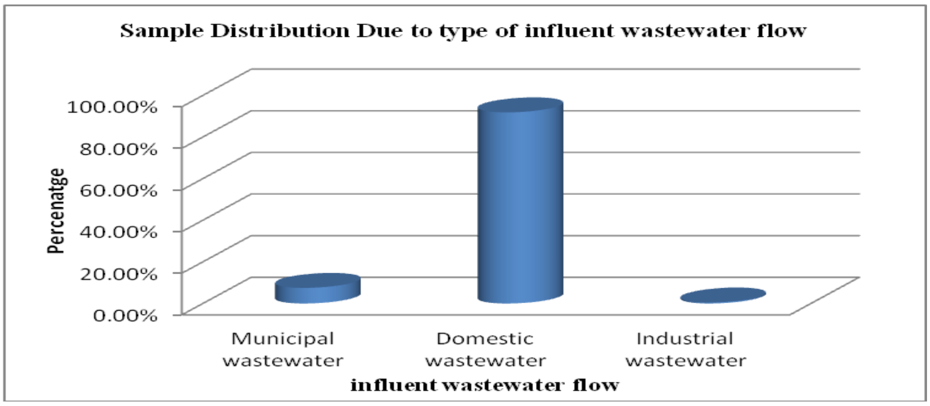


Figure 4.6: Sample Distribution due type of influent wastewater flow

The community and collective levels used municipal wastewater as influent flow estimated to 7.5%. The domestic wastewater is only the component of flow used as influent flow in onsite individual wastewater treatment plants in Palestinian rural areas; just one community onsite plant located in Nuba village is used the industrial wastewater sources which is originating from a mineral water bottling factory and a plastic factory.

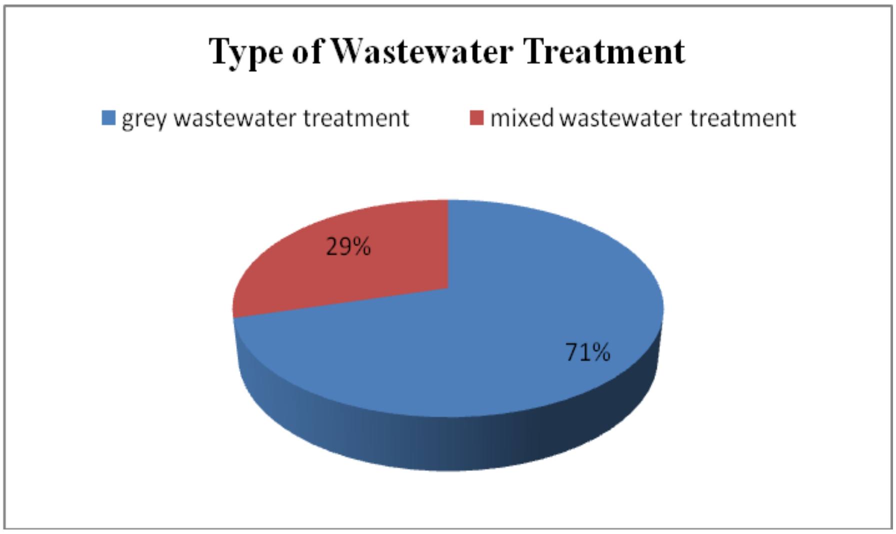


Figure 4.7: Type of Wastewater Treatment

The existing onsite wastewater treatment plants in Palestinian rural areas have designed to treat two types of components of raw domestic wastewater. The first one used grey wastewater as influent design plant which estimated at 71% of existing plants, while 29% of other existing types of technologies used to treat mixed (black) wastewater. As noted, the use of gray wastewater as influent design plant has the highest percentage. The main reason of that and as referred in previous studies, most people show their acceptance for treating grey wastewater in order to reusing it for irrigation purposes more than reusing of the treated mixed wastewater.

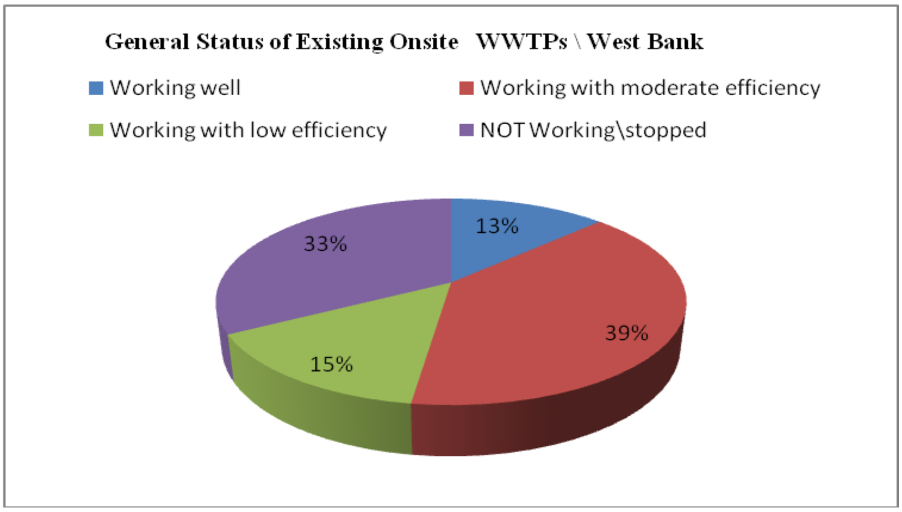


Figure 4.8: General Status of Existing Onsite WWTPs \ West Bank

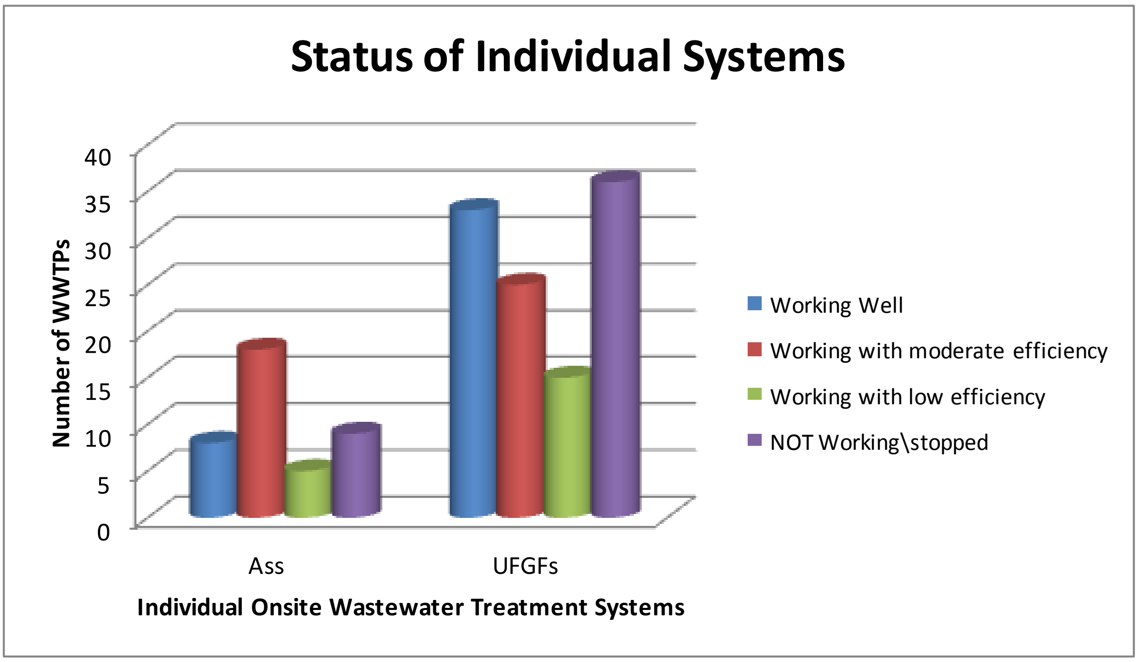
The general status or functional condition of existing onsite WWTP assess is used to be identified by working well, working with moderate efficiency, working with low efficiency and not working\stopped, where

Working Well; the plant, which is operated properly and provided appropriate influent, to produce irrigation quality water.

Working with Moderate Efficiency; the plant, which is operated with less efficient than ever before and having some significant effect on performance due to the functioning of wastewater treatment processes and the purity of irrigation quality water but provided appropriate influent.

Working with Low Efficiency; the plant, which is has a serious deterioration on its performance due to leakage or other problems, which could lead to effect on life of plant exceeded; and requires significant maintenance to remain operational.

Not Working\Stopped; the plant, which is stopped due to firm malfunction. No effluent or influent is present.



Accordingly, 13% of the existing onsite wastewater treatment plants in Palestinian rural areas which are working well, while 39% working with moderate efficiency, the plants which work with less efficiency estimated as much as 15%, whilst the rest of the plants had been stopped. In addition, 10% of the onsite household activated sludge systems were stopped because of operational expenditure in terms of high consumption in the electricity for each plant which estimated 30 Nis per month, which lead bothering the owners of the plants and led them to stop these plants by themselves. While the second technology of the onsite household up-flow gravel filter systems consume the same amount for operating expenditure, but no one expressed his disturbed of these expenses.

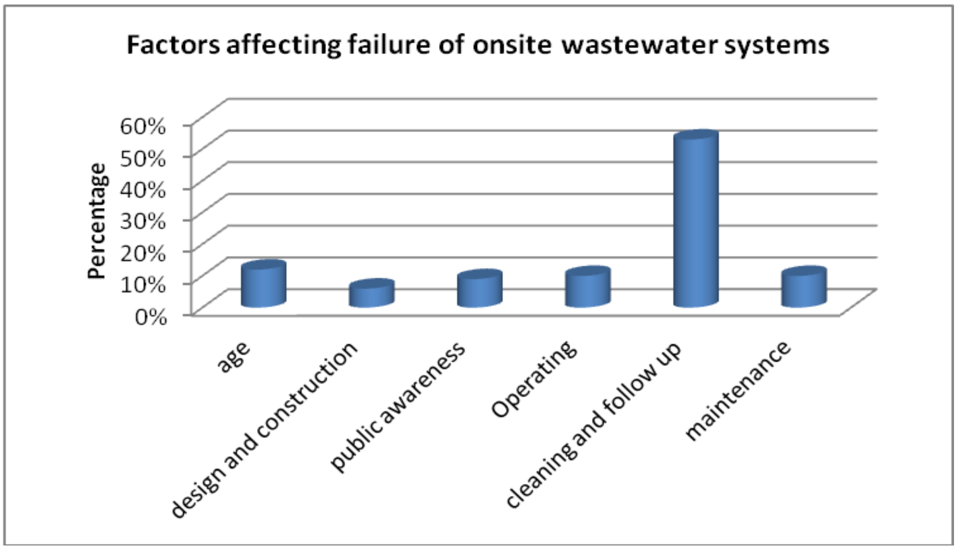


Figure 4.9: Factors affecting failure of Existing Onsite WWTPs \ West Bank

According to the general status of existing monitoring plants, the periodic follow up of operation is the main factor that may affect the failure of onsite wastewater treatment plants, where 53% of the owners of the plants confirmed their dissatisfaction from the periodic cleaning and follow-up operation of the plant in terms of the removal of the scum layer which formed and accumulated sludge on the septic Tank. It seems clear there was a lack of awareness about follow-up operation of the plants by their owners, where it noticed as a good number of these owners were adding fertilizer either on the influent or effluent flow in the treatment plant, as they thought that helps for increasing the efficiency of treated wastewater for crops irrigation.

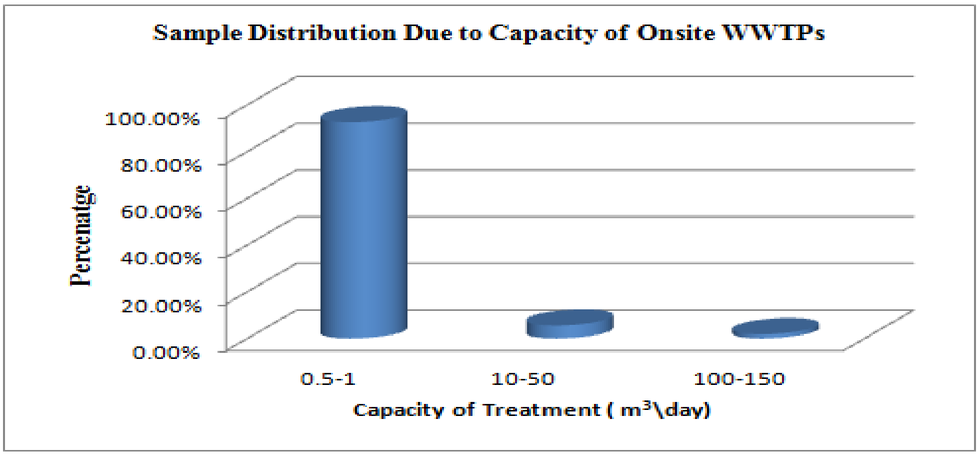


Figure 4.10: Sample Distribution due Capacity of Onsite WWTPs

It was observed that, the household onsite wastewater treatment plants which have been implemented in rural Palestine designed to treat a capacity of wastewater ranged from 0.5 to 1 m^3\day, while the collective onsite wastewater treatment plants designed to treat a capacity of wastewater ranged from 10 to 50 m^3\day. On the other hand, the result shows that the community onsite wastewater treatment plants which have been implemented designed to treat a capacity of wastewater ranged from 100 to 150 m^3\day. It is worth mentioning that most of community and collective onsite wastewater treatment plants are now overloaded.

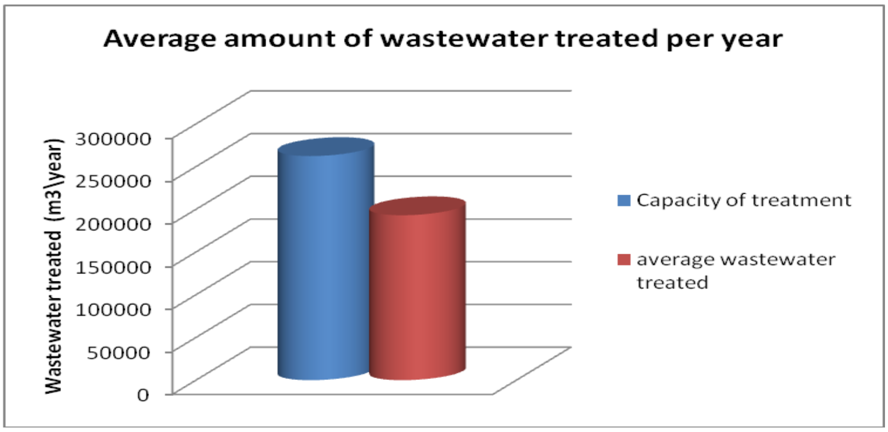


Figure 4.11: Average amount of Wastewater treated per Year

It is worth mentioning the wastewater treatment design capacity was not considered suitable for analysis or comparison because there are some of plants consider overloaded or stopped. The different sizes of onsite wastewater treatment plants are also reflected in the annual influent, where the total average of design capacity of all onsite wastewater treatment plants ranging approximately 262398.5 m^3/Yr. the average wastewater which is treated yearly estimated at 193122.3 m^3/Yr which has been calculating by collecting the water bills from the owners of the plants. This means the questionnaires cover onsite wastewater treatment plants treating influent ranging from approximately 193122.3 m^3/Yr. But that does not mean this amount of treated wastewater is considered suitable for agricultural purposes, most plants have proven their ineffective performance in the treatment.

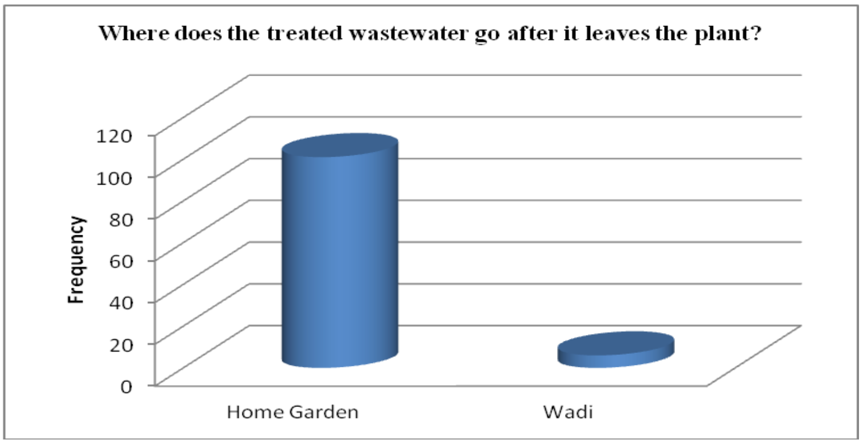


Figure 4.12: where does the treated wastewater go after it leaves the plant?

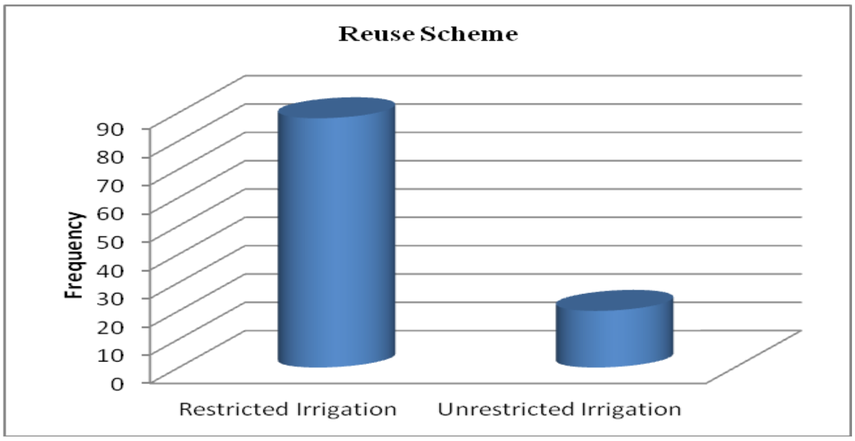


Figure 4.13: Reuse Scheme

The survey showed that around 88% of treated wastewater is reused for restricted irrigation such as citrus, cherries, lemon, apricot and plum, while the rest of the treatment plants are reused for unrestricted irrigation such as vegetables, vineyards, and crops. Despite of the large numbers of gray onsite household wastewater treatment plants which have been covered by the study, but it should be mentioned that according to the WHO guidelines for the use of reclaimed wastewater, effluent coming from the gray onsite plants is suitable for unrestricted irrigation. However, most of people were oriented to use reclaimed wastewater for restricted irrigation only. This is mainly due to the fact that the psychologically unacceptable of use reclaimed wastewater for unrestricted irrigation regardless of its quality, and also to minimize potential health risks that could arise due to improper operational manners.

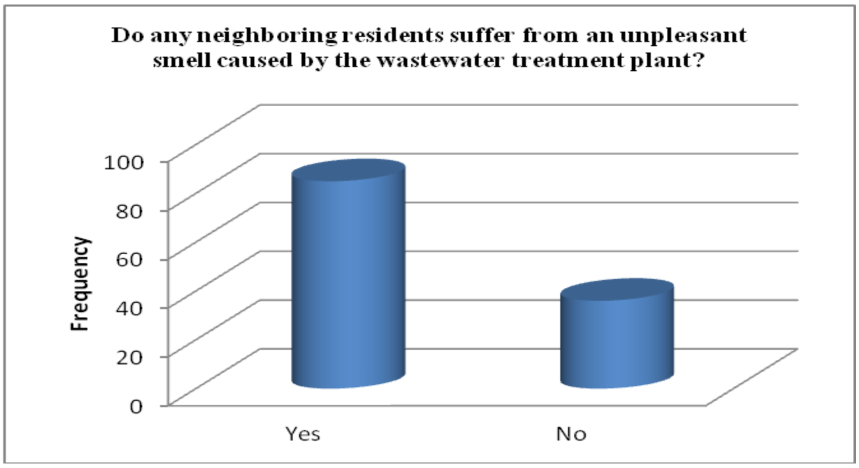


Figure 4.14: Presence of unpleasant smell caused by the wastewater treatment plant

The operation of different technologies of onsite wastewater treatment plants causing an emission of objectionable odors which release from various wastewater treatment processes which in turn results to complaints from owners and neighbors. Depending on the observation it was found that about 85% of neighboring residents near the onsite treatment plants are suffering from unpleasant smell caused by these plants. The rest of responders did not notice any emission of unpleasant odors from their onsite plants during the operating. It is worth mentioning that it was observed that those most who was their answer is no, are the owners of the onsite household activated sludge system.

4.2.2 Wastewater Treatment Information

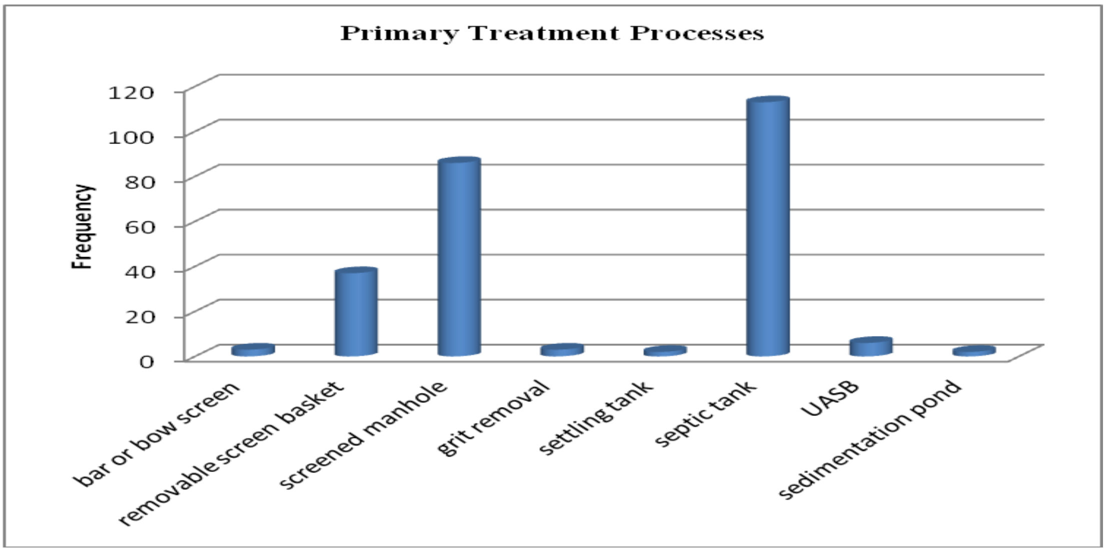


Figure 4.15: Primary Treatment Processes

The first step in wastewater treatment involves separating pieces of debris when the wastewater first enters the treatment plant by using screening or grit removal or etc. This is referred to as pre-treament. the wastewater then is held in a primary sedimentation tank process which containing either septic tank, imhoff tank, sedimentation basin or etc. in case of rural Palestine the systems which have been used at the different levels of plants for the primary process are shown in the figure 4.15. According to the result of survey the septic tank has been adopted much more than another kind of sedimentation tank estimated at 70.63% in onsite rural Palestine. It has been used removable screen basket followed by small scale Activated sludge systems and screened manhole followed by up-flow gravel filter systems.

