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ADDIS ABABA UNIVERSITY
INSTITUTE OF TECHNOLOGY

CIVIL ENGINEERING DEPARTMENT

Water Supply and Environmental Engineering

MSc THESIS

ON

WATER TREATMENT PLANT PERFORMANCE EVALUATION USING TURBIDITY AS A PARAMETER

A case study on Gafarsa and Lagadadi

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WATER TREATMENT PLANT PERFORMANCE EVALUATION USING TURBIDITY AS A PARAMETER:

A CASE STUDY ON GAFARSA AND LAGADADI WATER TREATMENT PLANT (ADDIS ABABA, ETHIOPIA)

A THESIS SUBMITTED TO THE SCHOOL OF GRADUATE
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ENVIRONMENTAL ENGINEERING PROGRAM

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Doctor Asse Kemal _____________________________________

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ACRONYMS

AAWSSA	: Addis Ababa Water Supply and Sewerage Authority
AD	: Anno Domini
AOC	: Assimilable Organic Carbon
ASCE	: American Society of Civil Engineers
AWWA	: American Water Works Association
BC	: Before Christ
BDOC	: Biodegradable Dissolved Organic Carbon
BPNP	: Biological Perchlorate/Nitrate Process
CSA	: Central Statistics Agency
DOC	: Dissolved Organic Carbon
GAC	: Granular Activated Carbon
GBA	: Carbon Biological Adsorption
GPM	: Gallon Per Minute
GS	: Galvanized Steel
JMP	: Joint Monitoring Program
MMC	: Million Metric Cube

NOM	: Natural Organic Matter
NTU	: Nephelometric Turbidity Unit
OEBF	: Ozone-enhanced biological filtration
PACL	: Polyaluminium Hydroxychloride
pH	: Potential of Hydrogen
RBF	: Rapid biological filtration
SBF	: Slow Biological Filtration
SDWA	: Safe Drinking Water Act
THM	: Trihalomethane
UN	: United Nation
UNICEF	: United Nation International Children Emergency Fund
WHO	: World Health Organization
WTP	: Water Treatment Plant

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ABSTRACT

Gafarsa and Lagadadi Water treatment plants play a vital role in supplying drinking water to Addis Ababa City since about 55 years. The two WTPs are located on opposite direction of the city. Groundwater sources developed at the southwest direction of the city augments the waters sources from the two treatment plants. This study focused on evaluation of performance efficiency of the two water treatment plants in view of turbidity removal. The evaluation was conducted by collecting primary turbidity data of water sample at outlet of clarifiers and filters where turbidity of water is removed. During rainy season due to high turbidity of raw water from the impounding reservoirs, daily production of the plants decrease as the surface loading rate is minimized to maintain the required water quality. Magnitude of floc carryover also increases during the wet periods. Lagadadi treatment plant is constructed in two stages. The second stage clarifiers are equipped with tube settlers while the first stage clarifiers have no any type of settlers installed. As observed from the turbidity removal efficiency evaluation of the two stages filters, the second stage filters are more efficient than that of the first stages. The study result revealed that turbidity removal efficiency of Lagadadi treatment plant filters is better than that of Gafarsa. The average turbidity removal efficiency of Gafarsa and Lagadadi filters are 53.13% and 88.45% respectively.

For improving rainy season output of the water treatment plants, this study recommended construction of bank filtration adjacent to intake from the impounding reservoirs which reduces turbidity of the raw water by half.

Non-uniformity is observed in orifice type weir of the two treatment plant clarifiers, which could be the most probable reason for floc carryover. For minimizing floc carryover, it is recommended to regularly clean the orifices inner surface to maintain uniformity of the cross sectional area which in turn regulate the flow velocity stabilizing the suspended flocs to settle.

1. INTRODUCTION

1.1 General

Globally over two billion people have gained access to improved sources of drinking water in the year 2012 and 116 countries have met their millennium development goal. More than half the world's population, almost 4 billion people, now enjoy the highest level of water access, a piped water connection at their homes.

According to WHO and UNICEF Joint Monitoring (JMP) report titled "Progress on Drinking Water and Sanitation 2014 update", trends in global drinking water coverage in the years 1990 to 2012 indicated that the percentage of piped on premises and other improved water sources are increased from 45 to 56 and 31 to 33 respectively. Even though this achievement is encouraging much remains to be done. More than 700 million people still lack ready access to improved drinking water sources of which nearly half are in sub Saharan Africa.

Regionally since 1990, drinking water coverage in developing regions has increased to 87% Eastern Asia, Southern Asia, South-Eastern Asia and Latin America and the Caribbean all reduced their population without access to improved drinking water sources by more than 50% achieving their millennium development goal target ahead of time. (UNICEF, 2014)

At national level as per the World Fact book, percentage of population having access to improved drinking water sources in the year 2015 is as follows.

Urban: 93.1% of population

Rural: 48.6% of population

Total: 57.3% of population

1.2 The study area

Addis Ababa is the diplomatic capital of Africa. More than 92 embassies and consular representatives cluster in the city where the Organization of African Unity and the UN Economic Commission for Africa have their headquarters.

Geographically the city is located 9^o 01'29" to north and 38^o44'48" to east. In the year 2007 the city has a population of 3,107,423 (CSA, 2007). Total area of Addis Ababa city is estimated to 540 square kilometer of which about 18 square kilometer is rural. The altitude above sea level ranges between 2000m at lowest point and 2800m as a peak of course the Entoto mountains rise up to 3000m.

Despite its proximity to the equator, Addis Ababa enjoys a mild, Afro-Alpine temperate and warm temperate climate. The lowest and the highest annual average temperature is between 9.89 and 24.64^oC. The city rambles pleasantly across many wooden hillsides and gullies cut through with fast flowing streams especially during the rainy seasons from July-September.

With regard to economic activities of the city, the population is predominantly based on different sorts of occupation. These activities include, manufacturing & industry; trade & commerce; home making of different variety; transport & communication; education; health & social services; hotel and catering services; and few agricultural activities. Few of the city dwellers carry out animal husbandry and cultivation of gardens.

1.3 The city water supply

1.3.1 The existing situation

Addis Ababa Water Supply and Sewerage Service Authority which is the sole responsible body for supplying water for the metropolitan is implementing additional projects to alleviate the problem. In addition to the previously constricted surface water abstraction systems, Gafarsa, Lagadadi and Dire, the authority has implemented and put into service ground water exploitation at the vicinity of the city in the south eastern direction to augment the existing supply source. However, the water supply shortage is increasing from time to time.

The city is suffering from serious domestic water supply system problems mainly associated with insufficient quantity up to the current period.

The enormous amount of migration of people to the capital city, growing number of investments and construction works are some of the reasons stated for the shortage.

a) Unaccounted for water

As stated in various researches and reports the main problem of the city water supply is high volume of water loss in the system. This is mainly due to old distribution networks and poor pressure management in the system. The unaccounted for water in the city was about 40% of which more than 25% was loss due to old pipes leakage. To tackle the problem, a leakage study was conducted on treatment plants, service reservoirs, distribution systems, water meters, and other processing points to assess the wastage and find a solution. As a result, old pipes are being changed and others are being properly maintained. However the limited budget is the major barrier to achieve the required goal as fast as possible.

b) Coverage

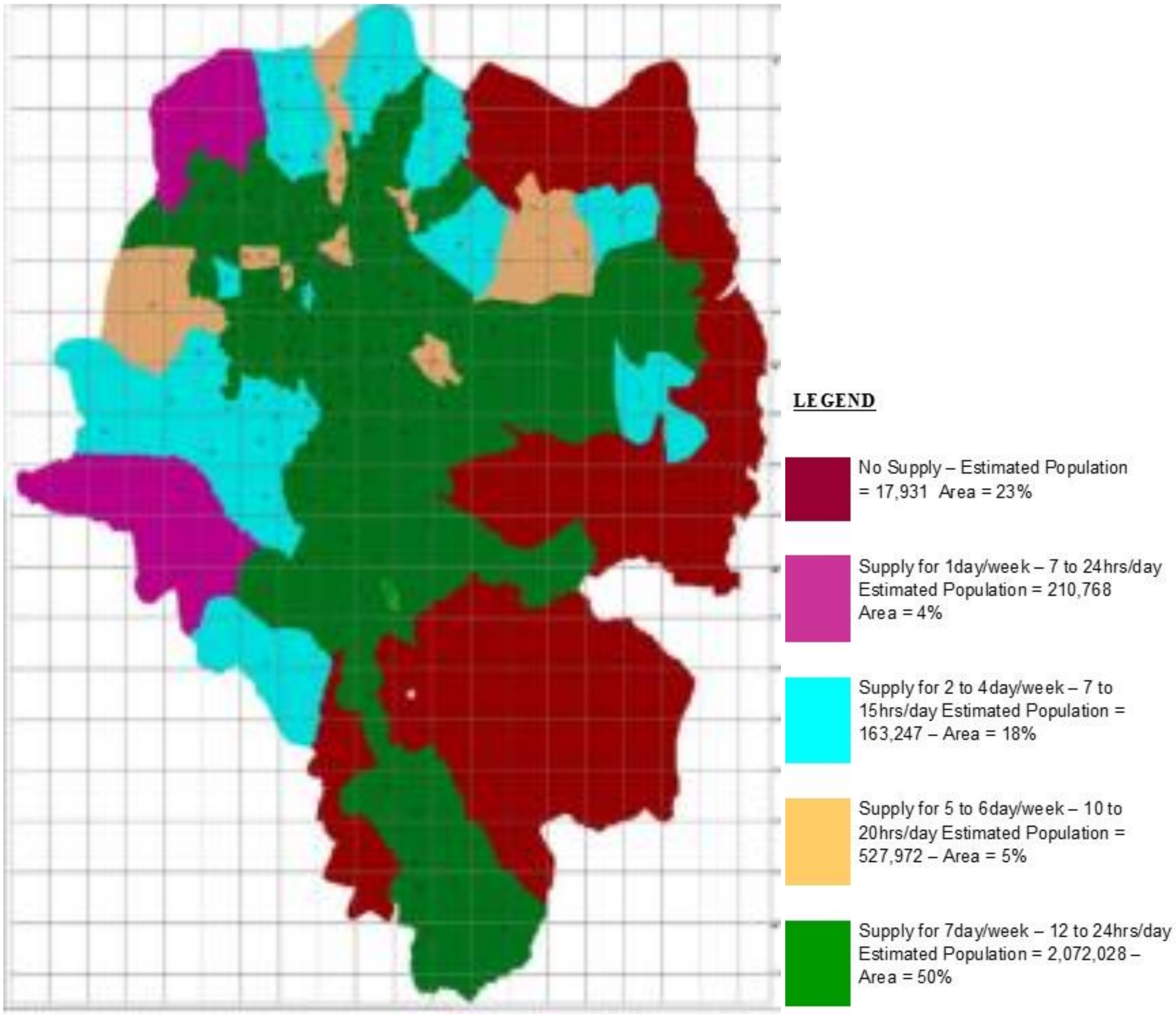
It is common to see people with long queue at public fountains in the periphery of the city, particularly the newly under development areas. Sometimes the old residential areas are also vulnerable to acute water shortage due to frequent service interruption.

Some people with better life standard as compared to others have their own overhead reserve water tanks which helps to cover their demand during service interruption.

In the city, the water supply area coverage and demand satisfaction varies due to topographical non uniformity. Accordingly, some areas of the city have better access to water supply than others being privileged by the low altitude. Those dwellers living at higher altitudes suffer from low water pressure which decreases the amount of water they receive in a day. Of course some of the inhabitants in higher altitudes could get better access to water by the booster pumps installed by AAWSA at various localities.

Based on the above stated fact, Shimeles Qabeto has prepared a map of average daily percapita consumption of the city in the year 2011 in his research paper. This areal coverage of the map is shown in the next page.

Addis Ababa Water Supply Situation in 2011



Source: Un-published MSc Thesis by Shimeles Qabeto (2011)

c) Water Quality Standards

i. Guidelines

The World Health Organization (WHO) published three editions of the Guidelines for drinking-water quality in 1983–1984, 1993–1997 and 2004, as successors to previous WHO International standards for drinking water, published in 1958, 1963 and 1971. From 1995, the Guidelines have been kept up to date through a process of rolling revision, which leads to the regular publication of addenda that may add to or supersede information in previous volumes as well as expert reviews on key issues preparatory to the development of the Guidelines. (Guideline for Drinking Water Quality, 2011)

ii. Regular water quality test in AAWSA

Water quality test is regularly conducted for samples collected from water treatment plants and borehole sites where the sources are from surface water and ground water respectively as per information obtained from the water quality laboratory head. In addition to this, water samples are collected from the user yards and examined for physical, chemical and biological fitness as per the WHO guidelines.

Water quality may be deteriorated due to encrustation in the old pipes, which impose objectionable odor and tastes to pipes water. This problem is observed particularly in the old areas of the city where the pipes are serving beyond their lifetime. The aged pipes also contribute to pollution of the treated water as leaking points may intrude external pollutants when the pressure in pipes are low or when there is service interruption.

Due to negligence of the plumbers and poor controlling mechanism of the system it is common that turbid water is tapped at the user yard immediately after pipe maintenance.

d) Distribution system

The existing water distribution system in Addis Ababa is divided into several distribution sub-systems due to the topography of the city. These sub-systems are interconnected to some extent and supplied from the major sources through separate transfer pipelines.

The existing water transfer and distribution system in Addis Ababa has evolved over several decades. The most recent significant upgrading took place in the late 1 980s under the Stage II Water Supply Project in conjunction with the expansion of the Lagadadi water treatment plant.

Currently except the newly developed areas of the metropolitan, almost all pipes in the distribution system are serving beyond their service life. This is contributing a lot to water loss in the system and deterioration of the water quality at the user's yard. The other problems with the distribution system is poor pressure management, which exacerbate leakage in the system both in the new network and in old network.

1.4 Gafarsa Water treatment plant

Gafarsa treatment plant receives its raw water from two consecutive dams constructed upstream and downstream adjacent to the treatment plant. Initially the main dam was constructed and secondary dam was constructed to obstruct additional surface water to cope up with the increasing demand of the city. The main dam is constructed in 1943 (Masson-Laval-FRANCE, 2008) and modified in 1954.This dam has the usable raw water storage of approximately 6.5MMC. The secondary dam is built in 1966 upstream of the main dam. Its usable raw water is estimated to be about 1.5MMC. The raw water from Gafarsa main reservoir is gravitationally fed into the WTP. Relatively high turbidity and color, low alkalinity and hardness characterize the water. The water treatment was constructed in two stages, stage 1 with production rate of 15,000 m^3/day in 1954, and stage 2 with production rate of 15,000 m^3/day in 1958.

The conventional treatment units incorporated in this water treatment plant include: Chemical dosing units, Flush mixing, Coagulation/Flocculation, Clarification, Filtration, Chlorination and Stabilization (pH adjustment). The clarifiers are of circular up-flow type. There are two similar units of clarifiers in the treatment plant. Both clarifiers are equipped partially with four plate settlers with size 1.24m X 3.68m each. The plates are made of asbestos and supported by metal frame anchored into concrete wall of the clarifiers. The weirs outlet of the clarifiers are of orifice type made by embedding ½" GS pipes at similar level on upper edge of the weir.

There are twelve (12) units of Rapid Sand Filters arranged in parallel to receive effluent from the two clarifiers to remove suspended solids and other microorganisms to an acceptable level.

Some of the other important features of these filters are:

- A filter bed of one grain size only; about 1 mm.
- Washing by simultaneous backwash and air scour, with minimal expansion of the filter bed, followed by rinsing with water only.
- Two wash water pumps giving 480 m^3/hr.
- Two electric air blowers, operating together, to provide scour air for backwash.

Each of the Aquazur filters installed at this plant is fitted with a minimum of 50 long-stemmed nozzles per m^2, screwed through a thin concrete false floor.

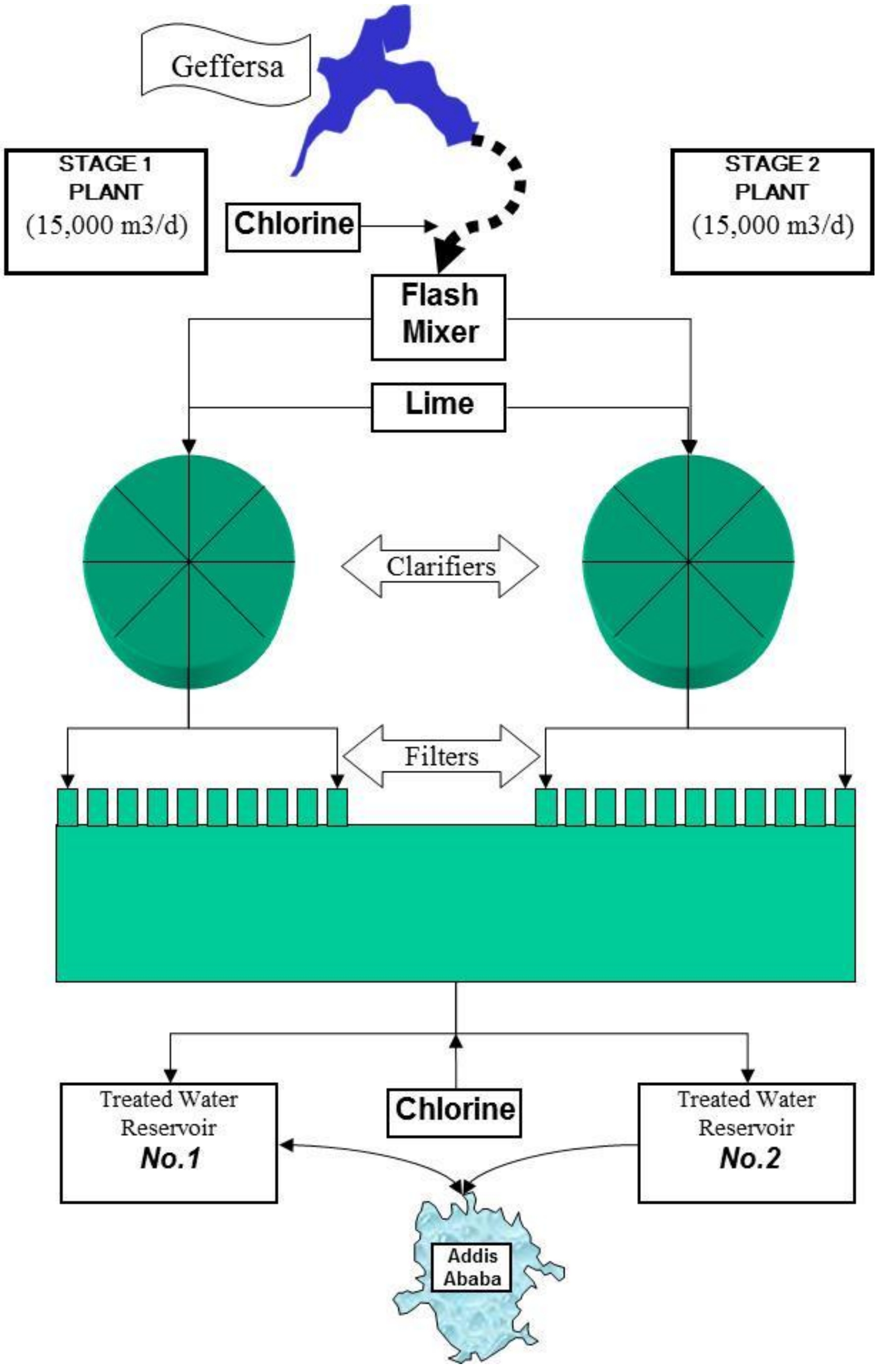


Figure 1-1: Gafarsa Treatment Plant Schematic layout (Source: Previous Reports)

1.4.1 Location of Gafarsa Treatment Plant

This water treatment plant is located in the western outskirt of the city. The geographical location is shown in the map below.

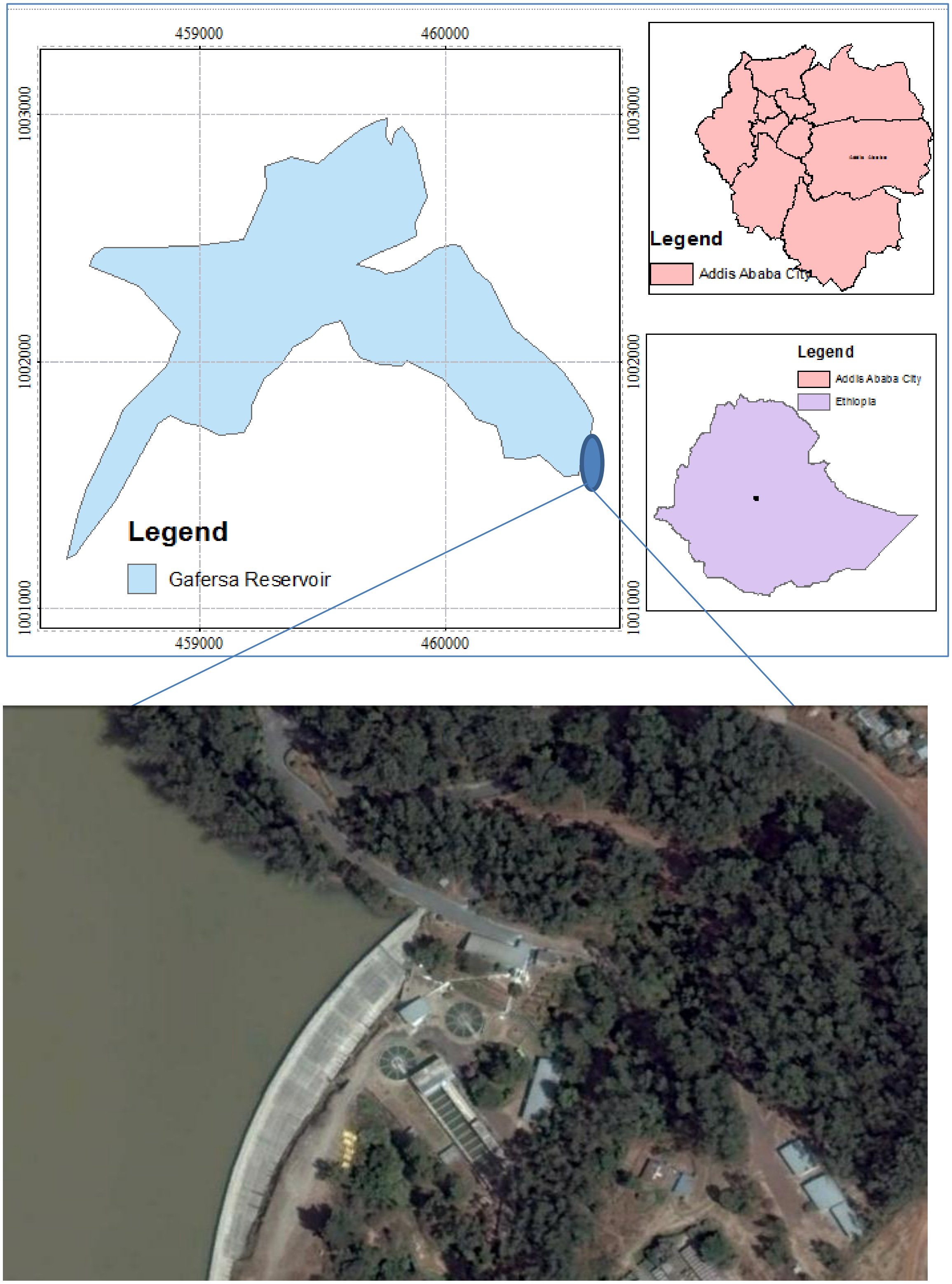


Figure 1-2: Gafarsa treatment plant location

1.4.2 Unit Process Descriptions

The Solids-Contact Clarifiers used at the Gafarsa Water Treatment Plant have a centrally located primary mixing zone above which is located a secondary mixing zone. These mixing zones are completely surrounded by the exterior settling zone, which is connected to them both at the top and the bottom.