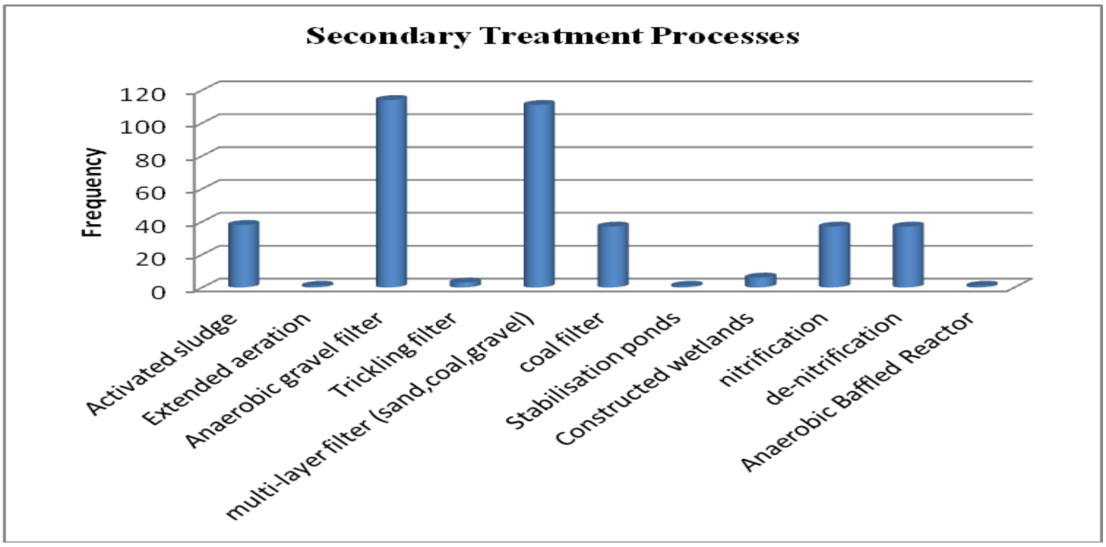


Figure 4.16: Secondary Treatment Processes

Secondary treatment involves biological treatment of wastewater usually following the primary treatment stage. If secondary treatment is the final level of treatment, the clarified wastewater is disinfected and then discharged into the surrounding environment for irrigation purpose. As shown in figure 4.16, the survey revealed that most of the secondary treatment system which has been adopted in case of onsite rural Palestine is the anaerobic gravel filter followed by multi-layer filter of sand, gravel and coal, while small scale household activated sludge occupies the second ranks of the systems which has been adopted in rural Palestine using nitrification and de-nitrification process.

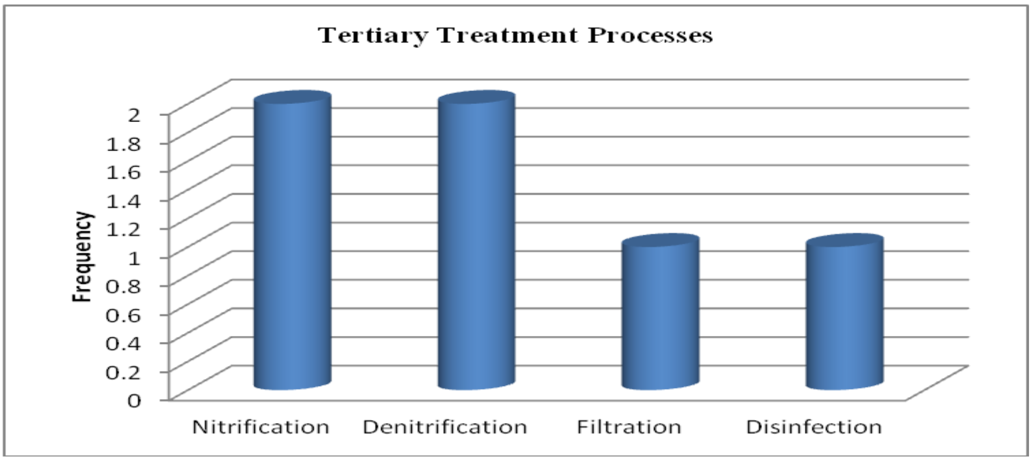


Figure 4.17: Tertiary Treatment Processes

Tertiary treatment of wastewater uses additional processes to further increase the quality of the wastewater effluent. These processes can be physical (filtration), biological, or chemical, based on the substances to be removed. This step further reduces the level of organic chemicals, nutrients, pathogens, and suspended solids in the treated effluent. Tertiary treatment is needed if wastewater must be treated to very high levels. Few plants that were built in rural Palestine used the tertiary treatment, which represents by using the collective onsite activated sludge systems including nitrification, de-nitrification, filtration, and disinfection process.

4.2.3 Control and Monitoring Status of Existing Systems

Little information is available regarding the most critical parameter that may affect the efficiency of the wastewater treatment plant. Depending on the observance of the status of the treatment plants, it can be concluded that the Hydraulic retention time (HRT) in the septic tank which is followed by up-flow gravel filters could be the most critical primary process parameter that may affect the efficiency of the plant. Septic Tanks should design with a sufficient volume for providing a retention time required in order to reserving an adequate volume for sludge storage by sedimentation of the suspended solids. The volume required for sludge storage is the determining factor in sizing the septic tank which depends on the potential occupancy of the dwelling, which can be estimated from the maximum number of people that the house can accommodate. Most of the existing septic tanks are designed to have 48 hours of hydraulic retention time but the problem is when the consumption of water in the targeted household is suffer from fluctuates of wastewater generation according to interruption of water supply lead to disrupted on the operation of plant and that affect on the hydraulic retention time which in turn affect the efficiency of treatment.

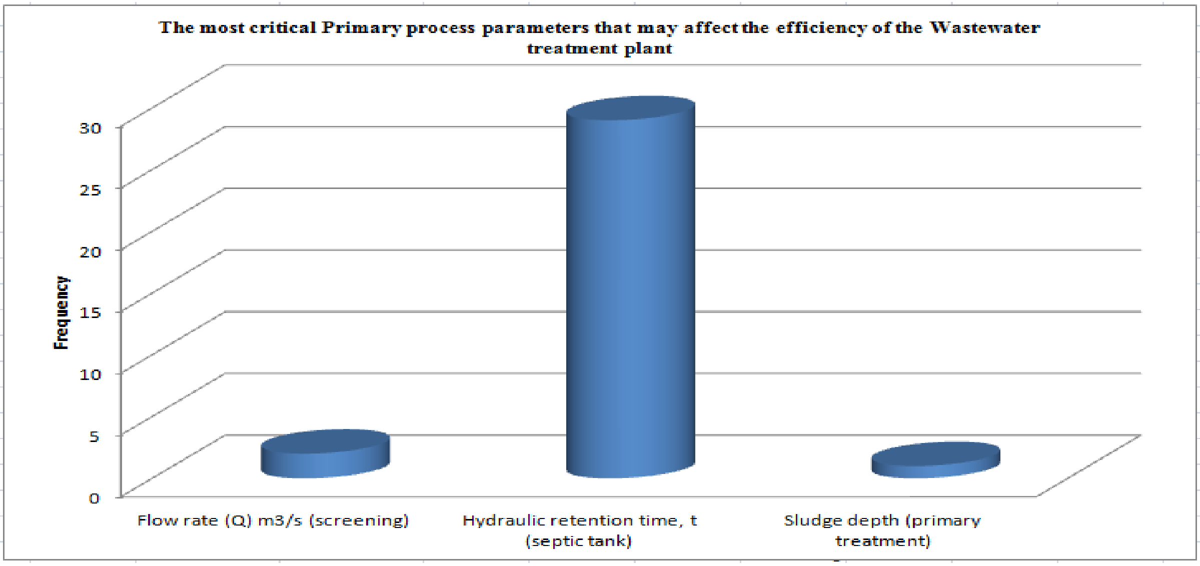


Figure 4.18: The most critical primary process parameters that may affect the efficiency of the wastewater treatment plant.

On the other hand, and depending on the observance of the status of the treatment plants, it can be concluded that the organic loading rate in the up flow gravel filter and multi-layer filter of sand, coal and gravel could be the most critical secondary process parameter that may affect the efficiency of the plant, where using this type of filters for a long period leads to wear or clogging of gravel, sand and coal, due to organic materials that pass through, therefore its need to be removing, clean, or re-change these components of the filter from time to time estimated at 3-5 years. In addition, the evaluated activated sludge systems reported that the sludge loading rate is the most critical parameter that may affect the efficiency of the treatment plants due to the interruption of water consumption, in relation with their design values. As such, the aeration of air lift pump could probably not provide the amount of oxygen required by the stabilization of organic matter and, in addition, the increasing in HRT resulting from the decreased flow rate which may affect the organic matter oxidation, TSS solids separation and consequently affect the value of the solids retention time (SRT).

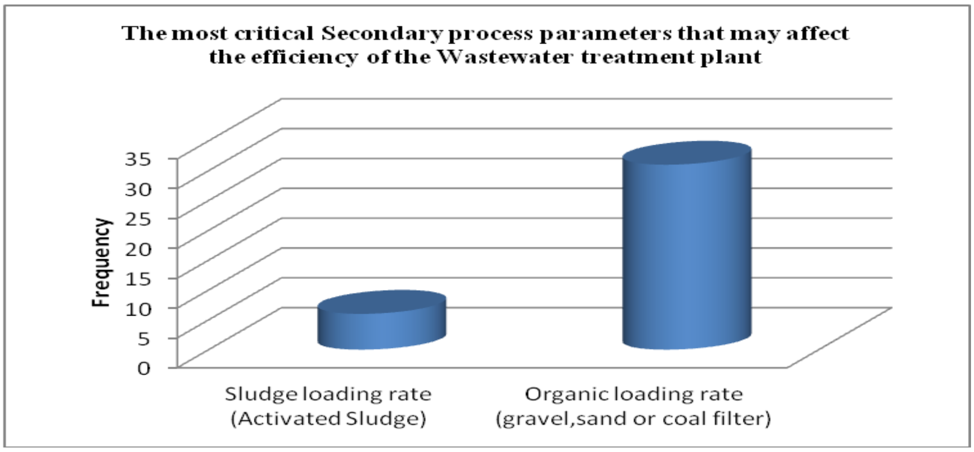


Figure 4.19: The most critical secondary process parameters that may affect the efficiency of the wastewater treatment plant.

From this point of view, the situation is not different in the other technologies plants; even they are subjected to the same problem of interruption of water supply, especially in the plants which are located in the middle or southern part of West Bank. This is result from the nature of the event in Palestinian rural region that suffer from lack and sharp decrease of water supply due to the current situation by Israeli occupation.

Table 4.1 Analytical observed results of household onsite level of Activated Sludge plants presented the influent and effluent mixed wastewater concentrations and removal efficiencies (%). Except at AS$_{B.O}$ plant presented the influent and effluent grey wastewater concentrations. All parameters are in (mg/l), except pH (no unit) and FC/TC (log).

Parameter	Sample #	AS$_{.B.O.}$ * Concentration			AS$_{.B.}$ * concentration			AS$_{.H.}$ * concentration			AS$_{.N.}$ * concentration		
		Influent Value**	Effluent Value**	Removal Efficiency (%)	Influent Value**	Effluent Value**	Removal Efficiency (%)	Influent Value**	Effluent Value**	Removal Efficiency (%)	Influent Value**	Effluent Value**	Removal Efficiency (%)
pH	3	8.74±0.95	8.53±0.69	-	9.19±1.23	9.23±0.99	-	9.34±0.74	9.34±0.43	-	9.52±0.45	9.66±0.39	-
BOD (mg/L)	3	60.60±18.65	17.40±9.33	71.30±5.03	322.8±100.5	282.6±50.4	12.50±8.83	400.2±75.95	89±32.3	77.8±3.28	540±65.33	94±87.98	82.6±12.65
COD (mg/L)	3	62.4±17.88	18.74±12.2	69.9±8.44	756±96.99	569±47.97	24.74±2.92	774±105.5	106±44.64	86.3±3.52	759±123.45	195±109.9	74.31±8.86
TS (mg/L)	3	938±64.99	486±25.92	48.19±5.94	2560±260	1657±100	35.27±2.53	1669±187.67	857±91.29	48.56±1.59	4007±210.8	2113.7±162.8	47.25±0.25
TSS (mg/L)	3	244±53	50±20.23	79.5±2.89	1507±200.3	1000±101.1	33.6±0.24	909±112.2	150±34.99	83.5±1.59	2520±108.8	1097.7±91.7	56.`45±1.69
TDS (mg/L)	3	496±12.3	436±6.45	12.1±0.85	655±68	657±15	-0.31±10.1	610±75.56	707±56.3	-15.9±11.34	1002±102	1016±71.1	-1.4±1.53
EC (µs)	3	852±23.32	879±13.33	-3.2±1.12	1309±120	1314±39.9	-0.4±5.26	1220±132.93	1406±89.32	-15.25±10.52	2008±204	2030±124	-1.1±2.68
PO4 (mg\L)	3	8.71±2.45	2.6±1.5	70.15±6.89	165.31±34	72.92±19.01	55.9±19.05	45.6±17.3	15.09±5.34	66.91±1.66	24.51±12.91	20.23±9.33	17.5±3.51
SO4 (mg\L)	3	2.2±0.94	1.5±0.59	31.82±25.93	89.6±21	62.81±13	29.9±1.56	38.31±12.5	24.64±4.96	35.7±6.21	59.9±20.3	51.24±18.98	14.5±4.04
NKj (mg\l)	3	40±11.2	13.4±3.5	66.5±5.65	57.12±7.45	47.04±4.73	17.65±2.83	52.64±5.5	33±7.43	37.31±8.45	93.8±33.4	43.8±13.53	53.3±3.42
NH4 (mg\L)	3	33±4.48	2.7±2.55	91.82±10.23	22.3±4.3	13.55±3.34	39.24±4.04	12.6±1.99	3.7±1.05	70.6±2.28	45.2±12.77	30.94±7.87	31.55±2.69
NO3 (mg\L)	3	6.7±2.32	10.1±0.59	-	14.61±2.22	10.41±0.99	-	31.01±4.59	17.5±3.44	-	33.5±3.79	11.9±2.78	-
TC (Log)	3	5.3±0.51	3.9±0.39	1.8±0.22	6.2±0.43	4.9±0.33	1.12±0.41	6.5±0.43	5.8±0.44	1.4±0.36	6.5±0.68	4.15±0.61	1.61±0.66
FC (Log)	3	5.1±0.43	3.5±0.31	1.9±0.12	5±0.69	4.3±0.45	1.3±0.39	5.4±0.62	4.8±0.36	1.5±0.27	5.5±0.53	4.3±0.32	1.71±0.43

* AS$_{.B.O.}$: Activated Sludge plant located in Beit Omer Village
AS$_{.B.}$: Activated Sludge plant located in Battir Village
AS$_{.H.}$: Activated Sludge plant located in Halhul Village
AS$_{.N.}$: Activated Sludge plant located in Nahhalin Village
** Value = (Average ±Standard Deviation)

Table 4.2 analytical observed results of household onsite level of Septc Tank - Up-flow Gravel Filter plants presented the influent and effluent grey wastewater concentrations and removal efficiencies (%). All parameters are in (mg/l), except pH (no unit) and FC/TC (log).

Parameter	Sample #	UFGF.Q * concentration			UFGF.M.Q * concentration			UFGF.B.L * concentration			UFGF.B.A * concentration		
		Influent Value**	Effluent Value**	Removal Efficiency (%)	Influent Value**	Effluent Value**	Removal Efficiency (%)	Influent Value**	Effluent Value**	Removal Efficiency (%)	Influent Value**	Effluent Value**	Removal Efficiency (%)
pH	3	7.93±0.39	8.6±0.29	-	7.01±0.82	8.64±0.7	-	9.38±0.18	9.41±0.03	-	8.02±0.61	9.14±0.21	-
BOD (mg/l)	3	172.8±54.3	68±10.5	60.65±4.78	218.4±92.6	177.9±45.8	40.23±3.08	184.5±67	51.7±30	71.98±4.5	296.4±115.2	91.2±66	69.2±7.4
COD (mg/l)	3	323.5±106.1	111.5±45.8	65.53±2.15	389.5±130	349.2±88.7	49.81±12.4	268.3±110	107.7±67	59.9±6.1	391±200	184.25±120	52.9±4.4
TS (mg/l)	3	912±131.11	767±59.2	15.9±4.89	2802±300	1041±140.5	62.85±0.94	654±170	494.2±96.4	24.43±18.8	4624±1145	906±176.6	80.41±6.95
TSS (mg/l)	3	116±90.67	40±35.6	65.52±2.1	742±120	296±60.45	60.11±1.47	122±66.6	20.2±19.9	83.44±13.4	3958±1010	121±99.9	96.9±1.35
TDS (mg/l)	3	796±40.44	727±23.6	8.7±1.56	1029±180	969±80	5.83±7.41	532±103.4	474±76.5	10.9±9.7	666±135	785±76.7	-17.9±10.3
EC (µs)	3	1592±23.3	1450±20.1	8.92±0.33	2060±93	1941±100	5.8±5.2	1065±123	946±74.6	11.2±2.89	1376±177.5	1893±99.99	-37.6±9.9
PO4 (mg\L)	3	11.8±4.5	8.1±2.9	31.4±1.12	8.2±2.6	6.93±1.7	15.5±4.59	1.94±0.99	1.01±0.32	47.94±6.67	11.3±1.1	6.41±0.6	43.3±0.17
SO4 (mg\L)	3	22.1±6.6	17.72±4.3	19.5±3.78	37.98±3.99	16.8±3.1	55.8±3.21	37.19±17.5	33.93±10.4	8.8±2.5	39.62±20.6	33.48±16.4	14.7±2.3
NKj (mg\l)	3	17.92±2.4	19.04±4.6	-6.25±9.99	39.2±9.2	26.9±6.1	31.4±3.9	22±7.89	11±5.4	50±4.87	48±2.3	35±1.33	27.1±5.78
NH4 (mg\L)	3	9.3±2.3	1.12±1.02	87.95±6.39	37.19±6.9	22.3±4.4	54.83±2.69	16.7±3.4	1.43±0.7	91.44±2.04	9.4±0.9	0.5±0.05	94.7±0.93
NO3 (mg\L)	3	0.8±0.03	17.3±2.22	-	1.05±0.5	3.3±1.5	-	3.92±1.2	9.8±0.9	-	17.1±2.33	13.3±1.05	-
TC (Log)	3	5.61±0.55	3.62±0.35	1.78±0.42	TMTC	6.3±0.65	-	3.9±0.32	3.7±0.29	1.2±0.24	3.7±0.29	3.3±0.23	1.34±0.22
FC (Log)	3	3.9±0.30	3.2±0.23	1.7±0.24	TMTC	5.6±0.54	-	3.6±0.23	3.3±0.22	1.3±0.20	2.8±0.25	2.6±0.20	1.32±0.20

* $UFGF_{.Q.}$: Household onsite upflow gravel filter plant located in Qebia Village
$UFGF._{M.Q.}$: Collective onsite upflow gravel filter upflow gravel filter plant located in Al Mazr'a Al gharbiya Village
$UFGF_{.B.L..}$: Household onsite upflow gravel filter plant located in Beit Leed Village
$UFGF_{.B..A}$: Household onsite upflow gravel filter plant located in Beit Anan Village
** Value = (Average ±Standard Deviation)
TMTC = Too Many To Count.

Table 4.2 analytical observed results of household onsite level of Activated Sludge plants presented the influent and effluent grey wastewater concentrations and removal efficiencies (%). All parameters are in (mg/l), except pH (no unit) and FC/TC (log). (Continue)

Parameter	Sample #	UFGF.S. * concentration			UFGF.B.S * Concentration		
		Influent Value*	Effluent Value*	Removal Efficiency (%)	Influent Value*	Effluent Value*	Removal Efficiency (%)
pH	3	9.37±1.1	9.36±0.99	-	9±2.8	9.07±0.4	-
BOD (mg/l)	3	293±35.69	149±23.45	49.2±1.67	114±29.7	54.6±11	52.11±2.24
COD (mg/l)	3	551.5±43.78	314.2±24.4	43.03±8.29	359.5±12	122±13	66.1±2.44
TS (mg/l)	3	1121±582	1465±472	-30.69±16.95	1046±144.34	1002±91.23	4.21±3.95
TSS (mg/l)	3	600±130	920±60	-53.3±19.05	144±35.34	68±13.23	52.84±1.87
TDS (mg/l)	3	521±452	545±412	-4.61±2.97	902±109	934±79	-3.55±1.56
EC (µs)	3	1043±982	1087±934	-4.22±4.02	1807±846	1865±187	-3.21±12.43
PO4 (mg\L)	3	3.92±2.34	6.82±0.44	-73.98±58.01	1.94±0.95	1.6±0.56	17.53±7.73
SO4 (mg\L)	3	10.9±3.88	34.3±4.58	-86.24±76.82	14.6±4.34	6.7±2.84	54.11±4.48
NKj (mg\l)	3	45±15.67	40±13.36	11.11±0.94	43.7±20.23	38.1±12.47	12.81±8.09
NH4 (mg\L)	3	24.5±10.3	14.8±6.78	39.6±1.62	30±16.88	1.8±0.98	94±0.07
NO3 (mg\L)	3	16.3±5.78	19.3±3.65	-	0.8±0.56	13.7±1.36	-
TC (Log)	3	3.8±0.65	3.7±0.53	1.2±0.60	3.9±0.43	3.3±0.39	1.6±0.40
FC (Log)	3	`3.3±0.62	3.1±0.43	1.4±0.45	3.5±0.41	2.5±0.30	1.7±0.33

* UFGF.S. : Household onsite upflow gravel filter upflow gravel filter plant located in Sanur Village
UFGF.B.S.: Household onsite upflow gravel filter upflow gravel filter plant located in Beit Sira Village
** Value = (Average ±Standard Deviation)

Table 4.3 analytical observed results of collective and community onsite level of wastewater treatment plants presented the influent and effluent mixed wastewater concentrations and removal efficiencies (%). All parameters are in (mg/l), except pH (no unit).

Parameter	Sample #	DWP.$_{Ar.}$ * Concentration			EAP.$_{N.}$ * concentration			CW.$_{Nu.}$ * concentration			UFGF.$_{Sr.}$ * concentration		
		Influent Value**	Effluent Value**	Removal Efficiency (%)	Influent Value**	Effluent Value**	Removal Efficiency (%)	Influent Value**	Effluent Value**	Removal Efficiency (%)	Influent Value**	Effluent Value**	Removal Efficiency (%)
pH	2	7.2±0.12	7.3±0.10	-	7.3±0.5	7.5±1.02	-	7.7±0.1	7.1±0.02	-	7.9 ±0.55	8±0.13	-
BOD (mg/L)	2	203±42.54	92±20.33	54.68±2.13	309±99.5	145±30.2	53.1±13.55	197±25.55	112±12.3	43.15±8.37	970±55.45	315±20.18	66.37±8.83
COD (mg/L)	2	638±69.08	333±32.3	47.81±5.43	842±77.89	308±37.87	63.4±11.52	423±19.5	233±8.44	44.92±9.32	1799±133.4	605±89.9	67.53±3.66
TS (mg/L)	2	1039±12.91	909±9.93	12.5±9.98	3130±760	1363±130	56.5±18.39	1468±11.7	1043±6.27	28.95±11.04	1848±200	991±89.8	46.37±8.75
TSS (mg/L)	2	102±4.19	56±3.24	45.1±12.32	721±210.5	189±44.05	73.8±4.99	376±3.21	256±2.09	31.9±3.84	705±101	80±20	88.65±8.19
TDS (mg/L)	2	932±8.2	845±5.33	9.33±5.25	2387±544.9	1145±85.5	52.03±22.10	1084±7.56	787±4.01	27.4±7.37	1142±99.5	911±69.8	20.23±9.83
EC (µs)	2	1866±16.01	1690±9.03	9.43±6.10	4774±1055	2290±169	52.03±22.22	2168±15.3	1575±4.2	27.4±8.52	2288±205	1820±122	20.45±9.62

* DWP.$_{Ar.}$: Collective onsite Duckweed-based pond system located in Al Aroub agriculture school
EAP.$_{N.}$: Collective onsite consists of Extended Aeration Process – Chlorine Disinfection and Sand Filtration plant located in Nahhalin Village
CW.$_{Nu...}$: Community onsite Up-flow Anaerobic Sludge Blanket following by Horizontal Flow Constructed Wetlands plant located in Nuba Village
UFGF.$_{Sr.}$: Collective onsite consists of aerobic and anaerobic gravel filters followed by polishing sand filters plant located in Seer Village
** Value = (Average ±Standard Deviation)

4.3 Technical/Analytical Performance Evaluation of existing wastewater treatment systems
4.3.1 Influent (mixed and gray) wastewater Characteristics

As a rule, the influents of mixed and grey wastewater have very different characteristics this is due to the difference of components of each of them. It has noticed during the laboratory analysis that the grey wastewater generation in the targeted onsite grey wastewater treatment plants fluctuates from household to another according to the indoor activities.