The raw water (to which the chemical reagents have previously been added) enters via the influent pipe, passes through a circular, raw water distribution channel, which surrounds the central stirring shaft.

This channel is open and accessible so that there is no risk of being blocked. The chemically treated raw water flows down through pipes to the primary mixing and reaction zone where a rotor-impeller stirring device, located at the top of the primary mixing and reaction zone circulates the flocculated water through the secondary mixing and reaction zone to the settling zone.

The heavy floc formed in the two mixing zones begins settling in this zone and returns to the primary mixing zone, under the hood, where it forms a nucleus for new floc formation. The resultant sludge concentration facilitates rapid flocculation and the formation of a more dense precipitate (floc).

Clarified water, rising through the settling zone, escapes the sludge blanket and is discharged via the clarified water outlet channel, through the clarified water effluent piping to the filters. The layer of sludge, sometimes called the “Sludge blanket” tends to increase in volume as a result of the impurities contained in the raw water and coagulant flocculant materials introduced, and its level rises gradually. When a certain height is reached, the excess sludge must be drawn off.

A special compartment, or zone, called a concentrator, is set aside for this purpose. Excess sludge collects in the concentrator and is drained off at regular intervals by means of automatic valves controlled by timed switches.

1.5 Lagadadi Water treatment plant

This treatment plant receives raw water from the impounding reservoir formed by obstructing surface runoff with the help of dam constructed in 1967.

The treatment plant was designed for a maximum production of 150,000m^3/day. It was constructed in two stages, stage 1 with production rate of 50,000 m^3/day in 1970, and stage 2 with production rate of 100,000 m^3/day in 1986.

The clarifiers constructed are of rectangular type in shape. The second stage has clarifiers equipped with tube settlers for enhancement of floc settlement. Tube settlers have not been installed in the clarifiers of the first stage. Recently the third stage expansion of the water treatment plant is underway. At completion of this third stage expansion works, there is plan to install tube settlers for the previously constructed stage -1 clarifiers together with the new one as per information obtained from the plant operators. Orifice types of weirs are provided to pass effluent from the clarifiers to the sand filters to reduce suspended matter and turbidities to a level where the filtration process economically and practically can handle the filter loadings and reductions required. Previously, sludge from the existing treatment units have been discharged to the nearby streams without treatment. However, at present sludge treatment unit is under construction to receive and stabilize the sludge from the treatment plant.

Treatment units of Lagadadi plant includes: Flush mixing unit, Pre-chlorination, Coagulation, Flocculation, Clarification, Gravitational sand filtration, Post-chlorinating and Final pH adjustment. There are twelve and twenty units of filters in the first stage and second stages respectively. The filters are backwashed periodically when the head loss of the filter system reaches the maximum limit or when clogged by sludge coming from the clarifiers.

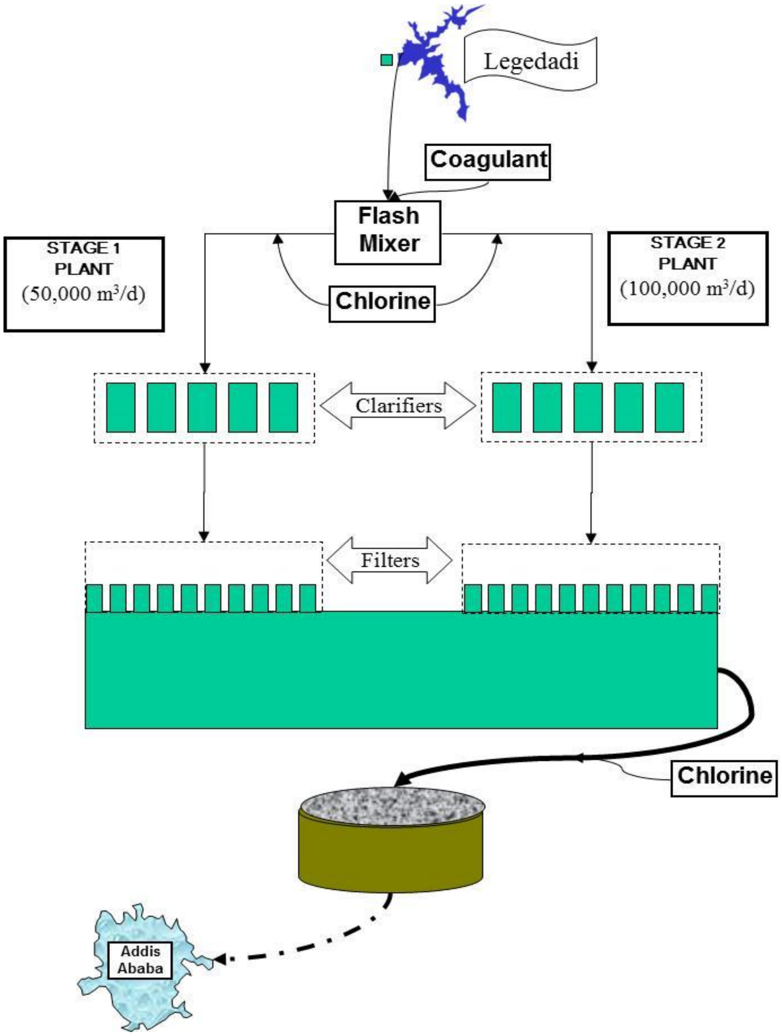


Figure 1-3: Lagadadi Treatment Plant Schematic layout (Source: Previous Reports)

1.5.1 Location of Lagadadi Treatment Plant

This treatment plant is located at the eastern direction of the city. The geographical location is shown in the map below.

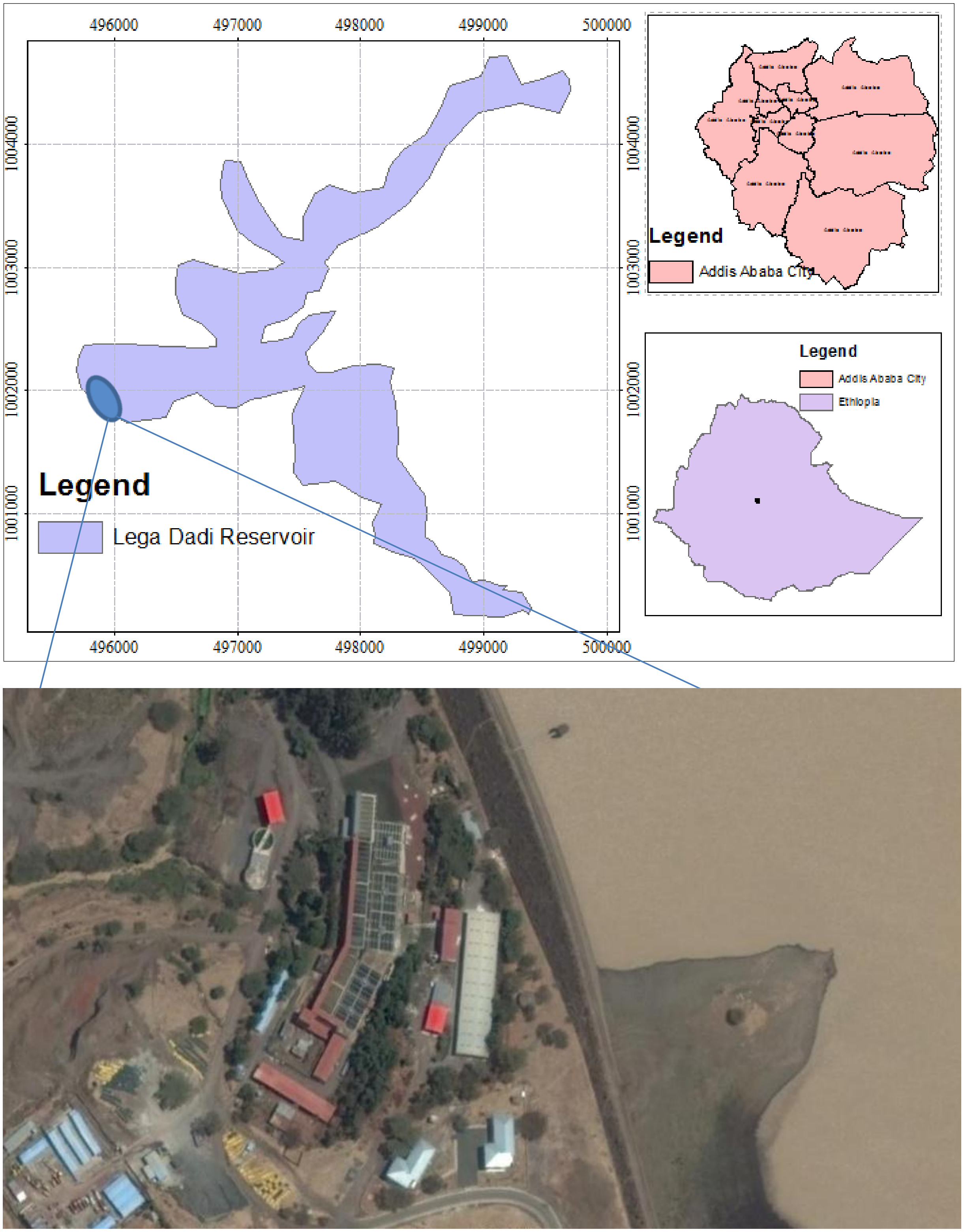


Figure 1-4: Lagadadi Treatment plant location

1.5.2 Unit Process Descriptions

The unit process in Lagadadi treatment plant differs from that of Gafarsa in that the clarifiers are horizontal type and the water flow is horizontal. The other difference is that the clarifiers are equipped with super pulsator at both clarifiers. The other units like filters and backwashing processes are similar in both treatment plants. In general, the Lagadadi treatment plant unit processes are described as follows.

The raw water from Dire dam and Lagadadi dam are mixed and conveyed to chemical dosing and rapid mixing system.

The coagulated water from a rapid mix system is transferred to the Super Pulsator vacuum chamber. The base of the vacuum chamber is open to a tapered distribution channel where coagulated water is uniformly distributed to a series of laterals. Orifices in the distribution laterals point downward to promote scouring of the basin floor and impart energy for flocculation.

The flocculated water is directed upward through a series of inclined settling plates. A sludge blanket is retained from the basin floor and between the settling plates. The sludge mass is directed downward on the front side of the settling plates. Lighter particulate is circulated between the plates, promoting internal solids contact. The lighter solids are continually swept up by the downward moving slurry.

An internal sludge concentrator collects sludge on a sludge blanket expansion. Each sludge collector header is opened upon a prescribed frequency and duration in the control panel. The operating head over the sludge pipes allows the sludge to be removed by gravity. Clarified water is collected uniformly across the length of the settling area. Submerged orifice laterals convey clarified water to a center collection trough. The collector troughs direct the clarified water to sand filters uniformly.

1.6 Problem Statement

As described in the preceding subsections Addis Ababa Water Supply system has many problems, which degrade the service provided for the dwellers.

Raw water quality deterioration is observed from time to time particularly during the period between 1986 and 1995 (Consult, 1998). This water quality deterioration obviously impact performance of the water treatment plants. Especially during rainy season daily output of the treatment plants decrease while the chemical consumption increases.

Another problem observed was the carryover of flocs to filter beds rather than settling to bottom of the sedimentation tank. This problem increased frequency of filter backwashing resulting in wastage of treated water.

Table 1-1: Monthly Water production and backwashing water in 2007 Eth. Calendar

Month	**Gafarsa**				**Lagadadi**		
	Water Produced(m^3)	**Water used for Backwashing**			**Water Produced(m^3)**	**Water used for Backwashing**	
		m^3	**%**			**m^3**	**%**
September	979,130	25,200	2.57%		4,951,881	212,535	4.29%
October	965,632	16,800	1.74%		4,956,921	207,495	4.19%
November	953,206	16,800	1.76%		4,966,056	198,360	3.99%
December	919,848	16,800	1.83%		5,460,000	188,280	3.45%
January	873,891	16,800	1.92%		5,103,135	194,895	3.82%
February	870,452	16,800	1.93%		5,460,000	193,635	3.55%
March	882,668	16,800	1.90%		5,460,000	193,635	3.55%
April	885,583	16,800	1.90%		5,460,000	193,635	3.55%
May	885,518	16,800	1.90%		5,460,000	193,635	3.55%
June	885,302	16,800	1.90%		6,305,333	223,614	3.55%
July	720,255	25,200	3.50%		5,835,885	206,965	3.55%
August	1,086,366	28,700	2.64%		5,864,598	207,983	3.55%

On average amount of water to be used for filter backwashing should not exceed 2% of the treated water. As observed in the above table, particularly at Lagadadi treatment plant, the percentage is above average throughout the year while that of Lagadadi is better.

On average about 19,192m^3 and 201,222m^3 of treated water are being consumed for filter backwashing at Gafarsa and Lagadadi treatment plant monthly i.e. about 640m^3 and 6707m^3 daily. Assuming an average per capita demand to be 100Lit/c/d, the water used for backwashing can serve more than 73,000 people.

In a city where there is scarcity of clean and safe water, this problem is crucial and need solution to minimize the water lost for filter backwashing.

1.7 Objective of the study

1.7.1 General Objective

The general objective of the study is to evaluate performance efficiency of Gafarsa and Lagadadi water treatment plant in terms of turbidity reduction. The unit process to be focused on are clarifiers and filter units, the main segment of the treatment plant process that plays a vital role in turbidity reduction. The desired outcome of the research is improvement of the plants performance efficiency and minimization of filter washing water quantity. The problem is mainly during rainy season when turbidity of the water increases. Performance efficiency results between similar units of the two treatment plants will be compared.

1.7.2 Specific Objectives

Specific objectives of the study are:

- Evaluating the clarifiers and filter units of both treatment plant
- To recommend both short term and long-term solutions and operation and maintenance alternatives aimed at improving the plants performance.
- Comparing performance of the treatment units at both plants
- Assessing the probable cause of floc carryover to filters

2. LITERATURE REVIEW

2.1 Background

Conventional surface water treatment plants have a standard sequence of processes. After screening out large objects like fish and sticks, coagulant chemicals are added to the water to cause the tiny suspended particles that make the water cloudy to be attracted to each other and form "flocs." Flocculation—the formation of larger flocs from smaller flocs—is typically achieved using gentle, constant mixing of the water to encourage particles and small floc to "bump" into each other, stick, and form larger floc. Once the flocs are large and heavy enough to be settled, the water moves into quiet sedimentation or settling basins. When most of the solids have settled out, some form of filtration either with sand or with membranes typically occurs. Disinfection is usually the next step. After disinfection, various chemicals may also be added to adjust pH, to prevent corrosion of the distribution system, or to prevent tooth decay. Ion exchange or activated carbon may be used during some part of this process to get rid of inorganic or organic contaminants. Groundwater sources generally have higher initial quality and tend to require less treatment than surface water sources.

Point-of-use and point-of-entry devices are typically simpler and employ a limited number of technologies. In most developed countries, pathogen-free drinking water that meets international standards is available at every consumer's tap. That being said, a significant number of consumers in the developed world elect to install point of use or point of entry devices as an extra precaution or to improve the aesthetic properties of the water in the public water system. In many parts of the developing world, however, public water systems that provide pathogen-free water are not available and success is measured primarily by reduced risk of diarrheal or other diseases. Thus, a point-of-use technology that is appropriate for one location is not necessarily recommended for another.

2.2 History of Water Treatment

In ancient Greek and Sanskrit (India) writings dating back to 2000 BC, water treatment methods were recommended. People back than knew that heating water might purify it, and they were also educated in sand and gravel filtration, boiling, and straining. The major motive for water purification was better tasting drinking water, because people could not yet distinguish between foul and clean water. Turbidity was the main driving force between the earliest water treatments. Not much was known about microorganisms, or chemical contaminants.

After 1500 BC, the Egyptians first discovered the principle of coagulation. They applied the chemical alum for suspended particle settlement.

After 500 BC, Hippocrates discovered the healing powers of water. He invented the practice of sieving water, and obtained the first bag filter, which was called the 'Hippocratic sleeve'. The main purpose of the bag was to trap sediments that caused bad tastes or odours.

During the Middle Ages (500-1500 AD), water supply was no longer as sophisticated as before. These centuries where also known as the Dark Ages, because of a lack of scientific innovations and experiments. After the fall of the Roman Empire enemy forces destroyed many aqueducts, and others were no longer applied. The future for water treatment was uncertain.

Then, in 1627 the water treatment history continued as Sir Francis Bacon started experimenting with seawater desalination. He attempted to remove salt particles by means of an unsophisticated form of sand filtration. It did not exactly work, but it did paved the way for further experimentation by other scientists.

Experimentation of two Dutch spectacle makers experimented with object magnification led to the discovery of the microscope by Antonie van Leeuwenhoek in the 1670s. He grinded and polished lenses and thereby achieved greater magnification. The invention enables scientists to watch tiny particles in water. In 1676, Van Leeuwenhoek first observed water microorganisms.

In the 1700s the first water filters for domestic application were applied. These were made of wool, sponge and charcoal. In 1804 the first actual municipal water treatment plant designed by Robert Thom, was built in Scotland. The water treatment was based on slow sand filtration, and horse and cart distributed the water. Some three years later, the first water pipes were installed. The suggestion was made that every person should have access to safe drinking water, but it would take somewhat longer before this was actually brought to practice in most countries.

In 1854 it was discovered that a cholera epidemic spread through water. The outbreak seemed less severe in areas where sand filters were installed. British scientist John Snow found that the direct cause of the outbreak was water pump contamination by sewage water. He applied chlorine to purify the water, and this paved the way for water disinfection. Since the water in the pump had tasted and smelled normal, the conclusion was finally drawn that good taste and smell alone do not guarantee safe drinking water. This discovery led to governments starting to install municipal water filters (sand filters and chlorination), and hence the first government regulation of public water.

In the 1890s America started building large sand filters to protect public health. These turned out to be a success. Instead of slow sand filtration, rapid sand filtration was now applied. Filter capacity was improved by cleaning it with powerful jet steam. Subsequently, Dr. Fuller found that rapid sand filtration worked much better when it was preceded by coagulation and sedimentation techniques. Meanwhile, such waterborne illnesses as cholera and typhoid became less and less common as water chlorination won terrain throughout the world.

However, the victory obtained by the invention of chlorination did not last long. After some time the negative effects of this element were discovered. Chlorine vaporizes much faster than water, and it was linked to the aggravation and cause of respiratory disease. Water experts started looking for alternative water disinfectants. In 1902 calcium hypo chlorite and ferric chloride were mixed in a drinking water supply in Belgium, resulting in both coagulation and disinfection. In 1906 ozone was first applied as a disinfectant in France. Additionally, people started installing home water filters and shower filters to prevent negative effects of chlorine in water.

In 1903 water softening was invented as a technique for water desalination. Cations were removed from water by exchanging them by sodium or other cations, in ion exchangers.

Eventually, starting 1914 drinking water standards were implemented for drinking water supplies in public traffic, based on coliform growth. It would take until the 1940s before drinking water standards applied to municipal drinking water. In 1972, the Clean Water Act was passed in the United States. In 1974 the Safe Drinking Water Act (SDWA) was formulated. The general principle in the developed world now was that every person had the right to safe drinking water.

Starting in 1970, public health concerns shifted from waterborne illnesses caused by disease-causing microorganisms, to anthropogenic water pollution such as pesticide residues and industrial sludge and organic chemicals. Regulation now focused on industrial waste and industrial water contamination, and water treatment plants were adapted. Techniques such as aeration, flocculation, and active carbon adsorption were applied. In the 1980s, membrane development for reverse osmosis was added to the list. Risk assessments were enabled after 1990.

Water treatment experimentation today mainly focuses on disinfection by-products. An example is trihalomethane (THM) formation from chlorine disinfection. These organics were linked to cancer. Lead also became a concern after it was discovered to corrode from water pipes. The high pH level of disinfected water enabled corrosion. Today, other materials have replaced many lead water pipes. (Enzler, n.d.)

2.3 Why Water needs to be treated

The water is found almost everywhere on Earth. Water resources like rivers, lakes, which provide water contain a lot of pollution, garbage unfit for consumption. To be clean, the water should undergo a number of treatments necessary to make it drinkable. Water purifiers designed to eliminate or reduce certain pollutants as well as improve the quality taste of water.

About 1 billion people or roughly 20% of the Earth's population lack access to safe drinking water, and about 5 million people die each year from poor drinking water. This is mainly due to water shortages.

Thus, water treatment is a vital process for human being and day-to-day activities in the routine life styles.

2.4 Significance of Turbidity Removal

Turbidity is one of the primary pollutants regulated in finished drinking water. To protect human health, the EPA has set the level of allowable turbidity in finished drinking water at: 0.3 Nephelometric Turbidity Units (NTU) in 95% of samples. Turbidity limits prevent drinking water from having excessive levels of suspended fine sediment (US EPA 1999). Suspended sediment is of concern for drinking water safety as it can reduce the effectiveness of disinfection treatments (LeChevallier et al 1981), harbor pathogens (e.g. Chang et al 1960, Tracy et al 1966, Sen & Jacobs 1969, Meschke & Sobsey 1998), contribute to formation of disinfection byproducts (Nikolaou et al 1999, US EPA 2002a), and carry nutrients, heavy metals, pesticides, and other toxic chemicals (Lick 2008).

Unpleasant tastes and odors frequently co-occur with excessive turbidity (US EPA 1998). Prevention or removal of fine sediment pollution from water reduces these risks to acceptable levels (US EPA 2001).

Most drinking water treatment facilities have the capacity to remove turbidity-causing sediments during treatment of raw water; however, the amount of turbidity that can be effectively removed depends on the treatment technology in use (US EPA 1999, personal communication with PWS managers).

Thus, removing turbidity from raw water is meant to accomplish more than 90% of water treatment.

2.5 Water Treatment Process

There are various physical, chemical and biological water treatment processes. These include pretreatment, Coagulation flocculation, sedimentation, filtration and disinfection. The treatment processes are discussed as hereunder.

2.5.1 Physical Processes

a) Screening

Initially, wood chips, leaves, aquatic plants and floating impurities are removed by the screening process. After the screening, a more compact suspended material will be removed to allow water to flow through the chamber in which it will settle to the bottom.

The purpose of screening process is to restrict the entry of suspended solids such as garbage in the water treatment plant, prevent pump, pipe and equipment from clogging or damage.

Screening is carried to out by a manually cleaned bar screen (large in size, in order to reduce the frequency of screenings collection operations) or, preferably, by an automatically cleaned bar screen (essential in cases of high flow rates of for water with a high solids content). The automatic bar screen is usually protected by a sturdy preliminary bar screen, which should also be provided with an automatic cleaning systems in large facilities and in case of raw water containing a high volume of coarse matter.

To reduce manual operations as much as possible, screening procedures have become increasingly automated, even in small facilities. Automation is essential in situations where large amounts of plant matter are carried by the water and arrive all at once at the bar screen, tending to mat the bars and completely clogging the screen in a few minutes. Fine screens must be automated. The collected refuse is stored in a container of given capacity, calculated according the acceptable frequency of refuse disposal operations.

Usual spacing are:

- For surface waters, between 20 and 40 mm (upstream the strainer)
- For some industrial effluents, especially agri-food effluents, fine bar screening (or at times, medium screening followed by straining)

b) Sedimentation/Clarification Processes

Sedimentation tanks type varies based on nature of working, location and shape. These are listed as follows:

Based on nature of working

- Fill and draw type
- Continuous flow type

Based on location

- Primary tank and
- Secondary tank

Based on shape

- Rectangular tank

- Circular tank
- Hopper bottom type
- Usual spacing are:

Detention time:

Detention time is the theoretical time that the water is detained in a settling basin. It is calculated as the volume of the tank divided by the rate of flow, and is denoted as $\theta = V/Q$.

Surface overflow Rate

Surface Overflow Rate is one of most important factors influencing sedimentation. It is often called Overflow Loading Rate and translates into a velocity which equals the settling velocity of the smallest particle which will be removed. If is simple to remember that loading rates are in gallons per square foot of surface area per unit of time, m^3day/m^2.

Generally, particles settle downward (in a direction opposite water flow) while water rises in a sedimentation basin.

- Particles with settling velocities greater than the Overflow Rate will be removed.
- Particles with settling velocities less than the Overflow Rate will be carried through and out of the basin.

The Surface Overflow Rate is also referred to as the Surface Loading Rate. Here is the equation:

$$Surface\ Loading(V_o), m^3/\text{day} = \frac{\text{Q(m3/day)}}{\text{Surface Area}(\text{m}^2)}$$

When flow increases through the basin, the surface overflow rate (i.e. surface loading rate) will increase and the detention time decreases.

Weir Loading

Outlet weirs draw effluent without disturbing quiescent condition of tank particularly secondary ones. For all tanks except secondary tank of activated sludge process it is 100 m^3/d/m otherwise it is 150m^3/d/m. To achieve these parameters long weirs are avoided, regular indentation are made to have distributed flow. A free fall of .05 to.15 is arranged on total head available.

A weir usually has notches, holes, or slits along its length. These holes allow water to flow into the weir. The most common type of hole is the V-shaped which allows only the top inch or so of water to flow out of the sedimentation basin. Conversely, the weir may have slits cut vertically along its length, an arrangement which allows for more variation of operational water level in the sedimentation basin.

Four Zones of Sedimentation tank

Based on their unique purposes there are four different zones in rectangular sedimentation tank. These are inlet zone, settling zone, sludge zone and outlet zone. Each zone has its own performance criteria to serve the intended purpose.

All kinds of sedimentation tanks have these zones for proper operation even though the location varies according to the shape of the sedimentation tank. The following figures show the four zones of rectangular and circular tanks.

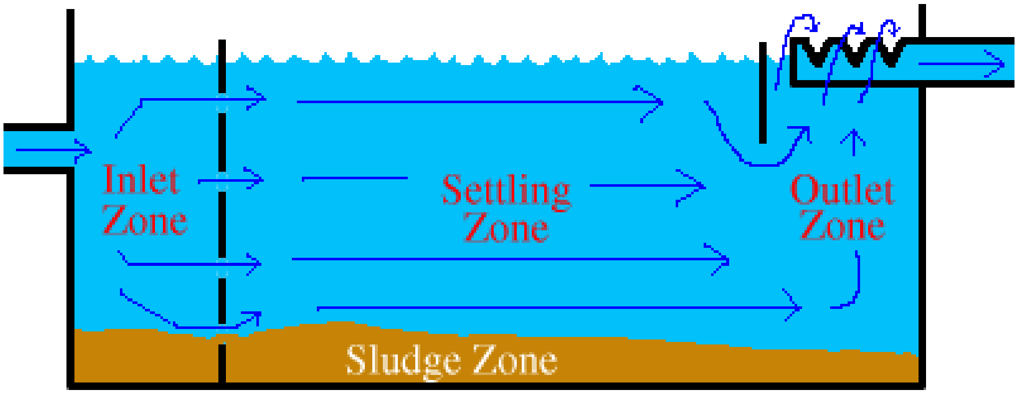


Figure 2-1: Zones in rectangular sedimentation tank

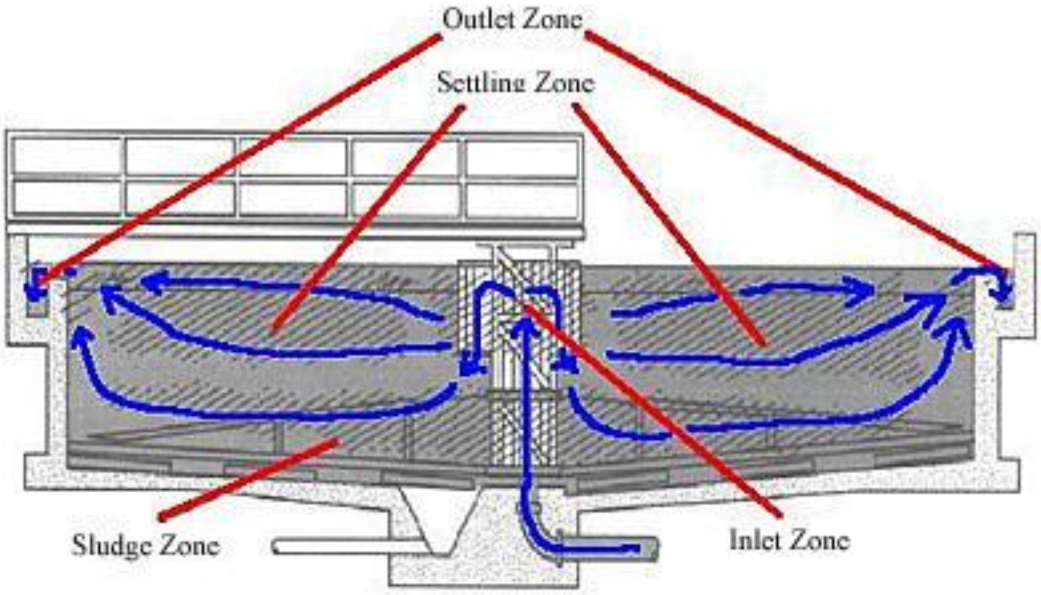
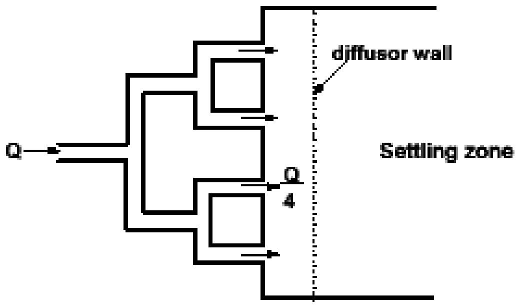


Figure 2-2: Zones in circular sedimentation tank

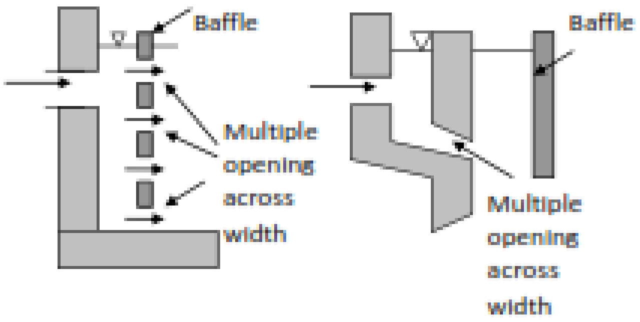
Inlet zone

The two primary purposes of the inlet zone of a sedimentation basin are to distribute the water and to control the water's velocity as it enters the basin.

In addition, inlet devices act to prevent turbulence of the water. Therefore, inlets should be designed to dissipate the inlet velocity, to distribute the flow uniformly and to prevent short-circuiting. Provisions shall be made for removal of floating materials in inlet structures having submerged ports. Orifices placed in walls at the inlets should be sized to produce velocities from 0.013 to 0.025 m/sec.



Flow Distribution



Typical baffle (diffuser) walls

Settling Zone

After passing through the inlet zone, water enters the settling zone where water velocity is greatly reduced. This is where the bulk of settling occurs and this zone will make up the largest volume of the sedimentation basin. For optimal performance, the settling zone requires a slow, even flow of water. The settling zone may be simply a large area of open water.

Outlet Zone

The outlet zone controls the water flowing out of the sedimentation basin - both the amount of water leaving the basin and the location in the basin from which the outflowing water is drawn. Like the inlet zone, the outlet zone is designed to prevent short-circuiting of water in the basin. In addition, a good outlet will ensure that only well-settled water leaves the basin and enters the filter. The outlet can also be used to control the water level in the basin. Outlets are designed to ensure that the water flowing out of the sedimentation basin has the minimum amount of floc suspended in it. The best quality water is usually found at the very top of the sedimentation basin, so outlets are usually designed to skim this water off the sedimentation basin.

Water flows over or through the holes in the weirs and into the launder. Then the launder channels the water to the **outlet**, or **effluent**, **pipe**. This pipe carries water away from the sedimentation basin and to the next step in the treatment process. A typical outlet zone begins with a baffle in front of the effluent. This baffle prevents floating material from escaping the sedimentation basin and clogging the filters. After the baffle comes the effluent structure, which usually consists of a launder, weirs, and effluent piping. A typical effluent structure is shown below:

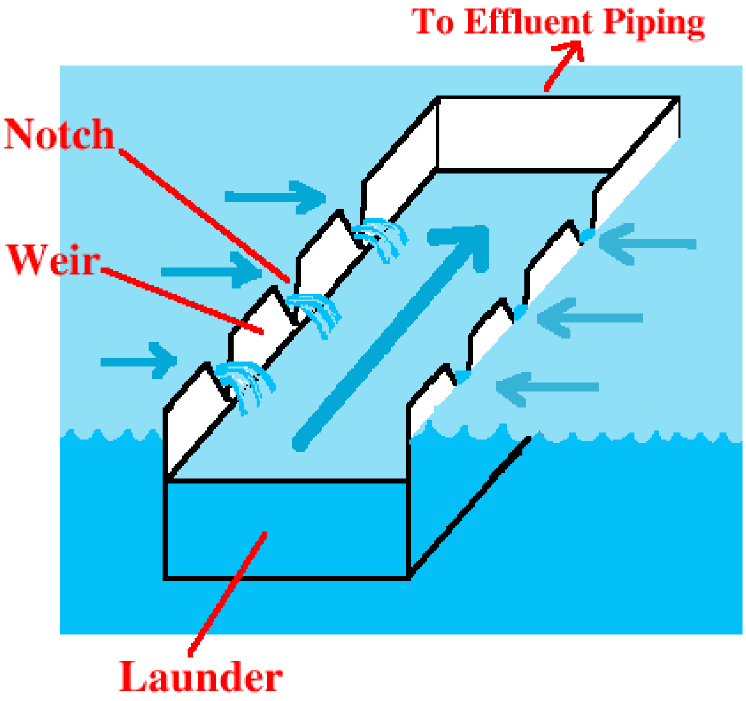


Figure 2-3: Weir and Launder at outlet zone

The primary component of the effluent structure is the effluent launder, a trough which collects the water flowing out of the sedimentation basin and directs it to the effluent piping. The sides of a launder typically have weirs attached. Weirs are walls preventing water from flowing uncontrolled into the launder. The weirs serve to skim the water evenly off the tank.

Sludge Zone

The sludge zone is found across the bottom of the sedimentation basin where the sludge collects temporarily. Velocity in this zone should be very slow to prevent re-suspension of sludge/settled flocs.

A drain at the bottom of the basin allows the sludge to be easily removed from the tank. The tank bottom should slope toward the drains to further facilitate sludge removal. In some plants, sludge removal is achieved continuously using automated equipment. In other plants, sludge must be removed manually. If removed manually, the basin should be cleaned at least twice per year, or more often if excessive sludge buildup occurs. It is best to clean the sedimentation basin when water demand is low, usually in April and October. Many plants have at least two sedimentation basins so that water can continue to be treated while one basin is being cleaned, maintained, and inspected.

If sludge is not removed from the sedimentation basin often enough, the effective (useable) volume of the tank will decrease, reducing the efficiency of sedimentation. In addition, the sludge built up on the bottom of the tank may become septic, meaning that it has begun to decay anaerobically. Septic sludge may result in taste and odor problems or may float to the top of the water and become scum. Sludge may also become re-suspended in the water and be carried over to the filters.

c) Filtration

Filtration is a process by which tiny non settleable floc remaining after chemical coagulation and sedimentation is removed. The rate of filtration that follows flocculation and sedimentation are in the range of 1.4Lit/m^{2}sec to 6.8Lit/m^{2}sec. For the practical implementation, the value of 3.4Lit/m^{2}sec is used for as the maximum design value.

Rapid Gravity Filters

Rapid gravity filters are open tanks where water passes through a filter medium, usually sand, by gravity. The tanks can be covered with a shed. Filters with a shed have the advantage that algae growth and dust accumulation in the filters is prevented.

The tank is generally rectangular, constructed of either masonry or concrete, and coated with water proof material. The depth of the tank may vary between 2.5 to 3.5m. Each unit may have a surface area of 20 to 50m^2. The tanks are arranged in series. The length to width ratio is normally maintained between 1.25 and 1.35 (Technical Guidelines for the construction and Management of Drinking Water Treatment Plant, 2009). In addition to under drainage systems, rapid sand filters have troughs made of corrugated iron or reinforced concrete spanning across the length or width of the walls for the distribution of influent water to be filtered during normal operation, and for collection of backwash water during the cleaning operation. During the normal filtration operation, the troughs remain submerged.

A rapid filter does not efficiently remove fine suspended solids. It is therefore essential that coagulation and flocculation should be performed prior to rapid filtration. A well-operated rapid filter will reduce turbidity to less than 1NTU and often less than 0.1 NTU (Technical Guidelines for the construction and Management of Drinking Water Treatment Plant, 2009).

Rapid filtration is not very efficient in removing all microorganisms, and should be followed by terminal disinfection. In order to force the water through the sand, a depth of about 1.5 to 2 (Technical Guidelines for the construction and Management of Drinking Water Treatment Plant, 2009) meters of water must be maintained above the sand. The filtration rate in rapid gravity filters varies widely and many installations are designed in the range of 4 to 12 meters per hour.

Filter medium

Filter media could be single, dual or multimedia. Coarse sand is the most commonly used filter medium in rapid filters. Some filters contain a mixture of sand and larger particles of anthracite (effective size about 1mm). After backwashing, the larger, lighter anthracite particles settle on top of the sand. Water is therefore filtered consecutively through coarse anthracite and then less coarse sand. This enables longer filter runs between cleaning.

The size of the sand is measured and expressed by effective size. The effective size, i.e. D_{10} may be defined as the size of the sieve in mm through which ten percent of the sample of sand (by mass) will pass. Selection of the correct effective size is very important because too smaller size will lead to very frequent clogging of filters and will give very low filtration rates. Similarly too large size will permit the suspended particles and bacteria to pass through it, with out being removed. The value of D10 for sand to be used in slow sand filter varies from 0.2 to 0.4mm , while for rapid gravity filters, this value varies from 0.35 to 0.55 mm.

The uniformity in size or degree of variations in sizes of particles is measured and expressed by uniformity coefficient. The uniformity coefficient, i.e. $\left(\frac{D_{60}}{D_{10}}\right)$ may be defined as the ratio of the sieve size in mm through which 60 percent of the sample of sand will pass, to the effective size of the sand. The value of $\left(\frac{D_{60}}{D_{10}}\right)$ for sand to be used in slow sand filters varies from 1.8 to 2.5; while for rapid gravity filters, this value varies from 1.3 to 1.7.

The specific gravity of sand varies from 2.55 to 2.65. Additionally the sand filter medium should satisfy the following criteria:

- The sand should be of hard and resistant quartz or quartzite and free of clay, fine particles, soft grains and dirt.
- Ignition loss should not exceed 0.7 percent.
- Soluble fraction in hydrochloric acid should not exceed 5% by weight.
- Wearing loss should not exceed 3%.

A coal-sand dual medium filter uses relatively coarser anthracite at the top of the medium with an effective size between 0.9 to 1.1mm and a specific gravity of 1.4 to 1.6 over a finer sand layer of effective size 0.45 to 1.0mm. The uniformity coefficient of anthracite should be less than 1.7. The upper layer of coarse anthracite has voids about 20% larger than the sand, and thus a coarse-to-fine grading of media is provided in the direction of flow. After backwashing, the bed stratifies with the heavier sand on the bottom and the lighter, coarser coal medium on the top. Larger floc particles are absorbed and trapped in the surface of the coal layer, while fine material is held in the sand filter; therefore, the bed filters in greater depth, preventing premature plugging.

The depth of the medium in the filter should be in the range of 0.5 to 1.0m. The filter sand media is supported on base material consisting of graded layers of gravel which should be free from clay, dirt, vegetable and organic matter, and should be hard, durable and round. The total depth of the base material varies from 45 to 60 cm and is layered as below:

Layer	Depth	Grade size
Top most	15cm	2 to 6mm
Intermediate	15cm	6 to 12mm
Intermediate	15cm	12 to 20mm
Bottom most	15cm	20 to 50mm

Filter under drains

The primary function of an under drain is to support the filter media, collect the filtered water, and distribute backwash water and air scouring. The various types of under drainage systems are shown below.

Under drain system	**Characteristics**
Pipe laterals with orifices	Deep gravel layer, medium head loss, no air scour
Pipe laterals with nozzles	Shallow gravel layer, high head loss, air scour
Vitrified tile block	Shallow gravel layer, medium head loss, no air scour
Plastic dual-lateral block	Shallow gravel layer, low head loss, air scour or concurrent air-and-water scour
Plastic nozzles	Shallow or no gravel layer, high head loss, air scour or concurrent air-and-water scour

Some under drain systems allow separate air scouring before water backwash some do not allow air scouring, whilst some allow concurrent air-and water-scouring. The following points need to be observed in designing under drainage systems with a perforated pipe system:

- Ratio of length to diameter of lateral should not exceed 60. The spacing of laterals closely approximates the spacing of orifices and shall be from 150 to 300mm.
- Diameter of perforations in the lateral should be between 5 and 12mm.
- The spacing of perforations varies from 80mm (for 5mm perforation) to 200mm (for 12mm perforations).
- The ratio of the total area of perforations in the under drainage system to total cross-sectional area of laterals should not exceed 0.5 (for 12mm perforations) and should decrease to 0.25 (for 5mm perforations).