The average values of the influent wastewater characteristics associated with the different monitored household, collective and community onsite systems are presented in tables 4.1, 4.2 and 4.3. The maximum and minimum average influent values of BOD_5, COD, TSS, NKj and TC of the total generated grey wastewater were (296.4, 114) mg/L at (UFGF.$_{B.A}$, UFGF.$_{B.S}$), (551.5, 268.3) mg/L at (UFGF.$_{S}$, UFGF.$_{B.L}$), (3958, 116) mg/L at (UFGF.$_{B.A}$, UFGF.$_{Q}$), (50, 17.92) mg/L at (UFGF.$_{B.A}$, UFGF.$_{Q}$), and (5.61, 3.7) log at (UFGF.$_{Q}$, UFGF.$_{B.A}$), respectively.

The maximum and minimum average influent values of BOD_5, COD, TSS, NKj and TC of the total generated mixed wastewater were (1799, 60.60) mg/L at (UFGF.$_{Sr}$, AS.$_{B.O}$), (970, 62.4) mg/L at (UFGF.$_{Sr}$, AS.$_{B.O}$), (2520, 244) mg/L at (AS.$_{N}$, DWP.$_{Ar}$.), (93.8, 40) mg/L at (AS.$_{N}$, AS.$_{B.O}$), and (6.5, 4.9) log at (AS.$_{N}$, AS.$_{B}$.), respectively.

It was observed that the influent of grey or mixed wastewater presented an average concentration within or systematically higher than that usually reported in the literature for raw domestic wastewater. The simpler treatment systems, that is, UFGF.$_{B.A}$, showed systematically much higher concentrations for TSS constituent. Possible explanations that could justify the high concentrations of raw wastewater treated by these processes even for grey or mixed wastewater could be: the behaviors of household's inhabitants, types and amount of detergent used by households, food style and meals patterns, or low per capita water consumption.

There are no significant variations between the characteristics of the influents of mixed wastewater and the characteristics of the influents of grey wastewater as monitored in the evaluated plants. In addition, high pathogenic counts were found in grey wastewater samples. The interpretation of this case could be via presence of Total and Faecal coliform input from hand washing after defecation and babies washing in hand washing basin were the key factors for this high numbers of E.coli.

4.3.2 Environmental conditions
4.3.2.1 pH

Values of pH for effluent were measured for different household, collective and community systems. The average effluent pH values were found (8.53±0.69), (9.23±0.99), (9.34±0.43) and (9.66±0.39) in the household onsite of AS.$_{B.O.}$, AS.$_{B.}$, AS.$_{H.}$, and AS.$_{N.}$, respectively. And the average effluent pH values were (8.6±0.29), (9.41±0.03), (9.14±0.21), (9.36±0.99) and (9.07±0.4) in the household onsite of UFGF.$_{Q.}$, UFGF.$_{B.L.}$, UFGF.$_{B.A.}$, UFGF.$_{S.}$, and UFGF.$_{B.S.}$, respectively. Also, the measurement average effluent pH values were (7.3±0.10), (8.64±0.7), (7.5±1.02) and (8.00±0.13) in the collective onsite of DWP.$_{Ar.}$, UFGF.$_{M.Q.,}$ EAP.$_{N.}$ and UFGF.$_{Sr}$, respectively. And it was (7.1±0.02) in the community onsite of CW.$_{Nu.}$ system. The results for the pH tests for different household, collective and community systems are shown in figures 4.20, 4.21 and 4.22.

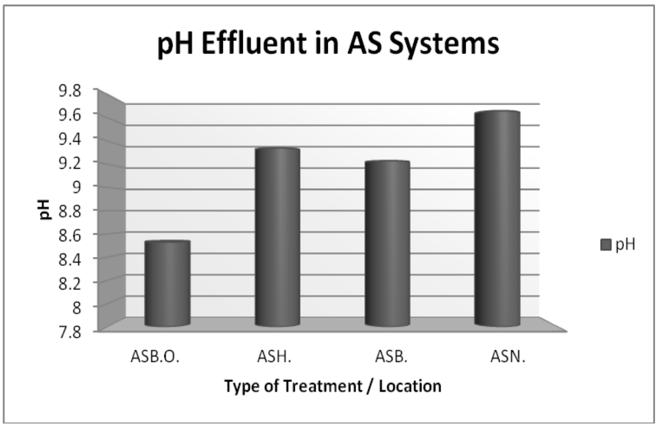


Figure 4.20: pH effluent in the household onsite AS systems

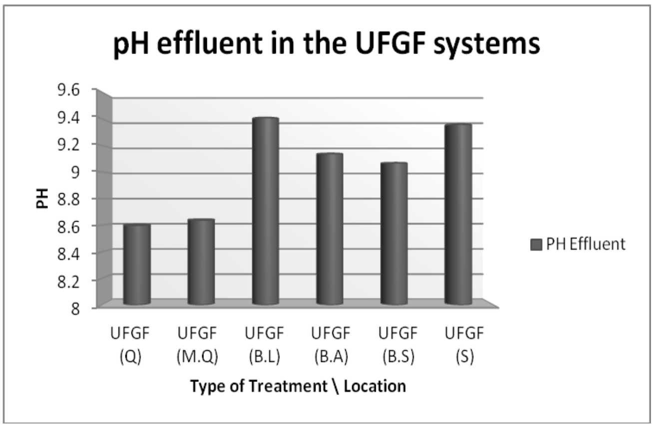


Figure 4.21: pH effluent in the household onsite UFGF systems

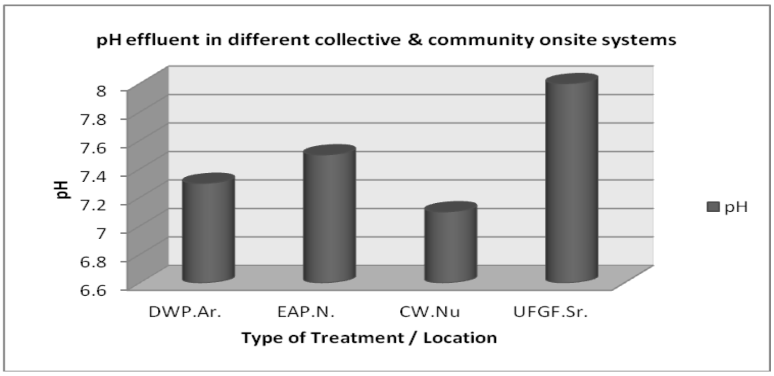


Figure 4.22: pH effluent in different collective onsite of $DWP._{Ar.}$, $EAP._{N.}$, $UFGF._{Sr.}$ and community onsite of $CW._{Nu.}$ Systems

Average values of effluent pH were slightly lower or higher than influent pH. The lowest value of pH was found in the effluent corresponding to $CW._{Nu.}$, which used constructed wetland technology at community level, while the highest values were found in the effluents of $AS._{N}$. which used activated sludge system at household level. According to recommended guidelines by the Palestinian Standards Institute (PSI) for Treated Wastewater Characteristics to pH parameter it should be from 6 to 9 for irrigational purposes (see Table A.4 Appendix-A).

4.3.2.2 Temperature

Temperatures of influent raw grey and mixed wastewater were measured for all different selected systems and levels. Temperature values ranged from 10 - 40 ^{0}C with average temperature values of (24.7 ± 9.95) ^{0}C.

Variation of temperature was due to the fact that 2 samples were taken during summer, and 1 sample was taken during winter for household onsite systems and 1 sample during summer for collective systems, 2 samples during winter for collective and community systems.

For samples taken during summer (from June to August, 2010), mean temperature was (31.8 ± 3.84) ^{0}C.

For samples taken during winter (from November, 2010), mean temperature was (13 ± 2.2) ^{0}C.

4.3.3 Organic removal
4.3.3.1 BOD$_5$ removal efficiency

BOD$_5$ measures the amount of dissolved oxygen required or consumed in five days at a constant temperature for the microbiological decomposition (oxidation) of organic material in wastewater. The average values of the influent to effluent BOD$_5$ concentration and the calculated removal efficiencies of the different monitored household, collective and community onsite systems are shown in tables 4.1, 4.2 and 4.3 and presented in the following Figures from 4.23 to 4.26. The average mixed influent BOD$_5$ concentration were (60.60±18.65), (322.8±100.5), (400.2±75.95) and (540±65.3) mg/l in the household onsite of AS.$_{B.O}$., AS.$_{B.}$, AS.$_{H.}$, and AS.$_{N.}$, respectively. The average grey influent BOD$_5$ concentration were (172.8±54.3), (218.4±92.60), (184.5±67), (296.4±115.2), (293±35.69), and (114±29.7) in the household onsite of UFGF.$_{Q.}$, UFGF.$_{M.Q}$., UFGF.$_{B.L}$., UFGF.$_{B.A.}$, UFGF.$_{S.}$, and UFGF.$_{B.S}$., respectively. While the average effluents BOD$_5$ concentrations were (17.4±9.33), (282.6±50.4), (89±32) and (94±87.98) mg/l in AS.$_{B.O.}$, AS.$_{B.}$, AS.$_{H.}$, and AS.$_{N.}$, respectively. The average effluents BOD$_5$ concentration were (68±10.5), (177.9±45.8), (51.7±30), (91.2±66), (149±23.45), and (54.6±11) in UFGF.$_{Q.}$, UFGF.$_{M.Q.}$, UFGF.$_{B.L}$., UFGF.$_{B.A.}$, UFGF.$_{S.}$, and UFGF.$_{B.S}$., respectively.

The Average BOD$_5$ removal efficiencies % were (71.3±5.03), (12.5±8.83), (77.8±3.28) and (82.60±12.65) mg/l in AS.$_{B.O.}$, AS.$_{B.}$, AS.$_{H.}$, and AS.$_{N.}$, respectively. The average BOD$_5$ removal efficiencies % were (60.65±4.78), (40.23±3.08), (71.98±4.50), (69.2±7.40), (49.2±1.67), and (52.11±2.24) in UFGF.$_{Q.}$, UFGF.$_{M.Q.}$, UFGF.$_{B.L.}$, UFGF.$_{B.A.}$, UFG$_{F.S.}$, and UFGF.$_{B.S.}$, respectively.

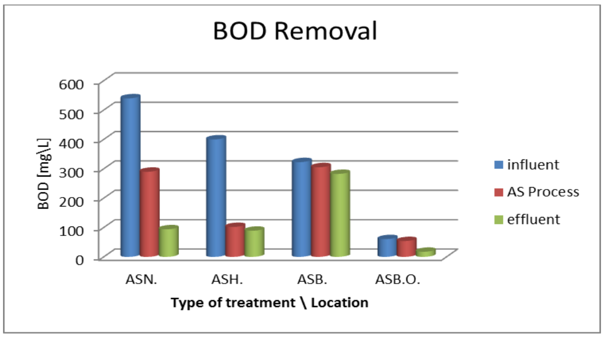


Figure 4.23: BOD$_5$ influent, AS process and effluent (mg/l)

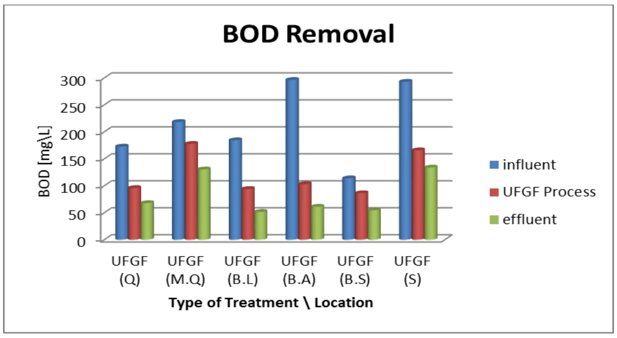


Figure 4.24: BOD$_5$ influent, UFGF process and effluent (mg/l)

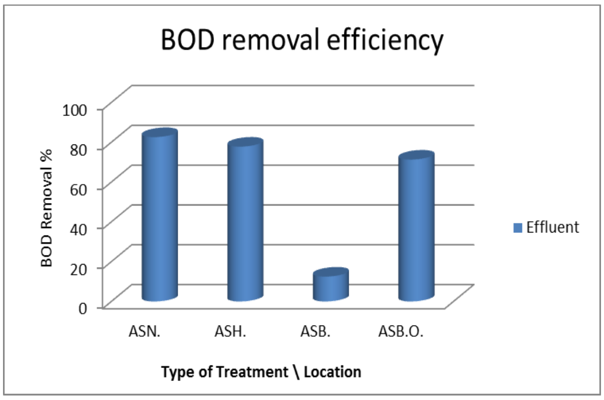


Figure 4.25: BOD$_5$ removal efficiency (%) in AS systems

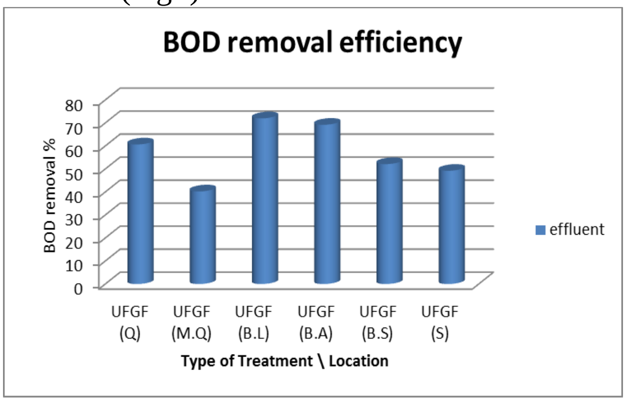


Figure 4.26: BOD$_5$ removal efficiency (%) in UFGF systems

On the other hand, the average influent BOD_5 concentrations were (203±42.54), (309±99.5) and (970±55.45) mg/l in the collective onsite of DWP.$_{Ar.}$, EAP.$_{N.}$ and UFGF.$_{Sr}$, respectively. And it was (197±25.55) in the community onsite of CW.$_{Nu.}$ While the average effluents BOD_5 concentrations were (92±20.33), (145±30.2) and (315±20.18) in the collective onsite of DWP.$_{Ar.}$, EAP.$_{N.}$ and UFGF.$_{Sr}$, respectively. The average effluents BOD_5 concentration was (112±12.3) in the community onsite of CW.$_{Nu}$. The Average BOD_5 removal efficiencies were (54.68±2.13), (53.1±13.55) and (67.53±3.66) in the collective onsite of DWP.$_{Ar.}$, EAP.$_{N.}$ and UFGF.$_{Sr}$, respectively. The Average BOD_5 removal efficiencies was (43.15±8.37) in the community onsite of CW.$_{Nu}$.

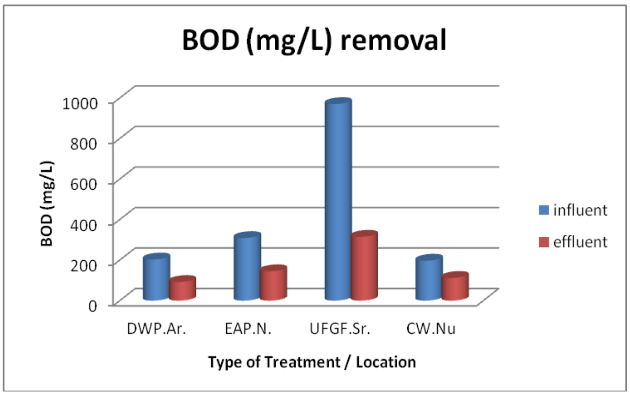


Figure 4.27: BOD_5 influent, and effluent (mg/l) in DWP.$_{Ar.}$, EAP.$_{N.}$, UFGF.$_{Sr}$ and CW.$_{Nu.}$ systems

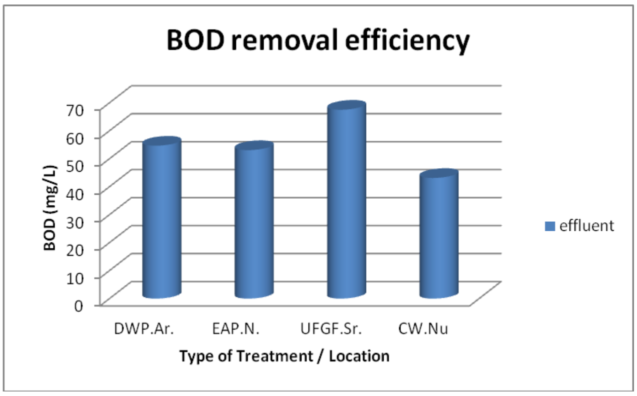


Figure 4.28: BOD_5 removal efficiency (%) in DWP.$_{Ar.}$, EAP.$_{N.}$, UFGF.$_{Sr}$ and CW.$_{Nu.}$ systems

The concentration of BOD_5 in the effluents decreased significantly compared to the average concentration of BOD_5 in the influents of all plants except for AS.$_{B.}$. The lowest BOD_5 concentrations being in the effluents of AS.$_{B.O}$. The efficiency of BOD_5 removal was higher than 80% in AS.$_{N.}$, while it was lower than 60% in AS.$_{B}$, UFGF.$_{M.Q}$, UFGF.$_{S}$, UFGF.$_{B.S.}$, DWP.$_{Ar}$, EAP.$_{N.}$, and CW.$_{Nu.}$.

It should be mentioned there, DO concentrations were observed higher than 3 mg/l in the effluents of UFGF.$_{Q}$; DO values in the range of 1–2 mg/l were found in the effluents of UFGF.$_{B.L}$, and UFGF.$_{B..A}$ and, finally, values lower than 1 mg/l were encountered in the effluents of AS.$_{B.O.}$, AS.$_{B.}$, AS.$_{H.}$, AS.$_{N}$, UFGF.$_{M.Q}$, UFGF.$_{S.}$, and UFGF.$_{B.S.}$

Most of the household onsite ST-UFGF-SF systems had a lower performance compared with the reference BOD_5 range reported in the literature (Table 3.4), considering both average BOD_5 effluent concentrations and BOD_5 removal efficiencies. This low performance was observed for most ST-UFGF-SF evaluated plants, except for UFGF.$_{Q}$, UFGF.$_{B.L,}$ UFGF.$_{B.A}$ and UFGF.$_{B.S}$, which presented average BOD_5 effluent concentrations within the expected range. The Average BOD_5 removal efficiencies % at the effluents of ST-UFGFs were 44.5, 18.5, 48.9, 65.2, 24.2, and 43 in UFGF.$_{Q.}$, UFGF.$_{M.Q.}$, UFGF.$_{B.L.}$, UFGF.$_{B.A.}$, UFGF.$_{S.}$, and UFGF.$_{B.S.}$, respectively. The lowest BOD_5 removal efficiencies appeared at the effluent of ST-UFGF for UFGF.$_{M.Q.}$, the reason of that may explained due to occurred malfunction in submersible pump which preventing and hardly transferred the flow of wastewater from septic tank stage to up-flow gravel filter stage making overloading at septic tank and it was observed the cover of the septic tank has been broken lead to convert it from anaerobic process to aerobic process that if the real treatment done. The Average BOD_5 removal efficiencies % at the effluents of multi-layer filters were 29.2, 26.6, 45.1, 11.6, 10.2, and 36.8 in UFGF.$_{Q.}$, UFGF.$_{M.Q.}$, UFGF.$_{B.L.}$, UFGF.$_{B.A.}$, UFGF$_{.S.}$, and UFGF.$_{B.S.}$, respectively. UFGF.$_{B.L}$ Plant showed the best efficiency in the removal of BOD_5 was 71.9 %. This is perhaps due the recent establishment of this plant even though it was found in the early days of the operation process was discovered a leak of the gray wastewater which entering surrounding the walls of the plant, which required re-restored again to repair the fault.

The household onsite activated sludge (AS) process at most the evaluated plants presented BOD_5 effluent concentration values higher than the reference values excepted at AS.$_{B.O}$. may because the owner of this plant rejected to connect the black wastewater to the plant preferring to connect only gray wastewater to be treated which in turns effect on the performance of the plant. However, considering BOD_5 removal efficiencies, the performance was below the expected for all activated sludge plants. This can be partially explained by the high influent concentrations, which makes the achievement of high removal efficiencies more difficult. The Average BOD_5 removal efficiencies % were 10.9, 5.2, 74.5 and 46.3 mg/l in the effluent of the Aerated zone at AS.$_{B.O.}$, AS.$_{B.}$, AS.$_{H.}$, and AS.$_{N.}$, respectively. The Average BOD_5 removal efficiencies % in the effluent of the settling or separation zone were 74, 7.5, 12.7 and 67.6 mg/l at AS.$_{B.O.}$, AS.$_{B.}$, AS.$_{H.}$, and AS.$_{N.}$, respectively. A good performance of BOD_5 removal achieved by Aerated zone and separation zone appearance in all Activated sludge plants except at AS.$_{B.}$.This may explained by the interruption and low consumption of water by the targeted house of this plant recorded once or twice a week.

The collective onsite UFGF.$_{Sr}$ plant using SF-AUFGF-ATF-PSF system showed good BOD_5 removal efficiency and low performance compared with the reference value reported in the literature. However, the actual BOD_5 effluent concentrations were significantly above the reference value (poor performance). The BOD_5 performance achieved by the collective onsite DWP.$_{Ar}$ using DWBP-BS-AT system was the lowest one comparing with the literature. Unfortunately, no literature data are available related to the performance of EAP-CD-SP system for doing comparison.

AS.$_{B.O}$. has achieved a level of treatment that exceeded the requirement of Palestinian standards PSI for the reclaimed wastewater reuse for irrigation, Where the average AS.$_{B.O}$ effluent value was 17.40 mg/l, while other plants have average values of the effluent slightly higher than 45 mg/l as the requirement of Palestinian standards PSI, but AS.$_{B}$ and UFGF.$_{Sr}$ have average values of the effluent to be highest more than doubled as recommended and that may affect on the characteristics of crops that are irrigated.

4.3.3.2 COD removal efficiency

Chemical oxygen demand (COD) is commonly used to indirectly measure the amount of organic compounds in wastewater by measuring the mass of oxygen needed for their total oxidation to carbon dioxide. COD has to be considered in relation to total suspended solids (TSS) since TSS removal efficiency affects the performance achieved with respect to COD (see section 4.3.3.3).

The average values of the influent to effluent of grey and mixed of COD concentrations and the calculated removal efficiencies of the different evaluated household, collective and community onsite systems are shown in tables 4.1, 4.2 and 4.3 and presented in the following figures from 4.29 to 4.34.

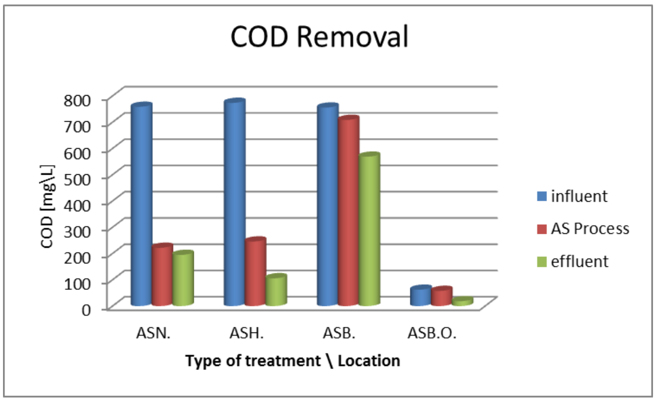


Figure 4.29: COD Influent, AS Process and Effluent (mg/l)

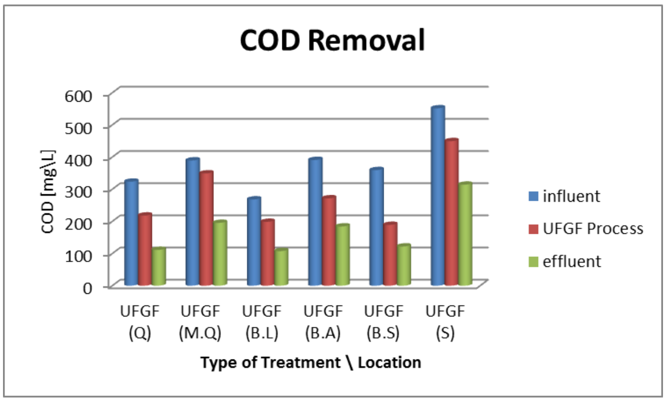


Figure 4.30: COD Influent, UFGF Process and Effluent (mg/l)

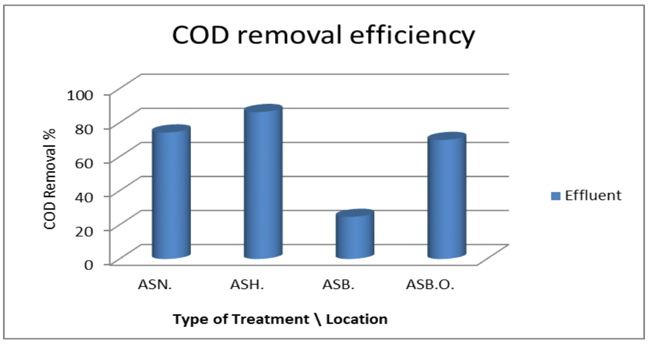


Figure 4.31: COD removal efficiency (%) in AS systems

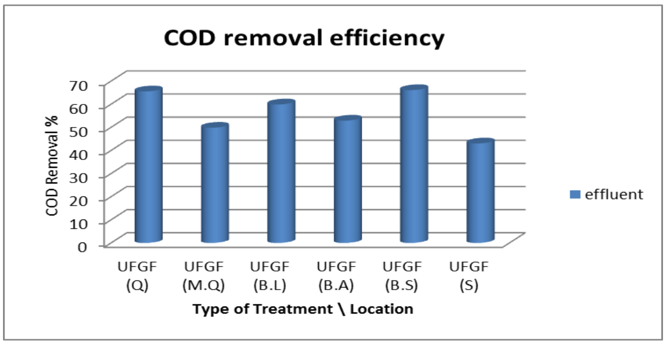


Figure 4.32: COD removal efficiency (%) in UFGF systems

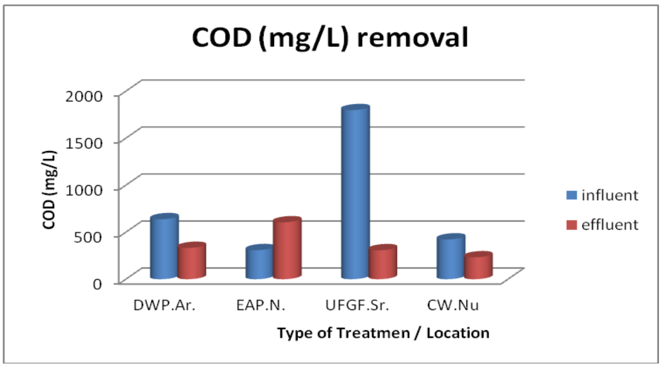


Figure 4.33: COD influent, and effluent (mg/l) in $DWP._{Ar.}$, $EAP._{N.}$, $UFGF._{Sr}$ and $CW._{Nu.}$ systems

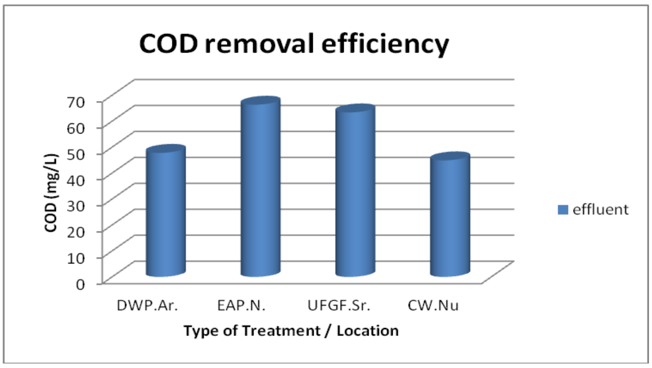


Figure 4.34: COD removal efficiency (%) in $DWP._{Ar.}$, EAP.N., $UFGF._{Sr}$ and $CW._{Nu.}$ systems

The recommended Guidelines by the Palestinian Standards Institute for Treated COD effluent value is 150 mg\l for reuse in irrigation. At $AS._{B}$, $AS._{N}$, $UFGF._{M.Q}$, $UFGF._{B..A}$, $UFGF._{S.}$, $DWP._{Ar}$, $EAP._{N}$, $CW._{Nu.,}$ and $UFGF._{Sr}$ systems, the measured average effluent COD is above 150 mg/l that means these selected evaluated systems are not acceptable for reuse in irrigation. The rest of evaluated plants have average effluent COD is less than 150 mg/l which are suitable for reuse in irrigation.

The ratio BOD_5/COD in the raw effluent cannot be used as an operative parameter for the wastewater treatment, but gives a rough indication of biodegradability. As a rule of thumb, BOD_5/COD ratios before treatment of <0.2 indicate relatively undegradable organic substances, ratios between 0.2 and 0.4 indicate moderately to highly degradable organic substances, and ratios of >0.4 indicate highly degradable organic substances. BOD_5/COD ratios in the grey and mixed influents and effluents of selected monitoring of onsite wastewater treatment plants are shown in Table 4.4-6.

Table 4.4 BOD_5/COD ratios in the influents and effluents mixed wastewater of onsite household Activated sludge systems. Except at $AS_{B.O}$ plant presented the influent and effluent grey wastewater ratios

Parameter	*** $AS._{B.O.}$**	*** $AS._{B}$**	*** $AS._{H}$**	*** $AS._{N}$**
BOD_5/COD ratios in influent	0.97	0.42	0.51	0.71
BOD_5/COD ratios in effluent	0.93	0.49	0.83	0.48

* $AS._{B.O.}$: Activated Sludge plant located in Beit Omer Village
$AS._{B.}$: Activated Sludge plant located in Battir Village
$AS._{H.}$: Activated Sludge plant located in Halhul Village
$AS._{N.}$: Activated Sludge plant located in Nahhalin Village

Table 4.5 BOD_5/COD ratios in the influents and effluents grey wastewater of onsite household up-flow gravel filters systems.

Parameter	***$UFGF._{Q.}$**	***$UFGF._{M.Q}$**	***$UFGF._{B.L}$**	***$UFGF._{B..A}$**	***$UFGF._{S}$**	***$UFGF._{B.S.}$**
BOD_5/COD ratios in influent	0.53	0.56	0.69	0.76	0.53	0.32
BOD_5/COD ratios in effluent	0.48	0.5	0.48	0.49	0.47	0.44

* $UFGF._{Q.}$: Household onsite upflow gravel filter plant located in Qebia Village
$UFGF._{M.Q.}$: Collective onsite upflow gravel filter upflow gravel filter plant located in Al Mazr'a Al gharbiya Village
$UFGF._{B.L..}$: Household onsite upflow gravel filter plant located in Beit Leed Village
$UFGF._{B..A}$: Household onsite upflow gravel filter plant located in Beit Anan Village
$UFGF._{S.}$: Household onsite upflow gravel filter upflow gravel filter plant located in Sanur Village
$UFGF._{B.S.}$: Household onsite upflow gravel filter upflow gravel filter plant located in Beit Sira Village

Table 4.6 BOD_5/COD ratios in the influents and effluents mixed wastewater of onsite collective and community wastewater treatment plants.

Parameter	*** $DWP._{Ar}$**	*** $EAP._{N}$**	*** $CW._{Nu.}$**	*** $UFGF._{Sr}$**
BOD_5/COD ratios in influent	032	0.36	0.46	0.53
BOD_5/COD ratios in effluent	0.27	0.47	0.48	0.52

* $DWP._{Ar.}$: Collective onsite Duckweed-based pond system located in Al Aroub agriculture school
- $EAP._{N.}$: Collective onsite consists of Extended Aeration Process -Chlorine Disinfection and Sand Filtration plant located in Nahhalin Village
- $CW._{Nu...}$: Community onsite Up-flow Anaerobic Sludge Blanket following by Horizontal Flow Constructed Wetlands plant located in Nuba Village
- $UFGF._{Sr.}$: Collective onsite consists of aerobic and anaerobic gravel filters followed by polishing sand filters plant located in Seer Village

As shown in tables 4.4-6, most of the detecting onsite grey and mixed wastewater evaluated plants treat wastewaters with BOD_5/COD ratios in influent >0.4. i.e. indicating wastewaters with highly degradable organic substances. Whilst, the BOD_5/COD ratios in the influent of UFGF.$_{B.S}$, DWP.$_{Ar}$, and EAP.$_{N}$ are between 0.2 and 0.4 indicate moderately to highly degradable organic substances. In the effluents, the BOD_5/COD ratios are mostly >0.4indicating wastewaters with relatively undegradable substances. Except the DWP.$_{Ar}$ plant, with its effluent BOD_5/COD ratios is between 0.2 and 0.4 indicate moderately to highly degradable organic substances. No plant has indicated to BOD_5/COD ratios <0.2 which means relatively undegradable organic substances. In case of the evaluated onsite grey wastewater treatment plants, the type of local manufactured detergent used by households, amounts of detergent used, food style and meals patterns as well as the low consumptive of water are the key factors that lead to the high organic loadings which all reflected on the performance of the treatment system used.

The household onsite ST-UFGF-SF systems showed fair COD removal efficiencies and a poorer performance compared with the reference ranges reported in the literature and presented COD effluent concentration values closer to the reference values except for UFGF.$_{M.Q}$, UFGF.$_{B.A}$ and UFGF.$_{S}$. The Average COD removal efficiencies % at the effluent of ST-UFGFs were 32.5, 10.3, 25.94, 30.5, 18.5, and 47.4 in UFGF.$_{Q.}$, UFGF.$_{M.Q.}$, UFGF.$_{B.L.}$, UFGF.$_{B.A.}$, UFGF$_{.S.}$, and UFGF.$_{B.S.}$, respectively. The lowest COD removal efficiencies appeared at the effluents of ST-UFGFs in UFGF.$_{M.Q}$ and UFGF$_{.S}$, while ST-UFGFs for UFGF.$_{Q}$, UFGF.$_{B.L}$, UFGF.$_{B.A}$, and UFGF$_{.B.S}$ showed low COD removal efficiencies. The Average multi-layer filter COD removal efficiencies % were 48.9, 10.3, 45.8, 32.2, 30.1, and 33.52 in UFGF.$_{Q.}$, UFGF.$_{M.Q.}$, UFGF.$_{B.L.}$, UFGF.$_{B.A.}$, UFGF$_{.S.}$, and UFGF.$_{B.S.}$, respectively. The best multi-layer filter COD removal was shown at UFGF.$_{B.A}$ and UFGF.$_{Q.}$ The lowest performance in UFGF$_{.S}$ explained by the existence of clogging in the gravel and multi-layer filters resulting from accumulation of organic materials that pass through the media of the filters because of the old lifetime of the treatment plant.

In case of household onsite activated sludge technology, the results of the performance of COD removal efficiencies was better than the previous system. The best performance of COD removal efficiencies was found in AS.$_{H}$. While in all cases showed lower performance compared with recorded reference in the literature and presented COD effluent concentration values higher than the reference values except for AS.$_{B.O.}$ The Average COD removal efficiencies % were 7.3, 6.22, 68.22, and 70.8 for aeration tanks in AS.$_{B.O.}$, AS.$_{B.}$, AS.$_{H.}$, and AS.$_{N.}$, respectively. The lowest COD removal efficiencies appeared at the effluents of aeration tanks of AS.$_{B.O}$ and AS.$_{B}$, while the performance of AS.$_{H.}$, and AS.$_{N}$ 'aeration tanks showed better COD removal efficiencies.The Average COD removal efficiencies % in the separation zone were 67.7, 19.74, 56.91, and 12.16 in AS.$_{B.O.}$, AS.$_{B.}$, AS.$_{H.}$, and AS.$_{N.}$, respectively..

The collective onsite SF-AUFGF-ATF-PSF system represented as UFGF.$_{Sr}$ showed a good COD removal efficiency but had a lower performance than expected, considering both average effluent concentrations and removal efficiencies. In addition, the collective onsite DWP.$_{Ar}$ represent the DWBP-BS-AT system had average effluent concentrations and removal efficiencies with performance below the expected range.

The evaluated community onsite UASB-HCWL system presented COD effluent concentration values higher than the reference values. However, considering COD removal efficiencies, the performance was below the expected at CW.$_{Nu.}$ plant. This can be partially explained by occurring a penetration on the bonders of constructed wetlands which leads to infiltrates the untreated wastewater flow into the surrounding layers and does not reach the effluent storage tank which makes the achievement of high performance removal efficiencies more difficult. (Photo B.2)

There is no clear performance trends associated with the use of certain selected techniques with respect to the removal of COD or BOD_5. Regardless of the type of treatment system selected, one of the keys to effective in the COD and BOD treatment is to develop and maintain an acclimated, healthy biomass, sufficient in quantity to handle maximum flows and the organic loads to be treated.

4.3.3.3 TSS removal efficiency

The average values of the influent to effluent TSS concentration and the calculated removal efficiencies of the different household, collective and community onsite systems are tabulated in tables 4.1, 4.2 and 4.3 and presented in the following Figures from 4.35 to 4.40.

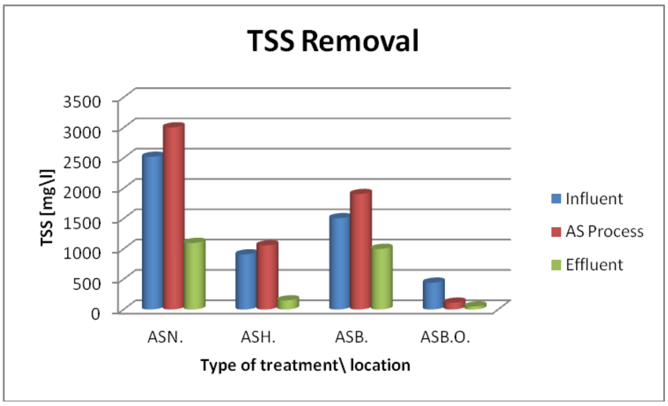


Figure 4.35: TSS influent, AS Process and effluent (mg/l)

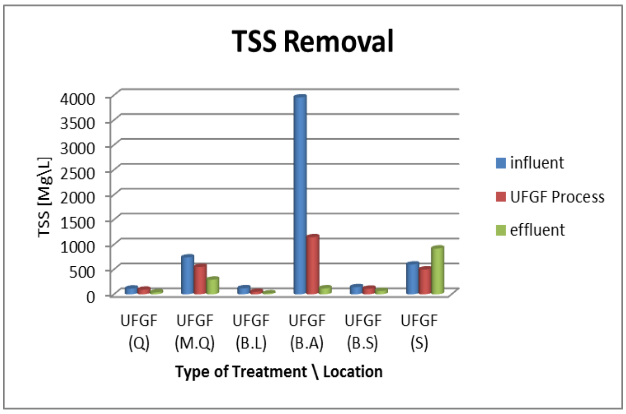


Figure 4.36: TSS influent, UFGF Process and effluent (mg/l)

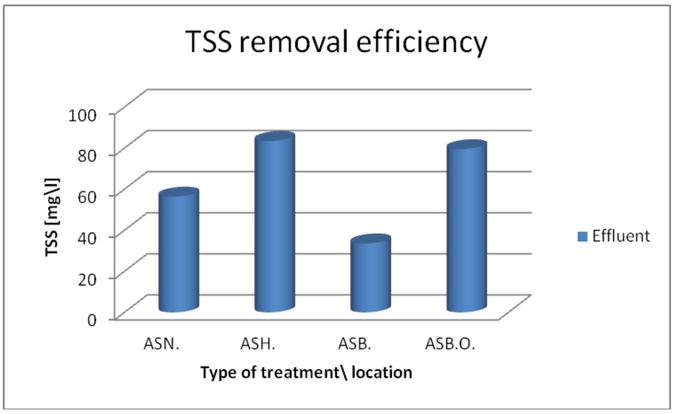


Figure 4.37: TSS removal efficiency (%) in AS systems

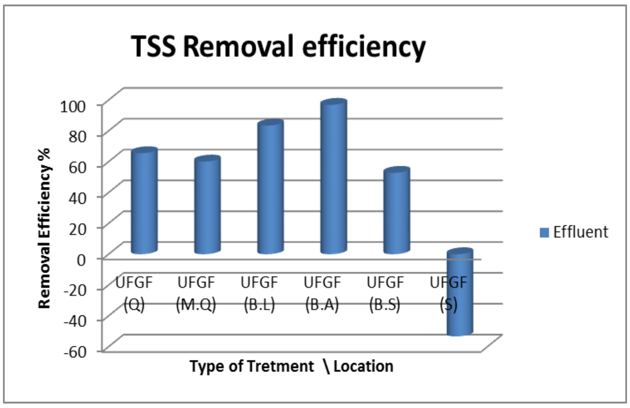


Figure 4.38: TSS removal efficiency (%) in UFGF systems

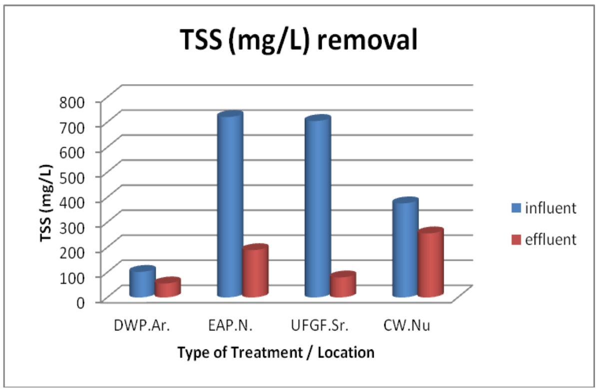


Figure 4.39: TSS influent, and effluent (mg/l) in DWP.$_{Ar.}$, EAP.$_{N.}$, UFGF.$_{Sr}$ and CW.$_{Nu.}$ systems

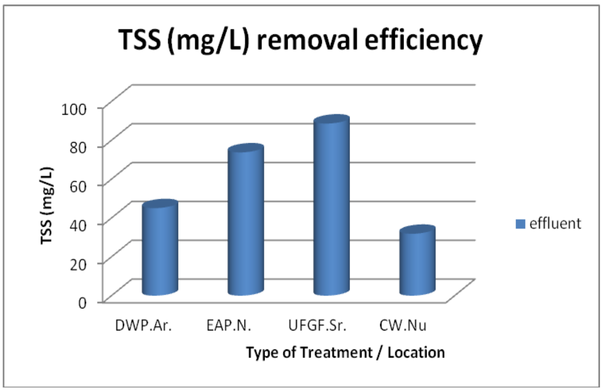


Figure 4.40: TSS removal efficiency (%) in DWP.$_{Ar.}$, EAP.N., UFGF.$_{Sr}$ and CW.$_{Nu.}$ systems

The efficiency of TSS removal was higher than 80% in AS.$_{H}$, UFGF.$_{B.L}$ UFGF.$_{B..A}$, and UFGF.$_{Sr}$ plants while it was lower than 60% in AS.$_{B}$, UFGF.$_{S}$, UFGF.$_{B.S.}$, DWP.$_{Ar}$ and CW.$_{Nu}$, which may be caused by the fact that insufficient influent wastewater had gone less beyond their limit. The highest efficiency for TSS removal was found in UFGF.$_{B.A}$ with average values higher than 96.9 %.

The recommended Guidelines by the Palestinian Standards Institute for Treated TSS effluent value is 40 mg\l for reuse in irrigation. The household onsite UFGF.$_{Q.}$ and UFGF.$_{B.L}$ plants have average effluent TSS is less 40 mg/l which means this selected evaluated systems are acceptable for reuse in irrigation.

The household onsite ST-UFGF-SF systems showed good TSS removal efficiencies and had a high percentage of them with a lower performance compared with the reference ranges reported in the literature, considering both average effluent concentrations and removal efficiencies. In all cases, the values of effluent TSS decreased with respect to the values observed in the influent excepted in the case of household onsite UFGF$_{.S.}$, where it is possible to have higher TSS values in the effluent than in the influent. The minimum values of TSS were found in the effluents of UFGF.$_{B.A}$ indicating a better performance of the up-flow gravel filter in this plant compared to the other technologies. The highest concentrations of TSS were found in the effluents from the UFGF.$_{S}$ as it has been mentioned above this is due the wear and clogging of the gravels in the plant. The UFGF.$_{B.L}$ presented TSS effluent concentration values closer to the reference values. However, considering TSS removal efficiencies, the performance was systematically below the expected limit value. The TSS removal efficiency % at UFGF.$_{B.A}$ showing a better performance than that reported in the literature. The Average TSS removal efficiencies % at the effluents of ST-UFGF were 18.96, 25.9, 59.02, 70.99, 16.7, and 20.83 in UFGF.$_{Q.}$, UFGF.$_{M.Q.}$, UFGF.$_{B.L.}$, UFGF.$_{B.A.}$, UFGF$_{.S.}$, and UFGF.$_{B.S.}$, respectively. The lowest TSS septic tank removal efficiencies appeared at UFGF$_{.Q}$ and UFGF$_{.S}$. The highest TSS septic tank removal efficiencies were found at UFGF.$_{B.L}$, and UFGF.$_{B.A}$. The Average multi-layer filter TSS removal efficiencies % were 57.45, 46.2, 59.6, 56.9, -84, and 31.94 in UFGF.$_{Q.}$, UFGF.$_{M.Q.}$, UFGF.$_{B.L.}$, UFGF.$_{B.A.}$, UFGF$_{.S.}$, and UFGF.$_{B.S.}$, respectively. The best multi-layer filter TSS removal was shown at UFGF.$_{B.A}$ and UFGF.$_{Q.}$ The lowest performance was found at multi-layer filter of UFGF$_{.S}$. The high effluent concentration of TSS that enters the up-flow gravel filter is the main factor of clogging. Colonization and growth of bacteria within the gravel and sand grains enhances the removal of SS but at the same time it may increase the risk of sand’s pores clogging.

The household onsite activated sludge (AS) process at $AS._B$ and $AS._N$ presented TSS effluent concentration values higher than the reference values and lower was found in the effluent of $AS._{B.O}$. and $AS._H$. However, considering TSS removal efficiencies, the performance was below the expected for all activated sludge plants. The Average TSS removal efficiencies % were 79.5.7, 33.6, 83.5 and 56.4 mg/l at $AS._{B.O.}$, $AS._{B.}$, $AS._{H.}$, and $AS._{N.}$, respectively. A good performance of TSS removal achieved showed in all Activated sludge plants but with high TSS effluent concentration. This can be explained as previously mentioned by the high influent concentrations, which makes the achievement of high removal efficiencies more difficult.

The collective onsite SF-AUFGF-ATF-PSF system represented by $UFGF._{Sr}$ showed a good TSS removal efficiency and had a higher performance than expected, considering both average effluent concentrations and removal efficiencies. The evaluated community onsite UASB-HCWL system presented TSS effluent concentration values lower than the reference values. However, considering TSS removal efficiencies, the performance of $CW._{Nu.}$ was below compared with the reference ranges reported in the literature.

4.3.4 Total Dissolved Solids (TDS)

The purpose of measured the Total Dissolved Solids (TDS) concentrations were to test if effluent quality could be used for irrigational purposes or not. According to recommended guidelines by the Palestinian Standards Institute (PSI) for Treated Wastewater Characteristics to TDS parameter it should be less than 1200 mg/l for irrigational purposes (see Table A.4 Appendix-A).

The average values of the influent to effluent TDS concentrations and the calculated removal efficiencies of the different household, collective and community onsite systems are tabulated in tables 4.1, 4.2 and 4.3. All of the results of measurements of TDS concentrations for all of wastewater treatment plants with different levels were less than 1200 mg/l. Extreme TDS effluent values were 1016 mg/l, 1145 mg/l found in $AS._{N.}$ and $EAP._{N}$, respectively. So, according to the effluent results of TDS concentration, the effluent could be used for irrigational purposes.

4.3.5. Nutrient removal

4.3.5.1 Total Nitrogen (Total-N) removal efficiency

4.3.5.1.1 TKN removal efficiency

TN_b (Total Bound Nitrogen) is a measure of the concentration of ammonia, ammonium salts, nitrites, nitrates and organic nitrogen components.