- The ratio of the total area of perforations in the under drainage system to the entire filter area may be as low as 0.002 to 0.003.
- Area of manifold should preferably be 1.5 to 2 times the total area of laterals, to minimize frictional losses and for best distribution

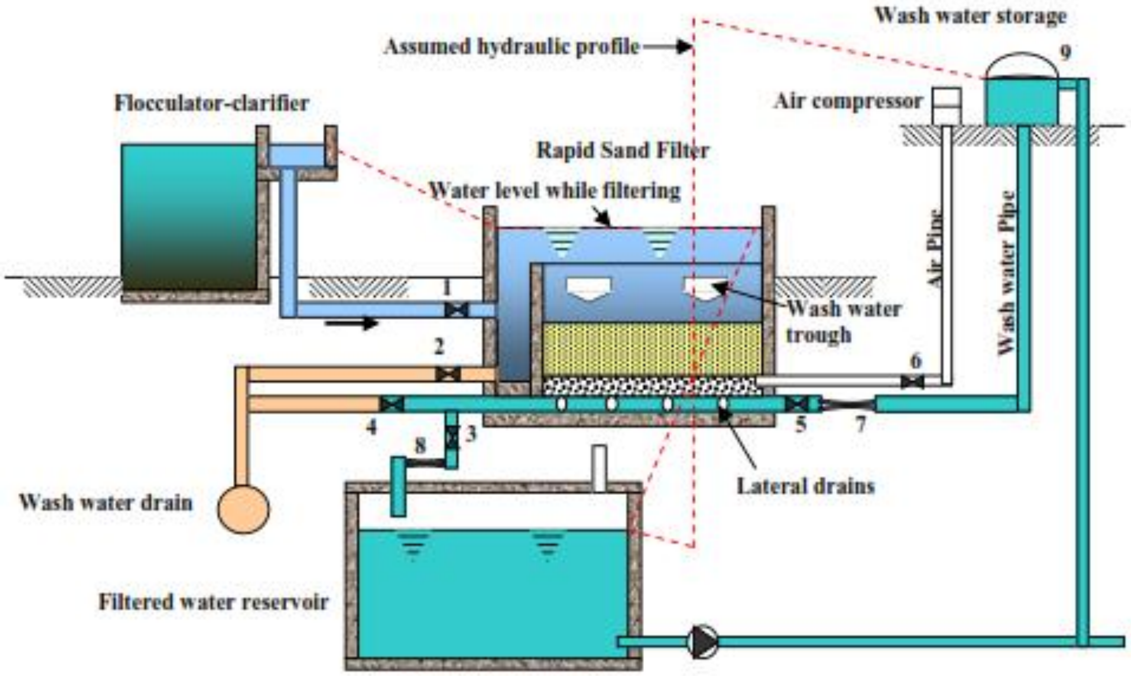


Figure 2-4: Schematic section of rapid gravity filter

Back washing:

The filter medium is washed when the loss of head through it has reached the maximum permissible. Rapid filters are washed by sending air and water upwards through the bed by reverse flow through the collector system in the sequence indicated below:

- Close influent valve 1. Allow the filter to operate till the water level reaches the edge of the troughs. Some water treatment operators permit the water level to fall to about 15cm from the top of the sand.
- Close effluent valve 3
- Open air valve 6, so that air blows back through the collector system at a rate of 1 to 1.5 m3 free air per minute per m2 of bed area at a pressure of 0.5 bar for about 2 to 3 minutes. This will break up the surface scum and loosen the dirt. Where air distribution system and water collector systems are separate, both air and water are circulated simultaneously.
- Close air valve 6, and open the wash water valve 5 gradually to prevent dislodgement of finer gravel. Open waste water valve 2 to carry the wash water to drain. Continue washing until the wash water appears fairly clear.
- Close wash water valve 5. Close waste water valve 2 after the water in the filter has drained down to the edge of the wash water trough. Allow a short period to permit material in the water to settle on the sand and form a very thin, sticky layer.
- Open influent valve 1 slightly. Open the valve 4 for few minutes.
- Close valve 4 and open effluent valve 3. Open influent valve 1 fully, to put the filter back in service.

Water used for back washing should be filtered water and the total wash water used should normally not exceed 2% of the treated water. The wash water applied is of such rates that expansion of stationary bed is in the order of 25 to 50% of its depth. The pressure at which wash water is applied is about 5m head of water as measured in under drains. The rate of application of wash water may be 600 Lit/m^2 of the filter surface, equivalent to a rise in the filter box of 60 cm/min for a period of 10 minutes.

Through repeated operations of back washing, the finer sand tends to stratify at the top. It is difficult to clean the finer sand by the conventional back washing system. Surface washing is used to clean this upper layer and also to prevent the formation of mud balls, dead or clogged area. In the surface wash system, clean filtered water is applied from above by jets of water through nozzles of special design, at a rate of 200 to 400 Lit/min per m^2 of area, under pressure of 0.7 to 1 kg/cm^2.

When a cleaned bed is put into operation, the loss of head through it will be small, usually of the order of 15 to 30cm. This increases as the water is filtered through it, by the impurities is arrested by the filter media, and the thickness of the suspended matter on the top of sand bed increases. This reaches a point when the frictional resistance exceeds the static head above the sand bed and the head becomes negative. Water is sucked through the filter media rather than being filtered through it. The formation of the negative head, allows the release of dissolved gases and air which fill the pores of the filter and the under drainage system. The filter becomes air bound; the head losses rise sharply, and the filter output capacity drops rapidly.

In rapid gravity sand filters, the permissible head loss may be between 2.5 and 3.5 m, and the permissible negative head may be 0.8 to 1.2m (Technical Guidelines for the construction and Management of Drinking Water Treatment Plant, 2009). When these limits are reached the filter must be back washed. Under normal conditions, the frequency between filter washes is one to four days.

2.5.2 Chemical Processes

Coagulation and Flocculation

Processes

Coagulation is the chemical reaction that changes soluble salts into insoluble salts. This is accomplished by adding certain chemicals, known as coagulants, to the raw water. Coagulant aids are used to adjust pH and help in the formation of insoluble salts.

These salts are referred to as floc. The formation of floc is accomplished by the even and thorough dispersal of the coagulant in the raw water by "rapid" or "flash" mixing. Changes in water quality or water temperature can have an adverse effect on the coagulation process.

Flocculation is a process of floc formation. Rapid mixing is followed by flocculation. The velocity of the water is reduced and a gentle mixing action is created to allow the formation of insoluble salts, clay, and other suspended matter into floc particles. The negatively charged colloids are attracted to a positively charged coagulant and begin colliding to form a large neutral floc particle that will settle out during the sedimentation process. This process of bringing positive and negative charged particles together to form a floc that has a neutral charge, and is large enough to settle is called agglomeration.

Flocculation units are often divided into two general groups: 1) hydraulic flocculators, and 2) mechanical flocculators. The hydraulic flocculators simply utilize cross-flow baffles or 180^0 turns to produce the required turbulence. Objective of hydraulic flocculators is to achieve gentle, uniform mixing that will not shear the floc. These types of flocculators are effective only if the flow rate is relatively constant. They are rarely used in medium and large sized water treatment plants, because of their sensitivity to flow changes. In mechanical flocculators, any type of mixers can also be used (at a reduced speed) for flocculation. The mixers typically used in flocculation basins are horizontal-shaft paddle wheel flocculators. In addition to mixer types, other common flocculators are the walking-beam type and the oscillating type.

Baffles or mechanically driven paddles in the flocculation tank are used for mixing. It is important to keep the velocity of the water slow enough to prevent "hydraulic shear". This will prevent the floc particles from breaking up before they reach the sedimentation tank. One of the more common problems occurring in flocculation basins is formation of pin floc. Low dosage or high dosage of coagulant chemicals usually causes pin floc. The addition of coagulant aids such as bentonite clay, activated silica, and polymers can improve the flocculation process.

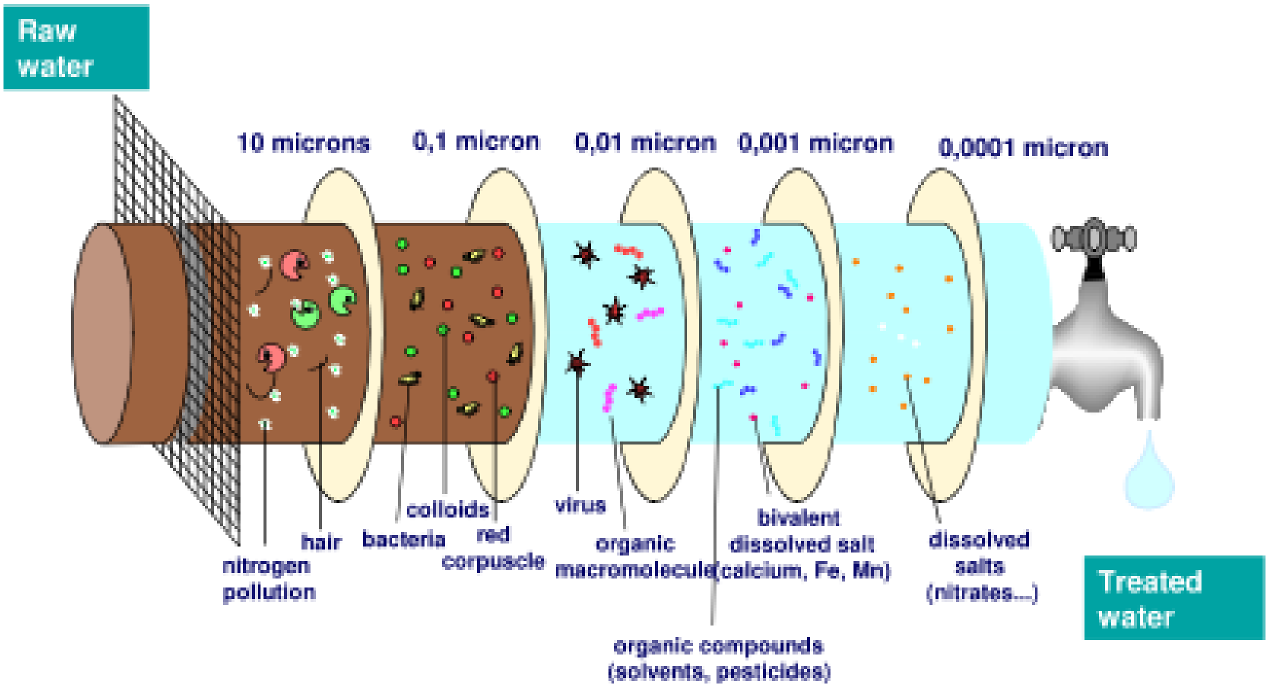


Figure 2-5: Principles of Coagulation-Flocculation

The purpose of coagulation and flocculation is to destabilize suspended solids in water to increase the size of these particles essentially colloidal and promote the formation of flocs by adsorption and agglomeration.

Flocculant aids also increase the volume, weight and cohesion of the flocs. These treatments favor the removal and extraction of particles and other organic matter in successive processes (clarification and filtration).

Coagulation is characterized by:

- Rapid mixing
- Rapid injection
- Short contact time (from few seconds to approximately 2 minutes)
- Iron and aluminum salts

Main characteristics of flocculation include:

- Slow mixing
- Rapid injection
- Long contact time (from 10 to 30 minutes).
- Use of flocculants aids (polymers).

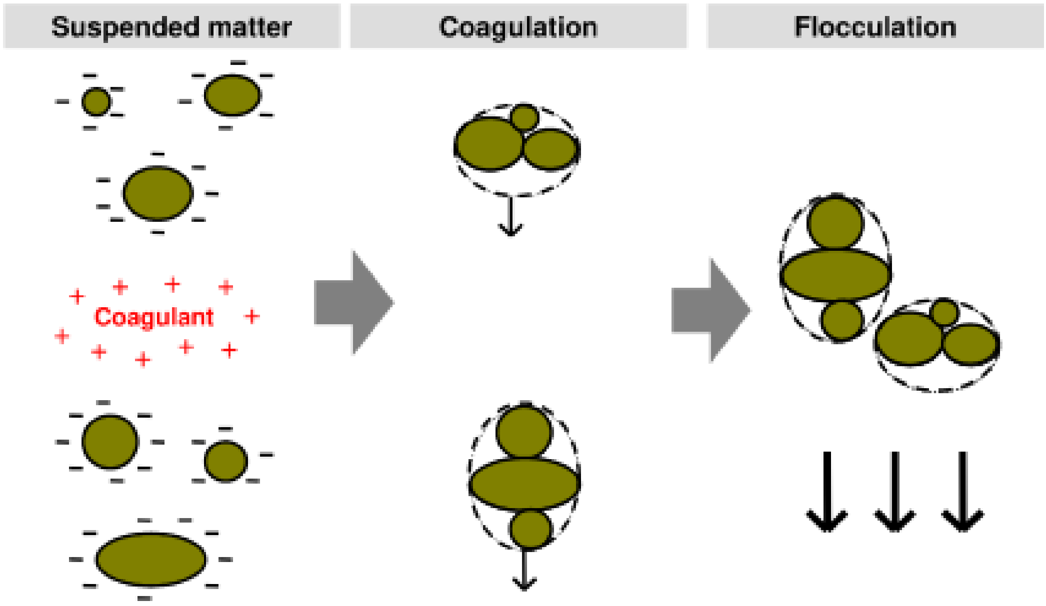


Figure 2-6: Coagulation-Flocculation Mechanisms

Metal salts, when added to water that has sufficient alkalinity, will hydrolyze into complex metal hydroxides of the form Meq(OH)p (Me=metal ion). The actual hydroxide formed is dependent upon the chemical makeup of the water, particularly its pH, and the coagulant dosage.

Coagulants typically destabilize colloids by a combination of five mechanisms: *compression of the double layer, counterion adsorption and charge neutralization, inter-particle bridging, enmeshment in a precipitate, and hetrocoagulation.* These mechanisms are discussed below.

Compression of the double layer: Compression of the double layer can be accomplished by the addition of a coagulant having a positive charge (to counteract negatively charged colloids). As the concentration of counter ions in the solution increases, the counterions cause the net charge in the diffused layer to neutralize and result in the compression of this layer. This compression affects the thickness of the entire double layers and so allows colloids to come closer together. When the colloids come close enough together for Van der Waals force to predominate, the colloids will agglomerate into a floc.

The Schulze-Hardy rule relates coagulating power of counterions to their valence state. This rule states that divalent ions are approximately 50 to 70 times more effective, trivalent ions approximately 600 to 700 times more effective, than monovalent ions.

Counterion adsorption and charge neutralization: The counterions from the coagulant can also be adsorbed onto the surface of the colloidal particles. In this way, the repulsive charges on the surface of the particles may be fully neutralized by the charges carried by the counterions. Therefore, the destabilized colloidal particles can adhere to each other to form colloidal-colloidal complexes, by Van der Waals attractions or by further adsorption of counterions. However, an excess addition of counterionic electrolyte may result in re-stabilization by a charge reversal. The net charge on the particles may be reversed by the adsorption of an excess of counterions.

Interparticle Bridging: If a synthetic organic polymer is utilized as the flocculation aid, the Interparticle bridging begins with adsorption of the polymer onto specific sites on the surface of colloidal and/or coagulant particles. The resulting structure grows into a single particle several times larger than any of its individual constituents. In order for this bridging to occur, the segment of a polymer chain must be adsorbed onto sites on more than one particle. The particle-polymer-particle aggregates are typically formed through this mechanisms, in which the polymer services as a bridge.

Enmeshment in a precipitate: The dosage of metal salts used in coagulation usually slightly in excess of the amount required for reduction of the zeta potential. The excess metal salts hydrolyze into the form $Me_q(OH)_p$. These hydroxides are extremely insoluble in water. As the hydroxide precipitate forms and accumulates, the small colloidal particles are entrapped or meshed in the hydroxide floc structures. This phenomenon is enmeshment in a precipitate or sweep-floc coagulation.

Hetro-coagulation: In water treatment practice, the surface charges on the surface of some naturally occurring particles may not be uniform. Oppositely charged sites may exist on the surface of the same particles, such as on plate-like clay particles. The coagulation of these colloidal particles can therefore occur via simple electrostatic interaction between these oppositely charged sites. The mechanism are known as hetrocoagulation.

The coagulation process chemically modifies the colloidal particles so that the stabilizing forces are reduced. To insure that a maximum amount of turbidity is removed, mixing condition and energy input must be properly provided after rapid mixing, to allow the aggregation of destabilized particles. The coagulated water must be gently stirred to promote growth of the floc. This process is flocculation. Flocculation is also important in precipitation processes. The precipitate initially forms into small particles that cannot readily be settled or filtered. In the flocculation process, the mixture is gently stirred to promote the growth of the floc to a size that can be removed by sedimentation and filtration. The typical floc size is in the range of 0.1 to 0.2mm.

Types of flocculators: Flocculation units are often divided into two general groups: 1) hydraulic flocculators, and 2) mechanical flocculators. The hydraulic flocculators simply utilize cross-flow baffles or 1800 turns to produce the required turbulence. Objective of hydraulic flocculators is to achieve gentle, uniform mixing that will not shear the floc. These types of flocculators are effective only if the flow rate is relatively constant. They are rarely used in medium and large sized water treatment plants, because of their sensitivity to flow changes. In mechanical flocculators, any type of mixers can also be used (at a reduced speed) for flocculation. The mixers typically used in flocculation basins are horizontal-shaft paddle wheel flocculators. In addition to mixer types, other common flocculators are the walking-beam type and the oscillating type.

Agitation Requirements: The degree of agitation employed in flocculators is much less than that used for rapid mixing. The purpose of flocculation is to cause particle contact while not creating sufficient turbulence to break up the floc particles already formed. The typical velocity gradients (G) for flocculators range from 15 to 60/sec. Flocculation basins are normally equipped with multiple mixing compartments in a series, with velocity gradients successively lower in each compartment. This type of arrangement is called tapered flocculation and has been found to produce a uniform and tough floc that will settle readily. The higher velocity value in the first compartment causes a rapid transformation of the primary particles into higher-density floc. The lower velocity values in subsequent compartments cause the buildup of progressively large size floc, for better settling. Compartmentalization and tapering improve the process of flocculation significantly. The velocity gradient for flocculators employing flow-induced turbulence is computed by Eqs. (2.1) below.

$$G = [\frac{\gamma\sqrt{h1}}{t\mu}] = [\frac{g\rho\sqrt{h_l}}{t\mu}] \qquad (2.1)$$

Where γ = Water density, N/m^3

g = acceleration due to gravity, m/s^2

μ = Dynamic viscosity of water, N-s/m^2

ρ = mass density of fluid, kg/m^3

h_L = total head loss through the mixer, m

t = detention time, s

Detention time in Rapid-Mix Basin: The detention time in rapid mixers should provide sufficient time for complete homogenization of the chemicals with the water and also provide sufficient time for the floc to reach particle size equilibrium. Particle size equilibrium refers to the condition where no additional rapid mixing will result in any further turbidity removal by settling alone. Such rapid mixing time can be determined only through laboratory tests or through experience with an existing treatment plant treating water with a similar chemistry and utilizing the same coagulant chemicals. Typical detention time for rapid mixers ranges from 10 sec to 5 minutes.

Shorter detention times require higher velocity gradients to achieve effective mixing. Conversely, longer detention times permit lower velocity gradients. The term Gt is often used to match the proper velocity gradient with detention time. Typical Gt values range between 30,000 and 60,000. (Reynolds, 1996) Optimum Gt values vary a great deal with the chemicals and dosage rates. Optimum design values are best determined by experiment.

Coagulant Dosage

In many plants, changing water characteristics require the operator to adjust coagulant dosages at intervals to achieve optimal coagulation. Different dosages of coagulants are tested using a jar test, which mimics the conditions found in the treatment plant. The first step of the jar test involves adding coagulant to the source water and mixing the water rapidly (as it would be mixed in the flash mix chamber) to completely dissolve the coagulant in the water. Then the water is mixed more slowly for a longer time period, mimicking the flocculation basin conditions and allowing the forming floc particles to cluster together. Finally, the mixer is stopped and the floc is allowed to settle out, as it would in the sedimentation basin.

The type of source water will have a large impact on how often jar tests are performed. Plants which treat groundwater may have very little turbidity to remove are unlikely to be affected by weather-related changes in water conditions. As a result, groundwater plants may perform jar tests seldom, if at all, although they can have problems with removing the more difficult small suspended particles typically found in groundwater. Surface water plants, in contrast, tend to treat water with a high turbidity which is susceptible to sudden changes in water quality. Operators at these plants will perform jar tests frequently, especially after rains, to adjust the coagulant dosage and deal with the changing source water turbidity.

The purpose of jar test is to determine the optimum concentration of coagulant to be added to the source water.

Materials required to conduct jar test include:

- Volumetric flask (1,000 mL)
- Analytical balance
- Coagulants and coagulant aids
- Magnetic stirrer (optional)
- A stirring machine with six paddles capable of variable speeds from 0 to 100 revolutions per minute (RPM)

- Beakers (1,000 mL)
- Pipets (10 mL)
- Watch or clock
- Turbidometer and sample tubes

2.6 Water Treatment Plant Performance

Since, the turbidity and coliforms have been considered as the most reliable and standard parameters for the performance evaluation of water treatment plants, attention was paid to one of these two water quality parameters, which is turbidity.

Unlike the developed countries, in the undeveloped countries there are various challenges in assessing the operation and performance of water treatment due to inefficient equipment, lack of skilled labor and inappropriate technologies. However, some minor routine activities like checking of final treated water turbidity and pH values as well as total coliform/ E.coli are usually performed. The performance of each unit processes are not assessed regularly.

Assessment of each unit process performance is vital for carrying out necessary maintenances on units with low performance.

2.7 Previous Studies

In various literatures, it has been recorded that difficulties of water treatment plants including problems associated with filter operational problems such as air binding and negative head are due to algae respiration and filter media cracking. Other operating considerations required for proper performance such as control needed for chemical dosing and analysis is also illustrated in a handbook of the water quality prepared by the AWWA, 1971.

ElDib has illustrated in different studies the importance of continuous monitoring and analysis of water quality before and after each treatment unit to evaluate performance efficiency of a water treatment plant. However as observed during this study, the operators have no experiences of recording influent and effluent lab test result for each units. The water samples are collected and tested only for raw water from the dam and treated water at clear water tank. Hence, the performance efficiency could not be evaluated based on the previous data only. Therefore, it was obligatory to collect water sample from in-out points of sedimentation and filters which plays the leading role in turbidity reduction of the water under treatment.

Sarika M.Mankar and Dr. P.B. Nagarnaik observed in their case study on performance evaluation of a water treatment plant that thickness of the sand in every bed was not as per the design resulting in low efficiency of the plant.

C.B.A. Ogutu and F.A.O Otieno have conducted assessment on the performance of drinking water treatment plant using turbidity as the main parameter. In their case study conducted at MOI University-Kenya, they confirmed that there is abnormality with the suspended solids removal across the treatment unit i.e. higher values at the tap water than the intake water to the treatment plant. Moreover, they indicated in their assessment that the treatment plant does not meet the WHO requirement of drinking water with respect to turbidity. They concluded that there is operation and maintenance problem because the improper coagulant dosage could be another probable cause for high turbidity in the water samples.

Herbert Mpagi Kalibbala confirmed in his case study on Ggaba II treatment plant of Kampala that inadequate sludge bleeding resulted in floc or sludge carryover to the filters reducing efficiency of the filter units (Kalibbala, 2007). He indicated that provision of adequate sludge blanket bleeding units could solve the problem.

Faris Hammoodi Al-Ani and Wassel Kadum (Faris Hammoodi Al-Ani, 2011)conducted evaluation of the Performance of Sharq Dijila Water Treatment Plant and found out that turbidly in supplied water is greater than turbidity of filtered water concluding that some impurities are intruding into the distribution system. Moreover, in their analysis they confirmed that pre-settling tanks are required for high turbid water during the rainy season.

Banff *et al*. presented in his study that common causes for poor clarifier performance are:

- density currents due to temperature variation within basins,
- excessive operating loading rates,
- entrained air- incidental flotation,
- poor hydraulics due to uneven inlet flow splitting or flocculation circulatory patterns,
- sudden changes in raw water conditions,
- chemical under-or over-dosing,
- inappropriate sludge removal rates and
- Insufficient air loading or sand concentrations (ActiFlo)

2.8 Raw Water Characteristics

Raw water or untreated water can be checked and analyzed by studying and testing their physical, chemical and biological characteristics. The type of treatment needed for surface water supplies depends on the contaminants present. These contaminants are grouped into five general categories: minerals, turbidity, bacteria, tastes and odors, and color. Prior to discussing processes of surface water treatment, the contaminants associated with each of these groups must be identified.

Turbidity

Turbidity refers to how cloudy water appears. It is a measure of how much light passes through water. Turbidity is result of existence of solid particles that scatter light beams. The solid particles may be one or combination of the more than one material/organisms such as microscopic plankton, eroded soil, silt, sand, clay, stirred up sediments or organic materials, industrial wastes, or sewage. Clear water may seem cleaner than turbid water; however the water may not be safe for human consumption.

Silt and clay are primarily causes of turbidity in water. However, any suspended material that will not readily settle is considered as turbidity. In most cases, some clay particles are so small that they float rather than settling. These particles in suspension are known as "colloids. “Most turbidity particles carry a slight negative electrical charge that causes them to repel each other. The treatment to remove these particles must neutralize these charges and bring them together until a particle is formed that is large enough to settle. Presence of algal blooms in water resulted from excess nitrogen from agricultural runoff, septic tanks or sewage outflow are also causes of excessive turbidity. The other causes are weather and seasons. Heavy rains or spring snowmelts can stir up soil particles, silts and sediments increasing turbidity of the water. Therefore, chemical consumption of treatment plant for turbidity reduction obviously increases during rainy season.

3. EXISTING CONDITIONS OF THE WATER TREATMENT PLANTS

3.1 Raw Water Quality

Surface waters tend to be variable in quality in terms of turbidity, color, and taste and odor causing substances. Surface waters are also exposed to waste inputs, including accidental spills, natural causes and man-made activities that may change the quality in a short time or over the lifetime of the project.

3.1.1 Lagadadi and Dire Raw water Turbidity

Lagadadi treatment plant receive raw water from the nearby impounding reservoir constructed to abstract rain water during wet season and Dire dam located at about 10km from the treatment plant.

The two surface water sources mentioned above for Lagadadi treatment plant has significant variation in turbidity. Turbidity of raw water from Lagadadi reservoir is considerably higher than that of raw water received from Dire dam. The under shown photos indicate that Dire reservoir water is more clear than that of Lagadadi.

Figure 3-1: Dire impounding reservoir

Figure 3-2: Lagadadi impounding reservoir

The raw water from Lagadadi dam is characterized by high turbidity and colour, low alkalinity and hardness. The suspended solids are very fine having colloidal nature. Water quality in the reservoir was deteriorated from year to year due to inadequate catchment protection upstream of the dam. As previous studies indicated, increment of more than 200NTU is observed between a period of ten years in the late 1900s. (Consult, 1998)

Taking the average turbidity of each month in the year 2001, 2002 and 2003 Eth. C the raw water data comparisons are shown here under. The same is shown in graphical representation.

Table 3-1: Year 2001 Eth. Calendar raw water turbidity variations

Year 2001 Eth. Cal.													
Months		**January**	**February**	**March**	**April**	**May**	**June**	**July**	**August**	**September**	**October**	**November**	**December**
Raw Water Turbidity (NTU)	**Lagadadi**	365.5	361.6	363.4	348	355	382.2	799.71	1240.83	555	409.2	371.8	380
	Dire	53.75	42	35.4	77.5	73.5	79.25	99.75	745.33	226	120.8	81.8	82

Source: AAWSA Daily Turbidity record

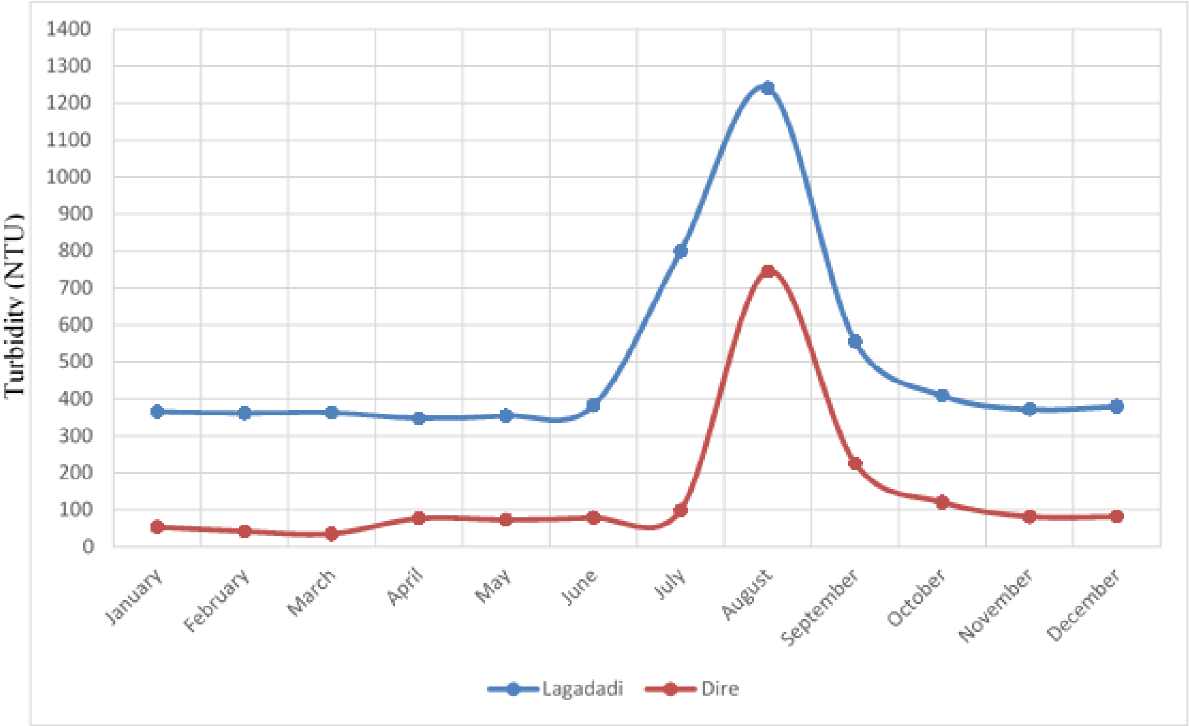


Figure 3-3: Year 2001 Eth. Calendar raw water turbidity comparison

Table 3-2: Year 2002 Eth. Calendar raw water turbidity variations

Year 2002 Eth. Cal.													
Month		**January**	**February**	**March**	**April**	**May**	**June**	**July**	**August**	**September**	**October**	**November**	**December**
Raw Water Turbidity (NTU)	**Lagadadi**	468.57	467.5	616	458.5	425.6	545	897.86	780	486.25	465.67	621	502.75
	Dire	93.25	108.75	99.25	96.5	88.5	92.25	221	185	125.67	110.67	98.4	68

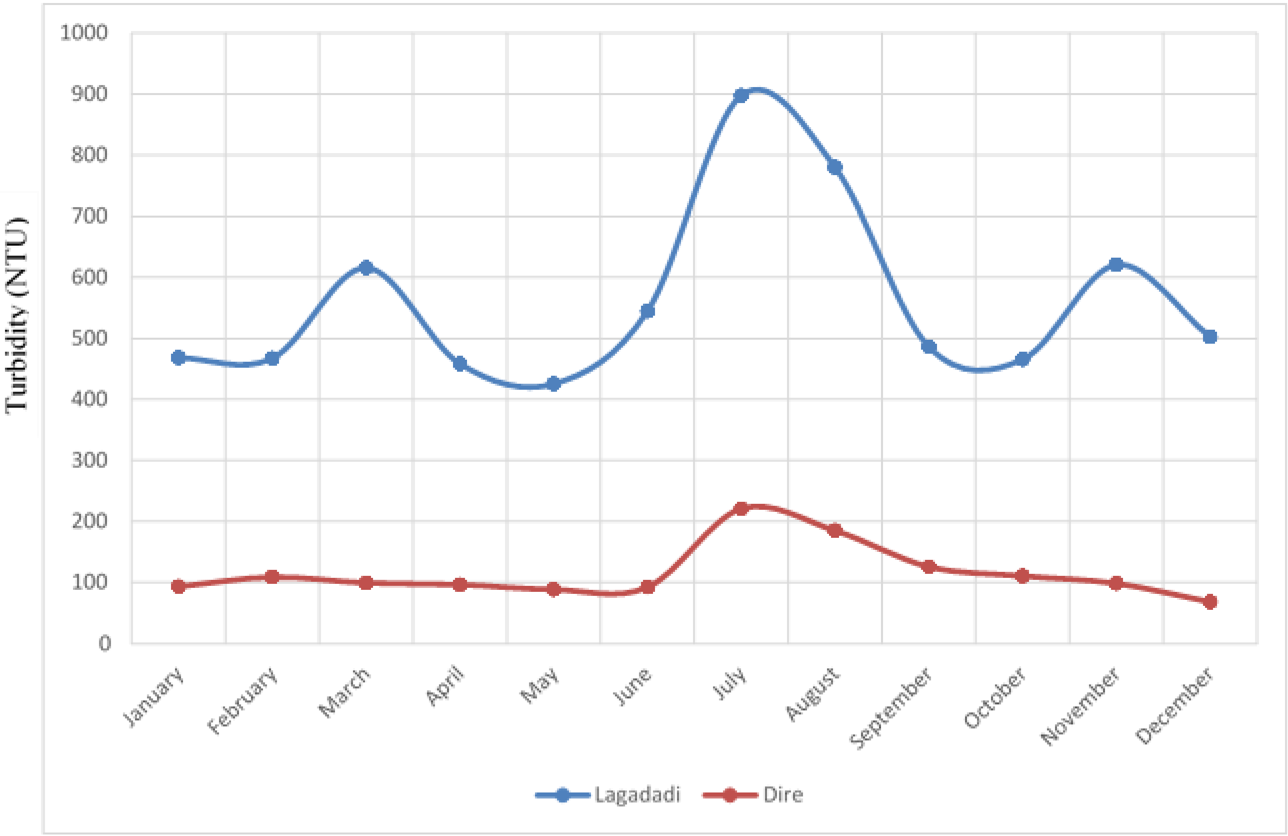


Figure 3-4: Year 2002 Eth. Calendar raw water turbidity comparison

Table 3-3: Year 2003 Eth. Calendar raw water turbidity variations

Year 2003 Eth. Cal.													
Month		**January**	**February**	**March**	**April**	**May**	**June**	**July**	**August**	**September**	**October**	**November**	**December**
Raw Water Turbidity (NTU)	**Lagadadi**	293.57	268.25	263	263.4	263.75	358	1269.4	693.57	452.33	322.5	310.40	293.57
	Dire	143.43	126.5	92	83.8	70.75	87	265.6	217.71	273	165	144.33	143.43

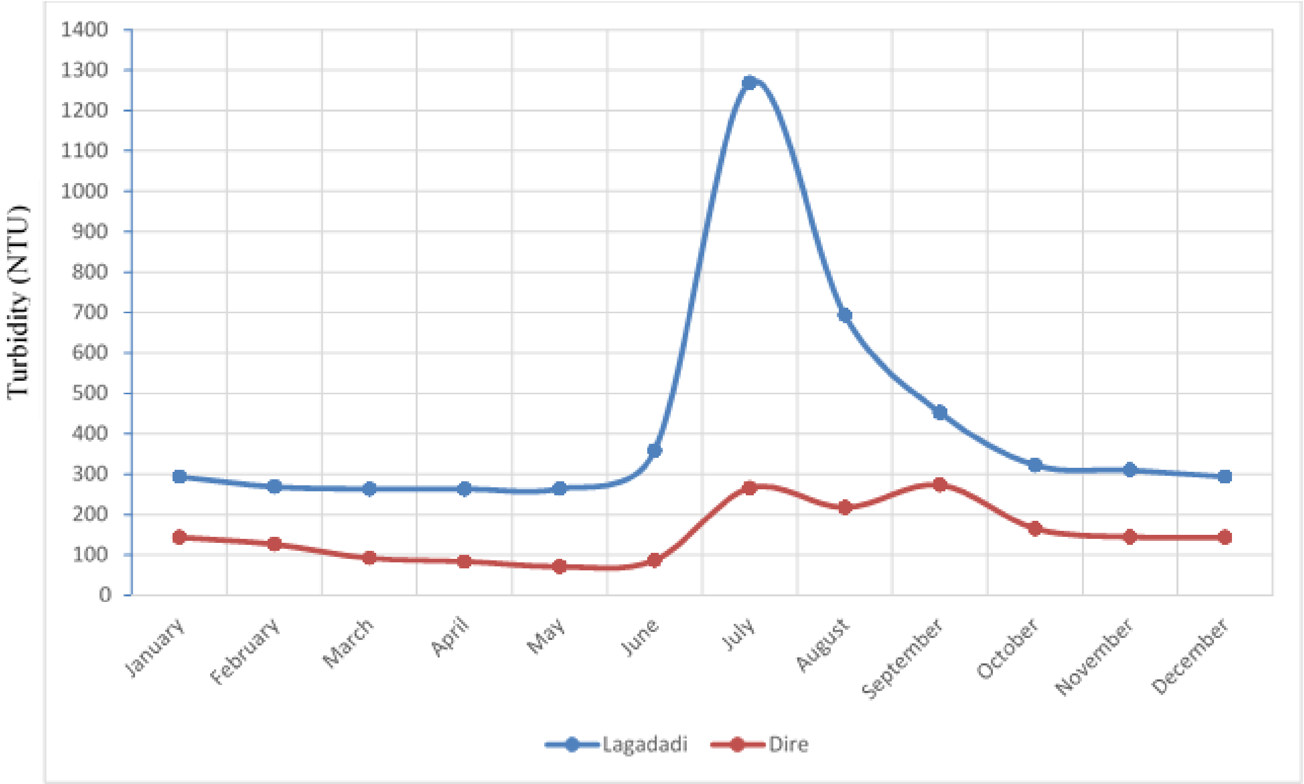


Figure 3-5: Year 2003 Eth. Calendar raw water turbidity comparison

As observed in the above comparison table and graph the turbidity variation is high during rainy season i.e. in the months of July and August.

Even though the raw water turbidity from both reservoirs vary, the treatment plant receive mixed raw water from both reservoirs. The optimum chemical dosage required to come up with acceptable turbidity level is determined by conducting jar test on the mixed raw water.

3.1.2 Lagadadi and Dire Raw water mixture Turbidity

Increasing human activities and poor catchment protection of Lagadadi dam site worsen the treatment difficulty of Lagadadi treatment plant. Relatively Dire dam raw water turbidity is much less than that of Lagadadi. Mixing raw water from Dire dam with raw water from Lagadadi in the ratio of 2:1 minimizes the output turbidity because the low turbidity source dilutes the raw water with high turbidity.

The raw water from Lagadadi impounding reservoir and Dire reservoir are mixed before entering into the treatment plant. When the raw water from both reservoirs are mixed in varying quantity/flow, the Turbidity has weighted values. The weighted average turbidity from the two sources were analyzed for the period 2001 to 2003 Ethiopian calendar. This result is tabulated and presented as follows.

Table 3-4: Year 2001 Eth. Calendar Raw water average flow and weighted average turbidity

Month	Raw Water Flow (Lit/Sec)		Average Turbidity (NTU)		Lagadadi and Dire weighted Average Turbidity (NTU)
	Lagadadi	Dire	Lagadadi	Dire	
January	450	900	365.5	53.75	157.7
February	500	850	361.6	42	160.4
March	500	815	363.4	35.4	160.1
April	560	815	348	77.5	187.7
May	560	815	355	73.5	188.1
June	375	950	382.2	79.25	165
July	360	980	799.71	99.75	287.8
August	350	1000	1240.83	745.33	873.8
September	360	950	555	226	316.4
October	360	970	409.2	120.8	198.9
November	360	975	371.8	81.8	160
December	350	950	380	82	162.2

Table 3-5: Year 2002 Eth. Calendar Raw water average flow and weighted average turbidity

Month	Raw Water Flow (Lit/Sec)		Average Turbidity (NTU)		Weighted Average Turbidity (NTU)
	Lagadadi	Dire	Lagadadi	Dire	
January	400	900	468.57	93.25	208.7
February	650	1000	467.5	108.75	250.1
March	650	950	616	99.25	309.2
April	450	950	458.5	96.5	212.9
May	450	950	425.6	88.5	196.9
June	375	900	545	92.25	225.4
July	350	950	897.86	221	403.2
August	350	900	780	185	351.6
September	320	950	486.25	125.67	216.5
October	400	900	465.67	110.67	219.9
November	400	900	621	98.4	259.2
December	400	900	502.75	68	201.8

Table 3-6: Year 2003 Eth. Calendar Raw water average flow and weighted average turbidity

Month	Raw Water Flow (Lit/Sec)		Average Turbidity (NTU)		Weighted Average Turbidity (NTU)
	Lagadadi	Dire	Lagadadi	Dire	
January	450	900	293.57	143.43	193.5
February	450	900	268.25	126.5	173.8
March	500	850	263	92	155.3
April	450	850	263.4	83.8	146
May	400	850	263.75	70.75	132.5
June	450	850	358	87	180.8
July	450	950	1269.4	265.6	588.3
August	450	900	693.57	217.71	376.3
September	500	850	452.33	273	339.4
October	500	850	322.5	165	223.3
November	500	850	310.4	144.33	205.8
December	500	850	293.57	143.43	199

The quantity of raw water entering into the clarifiers from Lagadadi and Dire dam has a ratio of 1:2 as shown above. The turbidity of Lagadadi raw water is about five times than that of Dire raw water.

In all the three years as normally expected the high turbidity is recorded in the months of July and August, the peak rainy season.

3.1.3 Gafarsa Raw water Turbidity

Gafarsa raw water Turbidity reaches its high value at the beginning of rainy season. This is due to high runoff which wash-down the surface impurities/dust from the surrounding. The raw water turbidity reaches as high as 354NTU. However as compared to Lagadadi dam raw water turbidity this value is low. The main reason for this is the better vegetation coverage (low land cultivation) cover within the catchment area as observed during the study. There are large area eucalyptus plantations in the catchment. The wooded shrubs grassland also plays a vital role in retaining impurities flowing down to dam during high rainfall periods.

Figure 3-6: Gafarsa impounding reservoir

As shown in the above figure, in the eastern catchment of the dam new settlements are developing which may impair the water quality particularly due to improper human excreta disposal. The sanitation facilities built in that area may not be to standard to enable proper waste disposal.

On the other hand cultivated areas are also increasing from time to time as per information obtained from the water treatment operators. This also need attention to properly manage the treatment plant areas. Large area cultivation can shorten the life of the dam by silting up of the reservoir volume which in turn reduce the volume of water abstracted from year to year apart from deteriorating the water quality. This also increase the plant operation due to high dosage of water treatment chemicals.

The following table shows the raw water turbidity distribution in the twelve months of the years 2001, 2002 and 2003 Eth. C. The graphical presentation is also shown hereunder.