TKN (Total Kjeldahl Nitrogen) is a measure of the concentration of ammonia/ammonium (NH_3/NH_4^+) and organic-Nitrogen. However, a series of organic compounds is incompletely detected by the Kjeldahl method.

Total-Inorganic-N as the sum of NH_4-N, NO_2-N and NO_3-N is commonly measured. TN_b, is therefore higher than Total-Inorganic-N

It has to be mentioned, that the analytical methods of Total-N as TN_b (by thermal oxidation) and Total-N by TKN (Kjeldahl hydrolysis and stripping) lead to non-comparable results.

Total-N can be quantified by direct determination or separate determinations of TKN, nitrite, and nitrate and by adding the three values.

Total Nitrogen = TKN + NO_3^- + NO_2^-
TKN = NH_4^+ + Organic Nitrogen

NO_3^- and NO_2^- influent concentration assumed to equal zero. So, influent total nitrogen concentration equals influent TKN concentration.

The average values of the influent to effluent TKN concentration and the calculated removal efficiencies of the different household onsite systems are tabulated in tables 4.1, 4.2 and 4.3 and presented in the following Figures from 4.41 to 4.44.

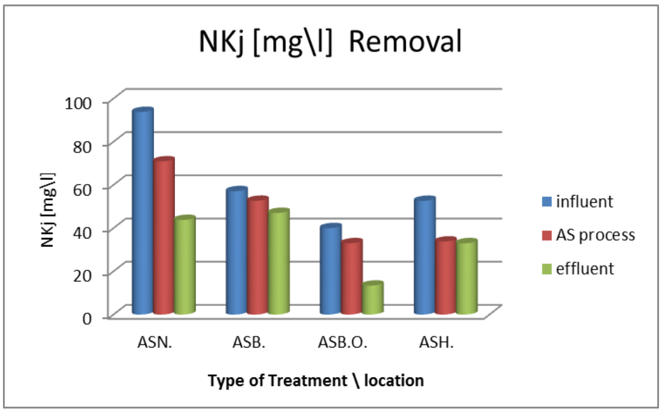


Figure 4.41: Nkj influent, AS process and effluent (mg/l)

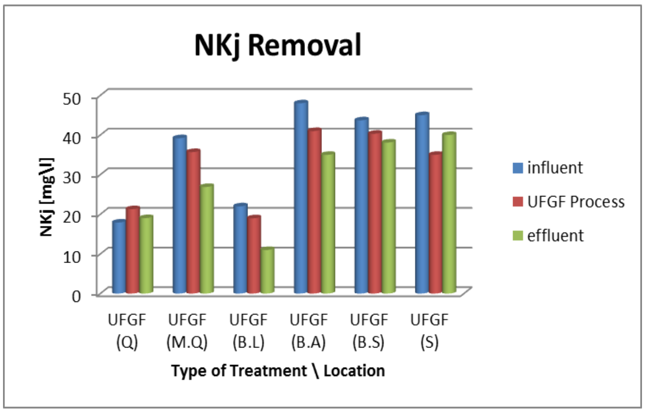


Figure 4.42: Nkj influent, UFGF process and effluent (mg/l)

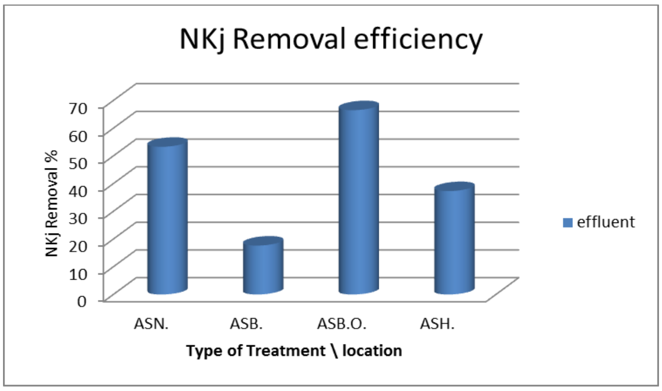


Figure 4.43: NKj Removal efficiency (%) in the AS systems

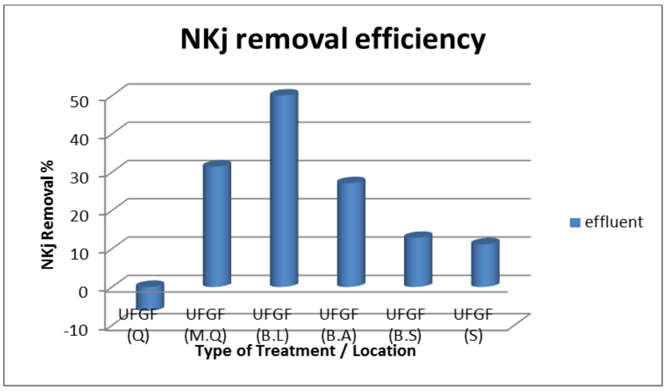


Figure 4.44: NKj Removal efficiency (%) in the UFGF systems

Most of the household onsite ST-UFGF-SF systems had a lower performance compared with the TKN removal efficiencies reference range reported in the literature. However, considering average TKN effluent concentrations values were closer to the reference in all cases. The Average TKN removal efficiencies % at the effluents of ST-UFGF systems were -18.9, 8.9, 13.6, 14.6, 22.2, and 7.7 in $UFGF_{.Q.}$, $UFGF_{.M.Q.}$, $UFGF_{.B.L.}$, $UFGF_{.B.A.}$, $UFGF_{.S.}$, and $UFGF_{.B.S.}$, respectively. The performance of ST-UFGF for $UFGF_{.Q}$ showed the lowest TKN removal efficiencies related to the penetration that found surrounding the cover of the septic tank which lead to entrance the air which in turns affected the de-nitrification process as well as the reason of increasing the concentration in the TKN effluent to be more than the concentration in the influent explained by the ammonification process as previously discussed (section 2.4.4.). In addition, The low TKN removal efficiencies appeared at the effluent of ST-UFGF for UFGF.$_{M.Q}$, the reason of that as previously explained due to occurred malfunction in submersible pump which preventing and hardly transferred the flow of wastewater from septic tank stage to up-flow gravel filter stage making overloading at septic tank and it was observed the cover of the septic tank has been broken lead to convert it from anaerobic process to aerobic process. The Average TKN removal efficiencies % at the effluents of multi-layer filters were 10.6, 24.6, 42.1, 14.6, -14.3, and 5.5 in UFGF.$_{Q.}$, UFGF.$_{M.Q.}$, UFGF.$_{B.L.}$, UFGF.$_{B.A.}$, UFGF.$_{S.}$, and UFGF.$_{B.S.}$, respectively. UFGF.$_{B.L}$ Plant showed the best efficiency in the removal of TKN. It should be noted here the observed low removal efficiencies in terms of TKN was expected, since none of these system has been designed for TKN removal.

The household onsite activated sludge (AS) process at all evaluated plants presented TKN effluent concentration values higher than the reference values excepted at AS.$_{B.O}$. but less or within than the recommended Palestinian standards. However, considering TKN removal efficiencies, the performance was below the expected for all activated sludge plants. Also here as mentioned above the observed low TKN removal efficiencies was expected, where none of these analyzed technologies has been designed for TKN removal. However, the good performance was observed at the AS.$_{B.O}$. Plant.

4.3.5.1.2 NH_4^+ removal efficiency

Ammonia/ammonium as mentioned in literature review chapter is formed as a first step in the removal of Total-N in onsite wastewater treatment plants using biological treatments (i.e. ammonifaction, where organic N is converted to ammonia/ammonium. NH_4^+ is on one hand assimilated to bacterial cells (leading thus to net growth) and on the other hand oxidized to nitrite and nitrate. Nitrifying organisms are present in almost all aerobic biological treatment processes, but usually their numbers are limited.

The average values of the influent to effluent NH_4^+ concentration and the calculated removal efficiencies of the different household onsite systems are tabulated in tables 4.1, 4.2 and 4.3 and presented in the following Figures from 4.45 to 4.48.

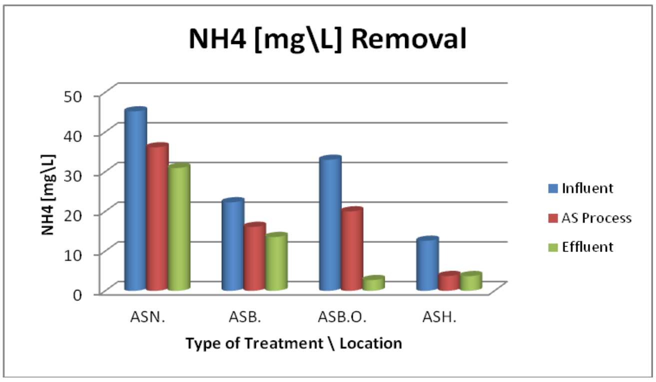


Figure 4.45: NH4+ Influent, AS process and Effluent (mg/l)

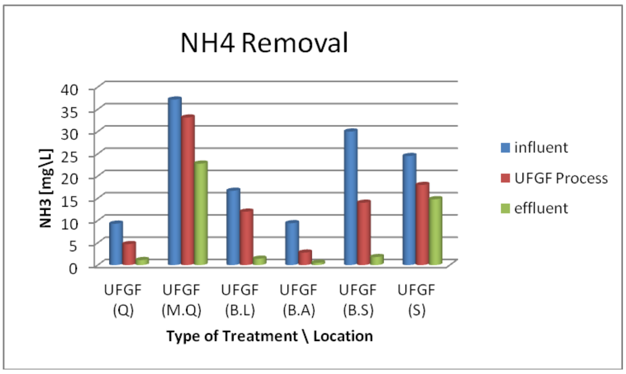


Figure 4.46: NH4+ Influent, UFGF process and Effluent (mg/l)

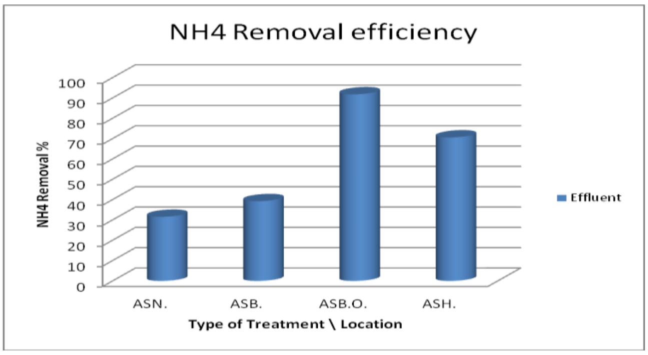


Figure 4.47: NH_4^+ Removal efficiency (%) in the AS systems

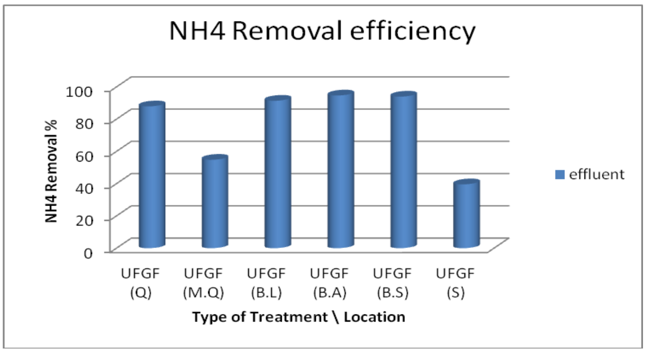


Figure 4.48: NH_4^+ Removal efficiency (%) in the UFGF systems

NH_4^+ was observed its presence in the influent of evaluated onsite wastewater plants, where it has reported it in the range 9.3-37.19 mg/l in case of ST-UFGF-SF systems and in the range 12.6-45.2 mg/l in case of activated sludge systems. The NH_4^+ concentration in the effluents decreased in all cases compared to the values observed in the influent. The lowest concentrations of NH_4^+ were found in the effluents of $UFGF_{.Q.}$, $UFGF_{.B.L,}$ $UFGF_{.B.A,}$ $UFGF_{.B.S,}$ $AS_{.H}$ and $AS_{.B.O}$ systems while the highest concentration of NH_4^+ was observed in the effluent of $AS_{.N}$.

The highest removal efficiencies % of NH_4^+ were found in $UFGF_{.B.L,}$ $UFGF_{.B.A,}$ $UFGF_{.B.S}$ and $AS_{.B.O}$ (more than 90%) while the lowest was found for $UFGF_{.S.}$, $AS_{.B.}$, and $AS_{.N.}$ (less than 40%).

High percentage of household onsite ST-UFGF-SF systems showed good NH_4^+ removal efficiencies with a higher performance compared with the reference ranges reported in the literature except at $UFGF_{.S}$. However, the actual effluent concentrations were significantly lower or within the reference value. The Average NH_4^+ removal efficiencies % at ST-UFGFs were 49.5, 4.04, 28.14, 70.21, 26.5, and 53.3 in $UFGF_{.Q.}$, $UFGF_{.M.Q.}$, $UFGF_{.B.L.}$, $UFGF_{.B.A.}$, $UFGF_{.S.}$, and $UFGF_{.B.S.}$, respectively. The broken covers of septic tanks of $UFGF_{.Q.}$ and $UFGF_{.M.Q}$ are the main reason of entrance the air to the septic tank which effected in the performance of their de-nitrification process. The Average NH_4^+ removal efficiencies % in the effluent of multi-layer filters were 76.2, 31.2, 88.1, 82.14, 26.5, and 87.14 in $UFGF_{.Q.}$, $UFGF_{.M.Q.}$, $UFGF_{.B.L.}$, $UFGF_{.B.A.}$, $UFGF_{.S.}$, and $UFGF_{.B.S.}$, respectively. The best ST-UFGF of NH_4^+ removal was shown at $UFGF_{.B.L}$ and $UFGF_{.B.S.}$ The lowest performance was found at $UFGF_{.S.}$ as previously discussed may because of the bad status of the this treatment plant makes the performance of operations is moving towards failure.

The performance achieved for household activated sludge plants was lower than the reference ranges reported in the literature. The NH_4^+ effluent concentrations for all cases were lower than the expected. The Average $NH4^+$ removal efficiencies % were 86.5, 27.8, 70.5 and 20.1 mg/l in the effluent of the Aerated zone at $AS_{.B.O.}$, $AS_{.B.}$, $AS_{.H.}$, and $AS_{.N.}$, respectively. The Average NH_4^+ removal efficiencies % in the effluent of the separation zone were 39.4, 15.8, 0.52 and 14.4 mg/l at $AS_{.B.O.}$, $AS_{.B.}$, $AS_{.H.}$, and $AS_{.N.}$, respectively. A good performance of NH_4^+ removal achieved by Aerated zone of $AS_{.B.O.}$ It should be mentioned, DO concentrations were observed 2.81, 0.27, 0.38, and 1.8 mg/l in the effluents of $AS_{.B.O.}$, $AS_{.B.}$, $AS_{.H.}$, and $AS_{.N.}$, respectively. The increase in DO concentration enhanced the organic matter oxidation, the hydrolysis of organic nitrogen compounds and the activity of the bacteria in general but also the metabolism and activity of the bacteria responsible for the nitrification process. This may explained by the interruption and low consumption of water leads to aeration air lift pump could probably not provide the sufficient amount of oxygen required by the stabilization of organic matter.

4.3.5.1.3 Nitrite (NO_2-N) Removal

Nitrite (NO_2^-) is seldom found in the treated effluent of wastewater from the onsite treatment plants. In addition, there are analytical problems to measure nitrite because of its fast conversion to nitrate. This is the reason why the monitoring of nitrite does not performed in the influent and effluent of the evaluated onsite plants.

Nitrite is generated when nitrification processes are used at treatment plant. Nitrification is a two-step process that first converts ammonia and ammonium to nitrite and then rapidly to nitrate, so that nitrite levels at any given time are usually low.

On the other hand, onsite wastewater treatment plants are not specifically designed to remove nitrite, just are possibly designed to remove Total-N, because nitrite is an 'intermediary' pollutant generated when using nitrification processes and is rapidly converted to nitrate.

4.3.5.1.4 Nitrate (as NO_3-N) Removal

As previously explained, the Nitrate is converted to nitrogen gas in the de-nitrification process. Effluents from UFGF.$_Q$, UFGF.$_S$ and AS.$_H$ had the highest concentration of NO_3^- while the lowest was found in the effluent of UFGF.$_{M.Q}$. At the household onsite activated sludge systems, it was observed that there are a relationship between the decrease in NO_3^- concentration and the decrease in pH in the effluent of activated sludge aerobic processes operating under certain conditions. This may be caused by the decrease in dissolved CO_2 concentration by stripping in the aeration step and by a reduction in the concentration of organic matter due to oxidation. NO_3^- concentrations in the influent of household onsite activated sludge systems was expected to be zero however, from the analysis it had an average value (maximum-minimum) of (33.5, 6.7) mg/l at (AS.$_N$, AS.$_{B.O}$), respectively. Nitrification at the subsequent treatment units raised the NO_3^- concentrations of the final effluent at all of cases. In case of ST-UFGF-SF systems, The high concentration of NO_3^- at the influent of UFGF.$_{B.A}$ could be because of the accumulation of grease at the upper layer of septic tank without removing it for long period of time or may because the owner of the plant was adding some of fertilizers at the influent of septic tank of the plant that depending on his admit.

According to Palestinian standards PSI that NO_3^- concentration must not exceed 50 mg/l in order to reuse treated wastewater for irrigational purposes. So, effluents from all the evaluated systems could be used for irrigational purposes to enrich the soil.

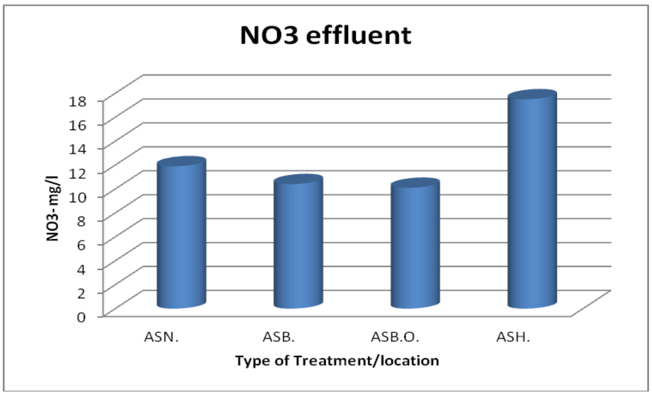


Figure 4.49: NO_3- Effluent (mg/l) at AS systems

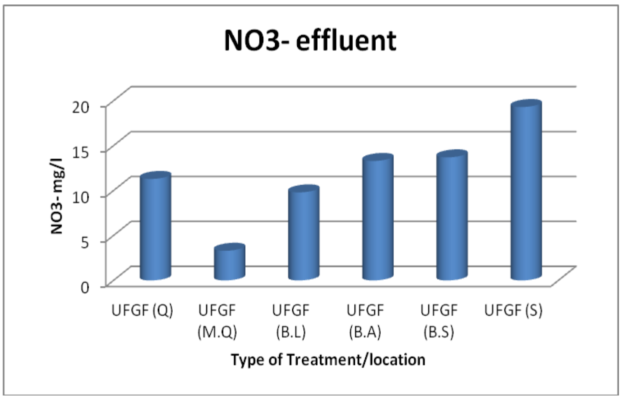


Figure 4.50: NO_3- Effluent (mg/l) at UFGF systems

4.3.5.2 PO_4^{-3} Removal Efficiency

Phosphorus is present in wastewater in inorganic and organic forms. The inorganic forms are orthophosphates (i.e. $HPO_4^{2-}/H_2PO_4^-$) and polyphosphates. Organically bound phosphorus is usually of minor importance. Polyphosphates can be used as a means of controlling corrosion. The average values of the influent to effluent PO_4^{-3} concentration and the calculated removal efficiencies of the different household onsite systems are tabulated in tables 4.1, 4.2 and 4.3 and presented in the following Figures from 4.51to 4.54.

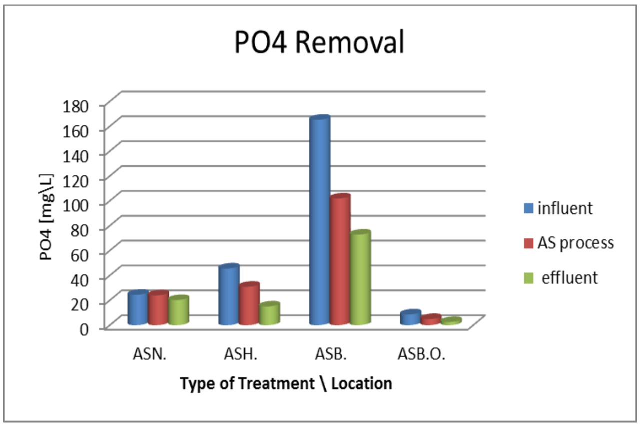


Figure 4.51: PO_4^{-3} influent, AS process and effluent (mg/l)

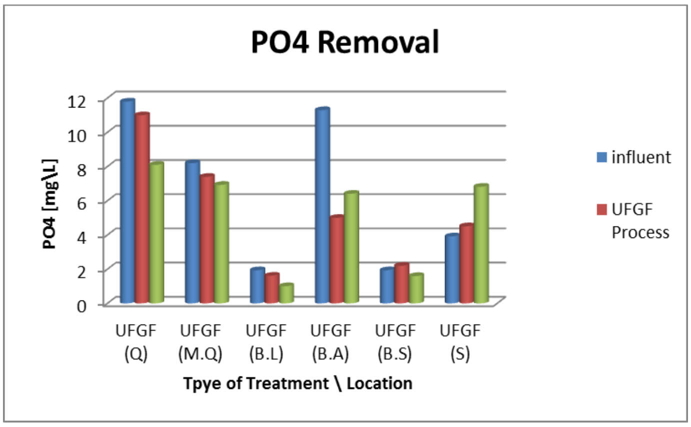


Figure 4.52: PO_4^{-3} influent, UFGF process and effluent (mg/l)

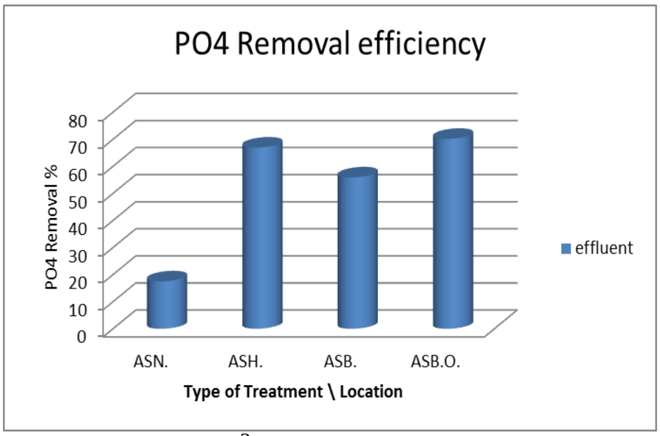


Figure 4.53: PO_4^{-3} Removal efficiency (%) in the AS systems

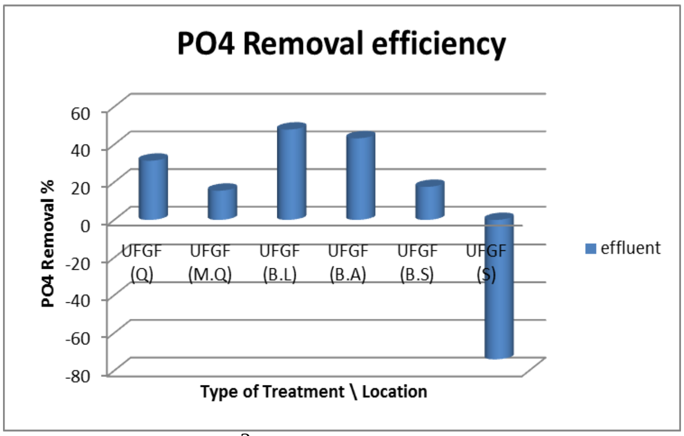


Figure 4.54: PO_4^{-3} Removal efficiency (%) in the UFGF systems

In all cases, the values of effluent PO_4^{-3} decreased with respect to the values observed in the influent excepted in the case of household onsite UFGF.$_S$ and UFGF$_{B.A}$, where it is not possible to have higher PO_4^{-3} values in the effluent than in the influent, but in the words of the owner of the plant that has been added a fertilizer materials to the final stage of the storage treated wastewater which may affect the result of the removal efficiency of PO_4^{-3}

At ST-UFGF-SF systems, all evaluated plants showed average effluent concentrations lower than the expected values. The performance of ST-UFGF treatment unit showed a slight increase in the effluent concentrations of PO_4^{-3} in UFGF.$_S$ and UFGF.$_{B.S}$. Plants, a slight decrease in the concentrations of PO_4^{-3} in UFGF.$_Q$, UFGF.$_{M.Q}$. and UFGF.$_{B.L}$ Plants, and a sharp decrease in the performance of UFGF.$_{B.A}$. Due to presence of disorder in each of the septic tanks of UFGF.$_Q$ and UFGF.$_{M.Q}$ that lead to entry of the oxygen which in turn effected on the efficiency of the performance of these units where the presence of oxygen affects on the presence of microorganisms which are responsible for the uptake or release of PO_4^{-3}**,** even the other units did not give satisfactory results, which demonstrates the flaw in the performance of functions of processes required. However, the values of influent PO_4^{-3} did not exceed the limit of the recommended Palestinian Institute; there is no proposal to remove it. Moreover, this type of technology is not designed to remove PO_4^{-3}.