Table 3-7: Year 2001, 2002 and 2003 Eth. Cal Gafarsa raw water turbidity monthly variations

Months	**Year 2001 (Eth. Cal)**	**Year 2002 (Eth. Cal)**	**Year 2003 (Eth. Cal)**
January	68.12	58.28	61.23
February	65.65	38.88	48.46
March	54.34	28.56	42
April	44.52	29.6	38.4
May	38.98	48.78	33.81
June	58.23	70.34	36.8
July	354.45	259.76	314.5
August	308.67	276.3	241
September	158.38	118.2	99.9
October	87.34	34.8	28.64
November	37.4	31.34	27.5
December	58.82	26.55	21.7

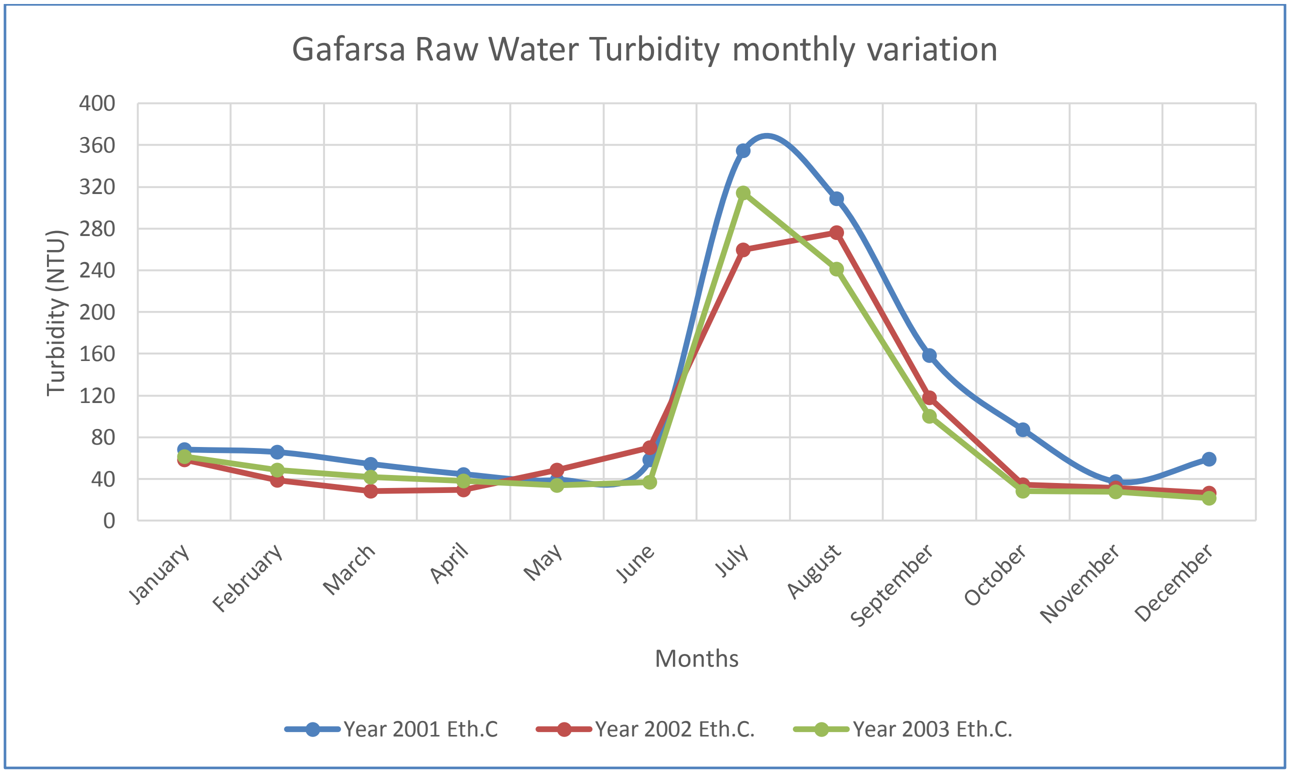


Figure 3-7: Gafarsa raw water turbidity monthly variation

3.2 Floc carryover

Floc carryover is an operation problem observed both at Gafarsa and Lagadadi treatment plant. Particularly during rainy season magnitude of the problem is high reducing daily production of the treatment plants.

Floc passing to filter through orifice type weir

Figure 3-8: Floc carryover Gafarsa treatment plant

3.2.1 Factors contributing to Floc carryover

The flocs in the sedimentation tank/clarifiers is affected by factors such as particle size, shape, density, electrical charge; number of particles; water temperature; sedimentation tank or clarifiers physical characteristics (shape, conditions such as wind and density currents). Smooth particles settle faster than irregular shapes; dense particles settle to the bottom faster than the light ones. Most of the particles are negatively charged. The main factors that cause floc carryover are briefly discussed hereunder.

a) Temperature

Temperature is one of the factors affecting floc settlement in clarifiers. Increase in water temperature minimizes floc settlement. This is due to reaction of water molecule to temperature changes. When the water temperature decreases the molecule of water come closer to each other resulting in density increases. In such cases, the density difference between water and solids particles becomes less; therefore, the particles settle more slowly. This phenomenon facilitates suspension of lighter flocs initiating floc carryover. Molecules of water become less denser as temperature increases (above 4° C) or contracts as temperature decreases (down to 4° C). Below 4° C the opposite is true.

b) Short circuiting

Short-circuiting in clarifiers is characterized by currents that move rapidly through and continue into the settling tanks. The floc removal problem is compounded then with flocculation, which is incomplete, and currents introduced into the settling process, which further inhibit removal. Properly operated entrance, curtain baffles and exit weirs and launders can significantly improve settling.

Observation of surface currents over the length of a rectangular basin/clarifier is usually evidence of short-circuiting. Circular basins, however, are harder to observe short-circuiting. These problems may be noticeable as pin floc in certain places around the edge of the tank and a patchy accumulation of sludge at the lower part of the tank. As water enters the settling basin, it should be evenly dispersed across the entire cross section of the tank. The flow must have the same velocity in all areas in the direction of the discharge end because when the velocity is greater in some sections than in others, serious short-circuiting may occur. The degree of short-circuiting in circular units can vary significantly, based on the type of inlet used. The most important factors to consider in controlling short-circuiting are dissipation of inlet velocity, protection of tanks from wind sweep and uneven heating, and reduction of density currents associated with high inlet suspended solids concentrations.

Insufficient weir length may result in floc carryover to the filters.

$$\text{Weir Loding Rate, m}^3/\text{day}/\text{m} = \frac{\text{Discharge, } (\text{m}^3/\text{day})}{\text{Weir Length, } (\text{m})}$$

Water leaves the clarifier by flowing over weirs and into effluent troughs (launders) or some type of weir arrangement. The number of lineal meter of weir in relation to the flow is important to prevent short circuits or high velocity near the weir or launder which might pull settling solids into the effluent. The weir overflow rate is the number of cubic meter of water that flow over one lineal meter of weir per day.

c) Density current

Several types of water currents may occur in the sedimentation basin:

- Density currents caused by the weight of the solids in the tank, the concentration of solids and temperature of the water in the tank.
- Eddy currents produced by the flow of the water coming into the tank and leaving the tank.

The currents can be beneficial in that they promote flocculation of the particles. However, water currents also tend to distribute the floc unevenly throughout the tank; as a result, it does not settle out at an even rate.

Some of the water current problems can be reduced by the proper design of the tank. Installation of baffles helps prevent currents from short circuiting the tank.

d) Detention time

Water should remain in the clarifier long enough to allow sufficient settling time for solid particles. If the tank is too small for the quantity of flow and the settling rate of the particles, too many particles will be carried out the effluent of the clarifier. The relationship of detention time to settling rate of the particles is important. Most engineers design settling tanks for about 2.0 to 3.0 hours of detention time.

The detention time is important in the case of a flocculent suspension where flocs development and aggregation proceeds with time.

This is, of course, flexible and dependent on many circumstances. Detention time can be calculated by use of two known factors:

i. Flow in cubic meter per day
ii. Tank dimensions or volume

3.2.2 Observed possible cause of floc carryover

As described earlier, the weir overflow type for both Gafarsa and Lagadadi treatment plants are of circular orifice type.

The weir surface (circumferential) areas of some orifices are narrowed as observed during clarifiers washing. There is no practice of dislodging attached matters from the orifices inner surfaces and the cross sectional areas are reduced increasing velocity near the lander. The following picture shows this phenomenon.

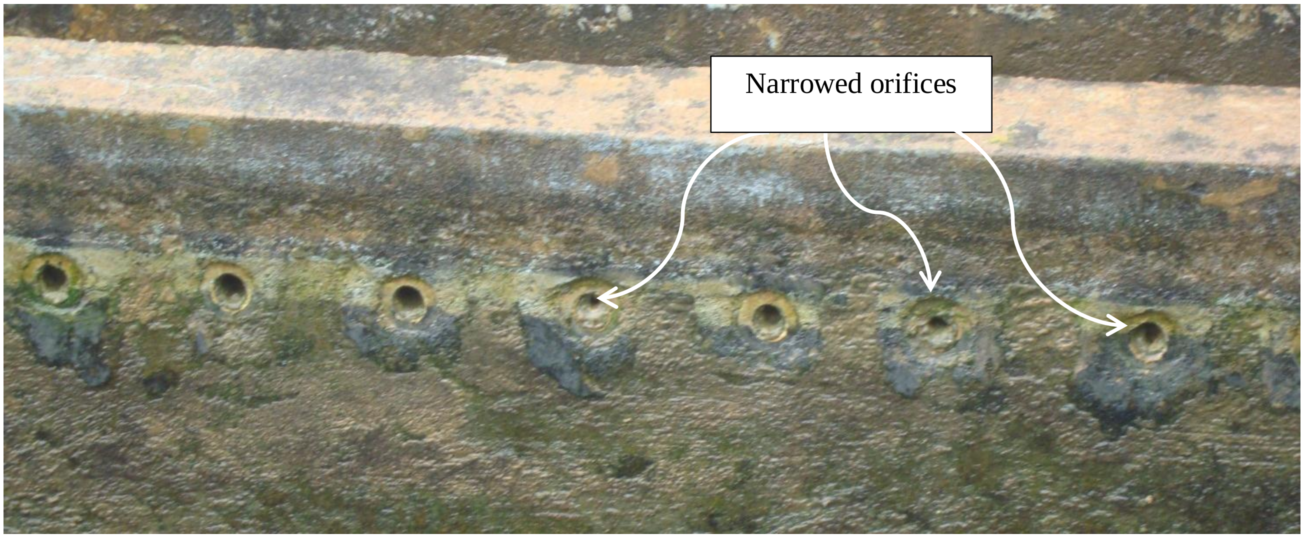


Figure 3-9: Narrowed orifices due to accumulated sludge/incrustation

3.3 Chemical used in the water treatment plants

In coagulation process, selection and optimization of primary coagulants and coagulant aids are very important to achieve best result in turbidity removal.

The process of coagulation may include using primary coagulants and may include using coagulant aids.

Primary coagulants are used to destabilize particles causing clumping together. As previously described the primary coagulants include metallic salts such as aluminum sulphates/Alum/, ferric sulphates and ferric chloride. Sometimes Cationic polymers may also be used as primary coagulants.

Coagulant aids increase density of slow-settling flocs and help improve floc formation in the process of turbidity removal from raw water. Organic polymers like polyaluminium hydroxychloride (PACL) are the most commonly used to enhance coagulation in addition to primary coagulants. These organic polymers have benefits of best efficiency at low dosages due to their natural characteristics of having a high positive charges. These organic polymers produces a small quantity of sludge.

The raw water characteristics will determine the type and dosages of chemicals used in surface water treatment turbidity removal process.

Variation of raw water characteristics like water pH, temperature, alkalinity, total suspended solids and turbidity may affect coagulation process and subsequently filtration and treated water quality. The commonly used method for determination of the proper coagulation type and dosage is using jar test which is the excellent way. This test allows a system to experiment with different coagulants, polymers and pH controllers. The jar test should simulate the actual plant operation conditions such as mixing rates and detention times.

3.3.1 Lagadadi Water Treatment Plant

As observed above the raw water turbidity of Lagadadi impounding reservoir is much higher as compared to the raw water from Dire and Gafarsa. Even though turbidity of raw water from Dire is comparatively low, since it is mixed with Lagadadi raw water before entering into the treatment plant, the turbidity is still high. Due to its high turbidity characteristics of the raw water Lagadadi water treatment plant uses Alum as a primary coagulant with polymer as to enhance turbidity removal efficiency.

Lagadadi raw water turbidity increased from time to time and the original chemical treatment based on Alum and polyelectrolyte was substituted with Alum and cationic polymer (CATFLOC-CL). Furthermore, the results from regular jar tests carried out indicated yearly increment of polymer demand. Other chemicals used in the water treatment plant are chlorine for pre and post disinfection as well as lime for pH correction.

Chemical consumption of three consecutive years from monthly recorded data of the utility is tabulated below. The monthly values are the cumulative of daily consumption in the month.

Table 3-8: Lagadadi WTP monthly chemical consumption for the physical year 2008/09 to 2010/11

Month	**July 2008 – June 2009**		**July 2009 – June 2010**		**July 2010 – June 2011**	
	Poly electrolyte (L/Hr)	**Chlorine Gas (Kg/Hr)**	**Poly electrolyte (L/Hr)**	**Chlorine Gas ((Kg /Hr)**	**Poly electrolyte (L/Hr)**	**Chlorine Gas (L/Hr)**
July	2400	20	2000	20	2000	19
August	2800	20	2200	24	2000	19
September	2500	20	2200	21	2200	19
October	2300	16	2500	20	1800	19
November	2100	16	2500	20	1700	19
December	2000	16	2500	20	1700	20
January	1800	16	2400	20	1600	20
February	1800	17	2200	20	1500	18
March	1800	17	1800	20	1400	20
April	1800	17	1800	20	1000	20
May	1400	17	1800	19	1000	20
June	1400	17	1800	19	1700	20
Total	**24100**	**209**	**25700**	**243**	**19600**	**233**

As observed in the above table high consumption of poly electrolyte is observed in the months of August, October to December and September in the years 2008/09, 2009/10 and 2010/11 respectively.

3.3.2 Gafarsa Water Treatment Plant

This WTP is designed to treat surface water based on pre-chlorination-coagulation-sludge blanket clarification-sand filtration and post-chlorination processes.

Due to raw water characteristics of Gafarsa impounding reservoir, the normal procedures i.e. coagulation by primary coagulant only is adopted. The other chemicals such as chlorine and lime are usually utilized for their respective roles in the treatment process. Unlike Lagadadi WTP the primary coagulant used is Aluminum Sulphates and hydrated lime with chlorine gas for disinfection. Below is the monthly chemical consumption tabulated for the years 2011/12 to 2013/14.

Table 3-9: Gafarsa WTP monthly chemical consumption for the physical year 2011/12 to 2013/14 in Kg

Month	**July 2011 – June 2012**			**July 2012 – June 2013**			**July 2013 – June 2014**		
	Aluminum Sulphates	**Hydrated Lime**	**Chlorine Gas**	**Aluminum Sulphates**	**Hydrated Lime**	**Chlorine Gas**	**Aluminum Sulphates**	**Hydrated Lime**	**Chlorine Gas**
July	97,350	50,075	396	77,750	37,500	5,448	99,500	44,375	4,320
August	79,500	56,500	4,344	99,000	52,750	5,676	104,000	44,250	4,320
September	51,000	35,000	3,720	77,500	43,750	4,608	73,500	35,250	3,545
October	37,500	30,500	3,240	58,000	24,500	3,600	52,750	27,700	3,600
November	46,250	29,000	3,240	49,500	21,900	3,384	45,000	17,700	2,880
December	52,500	29,500	3,240	45,000	18,600	2,860.6	43,500	18,000	2,880
January	45,000	30,000	3,936	45,000	18,000	2,880	40,500	17,700	2,880
February	41,500	22,700	3,960	45,000	18,000	2,824	39,750	18,000	2,852
March	36,750	16,200	3,960	45,750	18,300	2,892	45,750	18,300	2,892
April	40,500	17,100	3,960	45,000	18,000	2,928	45,000	15,900	2,880
May	39,000	16,800	3,960	45,000	18,000	3,576	54,750	22,400	3,072
June	44,000	18,450	4,932	44,000	18,450	4,932	75,000	33,250	4,320
Total	**610,850**	**351,825**	**42,888**	**676,500**	**307,750**	**45,609**	**719,000**	**312,825**	**40,441**

The yearly chemical consumption trend in the above table shows that quantity of Alum used increased linearly whereas that of hydrated lime and chlorine gas fluctuates.

4. MATERIALS AND METHODS

4.1 Data collection

In the study, it is planned to collect both primary and secondary turbidity data at each treatment units to evaluate the performance efficiency of clarifiers and filters for both treatment plants. The operators at both treatment plants measure turbidity of raw water and treated water after filtration. Turbidity value of the treated water/effluent from filtration units/ is always measured as per information obtained from the operators and water treatment laboratory head of AWSSA but there is no recorded data. Similarly, the operators do not check turbidity of effluent from the clarifiers. However, turbidity of raw water is regularly measured and recorded properly. Secondary data of raw water turbidity is collected from both treatment plant to assess the monthly variation of three years (2001, 2002 and 2003 Ethiopian calendar). There is no regular or occasional turbidity measurement for effluents from each unit of clarifiers and filters to know their efficiency.

4.1.1 Gafarsa

Gafarsa water treatment plant is small in capacity and number of units as compared to Lagadadi treatment plant. There are two clarifiers and twelve filter units. Water sample is collected from both clarifiers and each filter units.

For raw water sample, the tap is opened fully for reasonable period of time to collect representative sample. Samples from clarifiers and filters are collected at outlet zones of each unit.

The primary data are collected from August 25, 2013 to September 26, 2013. The data are collected and analyzed 11 times and the average values are taken for each unit processes.

4.1.2 Lagadadi

For collecting raw water sample for analysis, similar procedures are adopted with that of Gafarsa treatment plant.

Two clarifier units each from stage-1 and stage-2 are randomly selected for sampling. Similarly four filter units each from stage-1 and stage-2 are chosen arbitrarily.

Water samples are collected at inlet to and outlet from clarifiers and filters from February 21, 2014 to May 29, 2014. The data are collected and analyzed 11 times like that of Gafarsa and the average values are taken for each unit processes.

4.2 Materials used

For conducting this research, turbidity meter with complete accessories and plastic containers for sampling are used. For supporting pictures, Sony Digital camera was used to take some necessary photographs at field and laboratories.

4.2.1 Labelled Plastic bottles

The plastic bottles are labeled properly for identification of the samples particularly among the first stage and second stage clarifiers and filter units. As stated previously the clarifiers constructed during the first stage are not equipped with tube settlers. This necessitated tagging the bottles for identification. Accordingly, the plastic bottles used for sample collection are labeled. These plastic bottles are placed on the treatment units selected to be sampled.

Figure 4-1: Labeled Sampling Plastic Bottles

4.2.2 Turbidity meter

The other material used to conduct this research was digital Turbidity meter. The model of the turbidity meter used at Gafarsa Water treatment plant is HANNA LP 2000 whereas that of Lagadadi was HACH 2100AN. The turbidity meter type used to measure turbidity of water samples from Lagadadi treatment plant was more advanced than that of Gafarsa due to its capability to measure other water quality parameters like pH.

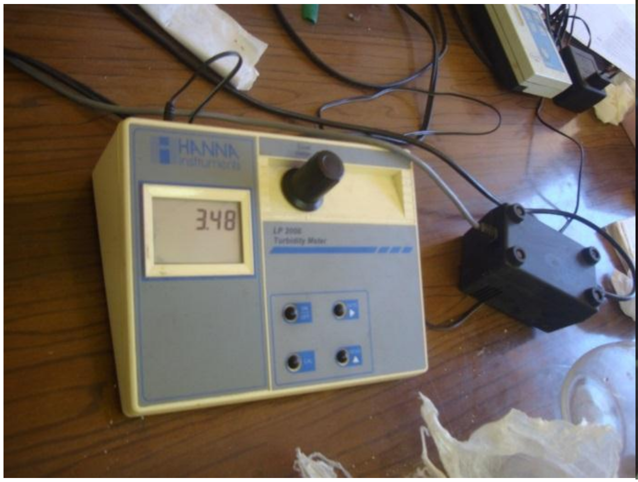


Figure 4-2: HANNA LP 2000 Turbidity Meter

Figure 4-3: HACH 2100AN Turbidity meter

During the study, laboratory at Gafarsa water treatment plant was not functioning. However, the turbidity meter in the laboratory works properly. Turbidity of Samples collected from raw water tap, each unit of the clarifiers and filters were measured with this instrument on the site and recorded.

At Lagadadi treatment plant there is a laboratory furnished with necessary basic equipment. However, the laboratory gives service occasionally for optimization of chemical dosage only when characteristics of raw water changes. Thus it was difficult to get easy access to measure turbidities of the samples on site. Therefore, samples from the treatment plant were collected and the test was conducted at head office laboratory. The plastic bottles shown in Figure 4-1 above were used for sample.

4.2.3 Turbidity meter operational check

Even though the Turbidity meter used in this study were under operation, for precision of the data, operational check has been conducted at both Water Treatment Plant sites. Distilled water is used for the check. The operational check procedures is described hereunder.

For periodical checking the probes were rinsed several times with distilled water and blot dried. Distilled water was filled into the sample tubes. The sample tubes were cleaned with dry cotton pieces of cloth to make sure that any water and fingerprints are removed. The sample tubes were inserted into the sample compartment. The reading was recorded several times to ensure consistency of the data.

4.3 Lagadadi, Dire and Gafarsa raw water Secondary data

As observed in each section above the raw water turbidity values vary from source to source i.e. the three impounding reservoirs have different monthly turbidity values. Of the three water sources the Lagadadi raw water turbidity is the highest in all times. The possible cause for this is the large catchment areas covered with crop cultivation as reported by different consultants who have studied the raw water quality of each dam in the past. The above figures of the dam areas indicate that there is no good vegetation cover and tree plantations in Lagadadi impounding reservoir catchment.

Taking monthly average raw water turbidity values from the three dams, the following table is presented hereunder.

Table 4-1: Lagadadi, Dire and Gafarsa raw water turbidity data

Months	**Year 2001 (Eth. C.)**			**Year 2002 (Eth. C.)**			**Year 2003 (Eth. C.)**		
	Gafarsa	**Dire**	**Lagadadi**	**Gafarsa**	**Dire**	**Lagadadi**	**Gafarsa**	**Dire**	**Lagadadi**
January	68.12	53.75	365.5	58.28	93.25	468.57	61.23	143.43	293.57
February	65.65	42	361.6	38.88	108.75	467.5	48.46	126.5	268.25
March	54.34	35.4	363.4	28.56	99.25	616	42	92	263
April	44.52	77.5	348	29.6	96.5	458.5	38.4	83.8	263.4
May	38.98	73.5	355	48.78	88.5	425.6	33.81	70.75	263.75
June	58.23	79.25	382.2	70.34	92.25	545	36.8	87	358
July	354.45	99.75	799.71	259.76	221	897.86	314.5	265.6	1,269.40
August	308.67	745.33	1,240.83	276.3	185	780	241.23	217.71	693.57
September	158.38	226	555	118.2	125.67	486.25	99.9	273	452.33
October	87.34	120.8	409.2	34.8	110.67	465.67	28.64	165	322.5
November	37.4	81.8	371.8	31.34	98.4	621	27.5	144.33	310.4
December	58.82	82	380	26.55	68	502.75	21.7	143.43	293.57

The graphical presentation of the above table is shown hereunder.