However, the removal of PO_4^{-3} in the Activated sludge plants was better than the previous technology, although the concentrations of the influent PO_4^{-3} was higher than the recommended Palestinian Institute as in case of AS.$_B$ and AS.$_N$. According to the reference values reported in the literature, the activated sludge process presented PO_4^{-3} effluent concentration values higher than the reference values. However, considering PO_4^{-3} removal efficiencies, the performance was below the expected for activated sludge plants. This can be presumably explained by the high influent concentrations, which makes the achievement of high removal efficiencies more difficult. In an anaerobic-aerobic activated sludge unit, as previously explained PO_4^{-3} is released under anaerobic conditions and taken up by microorganisms under aerobic conditions. The performance efficiency in the aeration zones found 42.5, 38.5, 32.1, and 1.9 % in AS.$_{B.O}$, AS.$_B$, AS.$_H$, and AS.$_N$, respectively. The lowest efficiency was found in the performance of AS.$_N$ Plant, that illustrate due to occurring a partial failure on the air lift pump affected on the amount of air supplied to aeration zone. Turning to the settling zones, the performance of these units was no different from their predecessors. However, the good performance presented by the AS.$_{B.O}$ considering PO_4^{-3} removal efficiency is somewhat unexpected, with presenting effluent concentrations lower than the expected. This explained because of the use of gray waterwater instead of mixed wastewater in the plant.

4.3.5.3 SO_4 Removal Efficiency

It is important to control sulphates in the influent of wastewater treatment plants because of the potential to create odour (formation of hydrogen sulphide) and corrosion problems in sewers (by oxidation of hydrogen sulphide to sulphuric acid).

The average values of the influent to effluent SO_4 concentration and the calculated removal efficiencies of the different household onsite systems are tabulated in tables 4.1, 4.2 and 4.3 and presented in the following Figures from 4.55 to 4.58.

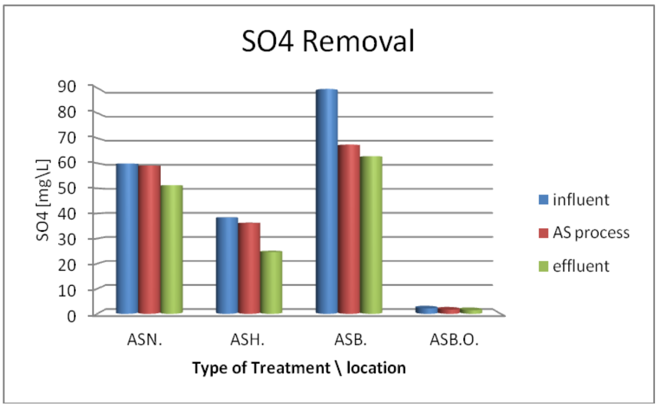


Figure 4.55: SO_4 influent, AS process and effluent (mg/l)

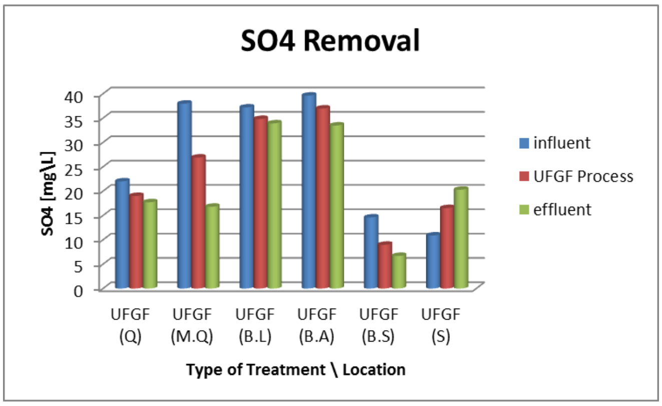


Figure 4.56: SO_4 influent, UFGF process and effluent (mg/l)

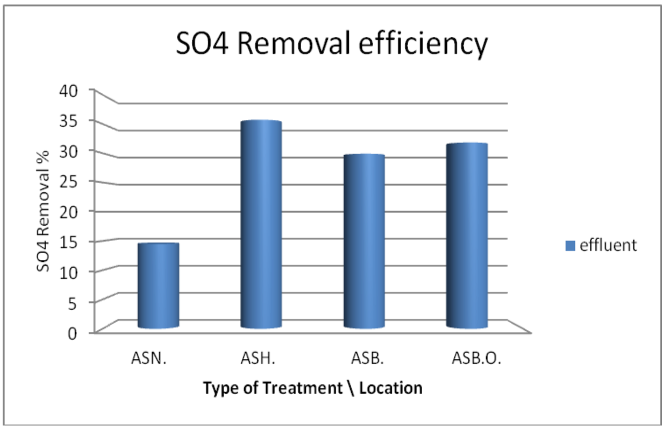


Figure 4.57: SO_4 Removal efficiency (%) in the AS systems

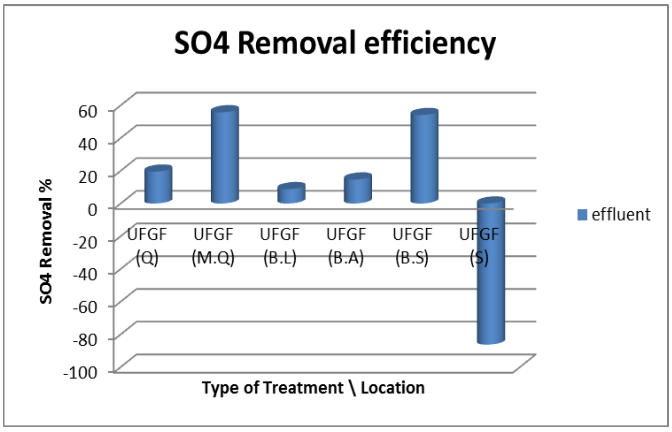


Figure 4.58: SO_4 Removal efficiency (%) in the UFGF systems

The concentration of SO_4 in the effluents decreased significantly compared to the average concentration of SO4 in the influents of all plants except at $UFGF_{.S}$, the lowest SO_4 concentrations being in the effluents of $AS_{.B.O}$. The removal efficiency of SO_4 was lower than 60% in all cases.

Concerning ST-UFGF-SF, most of evaluated Plants showed a low performance in terms of SO_4, considering both effluent concentrations and removal efficiencies. However, the actual effluent SO_4

concentrations were very small compared with what is required of the Palestinian Standards (poor performance). The performance of ST-UFGFs was less than 30% for all plants and the lowest one was at $UFGF_{.S}$. The performances in multi-layer filters were similar to ST-UFGFs for the same cases. The increasing in the SO4 effluent at $UFGF_{.S}$ was observed probably due to the wear situation of the plant. The results obtained were expected since this type of technology is not designed for the removal of SO_4.

It was observed that the performance of the plants which are running activated sludge process almost in all cases gave results similar to the former system. The activated sludge process presented SO_4 effluent concentration values very few to the Palestinian standards values. However, considering SO_4 removal efficiencies, the performance was below the expected for activated sludge plants. This can be explained by the low influent concentrations, which makes the achievement of removal efficiencies very weak. The performance achieved was expected, since this kind of technologies has not been designed for SO_4 removal.

4.3.6 Bacteriological removal
4.3.6.1 Total and Fecal Coliform removal

The average values of log removals of pathogen indicators of the different mixed and grey household onsite systems are tabulated in tables 4.1, 4.2 and 4.3 and presented Figures in 4.59 to 4.62.

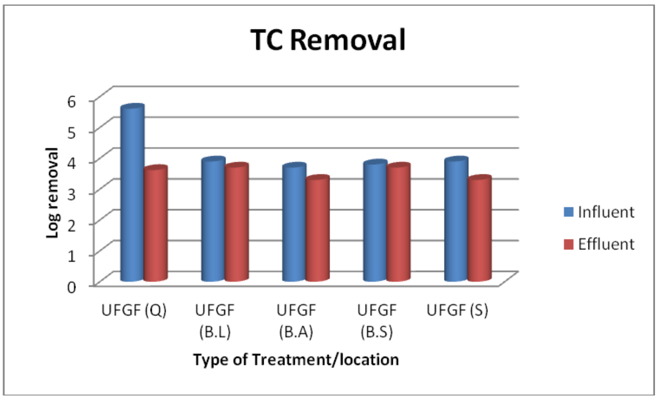


Figure 4.59: TC Removal at UFGF systems

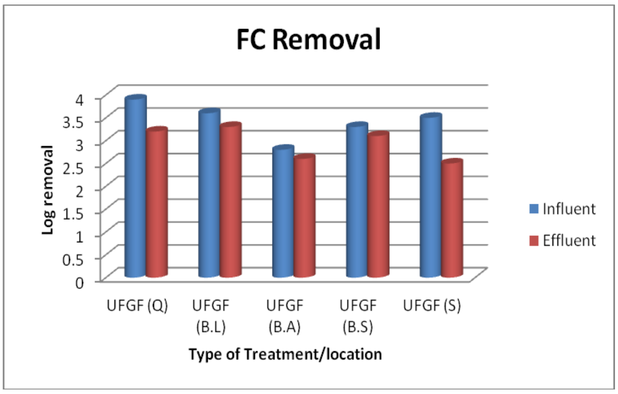


Figure 4.60: FC Removal at UFGF systems

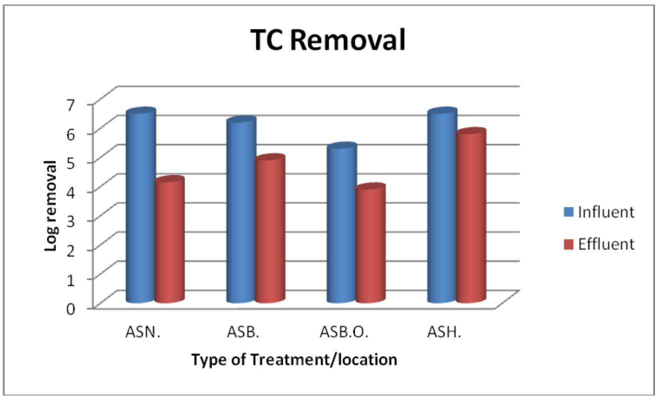


Figure 4.61: TC Removal at AS systems

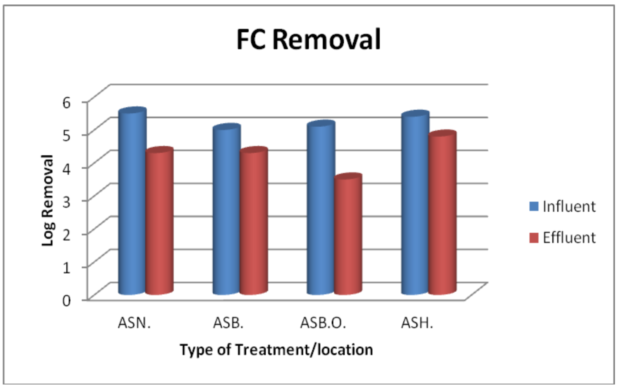


Figure 4.62: FC Removal at AS systems

The ST-UFGF-SF Systems for treating grey wastewater showed fair TC and FC removal efficiencies and a poorer performance compared with the reference ranges reported in the literature. However, the actual effluent concentrations of TC and FC were significantly above the upper reference value for all cases. The worst case appeared in the UFGF.$_{M.Q}$ Plant reaching TMTC. The high effluent concentrations of pathogenic counts in the grey wastewater treatment systems can be explained as previously interpretation could be during the high presence of TC and FC in the influent input from hand washing after defecation and babies washing in hand washing basin were the key factors for this high numbers of E.coli, making the performance of these plants very poor.

The activated sludge process for treating mixed wastewater presented TC and FC effluent concentration values higher than the reference values. However, considering TC and FC removal efficiencies, the performance was within the expected for activated sludge plants.

However, whether the AS or ST-UFGF-SF effluents systems have not a satisfactory bacteriological characteristics for reuse purposes in irrigation in terms of the concentrations of TC and FC are higher than standard values.

4.4 Operational Performance Evaluation of existing wastewater treatment systems

All data obtained from different levels of onsite wastewater treatment plants were evaluated in order to verify the existence of the relationship between design/operational parameters and the performance of the plants. It was not possible to analyze all monitored plants, which comprise the six technologies, because some of them did not have the required data to calculate the operational parameters. All results are discussed in the following sections, separated by technology type.

4.4.1 Individual (household) onsite wastewater treatment systems

4.4.1.1 Septic Tank – Up-flow Gravel Filter Systems

The design/operational parameters data which was provided for up flow gravel systems are not sufficient for the calculation to find the theoretical and practical operational parameters for each plant, so the comparison seems very difficult between design/operational parameters and the performance of the plants. However, the performance of the septic tank depends as well as on the Hydraulic retention time (HRT) and needs to regular desludging which estimated every 36 months which is never happened for any plant. Most of the existing septic tanks are designed to have 48 hours of HRT but the problem is when the consumption of water in the targeted household is suffer from fluctuates of wastewater generation according to interruption of water supply lead to downloading on the operation of plant and that affect on the HRT which in turn affect the efficiency of treatment.

On the other hand, filters are expected to operate without maintenance for 18-24 months, then the media of the filters need to be washed out by fresh water which also did not happened for any plant, which is affect on the voids space of filter medium by clogging it which also affect on the digester volume which is required to provide sufficient HRT.

The plant which is operating with overloading rate is UFGF.$_{M.Q}$ the reason of that is due to mixed the grey wastewater with black wastewater to be entering the plant while it just designed to be treat only grey wastewater. Also, it was found a faulty in the motor that runs on the withdrawals of treated wastewater from septic tank stage to the second treatment process without being repaired because of the financial situation of the owner of the plant causing percolation in the primary treatment process of septic tank, which affects the efficiency of the performance in removing of all the detected parameters. Finally, $UFGF._{S.}$ has the worst performance compared with other of its peers of the same system; the reason of that is according to its long life cycle period assumed to 10 years. The plant is completely destroyed inside.

4.2.1.2 Individual Household Activated Sludge systems

Wastewater loading rate is a critical design factor for wastewater treatment systems. Table 4.7 and Figure 4.63 show the result of Volumetric COD Loading Rate and Organic Loading Rate of the selected four household onsite AS systems were evaluated.

Table 4.7: Volumetric COD Loading Rate and Organic Loading Rate of the AS systems

Type of Treatment / Location	**Actual Flow (m^3/day)**	**Hydraulic retention time (HRT) (d)**	**Ave. Vol. COD Load. Rate (kg COD/m^3.day)**	**Ave. OLR (kg BOD_5/m^3.day)**
$AS_{B.O.}$	0.3	3.12	0.02	0.02
$AS_{H.}$	0.5	2.15	0.36	0.19
$AS_{B.}$	0.25	4.2	0.18	0.08
$AS_{N.}$	0.62	1.7	0.44	0.32

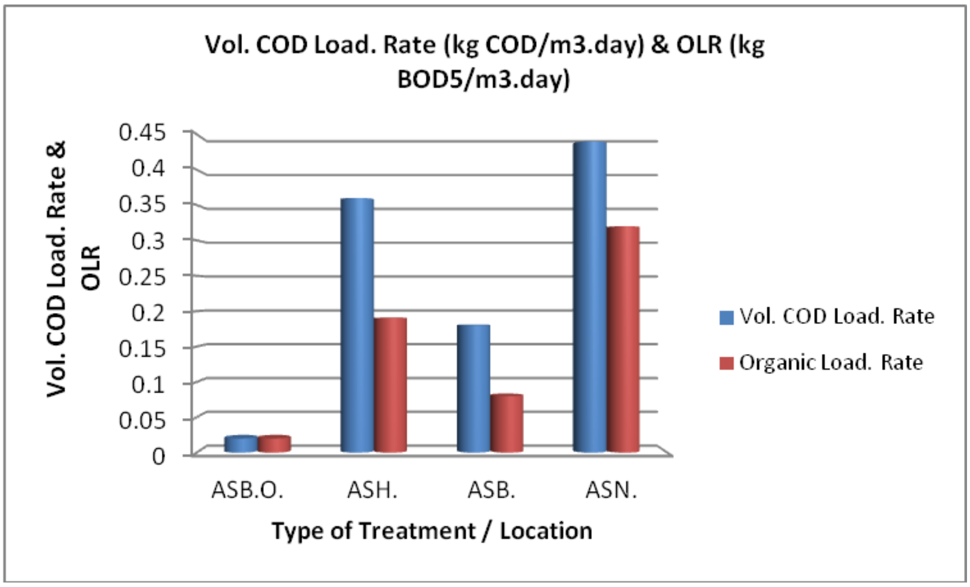


Figure 4.63: Volumetric COD Loading Rate and Organic Loading Rate of the AS systems

From Figures, it can be emphasized that the four different aerated tanks have different operating conditions, sometimes with strong fluctuations in terms of biomass contents or sludge volume index (SVI) values.

One likely explanation of the high values of OLR and volumetric COD loading rate entering the Aeration zone at $AS_{N.}$, the increasing in BOD_5 removal efficiency and this kind of systems assures good removal efficiency of organic matter.

Table 4.8: SV, SVI, MLSS, MCRT and F/M Ratio

Type of Treatment / Location`	MLSS (mg/l)	S.V (ml/l)	S.V.I (ml/g)	F/M (day-1)	MCRT (day)
$AS_{B.O.}$	442	8	18.1	0.038	26.3
$AS_{H.}$	1059	120	113.3	0.178	5.6
$AS_{B.}$	1905	160	84	0.039	25.6
$AS_{N.}$	3005	240	79.9	0.105	9.5

While Figures 4.21, 4.22, 4.23 and 4.24 show MLSS in AS, S.V. in AT, S.V.I in AT and F/M ratio in AT, respectively:

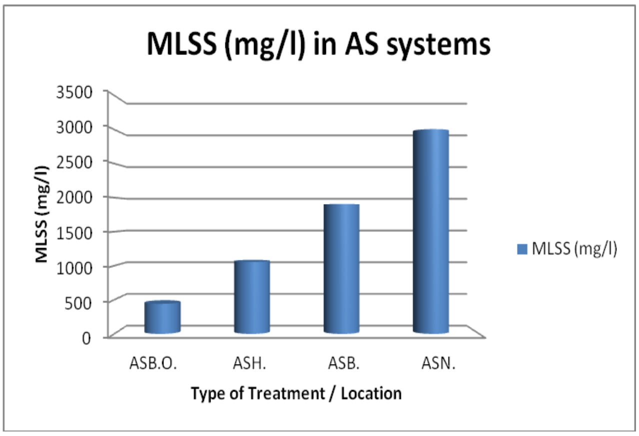


Figure 4.64: Mixed Liquor suspended Solids (MLSS) in the Aeration Tank (AT)

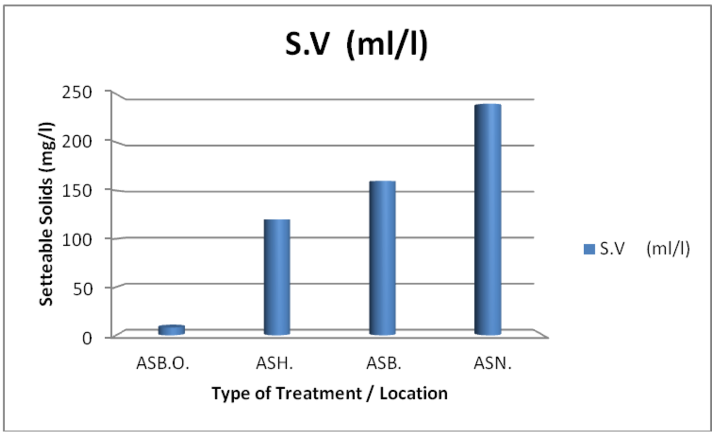


Figure 4.65: Sludge Volume (SV) in the Aeration Tank (AT)

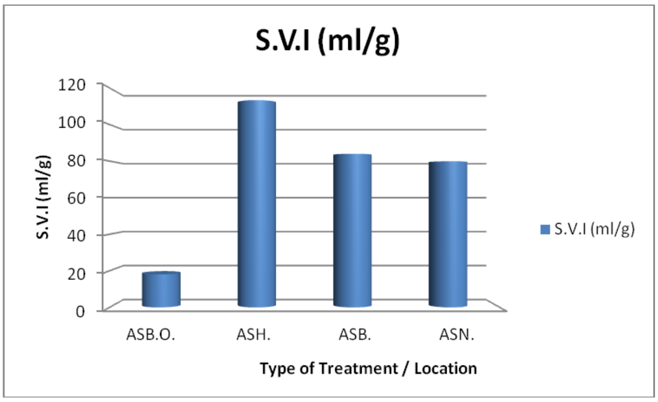


Figure 4.66: Sludge Volume Index (SVI) in the Aeration Tank (AT)

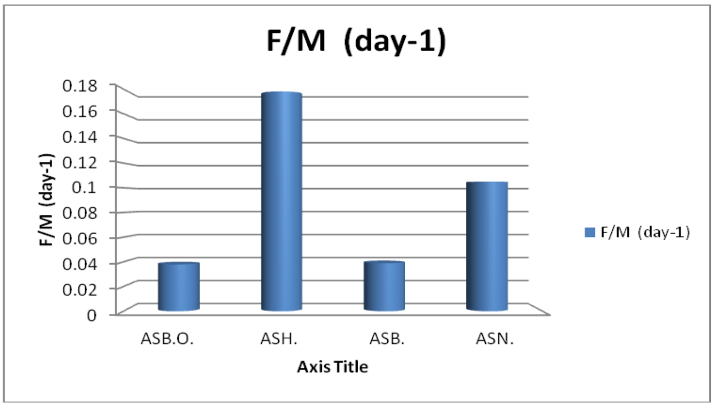


Figure 4.67: F/M Ratio in the Aeration Tank (AT)

According to the measured values of SVI, it was observed that substantial decreases in SVI at $AS._{B.O.}$, indicating pinfloc potential, $AS._{B}$ and $AS._{N}$ have a good values of SVI ranging from 50 to 100 ml/g , filament growth could appears at $AS._{H}$ has a value of SVI equal 113 ml/g indicating potential growth of filaments.

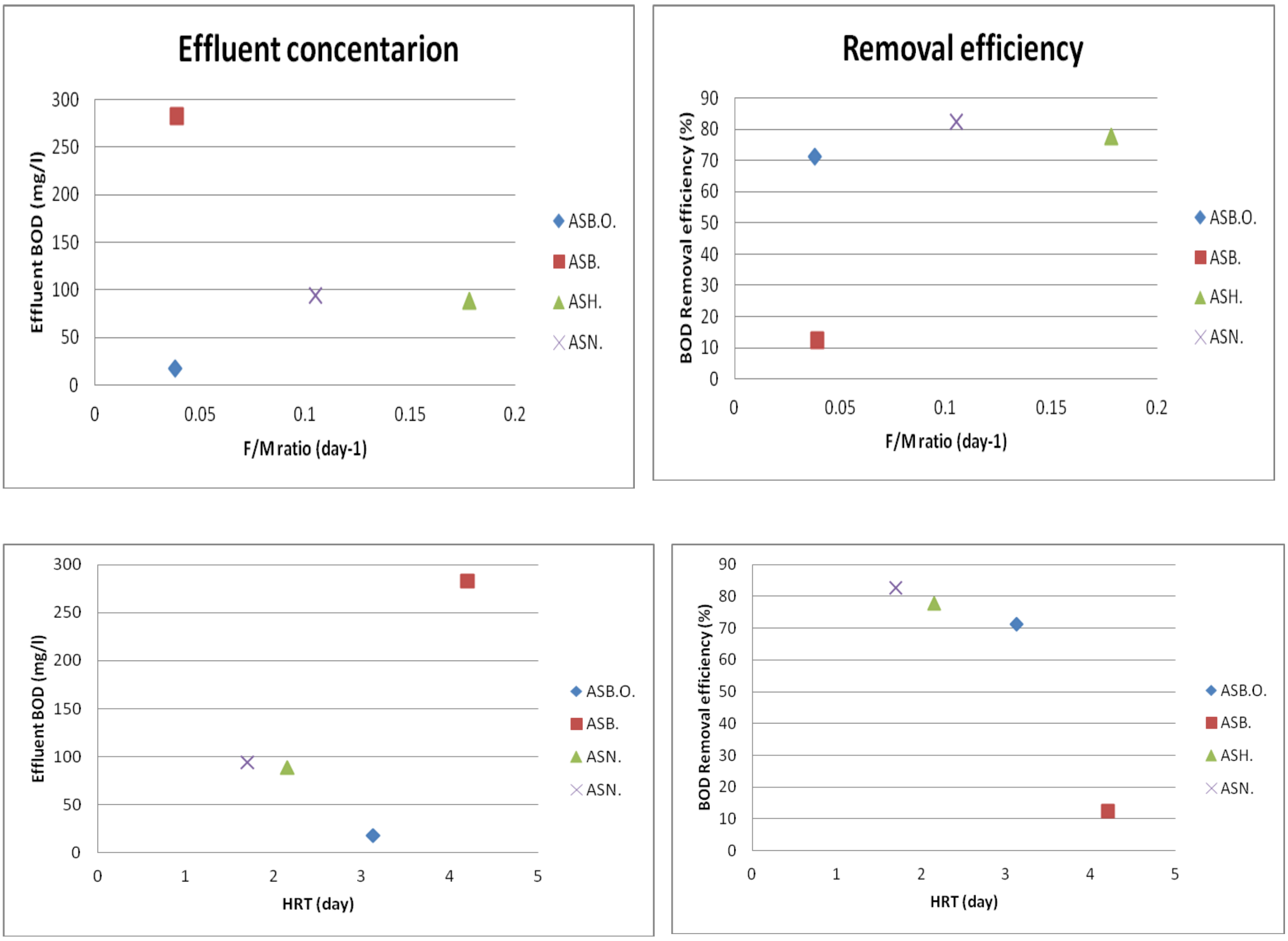
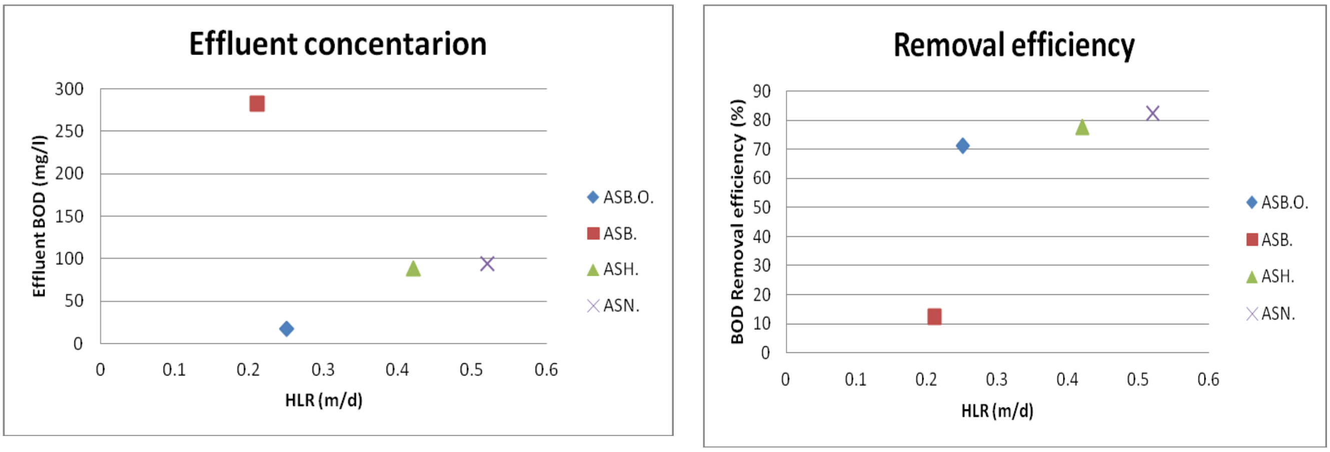


Figure 4.68 Relationship among F/M, HRT (aeration zone), and effluent BOD concentration, BOD removal efficiency – household onsite activated sludge systems.



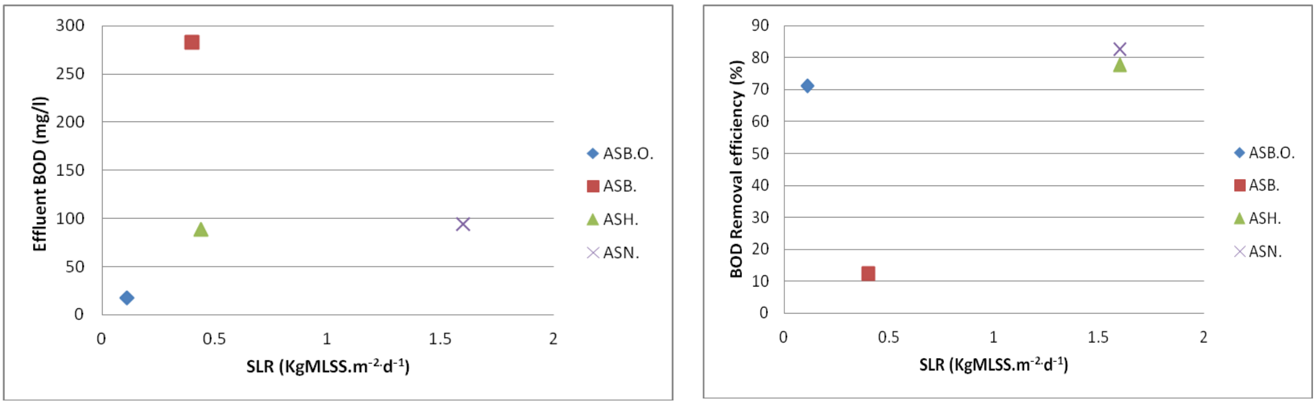


Figure 4.69 Relationship among HLR, SLR (separation or clarifier zone), and effluent BOD concentration, BOD removal efficiency – household onsite activated sludge systems.

The results illustrated that as shown in figure 4.68, the different F/M ratios and HRT values did not influence substantially the performance of the aeration zone for $AS_{B.O.}$, AS_H and AS_N., but observed a clear decline in the performance of the aerobic zone at AS_B.. This is a result of the operating at underloading conditions with high BOD effluent concentration. Comparing with typical design and operational parameters values reported in the literature, the F/M ratio were significantly lower with reference values, considering the actual HRT, the values were highly significantly in all cases. According to the typical reported values for HLR and SLR, all plants presented HLR values lower than the reported values. However, considering SLR, the values were below the expected for all cases. The difference shown between the influent flows of $AS_{B.O.}$, AS_H and AS_N did not influence significantly the plants' performance, considering the effluent quality. No plants operating at overloading due to lack of water consumption, while AS_H and AS_N operating at critical loading, as for $AS_{B.O}$ and AS_B are operating at underloading conditions.

4.4.2 Collective onsite wastewater treatment system

4.4.2.1 Al-Aroub College Duckweed Based Pond System

As mentioned in literature review, the ponds were operating as semi-continuous flow reactors. The influent wastewater depends mainly on the seasons where attendance of students and staff are active at Al-Aroub College, so the wastewater flows are varies from day to night and from season to season may be almost non-existent in the summer season. It is seen that, as expected, when the ponds operated under downloading conditions (high HRT, it was calculated approximately equal to 68 days), there was a tendency to a decreased effluent BOD concentration, with results confirmed by the analytical tests. Taken into account the results obtained when the plant was operating at critically conditions, as shown in the reference.

4.4.2.2 Extended Aeration Process – Chlorine Disinfection and Sand Filtration

The treatment plant is designed to treat 50 m^3 of domestic wastewater per day which collected using a 7 m^3 vacuum truck which conducts around 7 trips per day to deliver the required amount of wastewater that emptied from cesspits. Problem depends on the commitment of the truck driver to collect the required amount, where it was noted that the plant was not working on regularly, which refers to fluctuation in the wastewater flow. On summer of 2010, the plant was working within downloading conditions, the same period where the samples were collected, this indicates that the performance of the plant when it working on downloading conditions is scanty depending on the removal efficiency achieved for BOD.

Unfortunately, there is no data about the typical design of the plant and its efficiency, so the comparison of its performance was not actually done.

4.4.2.3 Septic Tank - Anaerobic Upflow Gravel Filter - Aerobic Trickling Filter followed by Polishing Sand Filter

Actually no data are available related to the typical design and operation parameter for $UFGF._{Sr}$ plant. However, the Plant is working at critical conditions and provided a good BOD removal efficiency but with increased effluent BOD concentration.

4.4.3 Community onsite wastewater treatment system

4.4.3.1 Up-flow Anaerobic Sludge Blanket (UASB) - Horizontal Flow Constructed Wetlands

As previously mentioned in literature, the community onsite $CW._N$ plant is overloaded. The actual daily of wastewater flow was estimated at 200 m^3. Moreover, the sewage that reaches the constructed wetlands infiltrates into the surrounding layers and does not reach the effluent storage tank. The reason of that is because of the wetlands' lagoons which lined in base and sides with high density polyethylene (HDPE) do not bear to the enormous pressure because of the increased flow of wastewater more than expected. Even though the performance of the plant seems much deteriorated.

It was calculated (Table 4.9) the theoretical, design and operational parameters for community onsite $CW._N$ WWTP, where it was found that the original design of the unit operations of the $CW._N$ was not based on real data of the quantity and analysis of wastewater but assumed values. The actual design data was calculating depending on the actual flow rate obtained. The theoretical design data was calculating depending on the actual design capacity as reported and the reported design data was not similar with the origin one, which, lead us to the following truth that the design of $CW._N$ was made by a non-experienced engineering office. Based on the results obtained from the monitoring phase of the $CW._N$, Table 4.6 illustrates the reported, theoretical calculated, and the actual design data for an adequately treated effluent for $CW._N$.

Table 4.9 Theoretical, reported and actual design data for $CW._N$.

Design Parameter	Unit	Up-flow Anaerobic Sludge Blanket (UASB)	Horizontal Flow Constructed Wetlands (HFCWs)		
			Reported	Theoretical	Actual
Design capacity	(m^3/d)	25-50	120 but 90	120	200
Surface area (A_s)	(m^2)	16	1000	284.1	473.5
Cross-section area (A_c)	(m^2)		18	24	40
Bed length (L)	(m)	4	50	11.84	11.84
Bed width (W)	(m)	4	18	24	40
Bed depth (d)	(m)	5	1	1	1
Hydraulic residence time (t)	(d)	1.6	7 but 2.1	0.83	0.83

4.5 Evaluation of different treatment systems performance

In order to compare the overall performances of the different plants, a general efficiency indicator was determined as an average of calculated different removal efficiencies of evaluated parameters, as follows:

$EG = 1/NE_p\,[E_{p1} + E_{p2} + \ldots + E_{pn}]$ …………………….…………………………..…….. 4.1

Where EG is the general efficiency indicator of removal (%), E_{p1} is the average removal efficiency of first parameter (%), E_{p2} is the average removal efficiency of second parameter (%), NE_p is the number of the parameters. Figures 4.70 – 4.72 illustrate the general efficiency indicators for each evaluated onsite wastewater treatment plants.

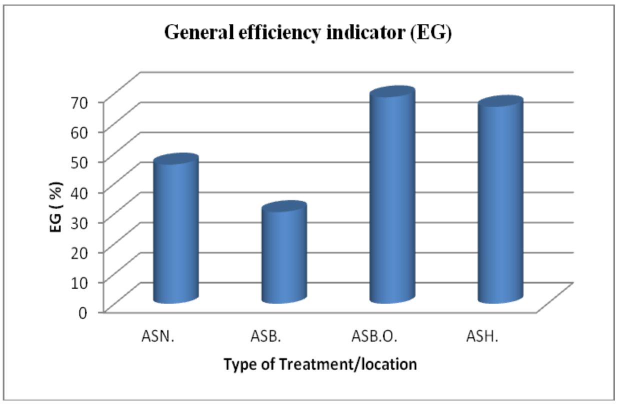


Figure 4.70: General efficiency indicator (EG) values of AS systems

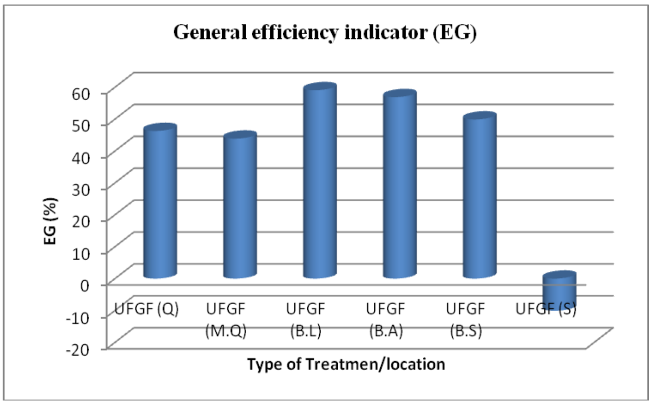


Figure 4.71: General efficiency indicator (EG) values of UFGF systems

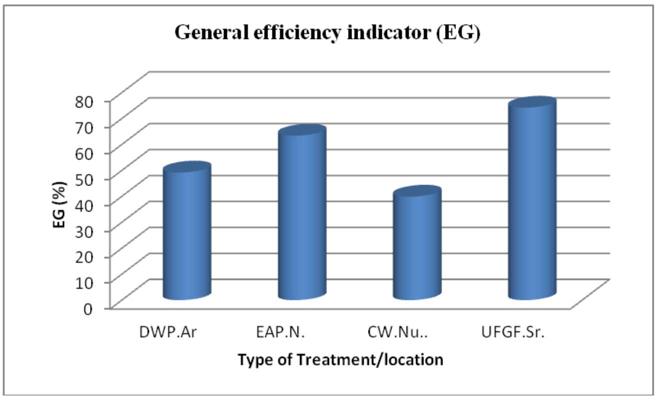


Figure 4.72: General efficiency indicator (EG) values of $DWP._{Ar.}$, EAP.N., $UFGF._{Sr}$ and $CW._{Nu.}$ systems

The highest values of EG were found in $UFGF._{Sr}$ plant using technology of aerobic and anaerobic gravel filters followed by polishing sand filter at collective level with efficiency indicator value of 74.2%. On the other hand, the plants using Activated sludge systems at household level had values of EG in a range of 32.5–70.03%, while the plants using up-flow gravel filter technology at household level had a values of EG in a range of -10.08-59.07%. The plants which have a values of EG in a range of 50-60% are $AS._{H}$, $AS._{B.O}.$, $UFGF._{B.L.}$, $UFGF._{B.A}$ and $UFGF._{B.S}$. The plant $EAP._{N}$ using Extended Aeration Process at collective level had values of 63% EG. While the Duckweed-based pond systems and up-flow Anaerobic Sludge Blanket following by Horizontal Flow Constructed Wetlands were found with general efficiency indicator values less than 40%. The differences of values of EG among the different technologies reflect the status of environmental and the operational conditions for each plant.

Sum up the end, there were cases of very poor effluent quality with comparison between removal efficiencies and expected values according to the literature. However, the poor or good Performance of different evaluated onsite wastewater treatment technologies was observed on underloading conditions and also good effluent quality with overloading conditions, there were cases of plants operating within the critical loading, but without a good suitable effluent quality.

CHAPTER FIVE

CONCLUSION AND RECOMMENDATION

5.1 Conclusion

After an in depth analytical discussion for laboratory results and analysis of the questionnaire statistically the researcher arrives for the following concluding remarks:

In technical terms,

- The aerobic and anaerobic gravel filters followed by polishing sand filters and activated sludge technologies were more efficient and gave more stable operation than the others. Extended Aeration plant had lower efficiency, probably as a consequence of problems derived from the suspended solids separation, increasing of HRT and incomplete aeration. This was probably as a result of operating it at a lower loading than its original design intended. Duckweed-based pond and up-flow Anaerobic Sludge Blanket following by Horizontal Flow Constructed Wetlands technologies was found to be inadequate in meeting the effluent irrigation standard.

- Depending on the interruption of water supply, the low consumption rate of water in the targeted households has resulted in high pollution loads of the influents leaded in inefficiency of most of plants.

- Failure of up-flow gravel filter due to clogging as in case $UFGF._S$ is the main concern in the long term operation of the treatment system.

- The original design of the unit operations of the $CW._N$ was not based on real data of the quantity and analysis of wastewater but assumed values.

- The variability of performance for each plant is mainly influenced by temporary changes of the raw loads, different technologies used, the elimination rate, and the current situation for each.

- The availability of experienced engineering designer, skilled personnel, spare parts for repair, and effective operation, maintenance and monitoring are more crucial than the type of technology.

In statistical terms,

- The good point that found through the analysis of the questionnaire that a high percentage of onsite wastewater treatment plants which are working on bad situation affected by the periodic follow up of operation which is the main factor that affecting on the failure of these plants, and the reason behind that the people have a bad image and they confirmed on their dissatisfaction about the periodic cleaning and follow-up operation of the plant.

- 13% of the existing onsite wastewater treatment plants in Palestinian rural areas which are working well, while 39% working with moderate efficiency, the plants which work with less efficiency estimated as much as 15%, whilst the rest of the plants had been stopped.

- A good percentage (10%) of respondents who have onsite household activated sludge systems say that they stopped their plants because of the high of consumption in the operational expenses of electricity for the plant which estimated 30 Nis per month.

- Many people have a lack of awareness for operating and follow-up of the plants, where it was seen as a good number of them were adding fertilizer on the treatment plant, as they thought that helps for increasing the efficiency of treated wastewater for irrigate crops.

- The total cost of capital expenditure for construction the onsite wastewater treatment plants which have been covered during the questionnaire survey estimates at 1,075,800$, including 354,200$ have been exploited in the plants which have stopped working shortly after its construction and 163,000$ is the cost of the plants which still working well while the remaining amount is estimated 558,600$ is the cost of the plants which need to a real maintenance and rehabilitation.

- The Hydraulic Retention Time could be the most critical process parameter that may affects on the efficiency of the different technologies plants, the reason behind that is these plants are subjected to the fluctuate consumption of water, especially in the plants which are located in the middle or southern part of West Bank. This is result from the nature of the event in Palestinian rural region that suffer from lack and sharp decrease of water due to the current situation by Israeli occupation.

5.2 Recommendation

Based upon the above concluding remarks the researcher will try to suggest some recommendations that may enhance the technical implementation of onsite wastewater treatment plants in Rural Palestine: and these recommendations can be summarized in the following points:

- That 6000 euros is not a sufficient amount of funding to conduct the appropriate laboratory tests and doing statistical analysis of the questionnaires where based on that, it was reduced the period of time which was estimated 12 times to be 3 times in order to conduct periodically laboratory tests for each plant intended to be monitored, which in turn may also affect on the efficiency of results obtained.

- Because influent characteristics are changeable for all monitored plants so, much experimental tests for paired influent and effluent data may be needed to adequately characterize performance.

- The comparison between the different technologies under different circumstances for each plant does not mean that the results of the comparison hardly be accurate and that we can determine exactly which is the best technology can adopted, but this indicates that each technology depending on its different design capacity, it must be follow-up, monitor and on-site maintenance in order to make sure of its effectiveness in order to preserve the lives of families who are using the crops that are irrigated by reclaimed wastewater. Otherwise, these plants may become a real health disaster.

- The NGO's which have designed and implemented these plants did not provide us the manual design for each used technology to know the ability of the plant for removal efficiency for each parameter. So, the real comparison did not work.

- Must be issued a license or permit to be explicit by the Palestinian Water Authority for design and construct such plant, either at the household, collective or community level.

- It recommend using large levels of wastewater treatment plants more than small-sized or household levels because of their effectiveness of efficiency are better compared with small-scale as household plants as well as most previous studies indicated that people have a desire to connect with large plants rather small-scale.

- Must take clear measures in order to reduce the problem of wasting the money used for capital, operational and maintenance expenditure for these plants and it has to be a clear strategy for management these plants in various Palestinian rural areas.

- Future studies can use the results of this thesis to identify a useful data about the monitoring and evaluation of the existing onsite wastewater treatment plants and take a look at the overall situation in the sanitation sector in Palestinian rural areas.

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Appendix A: Palestinian Standards

Table A.1 Number and type of barriers for different crops and effluent qualities

Class A: High quality	Class B: Good quality	Class C: Medium quality	Class D: Low quality	Crop	Sand filtration or long retention or 10% effluents .	Effluent disinfection	Distance from drip-irrigation.	Plastic ground cover.	Subsurface irrigation	Inedible pill or shell.
The number of required barriers					One of the 3					
Zero	Forbidden	F	F	Gardens, Play grounds, Parks						
Zero	Zero	Zero	F	Groundwater recharge by infiltration						
Zero	Zero	Zero	F	Discharge to the Sea, 500m far						
Zero	Zero	Zero	Zero	Seeds crops						
Zero	3	3	4	Artichokes	+	+	++	+	++	
Zero	3	3	4	Corn (edible).	+	+	++	+	++	+
Zero	Zero	Zero	F	Green Fodders						
Zero	Zero	Zero	Zero	Dry Fodders						
Zero	2	2	3	Citrus, with Drip irrigation	+	+	++			+
Zero	3	3	4	Citrus, without Drip irrigation	+	+	+			+
Zero	2	2	3	Crops with Inedible pill or shell almonds, pomegranate, pistachios.	+	+	++		++	+
Zero	2	2	3	Deciduous trees (apple, prune, plum, pear, peaches, apricot) and cherry.	+	+	++			
Zero	2	2	3	Tropical fruits (mango, avocado, persimmon).	+	+	+	+	++	+
Zero	2	2	3	Grapes with high trellis.	+	+	++	+	++	
Zero	2	2	3	Grapes with regular trellis.	+	+	+	+	++	
Zero	2	2	3	Sabras	+	+	++	+	++	+

				(cactus)						
Zero	2	2	3	Dates.	+	+	+++	+	++	
Zero	2	2	3	Olives.	+	+	++	+	++	
Zero	2	2	3	Flowers.	+	+	+	+	++	+
Zero	Zero	Zero	Zero	Forest with no public access						
Zero	Zero	Zero	Zero	Industrial and cereal crops						

Table A.2 Reclaimed wastewater Quality Monitoring

No	Indicator	Frequency	Assessment period
1	Total & Fecal Coliforms	One sample/two days	Three months
2	Pathogens	One sample/2 weeks May-October. One sample/month November-April	Additional 2 samples after 2 days if Pathogens are detected. Stop irrigation if the additional samples are positive.
3	Intestinal nematodes & protozoa	One sample/2 months	One year.
4	Regular chemicals	One sample/month	One year.
5	Heavy metals	One sample/year	One year.