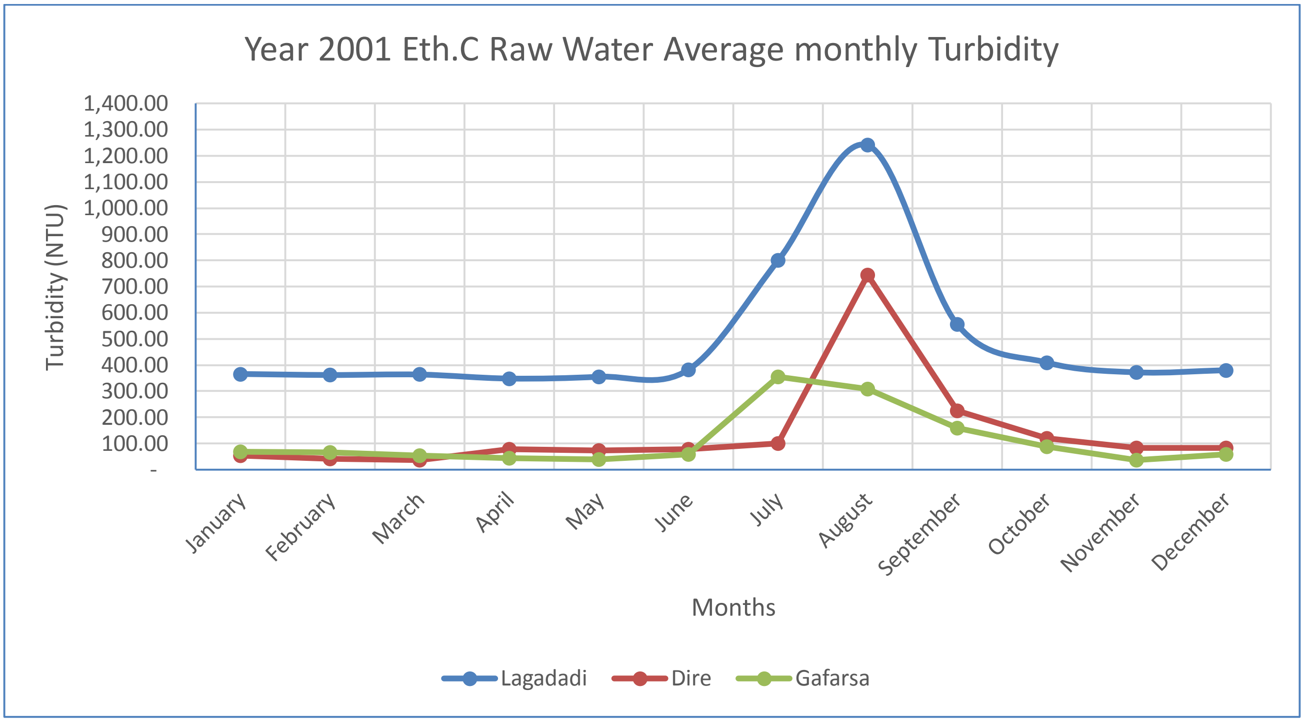


Figure 4-4: Year 2001 Eth. C. raw water turbidity

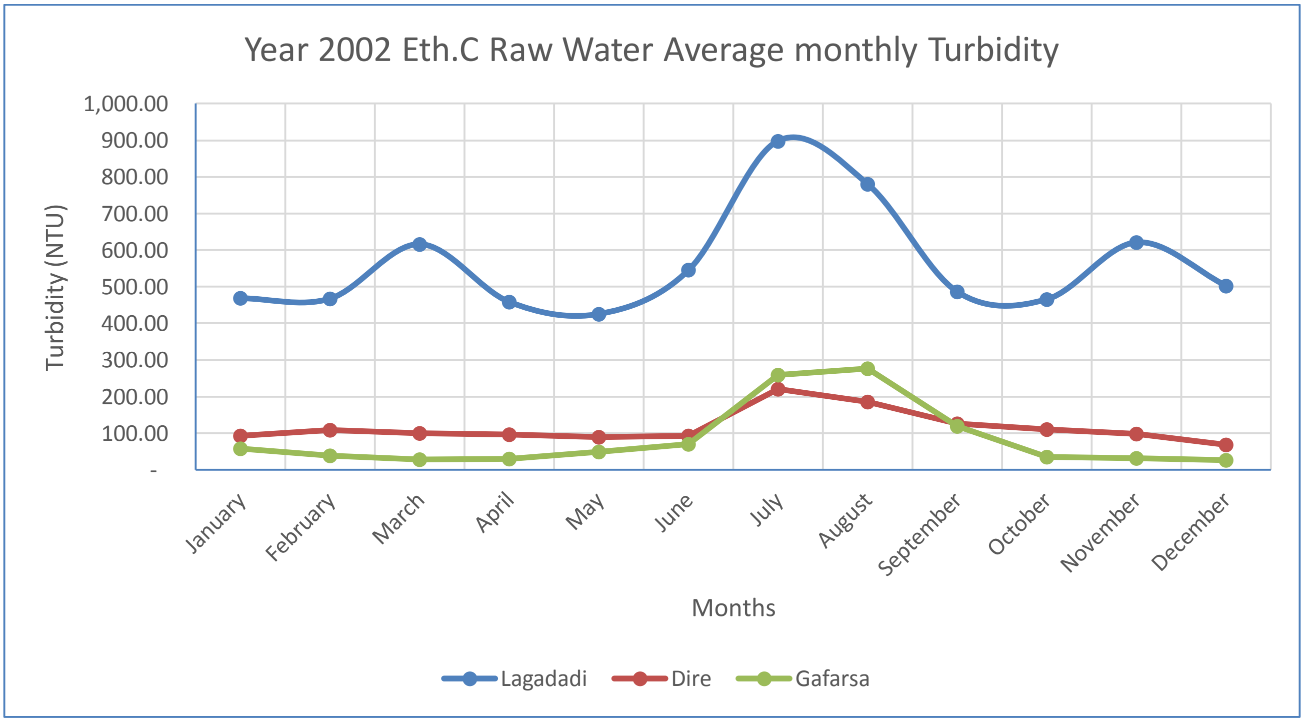


Figure 4-5: Year 2002 Eth. C. raw water turbidity

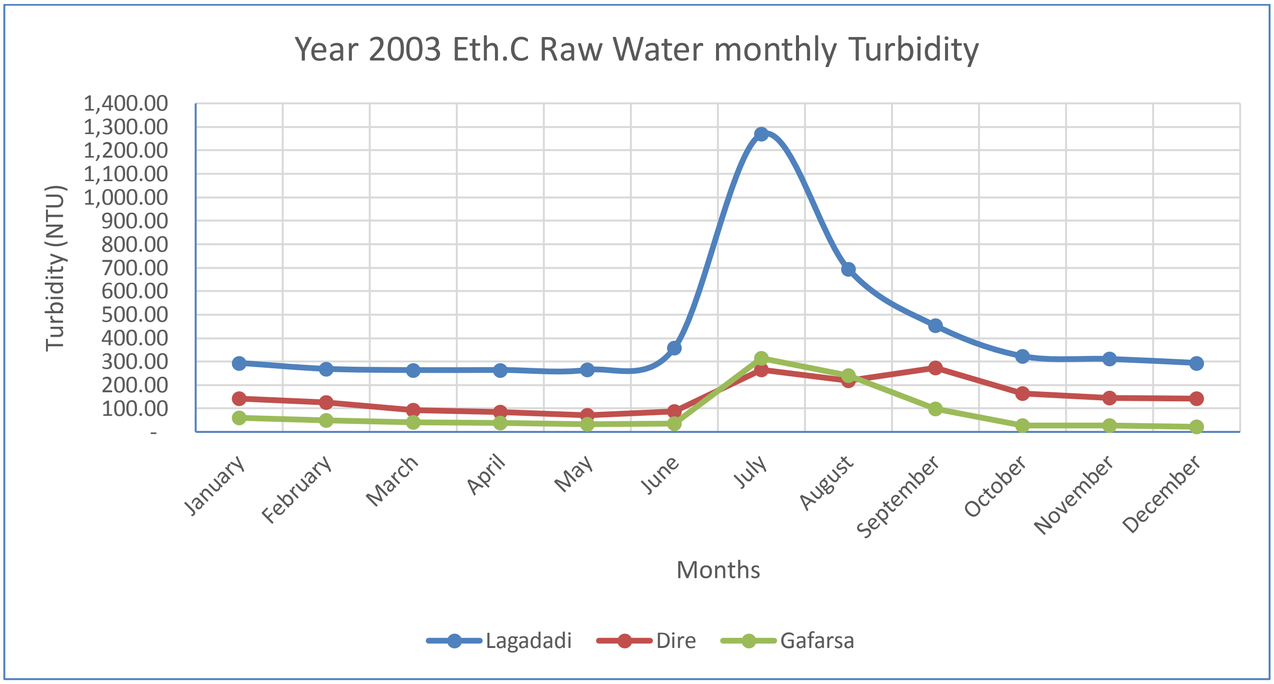


Figure 4-6: Year 2003 Eth. C. raw water turbidity

4.4 Laboratory Primary Data collection

4.4.1 Gafarsa

During the study, laboratory at the water treatment plant is not functioning. However, the turbidity meter in the laboratory was functioning. Turbidity of Samples collected from raw water tap, each unit of the clarifiers and filters were measured with this instrument on the site and recorded.

Water samples are collected from both clarifiers and all the 12 filters beds are also sampled for measurement of influent and effluent turbidity of the water.

Raw water sample was taken from entry point to the water treatment plant following appropriate sampling techniques.

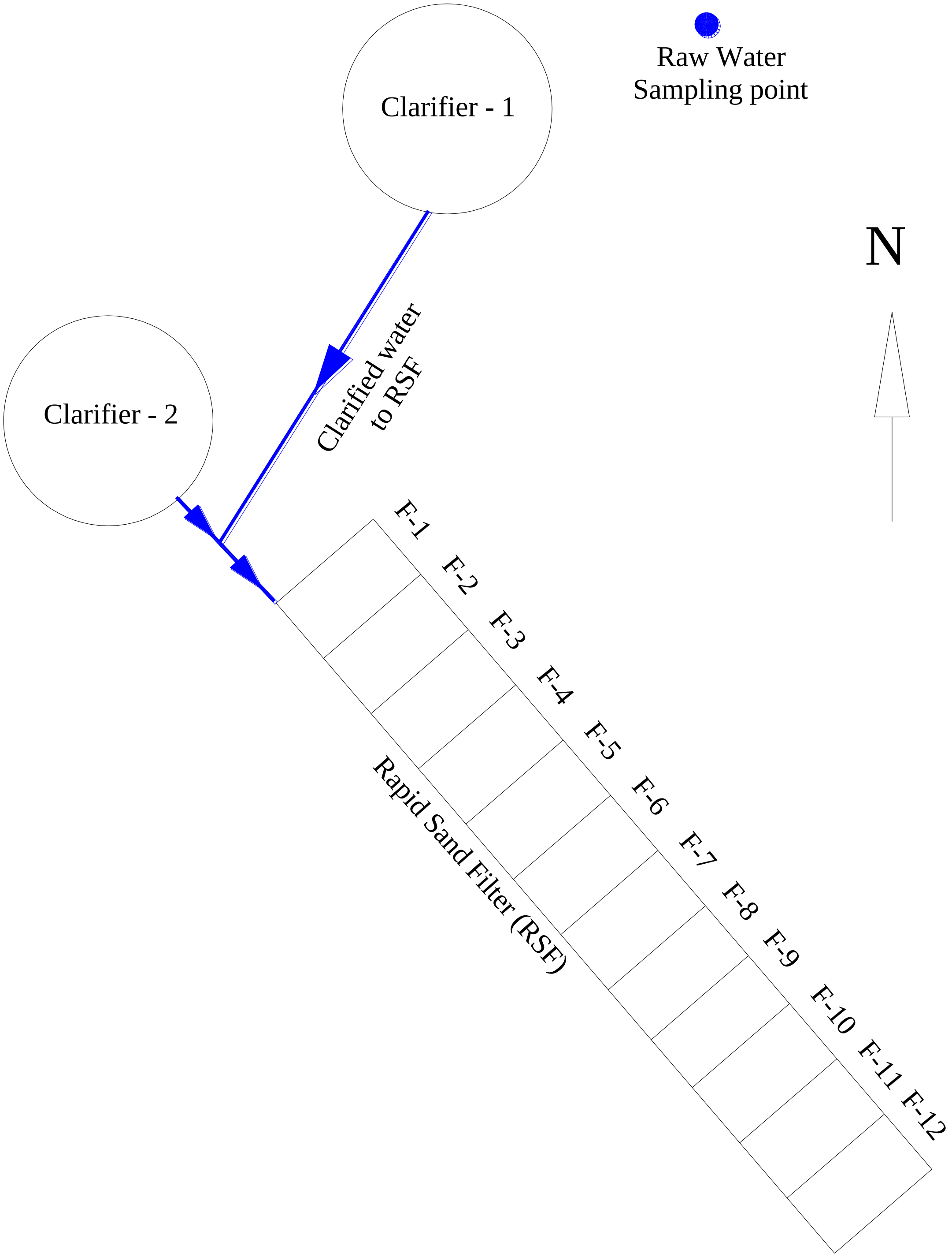


Figure 4-7: Gafarsa sampling labels

4.4.2 Lagadadi

At Lagadadi treatment plant there is a laboratory furnished with necessary basic equipment. However, the laboratory gives service occasionally for optimization of chemical dosage only when characteristics of raw water changes. Thus it was difficult to get easy access to measure turbidities of the samples. Therefore, samples from the treatment plant were collected and the test is conducted at head office laboratory. The plastic bottles shown in Figure 4-1 above were used.

To come up with realistic result representative samples are collected from each treatment units mainly related to turbidity removal. Samples are collected from each clarifiers as there only four units, two each in both construction stages.

There are 32 units of filter beds in the system of which 12 are constructed in the first stage and 20 are constructed during the second stage. Sampling locations are randomly selected from both stages and four filters beds are selected from each.

The figure below shows sampling location of the filter units.

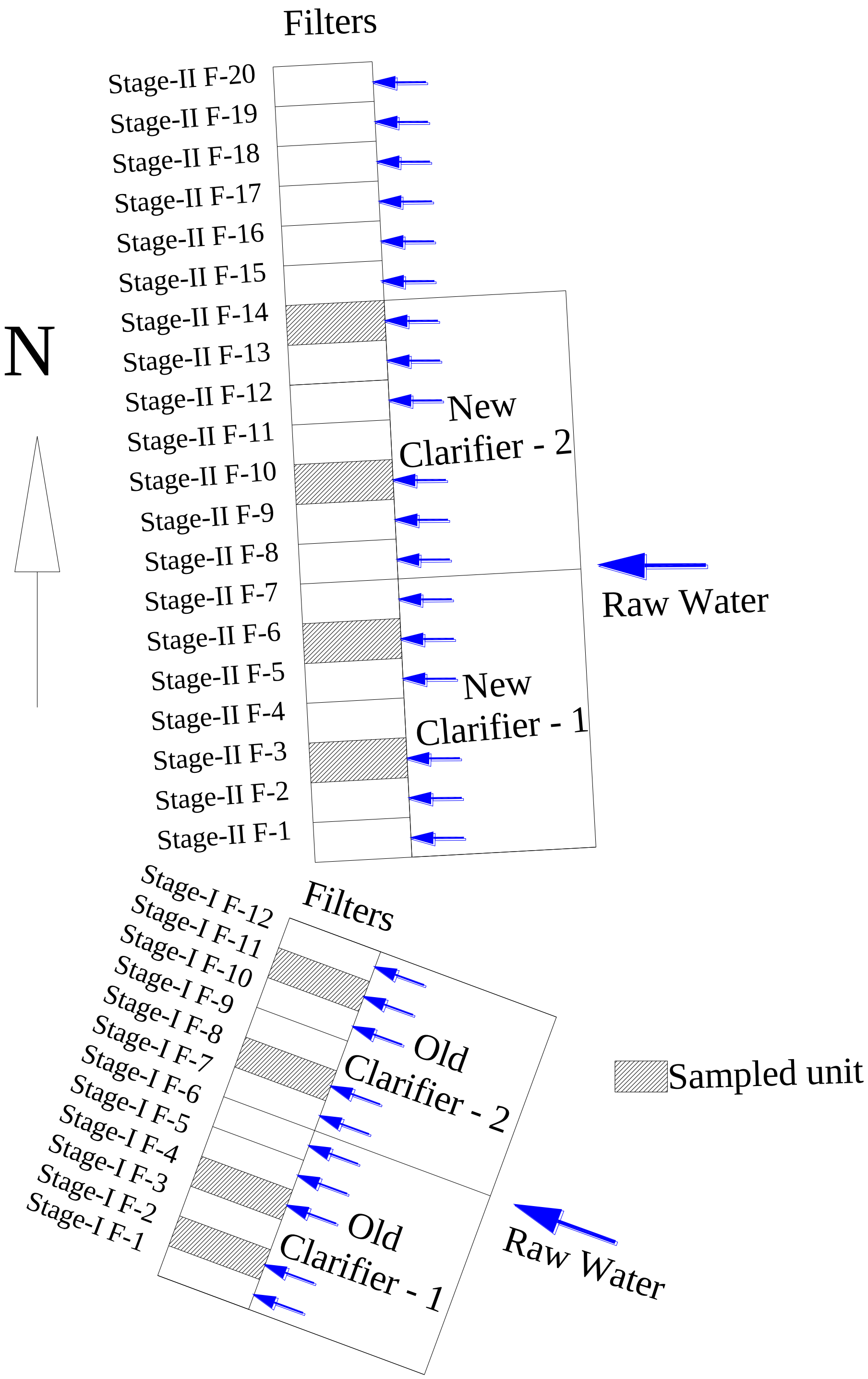


Figure 4-8: Lagadadi sampling Labels

5. RESULTS AND DISCUSSION

5.1 General

Silt and clay are primarily responsible for the turbidity in water. However, any suspended material that will not readily settle is considered to be turbidity. Some clay particles are so small that they will not settle at all. These particles in suspension are known as "colloids."

Most turbidity particles carry a slight negative electrical charge that causes them to repel each other. Zeta Potential is the term used to identify this electro-chemical repulsion. The treatment to remove these particles must neutralize these charges and bring them together until a particle is formed that is large enough to settle.

Pre-sedimentation is commonly used for water supplies where raw water turbidity is continually high, is high on a seasonal basis, or is high sporadically due to storms or other environmental events within the watershed. Pre-sedimentation may also be used in situations where substantial amounts of sand and gravel may be present in the source water. Depending on the purpose of the pre-sedimentation process, the pre-sedimentation basins may be relatively large settling ponds or small concrete basins. When ponds are utilized, they are generally designed to remove large quantities of silt from the raw water and typically provide hydraulic detention times ranging from a few hours to a few days.

Smaller concrete basins that provide less than 20 minutes detention time are sometimes used to provide grit removal. The larger settling ponds are generally not equipped with mechanical sludge removal facilities and must be periodically cleaned by dredging or other means. The concrete pre-sedimentation basins may be equipped with mechanical equipment to remove solids from the basin bottom, or they may be designed to promote manual cleaning using a fire hose or other equipment.

When pre-sedimentation is intended to remove silt and other fine suspended solids, chemical addition is often used to enhance process performance. Organic polymers are the chemicals most commonly added prior to pre-sedimentation to enhance solids removal, but alum and ferric chloride are also sometimes used. The chemicals are added to the raw water as it enters the pre-sedimentation basin to promote solid separation.

In this study the two main treatment unit processes evaluated are clarifiers and filters of both water treatment plants. These are the two major units playing a vital role in turbidity removal.

5.2 Gafarsa Treatment Plant

The Gafarsa WTP designed to treat surface water based on prechlorination-coagulation-sludge blanket clarification-sand filtration and post-chlorination processes. The maximum design production rate of 30,000 cum/day cannot be achieved in the recent years, besides the deterioration of the filtered water quality in terms of turbidity and color. The main concern of AWWSA is the shortage in water supply rather than water quality, because filtered water quality comply with the guidelines of the World Health Organization (WHO) that are adopted by Ethiopia.

Gafarsa water treatment plant has a circular up flow type. The clarifiers shape matters its turbidity removal efficiency. When shapes are considered rectangular clarifiers are more efficient than circular ones.

The designer recommended in the operation manual that sludge draw off valves be opened for 10 seconds every 2 minutes. However in practice, Clarifier 1 has three draw off valves that open every 5 minutes for 30 seconds. The valves leak during the 5 minutes of closure. Clarifier 2 has three draw off valves that open every 3 minutes for 20 seconds.

5.2.1 Clarifiers performance

Table 5-1 below depicts removal percentages of clarifier No. 1 and No. 2. Raw water turbidity varies between 95NTU and 151NTU. Effluent water turbidity varies from 2.20 NTU to 3.25 NTU and 2.45 NTU to 3.40NTU for Clarifier-1 and Clarifier-2 respectively.

Table 5-1 Turbidity Reduction Efficiency of Gafarsa Clarifiers

Sample No.	**Turbidity in Clarifier-1**		**Turbidity in Clarifier-2**	
	Before	**After**	**Before**	**After**
1	151	3.03	151	3.40
2	148	3.00	148	2.52
3	140	2.85	140	2.60
4	136	3.20	136	2.90
5	130	2.95	130	3.01
6	128	3.25	128	3.00
7	120	3.12	120	2.98
8	110	2.87	110	2.65
9	106	3.01	106	2.84
10	103	2.50	103	2.65
11	101	2.80	101	2.70
12	95	2.20	95	2.45

As shown in the above table, clarifier-1 reduced average turbidity of raw water from 122.33 NTU to 2.90 NTU whereas Clarifier-2 reduced average turbidity of raw water from 122.33 NTU to 2.81 NTU. Even though it is not significant clarifier-1 is performing better when compared to clarifier-2 as observed in the result.

5.2.2 Filters Performance

There are twelve filters operating simultaneously receiving effluent from the two circular up flow clarifiers in the treatment plant. These filters are backwashed when the head loss in the system become above the limit set during the design.