Table A.3 Recommended Guidelines by the Palestinian Standards Institute for Treated Wastewater Characteristics according to different applications

Quality Parameter mg/l except otherwise indicated	Fodder irrigation		Gardens, play grounds, parks.	Industrial and cereal crops	Ground water recharge infiltration	Drainage to sea 500m far	Wood land and forests	Fruiting Trees		
	Dry	**Green**						**Citrus**	**Olives**	**Almonds**
BOD_5	60	45	40	60	60	40	60	45	45	45
COD	200	150	150	200	200	150	200	150	150	150
DO	> 0.5	> 0.5	> 0.5	> 0.5	> 0.5	> 1	> 0.5	> 0.5	> 0.5	> 0.5
TDS	1500	1500	1200	1500	1500	1500	1500	1500	1500	1500
TSS	50	40	30	50	50	50	50	40	40	40
Ph	6-90	6-9	6-9	6-9	6-9	6.0-9.0	6-9	6-9	6-9	6-9
COLOR (PCU)	Free	Free	Free	Free	Free	Free from color	Free	Free	Free	Free
FOG	5	5	5	5	5	0	5	5	5	5

PHENOL	0.002	0.002	0.002	0.002	.0002	0.002	0.002	0.002	0.002	0.002
MBAS	15	15	15	15	15	5	15	15	15	15
NO_3	50	50	50	50	50	15	50	50	50	50
NH_4	-	-	50	-	-	10	-	-	-	-
O.KJ.N	50	50	50	50	50	10	50	50	50	50
PO_4	30	30	30	30	30	15	30	30	30	30
Cl	500	500	350	500	500	600	500	400	600	400
SO_4	500	500	500	500	500	1000	500	500	500	500
Na	200	200	200	200	200	230	200	200	200	200
Mg	60	60	60	60	60	150	60	60	60	60
Ca	400	400	400	400	400	-	400	400	400	400
CAR	9	9	10	9	9	-	9	9	9	9
Al	5	5	5	5	1	5	5	5	5	5
Ar	.01	0.1	0.1	0.1	0.05	0.05	0.1	0.1	0.1	0.1
Cu	.02	0.2	0.2	0.2	0.2	0.2	.02	0.2	0.2	0.2
F	1	1	1	1	1.5	-	1	1	1	1
Fe	5	5	5	5	2	2	5	5	5	5
Mn	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2
Ni	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2
Pb	1	1	0.1	1	0.1	0.1	1	1	1	1
Se	0.02	0.02	0.02	0.02	0.02	0.02	0.02	0.2	0.2	0.2
Cd	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01
Zn	2	2	2	2	5	5	2	2	2	2
Cn	0.05	0.05	0.05	0.05	.01	0.1	0.05	0.05	0.05	0.05

Cr	0.1	0.1	0.1	0.1	0.05	0.5	0.1	0.1	0.1	0.1
Hg	0.001	0.001	0.001	0.001	0.001	0.001	0.001	0.001	0.001	0.001
Co	0.05	0.05	0.05	0.05	0.05	1	0.05	0.05	0.05	0.05
FC (CFU/100ml)	1000	1000	200	1000	1000	50000	1000	1000	1000	1000
Pathogens	Free	Free	Free	Free	Free	Free	Free	Free	Free	Free
Amoeba & Gardia (Cyst/L)	-	-	Free	-	Free	Free	-	-	-	-
Nematodes (Eggs/L)	< 1	< 1	< 1	< 1	< 1	< 1	< 1	< 1	< 1	< 1

Appendix B: Photos

The following photos were taking during the field survey and monitoring period.

Photo B.1 Community onsite of waste stabilization bond located in Talita Kumi Shchool in Beit Jala.

Photo B.2 Community onsite wastewater treatment plant consists of Up-flow Anaerobic Sludge Blanket (UASB) - Horizontal Flow Constructed Wetlands located in Nuba Village in Hebron Governorate. The right photo shows a penetration on the bonders of Constructed Wetlands which leads to infiltrates the sewage into the surrounding layers and does not reach the effluent storage tank

Photo B.3 Collective onsite wastewater treatment plant consists of Septic Tank - Anaerobic Upflow Gravel Filter - Aerobic Trickling Filter followed by Polishing Sand Filter located in Attil Village in Tulkarem Governorate

Photo B.4 Collective onsite wastewater treatment plant consists of Septic Tank - Anaerobic Upflow Gravel Filter - Aerobic Trickling Filter followed by Polishing Sand Filter located in Sir Village in Qalqilya Governorate

Photo B.5 Collective onsite wastewater treatment plant consists of Duckweed-based pond system located at Al Aroub agriculture school in Hebron Governorate

Photo B.6 Collective onsite Upflow Gravel Filter wastewater treatment plant in Al Mazr’a Al gharbiya Village in Ramallah Governorate. Right photo shows broken cover of septic tank.

Photo B.7 Household onsite Activated Sludge wastewater treatment plant in Battir Village in Bethlehem

Photo B.8 Household onsite Activated Sludge wastewater treatment plant in Nahalin Village in Bethlehem

Photo B.9 Household onsite Activated Sludge wastewater treatment plant in Beit Omer Village in Hebron Governorate

Photo B.10 Household onsite Activated Sludge wastewater treatment plant in Halhul Village in Hebron Governorate

Photo B.11 Household onsite Upflow Gravel Filter wastewater treatment plant in Sanur Village in Jenin Governorate

Photo B.12 Household onsite Upflow Gravel Filter wastewater treatment plant in Beit Leed Village in Tulkarm Governorate

Photo B.13 Household onsite Upflow Gravel Filter wastewater treatment plant in Beit Anan Village in Jerusalem Governorate. Left photo shows accumulation of oils in the septic tank for several months without removing it which affects on the performance of the treatment plant.

Photo B.14 Household onsite Upflow Gravel Filter wastewater treatment plant in Qibya Village in Ramallah Governorate. Right photo shows penetration in the septic tank which leads to allow entrance of air to be aerobic tank instead of anaerobic status.

Photo B.15 Onsite household Upflow Gravel Filter wastewater treatment plant in Beit Sira Village in Ramallah Governorate

Appendix C: Data Collected

Table C .1 Implemented Technologies of onsite community level in the Rural West Bank.

Treatment Type	Implementing Agency	year of construction	Design Flow (m3\day)	Actual Flow (m3\day)	LOCATION		Status
					Village	Governorate	
Contact Stabilization Pond	Beirzeit University		6,000 PE 100 m^3 per day	60-80	Beirzeit University	Ramallah	working
Extended Aeration Process – Chlorine Disinfection and Sand Filtration	Al-Quds University		350 PE		Abu Dees	Jerusalem	working
Waste stabilization bonds	PHG	2001	130 m^3 per day 1000 PE	-	Talita Kumi School	Bethlehem	Not working \ stopped
Up-flow Anaerobic Sludge Blanket (UASB) + Horizontal Flow Constructed Wetlands		2002 - 2003	120 m^3 per day 200-300 houses 2,000PE	100	Kharas	Hebron	Not working
			120 m^3 per day 200-300 houses 2,000 PE	200	Nuba	Hebron	Malfunctioning overloaded

Table C .2 Implemented Technologies of onsite Collective level in the Rural West Bank.

Implementing Agency	year of construction	Capacity of the Plant	LOCATION		Type of Raw Wastewater	Status
			Village	Governorate		
Septic Tank - Anaerobic Upflow Gravel Filter - Aerobic Trickling Filter followed by Polishing Sand Filter						
PARC	December 2006	14 m^3 per day	Seer Village	Qalqilya	mixed wastewater	Working well with moderate efficiency
	March 2007		Attil City	Tul Karim		Malfunctioning with low efficiency
	March 2008		Zeita City			Working well with moderate efficiency
Anaerobic Gravel Filters followed by Polishing Sand Filters						
PARC	April 2000	15 m^3 per day	Beit Duqqo	East Jerusalem	gray wastewater	Stopped in 2005
PARC	September 2001	15 m^3 per day	Izbet shufa	Tul Karim	gray wastewater	Stopped in 2002 Problems in Pumps and Electricity
PARC	December 2002	15 m^3 per day	Nuba	Hebron	gray wastewater	Stopped in 2006 Clogging problems Pipeline still working
PARC	December 2002	15 m^3 per day	Al Shokeh	Gaza	gray wastewater	Stopped because it was destroyed by the Israeli invasion 2005-2006
PARC	August 2002		Beit Lahia	Gaza	gray wastewater	Stopped in 2005
PARC			Deir Alballah	Gaza	gray wastewater	
PARC			Khanyounis	Gaza	gray wastewater	
Small Scale Activated Sludge (Extended Aeration Process – Chlorine Disinfection and Sand Filtration)						
ARIJ	2006	50 m^3 per day	Nahalin	Bethlehem	Mixed wastewater	working with moderate efficiency
Anaerobic Baffled Reactor – Activated Sludge process – Multimedia Granule Filtration – Ultraviolet Disinfection						
Birzeit university	2007	10 m^3 per day	Ein Siniya	Ramallah	Mixed wastewater	Not working – stopped since 2009

Septic Tank - Constructed Wetland						
PHG	2004	600 PE 40 m^3 per day	Sarra	Nabuls	Mixed wastewater	Not working - stopped since 2006
United Nations Development program (UNDP)	2004	400 houses	Zeita	Tulkarm	Mixed wastewater	Malfunction well with low efficiency
Septic Tank – Horizontal Flow Constructed wetlands						
PARC	September 2007	11.2 m^3 per day	Biddya city	Salfit	mixed wastewater	Malfunction with low efficiency
PHG	2004	40 m^3 per day	Hajja	Qalqiliya	mixed wastewater	Working well with moderate efficiency
Upflow Anaerobic Sludge Blanket (UASB) – Horizontal Flow Constructed wetlands						
PHG	2005	100 houses	Bani Zeid (Beit Reema Deir Ghassaneh)	Ramallah	gray wastewater	working will low efficiency
Septic tank (ST) and Bio-filter hybrid (BF) Anaerobic Upflow Gravel Filter						
PHG	2001	20 m^3 per day 40 houses 400 PE	Deir Samet	Hebron	mixed wastewater	Malfunction Working with low efficiency (non-organic and pathogens)
Septic tank - Anaerobic filter						
PHG	2002	70 families . 250 PE 30 m^3 per day	Ijnisinya	Nablus	gray wastewater	Not working
Septic Tank followed by Trickling Filter						
PHG	2001	12 m^3/day	Abassan region	Gaza Strip	Black wastewater	Not working
Septic Tank (ST) + Multilayer Trickling Filter (TF) + Polishing Pond (PP)						
		50 PE	Turmus Ayya school	Ramallah	Black wastewater	Not working
		50 PE	Al-Samu'school	Hebron	Black wastewater	Not working
Septic Tank (ST) + Trickling Filter (TF) + Sand Filter (SF)						
		500 PE	Aba	JENIN		
Duckweed-based pond system - Small-scale biochemical system - Aeration tank						
EQA	1997	8 m^3/day	Al Aroub agriculture school	Hebron	Black wastewater	Malfunctioning with low efficiency
Duckweed and Algae based ponds						
BZU	1998		Beirzeit University	Ramallah	Black wastewater	Under evaluation

UASB-Septic Tank						
BZU			Beirzeit University	Ramallah	Black wastewater	Under evaluation
Sequencing Batch Reactors						
		200 PE	Jericho Casino	Jericho	Black wastewater	working
Septic tank - Up-Flow Gravel filter						
PWEG		60 PE	Al Mazr'a Algharbiya	Ramallah	Black wastewater	Not Working

Table C .3 Implemented Technologies of onsite household level in the Rural areas of West Bank and Gaza Strip.

Implementing Agency	year of construction of the plant	Capacity of the Plant	LOCATION		Number of treatment plants	Type of Raw Wastewater	Status
			Village	Governorate			
Septic tank - Up-Flow Gravel filter – aerobic filter							
PARC	1997	7,10,14 persons	Terqoia Hosan 5 villages	Hebron	20	Gray wastewater	Not working
ARIJ	2006	0.75 m^3 per day	Bani Na’eem		14		working
			Al Reheya		5		
			Beit Ummar		6		
			Yatta		75		
PHG	2005	0.5 m^3 per day	Halhul School Tarqumia School Idhna School Ash Shuyukh School Bani Na'im School Dura School Kharsa School		10		Not working
FAO	2008		Yatta		15		working
			Samou		10		
	2009		Izna		8		
UWAC + PWEG	2008	9-7 PE 10-15 PE	Bedouin yatta- (AnNajadah and AzZuweidin)		20		working
PARC	1997	7,10,14 persons	Foqeen Husan 3 villages	Bethlehem	15	Gray wastewater	Not working
ARIJ	2006	0.75 m^3 per day	Dar Salah		4		working
			Al Ubedeya		8		
			Z’tara		3		
PHG		0.5 m^3 per day	Al Ubeidiya School		1		Not working
			Beit Sahur School		1		
			Nahhalin School		1		

PARC	1997	7,10,14 persons	2 villages	Jerusalem	10	Gray wastewater	Not working
	2005		Bet Seira		12		
PWEG	2007	9-7 PE 10-15 PE	Bet Inan		7		
	2008		Qatannah		12		working
			Alqubeba		1		
			Beit Hanina		1		Not working
	2009		Dayr Rafat		2		
PARC	1997	7,10,14 persons	8 villages	Nablus	35	Gray wastewater	Not working
PHG	2002	0.5 m^3 per day	Aqqaba School		1		
			Awarta School		1		
			Jamal Abdel Naser School		1		
			Sabastyia School		1		
			tallouza school		1		
			Al-Badhan School		1		
FAO	2009		Talfit		6		working
PARC	1997	7,10,14 persons	Bilien	Ramallah	4	Gray wastewater	Not working
			Biddu 4 villages		16		
	2005		Qebia		18		
			Kufr Al-Dik		12		
PWEG	2008	9-7 PE 10-15 PE	Jifna		5		working
			Dura Al Qaraa'		17		
	2009		Ein Seenya		5		
	2008		Kharbatha Almusbah		12		
PHG	2005	0.5 m^3 per day	Bil'in Ras Karkar Deir Ibzi' Kharbatha Al Misbah		12		Not working
FAO - QWC		1 m^3 per day	Qebia		50		working
PARC		7,10,14 persons	Masha	Salfeet	10	Gray wastewater	Not working
	1997		8 villages		26		
FAO	2008	1 m^3 per day	Zawea		5		
			Haris		5		

PARC		7,10,14 persons	Jayyous	Qalqilya	10	Gray wastewater	Not working
			Kufr Thelth		8		
	1997		8 villages		26		
PHG	2002	0.5 m^3 per day	kafr thulth school		1		
PARC	1997		5 villages	Tulkarem	25	Gray wastewater	Not working
FAO	2009	1 m^3 per day	Beit Leid		6		working
PARC	2005	7,10,14 persons	ALjadeedah	Jenin	50	Gray wastewater	Not working
	1997		10 villages		140		
PHG	2002	0.5 m^3 per day	Meselyia Rabah Seir Al Jadida Tayaseer		60		
			Sanur		57		
			Jenin Schoool		1		
FAO	2008		Jibaa		9		working
PARC	1997	7,10,14 persons	5 villages	Tubas	20	Gray wastewater	Not working
PARC	1997	7,10,14 persons	4 villages	Jericho	15	Gray wastewater	
PARC	1997	7,10,14 persons	3 villages	Gaza	12	Gray wastewater	
PHG	2004 -2005	0.5 m^3 per day	Abbasan Al Kabeera Abbasan Al Sagheera		7		
PARC	1997	7,10,14 persons	5 villages	Dier Alballah	17	Gray wastewater	Not working
PARC	1997	7,10,14 persons	8 villages	Khanyounis	20	Gray wastewater	Not working
PHG	2004 -2005	0.5 m^3 per day	Bani Suhalla		7		
Activated Sludge followed by sand filter							
ARIJ	2006	1 m^3 per day / 5-10 PE	Sa'ir	Hebron	11	mixed wastewater	working
			Shuyukh		11		
			Halhul		15		
			Beit Ummar		15		
			Beit Kahil		15		
			Taffuh		15		
ARIJ	2006	1 m^3 per day / 5-10 PE	Nahalin	Bethlehem	11	mixed wastewater	working
			Bateer		15		
			Al walaja		15		
			Al Khadir		12		

			AL Shawawra		15		
			AL abedea		15		
			Dar Salah		15		
Constructed wetland							
WEDO + FOEME		1 m^3 per day	Baqa al sharqiya ‘school	tulkarem	1	Mixed wastewater	Not working
			adaweya school		1		
Subsurface Drainage technique (SDT)							
SCF	1989 -1998		Tamoun		100	Mixed wastwater	Not working
			Oareen	nablus			
			aldowareh				
			Sair				
			Bani naim	hebron			
			Alwalajeh	bethlehem			
Septic tank- subsurface treatment							
ANERA			Few schools in some Palestinian villages		9		Not working
Septic Tank (ST) – Trickling Filter (TF) – Sand Filter (SF)							
		20 PE	Aba School	Jenin	1		Not Working

Appendix - D

Questionnaire: monitoring of wastewater treatment plants

This questionnaire has been designed to provide a basic information about each existing wastewater treatment plant which have been selected by Excel Selector within the stratified sample in Palestinian rural areas, in terms of assessing and monitoring their process performance in order to be able to choice the best system to be adapted in the case of Palestinian rural areas, and to be able to add an technical enhancement for every process if possible.

1 BASIC DATA OF THE WASTEWATER TREATMENT PLANT

1. Name of owner ____________________________ Gender: _____________

Address: _________________ City: _____________ Country: _____________

Telephone number:_______________________

2. GPS Coordinates ____

3. Where is the wastewater treatment plant located? _________________________

4. What is the Cost of the treatment plant? _______________

5. In what year was the plant built? _____________________

6. What is the name of NGO which has constructed the treatment plant?

____

7. Wastewater treatment plant agreement? (If Exist) _______________"attach"

8. Wastewater treatment plant engineering design? (If Exist) _______________"attach"

9. PWA Permit #: ____________________ (A copy of permit must be attached.)

10. Electricity bill and Water bill: _______________ (A copy of bills must be attached.)

11. Name and Type of technology used ___________________________________

12. Level of wastewater treatment plant :

Individual	□	Collective	□
Community	□	Other	□

____

____

13. The major wastewater flow that enter the wastewater treatment plant

Municipal wastewater	□	Domestic wastewater	□
Industrial wastewater	□	Storm water	□

____

____

14. Type of raw wastewater treated?

Grey wastewater treatment	□	mixed wastewater treatment	□

15. What is the Objective of the treatment plant?

Reuse	□	Infiltrating groundwater	□
Environmental protection	□	Other	□

____

____

16. What is the Status of the treatment plant?

Working Well	□	Working with moderate efficiency	□
Working with low efficiency	□	Not Working \ Stopped	□

Note:____

____

17. Is/are any member of the family / neighboring residents suffering from illness due to eating vegetables irrigated with treated wastewater?

Yes	□	No	□

If Yes, what kind of disease that has been injured by?

____

____

18. How many stages of treatment does the facility use?

Primary	□	______________________________________
Secondary	□	______________________________________

Tertiary □ ______________________________

Other □ ______________________________

19. What is the capacity of the treatment plant?

Liters per day (average) ____________

Number of People and/or Employees ____________

Peak Daily Flow Estimate ____________

20. How often is the sludge removed? ___________

21. In which means the sludge is removed?

____

22. How is the sludge disposed of?

Burned □ Landfill □

Fertilizer □ Other □

____

____

23. Where does the treated wastewater go after it leaves the plant?

Home Garden □ Wadi □

Irrigation drip □ Other □

24. Reuse Scheme

Restricted Irrigation □ Unrestricted Irrigation □

Other □

____

25. Type the names of the Crops that are irrigated with treated wastewater?

____

____

26. Do any neighboring residents suffer from an unpleasant smell caused by the wastewater treatment plant?

Yes □ No □

27. What is the distance between the treatment plant and your / neighboring residents home? _________________

28. Have there been any modifications of the plant in recent years?

____

____

____

____

29. Are there any plans for additional improvements to the plant?

____

____

____

____

30. Wastewater analysis information (influent)

Wastewater BOD _____________

Wastewater COD _____________

Wastewater Suspended Solids _____________

31. Treated water- PWA requirement - If known (effluent)

Wastewater BOD _____________

Wastewater COD _____________

Wastewater Suspended Solids _____________

2. WASTEWATER TREATMENT INFORMATION

32. Primary Treatment Processes

	Processes	*Size (if know)*	*Main operational problems (if exists)*
□	Bar or bow screen	_____________________	_________________________________
□	Grit removal	____________________	_________________________________

□	Primary sedimentation	__________	________________
□	Comminution	__________	________________
□	Oil / fat removal	__________	________________
□	Flow equalisation	__________	________________
□	Ph neutralisation	__________	________________
□	Imhoff tank	__________	________________
□	__________	__________	________________
□	__________	__________	________________

33. Secondary Treatment Processes

	Processes	***Size (if know)***	***Main operational problems (if exists)***
□	Activated sludge	__________	________________
□	Extended aeration	__________	________________
□	Aerated lagoon	__________	________________
□	Trickling filter	__________	________________
□	Rotating bio-discs	__________	________________
□	Anaerobic treatment/UASB	__________	________________
□	Anaerobic filter	__________	________________
□	Stabilisation ponds	__________	________________
□	Constructed wetlands	__________	________________
□	Aquaculture	__________	________________
□	__________	__________	________________
□	__________	__________	________________

34. Tertiary Treatment Processes

	Processes	***Size (if know)***	***Main operational problems (if exists)***
□	Nitrification	__________	________________
□	Denitrification	__________	________________
□	Filtration	__________	________________
□	Chemical precipitation	__________	________________
□	Disinfection	__________	________________

- □ Chemical oxidation ____________ ____________
- □ Biological P removal ____________ ____________
- □ Stabilisation ponds ____________ ____________
- □ Adsorption ____________ ____________
- □ Ion exchange ____________ ____________
- □ electrodialysis ____________ ____________
- □ Constructed wetlands ____________ ____________
- □ Aquaculture ____________ ____________
- □ ____________ ____________ ____________

Other comments

____

____

____

____

____

____

____

3 CONTROL AND MONITORING SYSTEMS

35. Which are the most critical process parameters that may affect the efficiency of the Wastewater treatment plant?

	Parameter	*Process*	*Current Automatic Control?*
□	Horizontal flow velocity V_h (m/s)	Bar screen	Yes□ □No
□	Flow rate (Q) m^3/s	Bar screen	Yes□ No□
□	Horizontal flow velocity V_h (m/s)	Grit removal	Yes□ No□
□	Particle settling rate (v_s)	Grit removal	Yes□ No□
□	Hydraulic retention time, t	Grit removal	Yes□ No□
□	Surface loading rate (or overflow rate) (v_s)	Sedimentation	Yes□ No□
□	Hydraulic retention time, t	Sedimentation	Yes□ No□
□	Concentration of TSS in the influent flow	Sedimentation	Yes□ No□

□	Sludge depth	Primary treatment	Yes□	No□
□	Volumetric (organic) loading rate	Anaerobic treatment	Yes□	No□
□	Volumetric (organic) loading rate	Trickling filter	Yes□	No□
□	Volumetric (organic) loading rate	Anaerobic ponds	Yes□	No□
□	Surface loading rate	Rotating biological contactor	Yes□	No□
□	Surface loading rate	Stabilization ponds	Yes□	No□
□	Hydraulic retention time, t	Lagoons and ponds	Yes□	No□
□	Sludge loading rate	Activated sludge	Yes□	No□
□	Sludge residence time (sludge age)	Activated sludge	Yes□	No□
□	Dissolved oxygen concentration	Activated sludge	Yes□	No□
□	______________________	______________________	Yes□	No□
□	______________________	______________________	Yes□	No□
□	______________________	______________________	Yes□	No□

36. What are the main problems with the control system of the wastewater treatment plant?

____

____

____

____

____

____