In computation of the removal efficiency of the filters, average values of effluent turbidity from the clarifiers are considered as initial influent turbidity value. This value is compared with effluent turbidity from each filter.

Turbidity removal efficiency of the filters are evaluated and shown as here under.

Table 5-2: Turbidity Reduction Efficiency of Gafarsa filters

Sample No.	Turbidity in Filter - 1		Turbidity in Filter - 2		Turbidity in Filter - 3		Turbidity in Filter - 4	
	Before	After	Before	After	Before	After	Before	After
1	3.22	1.12	3.22	1.16	3.22	1.18	3.22	1.17
2	2.76	1.25	2.76	1.35	2.76	1.4	2.76	1.27
3	2.73	1.15	2.73	1.17	2.73	1.16	2.73	1.2
4	3.05	1.17	3.05	1.15	3.05	1.15	3.05	1.16
5	2.98	1.16	2.98	1.17	2.98	1.18	2.98	1.16
6	3.13	1.25	3.13	1.27	3.13	1.22	3.13	1.28
7	3.05	1.26	3.05	1.35	3.05	1.3	3.05	1.4
8	2.76	0.75	2.76	1.25	2.76	1.3	2.76	1.35
9	2.93	1.1	2.93	0.95	2.93	1.3	2.93	1.25
10	2.58	0.85	2.58	1	2.58	1.05	2.58	1.15
11	2.75	0.9	2.75	1.05	2.75	1.1	2.75	1.11
12	2.33	0.75	2.33	0.8	2.33	0.85	2.33	1

Sample No.	Turbidity in Filter - 5		Turbidity in Filter - 6		Turbidity in Filter - 7		Turbidity in Filter - 8	
	Before	After	Before	After	Before	After	Before	After
1	3.22	1.15	3.22	1.18	3.22	1.13	3.22	1.15
2	2.76	1.35	2.76	1.16	2.76	1.15	2.76	1.17
3	2.73	1.18	2.73	1.20	2.73	1.1	2.73	1.16
4	3.05	1.17	3.05	1.18	3.05	1.14	3.05	1.2
5	2.98	1.16	2.98	1.18	2.98	1.15	2.98	1.18
6	3.13	1.30	3.13	1.29	3.13	1.22	3.13	1.25
7	3.05	1.45	3.05	1.62	3.05	1.35	3.05	1.45
8	2.76	1.40	2.76	1.35	2.76	1.00	2.76	1.00
9	2.93	1.20	2.93	1.25	2.93	1.12	2.93	1.20
10	2.58	1.00	2.58	1.17	2.58	0.95	2.58	1.10
11	2.75	0.95	2.75	1.12	2.75	1.13	2.75	1.00
12	2.33	0.95	2.33	1.00	2.33	0.7	2.33	0.90

Sample No.	Turbidity in Filter - 9		Turbidity in Filter - 10		Turbidity in Filter - 11		Turbidity in Filter - 12	
	Before	After	Before	After	Before	After	Before	After
1	3.22	1.12	3.22	1.17	3.22	1.18	3.22	1.2
2	2.76	1.29	2.76	1.18	2.76	1.25	2.76	1.3
3	2.73	1.21	2.73	1.18	2.73	1.16	2.73	1.18
4	3.05	1.19	3.05	1.18	3.05	1.19	3.05	1.18
5	2.98	1.19	2.98	1.18	2.98	1.19	2.98	1.18
6	3.13	1.28	3.13	1.26	3.13	1.3	3.13	1.28
7	3.05	1.55	3.05	1.3	3.05	1.29	3.05	1.45
8	2.76	1.5	2.76	1.6	2.76	1.5	2.76	1.6
9	2.93	1.13	2.93	1	2.93	1.2	2.93	1.17
10	2.58	0.9	2.58	1.2	2.58	1.15	2.58	1.3
11	2.75	0.95	2.75	1.1	2.75	1.14	2.75	1.12
12	2.33	1	2.33	0.85	2.33	0.95	2.33	1

When average turbidity reduction is analyzed in the above table, maximum performance and minimum performance are observed in Filter-1 and Filter-12 respectively. Filter-1 reduced turbidity from average 2.86 NTU to 1.06 NTU while Filter-12 reduced turbidity of influent to 1.25 NTU.

5.3 Lagadadi Treatment Plant

Lagadadi treatment plant clarifies are of rectangular type and has better performance than that of Gafarsa due to its geometrical advantage.

Treated water is impounded in a storage tank at the site before it is pumped to the capital. Sludge drawn off from the clarification basins and waste backwash water are disposed without any treatment to the Akaki River.

Coagulation designed to be based on Alum combined with polyelectrolyte. Because of low alkalinity of the raw water Lime was added to maintain pH value for optimal performance of Alum. In the first two decades of operation of the treatment plant high doses of Alum (50-60 mg/l) combined with polyelectrolyte was needed for appropriate operation of the treatment plant. During the last decade there was a tremendous deterioration of raw water quality in regard of turbidity and apparent color. As a result of that Alum was abandoned as coagulant and substituted with an organic cationic coagulant (CATFLOC-CL).

Operational problems were encountered during operation period of the plan. Some of the problems were caused as a result of raw water quality deterioration, malfunction of some equipment and aging of the plant.

Flash mixing stage is a key component in the coagulation-flocculation process. This stage is more essential in the Lagadadi WTP because there is no separate coagulation step. The effect of flash mixing on flocculation-clarification can be demonstrated by the jar test. Turbidity removal efficiency can be improved with best/optimum flush mixing.

The Super Pulsators installed in the clarifiers operate improperly for many years, and as a result, flocs were released from the clarifier sludge blanket and reach the sand filters. The consequence of this is deteriorating filtered water quality and shortening filter runs period. Actual sludge thickness of 4.1 meters and boundary layer levels of 0.7 were reported in previous tests, which are significantly differ from the design values of 3.0 and 1.8 meter for sludge thickness and boundary layer respectively.

To overcome the leakage problem of flocs, operators open manually sludge draw off valves for extended periods to reduce sludge levels in the clarifier. As a result, the volume of wasted water increases and clarification efficiency decreases (because sludge density decreases).

Timing of sludge draw off valves is done by two adjustable timers installed in the controller board. One is to adjust duration of opening of the valves, and the other for adjusting intervals between two successive openings. The timer of opening intervals is connected to three different valves. That means the time interval of opening is three times the value set in the timer. The timer was set at 4 minutes and checking the intervals of openings is 12 minutes.

The 12 minutes intervals are higher than the 4 minutes recommended by the designer. One of the reasons that flocs were released is connected to the malfunction of the sludge draw off valves.

Two different waste waters are produced in the treatment plant: sludge drawn off the clarifiers and wasted filter backwash water. Both of the wastewater streams are disposed of to the near river without any treatment.

5.3.1 Clarifiers Performance

Lagadadi treatment plant is constructed in two stages. The first stage clarifiers are not equipped with tube settlers that enhance floc settlement. For the clarifiers constructed during the second stages, tube settlers are installed.

Water samples are collected from randomly selected units of the first and the second stage constructed clarifiers and filters. Based on the laboratory test result of the samples, turbidity removal efficiencies of the clarifiers are presented in table 5-3 below.

Table 5-3: Lagadadi First stage and Second stage Clarifiers Turbidity removal efficiency

Sample No.	**Old Clarifier-1**		**Old Clarifier-2**		**New Clarifier-1**		**New Clarifier-2**	
	Before	**After**	**Before**	**After**	**Before**	**After**	**Before**	**After**
1	395	3.48	395	3.16	395	1.63	395	1.49
2	156	4.11	156	3.33	156	1.59	156	1.54
3	161	2.296	161	2.646	161	1.886	161	1.816
4	158	4.091	158	3.411	158	1.981	158	1.951
5	155	3.718	155	3.023	155	1.765	155	1.558
6	162	3.124	162	2.943	162	1.668	162	1.775
7	159	3.644	159	3.126	159	1.805	159	1.761
8	154	2.995	154	2.531	154	1.746	154	1.698
9	160	2.754	160	2.367	160	1.94	160	1.745
10	157	2.631	157	2.175	157	2.03	157	1.735

As observed in the above table, turbidity of effluent from clarifiers with tube settlers is nearly double of that of clarifiers without settlers. Here role of tube settlers in enhancing floc settlement is magnificently observed. The removal efficiency of all clarifiers is greater than 97%.

Sample number -1 is collected in rainy season and turbidity of the raw water for this sample is more than double of the other samples turbidity. Efficiency of the new clarifiers is nearly similar for all samples tested whereas that of the old clarifiers has variations except for the first sample (Figure 5-1 below).

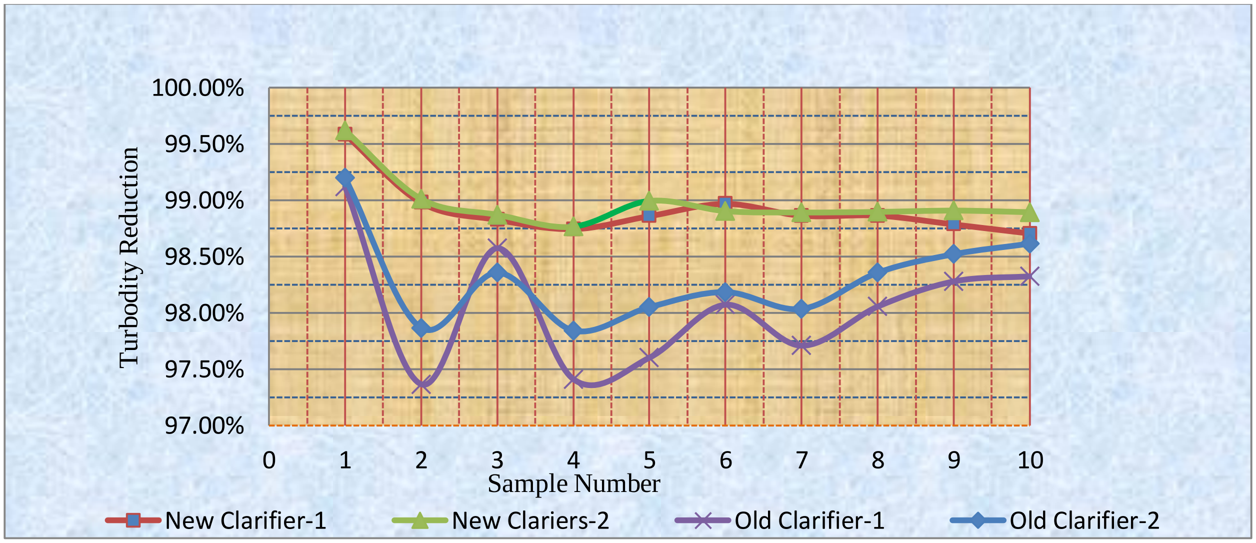


Figure 5-1: Turbidity reduction efficiency of Lagadadi old and new clarifiers

5.3.2 Filters Performance

In this study effluent from eight filter units, four from the first stage and four from second stage are analyzed. The turbidity removal efficiencies of each filter unit as measured are presented hereunder.

Table 5-4: Lagadadi first stage and second stages filter turbidity removal efficiency

Sample No.	Stage-I F-1		Stage-I F-2		Stage-I F-3		Stage-I F-4	
	Before	After	Before	After	Before	After	Before	After
1	2.44	0.11	2.44	0.13	2.44	0.21	2.44	0.31
2	2.64	0.68	2.64	0.54	2.64	0.23	2.64	0.42
3	2.16	0.152	2.16	0.302	2.16	0.244	2.16	0.479
4	2.86	0.164	2.86	0.266	2.86	0.279	2.86	0.027
5	2.52	0.112	2.52	0.315	2.52	0.166	2.52	0.503
6	2.38	0.188	2.38	0.403	2.38	0.208	2.38	0.427
7	2.58	0.155	2.58	0.328	2.58	0.218	2.58	0.319
8	2.24	0.152	2.24	0.349	2.24	0.197	2.24	0.416
9	2.20	0.165	2.20	0.16	2.20	0.208	2.20	0.387
10	2.14	0.157	2.14	0.379	2.14	0.238	2.14	0.374

Sample No.	Stage-II F - 1		Stage-II F -2		Stage-II F-3		Stage-II F-4	
	Before	After	Before	After	Before	After	Before	After
1	2.44	0.35	2.44	0.45	2.44	0.52	2.44	0.47
2	2.64	0.12	2.64	0.61	2.64	0.17	2.64	0.21
3	2.16	0.324	2.16	0.209	2.16	0.243	2.16	0.396
4	2.86	0.119	2.86	0.124	2.86	0.137	2.86	0.111
5	2.52	0.254	2.52	0.333	2.52	0.152	2.52	0.234
6	2.38	0.288	2.38	0.442	2.38	0.227	2.38	0.255
7	2.58	0.220	2.58	0.300	2.58	0.172	2.58	0.200
8	2.24	0.254	2.24	0.358	2.24	0.184	2.24	0.230
9	2.20	0.354	2.20	0.367	2.20	0.194	2.20	0.228
10	2.14	0.376	2.14	0.342	2.14	0.183	2.14	0.219

5.4 Turbidity reduction efficiency comparison

5.4.1 Clarifiers

The clarifiers and filter units of the two treatment plants are compared with regard to turbidity reduction efficiency.

As shown in the tables above the average turbidity of raw water from Lagadadi sources is reduced from 181.7 NTU to 2.41 NTU by the clarifiers while that of Gafarsa Clarifiers reduced average raw water turbidity from 122.33 NTU to 2.85 NTU. This implies that turbidity removal efficiency of Lagadadi and Gafarsa clarifiers in percentage are 98.67 and 97.67 respectively.

In comparison of clarifiers efficiencies the average turbidity removal efficiency of the old and new clarifiers are taken for Lagadadi treatment plant.

Turbidity removal efficiency of Lagadadi treatment new clarifier is higher than that of the old one due to presence of tube settlers which enhanced its performance. However, performance efficiency of Gafarsa clarifiers is lower than that of Lagadadi old clarifiers in most cases. When efficiency of Lagadadi new clarifier is compared to Gafarsa clarifier, all values are on the higher side.

With respect to turbidity removal efficiency, all clarifiers are performing well to standard as all values are above 97%.

5.4.2 Filters

For filters efficiency comparison, the overall turbidity removal of filter units at both treatment plants are evaluated.

The primary data collected from both treatment plant filter units indicated that Lagadadi filters reduced average turbidity of clarifiers' effluent from 2.42 NTU to 0.27 NTU i.e. 88.84% reduction while that of Gafarsa treatment plant filters reduced average turbidity of clarifiers' effluent from 2.85 NTU to 1.17 NTU which is 58.95%.

Study conducted on Gafarsa and Lagadadi treatment plants by a PHD student researcher named Samir Hatukai revealed that the Gafarsa treatment plant some of the filters have lost 30 – 36% and others between 20 - 27%. Moreover, Samir's study confirmed experimentally that due to non-uniform flow distribution of backwash water, filter media grains were piled up at different places negatively influencing the performance of filters. Turbidity removal efficiency of Gafarsa filters is much lower than that of Lagadadi filters due to these problems.

The following table shows average turbidity removal efficiency of Lagadadi and Gafarsa water treatment sampled filter units.

Table 5-5: Average turbidity removal efficiency comparison

Description	Turbidity Removal Efficiency							
Lagadadi WT Filters	91.70	86.83	90.85	84.44	88.63	85.24	90.84	89.20
Gafarsa WT Filters	62.60	59.58	58.07	57.55	58.22	56.98	61.00	59.68

As shown in Table 5-5 above, the turbidity removal efficiency of Lagadadi treatment plant is higher than that of Gafarsa.

The average turbidity removal efficiency of Lagadadi filter units and Gafarsa filter units are 88.47% and 59.21% respectively.

6. CONCLUSION AND RECOMMENDATION

6.1 Conclusions

The differences in the quality of inflow water cause to the distinct water quality in Lagadadi and Gafarsa reservoirs. Kaolinite dominantly observed in Gafarsa dam is a 1:1 (Al:Si) clay of large particle size (0.1-5.0 μm) which typically has a red color because of iron oxide coating, while montmorillonite is a 2:1 (Si:Al) smaller particle (0.01-2.0 μm) clay (Cuker, 1990). Montmorillonite causes more sustainable turbidity per unit of mass than kaolinite because its ability to form very stable suspensions in water by adsorption of monovalent cations on its surfaces (Consult, 1998) and because of its smaller particles. These different characteristics of the clay minerals have a powerful effect on the phisico-chemical and biological quality of the water in the reservoirs, although it must be noticed that in controlling limnological interactions the nature of the coating of the clay particles could be more important than the underlying mineral itself (Cuker, 1990).

The possible higher montmorillonite levels in Lagadadi reservoir suggest stronger light limitation, and better phosphate sink than in the kaolinite rich water of Gafarsa reservoir, thus limiting the algal growth in Lagadadi reservoir. Sinking of algal-clay flocs is also a possible mechanism for reducing algae levels in the reservoir.

The time needed to an alum floc in Gafarsa reservoir to settle down was three times less than in Lagadadi reservoir (BCEOM, 1980).

The relative increase in phytoplankton density in the dry season in Gafarsa reservoir seems to be correlated to the better light penetration and thicker productive layer in this reservoir although the nutrient load (especially phosphate) is lower than in Lagadadi reservoir (1.7 and 4.04 g/m^2/year in Gafarsa and Lagadadi reservoirs respectively).

Based on output of the study conducted on performance efficiency evaluation of Gafarsa and Lagadadi treatment plants, the following conclusions can be drawn:

i. Turbidity removal efficiency of stage II clarifiers of Lagadadi treatment plant is better than that of Stage I most probably due to presence of Lamella plates and tubes in stage II clarifiers.
ii. Turbidity removal efficiency of Lagadadi filters is better than Gafarsa treatment plant filters. The main reason can be use of polymers/coagulant aid at Lagadadi treatment plant.
iii. Gafarsa water treatment filters turbidity removal efficiency varies from 46.89% to 72.83% while removal capacity of Lagadadi treatment plant filters ranges between 76.92% and 95.84%.
iv. Floc carry over is common problem for both treatment plants particularly during wet season
v. Orifice type overflow weir at both treatment plants played a vital role in increasing floc carryover due to non-uniformity of orifices sizing resulted from incrustation/impurities built up at inside surfaces of some orifice weirs, affecting uniform flow of effluents to the receiving filters.
vi. Lack of regular plant cleaning particularly with understanding of the science is observed at both treatment plants. For instance more emphasis is given to sludge washing away and attention is not given to maintaining the weir level and size.
vii. There is no sufficient dam catchment protection at Lagadadi and Dire reservoirs.

6.2 Recommendations

Two studies were conducted by consultants named BCEOM and SEURECA in 1980 and 1991. The SEURECA (1991) report, which is the more comprehensive report, stated that turbidity of 100 FTU occurred near Lagadadi dam during the wet season, with a gradually improving during the succeeding dry season. BCEOM report (1980) stated that there is a slight temperature decrease between the top and the bottom of the reservoir but there is no thermal stratification with a pronounced thermocline. November 1998 echo sounding showed no temperature stratification as well. There are dissolved iron and manganese at all levels of the reservoir. It was found that the manganese concentration in the water increased quite dramatically with depth within the reservoir, from 0.2 mg/l close to the surface to over 2.0 mg/l at 15 m depth. The pH of the water was only slightly alkaline (pH: 7.3-7.4). The mean orthophosphate concentration in 1991 was about 0.35 mg/l and the phosphate loading was estimated as 1.7 g/year per square meter of this reservoir.

There are only very partial and not reliable data on the phytoplankton populations in Gafarsa reservoir. At the time period that BCEOM (1980) study was conducted only a little algae growth existed in the reservoir. Most of the algae were Microcystis species (Cyanobacteria) which create a film on the reservoir surface. The very big and heavy diatom algae Melosira spp. existed also in the reservoir. At the end of the dry season (April-May) algae blooms may happen in the reservoir and create a nuisance in the treatment plant that had to be remedied by special treatment (AAWSA personnel, personal communication). Cyprinidae species fish were fished by local fishermen at Gafarsa reservoir on August 2, 1998.

Improving raw water quality of the sources improve efficiency of water treatment plant and consecutively the chemical consumption will be minimized. Various water quality improvement measures can be taken for this. The primary tasks to be done are reducing turbidity load in the reservoir as well as reduction of nutrient load in the impounding reservoirs. Table below depicts possible measures to improve water quality in reservoirs.

Table 6-1: Possible measures to improve raw water quality

Catchment area	Reservoir
Changing the agricultural practices. Control of Livestock farming. Reforestation and re-vegetation. Upstream silt trap. Control of wastewater effluents.	Selective discharge of nutrient rich mud from the reservoir bottom. Use of algaecide. Biological control by fish.

Introducing the biological control of algae by fish (the bio-manipulation approach), at the reservoir level, is also a preventive measure which may suppress planktonic blooms at the dry season especially at Gafarsa reservoir. Although there is only limited information about fish populations in the reservoirs it gives a hint that the water quality enables fish growth. The most suitable fish species are the filter feeding fish, such as the silver carp or the big-head carp which consume phytoplankton and zooplankton without re-suspending the sediments, and not bottom feeders which probably assemble the reservoirs fish population now.

Using algaecides such as copper sulfate at the reservoir is a curative measure which can be selected according to the varying needs. Other means, at the reservoir level, such as reservoir aeration or phosphate precipitation are expensive and unsustainable.

For betterment of the treatment plants turbidity removal efficiency and minimization of floc carryover, the following can be recommended based on findings of the study:

i. Installation of plate/tube settlers preferably plate settlers on the remaining compartments of Gafarsa clarifiers
ii. Construction of bank filtration adjacent to raw water intake from the dams to minimize turbidity of raw water entering the sedimentation tank
iii. Frequently scrubbing inner circumferential surfaces of the orifice overflow weir to maintain uniformity of velocities through all orifices
iv. Capacity building trainings regarding operation of plant unit as well as record keeping shall be provided to the operators.
v. At Gafarsa treatment plant the field laboratory has to be established to conduct water sample analysis on the spot.
vi. Catchment protection shall be given attention at Lagadadi and Dire dams to prevent deterioration of the raw water.

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