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PERFORMANCE EVALUATION OF DRINKING WATER TREATMENT PLANT

(CASE STUDY: GAMBELLA TOWN DRINKING WATER TREATMENT PLANT)

By

Tariku G/Tsadik

Advisor

Dr. Ing. Berhanu Assefa

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Approved by Board of Examiners

Dr.Ing Berhanu Assefa	______________	28 March 2013
(Head of Chemical Engineering Department)	Signature	Date
Dr.Ing. Berhanu Assefa	______________	28 March 2013
Advisor	Signature	Date
Dr. Tassisa Kaba	______________	28 March 2013
Internal Examiner	Signature	Date
Dr.Eng. Shimelis Admassu	______________	28 March 2013
External Examiner	Signature	Date

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Acronyms

AWWA	American Water Works Association
CCP	Composite Correction Program
CPE	Comprehensive Performance Evaluation
CT	Concentration of disinfectant residual (C) times contact time (T)
ESWTR	Enhanced Surface Water Treatment Rule
GWTP	Gambella town Water treatment Plant
IESWTR	Interim Enhanced Surface Water Treatment Rule
mg/l	Milligrams per liter
MPA	Microscopic Particulate Analysis
NTU	Nephelometric Turbidity Unit
RPM	Rotation per Minute
RSF	Rapid Sand Filter
SWTR	Surface Water Treatment Rule
USEPA	United States Environmental Protection Agency
WHO	World Health Organization

Abstract

The conventional water treatment plant, especially in developing countries, faces major challenges in terms of assessing its operation and performance due to inappropriate technologies, insufficient equipment and deficiency in skilled expertise. Simple but efficient technologies are therefore necessary for reasonable evaluation of the daily performance of the plant. Turbidity is thought of as a convenient surrogate to give favorable indication of the biological and physical quality of the treated water thus by extension provide a fair gauge of the performance of the treatment plant with respect to water purification. Besides, it is fairly simple to measure, cheap and can easily be understood by the operators. In this study the performance of Gambella town water treatment plant was assessed. The study was conducted by assessing unit process capability, design, operation and maintenance potential to meet optimized goals. From results of the assessments, root factors limiting optimum performance were identified and improvement options were proposed. Major unit processes were evaluated to project their design capabilities to meet current peak water demand by selecting appropriate loading rates as basis criteria. The results of the assessment found that all units had the capability to satisfactorily treat water at peak daily demand of 2000 m^3/day. The study assessed turbidity performances of sedimentation and filtration units by setting optimized turbidity goals. The assessment results indicated that, settled water turbidity was measured less than 10 NTU. And filter turbidity spike of 6.5 NTU following backwash with a reduction to 0.6 NTU after one hour was observed. Generally optimized performance goals were not being achieved. This indicated high risk of microbial pathogens that could pass the filtration barriers in the finished water. Jar test experiments were conducted to evaluate the effectiveness of Aluminum Sulphate (recently used by the treatment plant), Ferric chloride and Ferric Sulphate by comparing the optimum dose at optimum pH for highest turbidity removal and relative costs. From the jar test results Aluminum Sulphate was found to be the effective chemical with 45 mg/l optimum dose at pH 7.1 and the treatment plant was recommended to continue using Aluminum Sulphate at the optimum dose for the raw water characteristics during the evaluation period. Treated water samples were collected from the clear-water well to test 14 water quality parameters according to the standard methods for water and waste water examinations. The collected samples were intended to show the characteristics of the finished water only during the evaluation period. The samples were analyzed at the laboratory of GWTP and results were compared with WHO standards and guidelines for drinking water. Results of the analysis showed that all of the measured parameters were within the acceptable range. In the assessment of factors limiting performance of the treatment plant; major factors were categorized as design, operational and maintenance. No single factor was responsible for poor plant performance, although in general the study found that all factors influence the plant’s ability to work properly. Some of the primary operational problems and the intake structure’s adequacy significantly affected performance. Operational factors were found to have the highest rank. This finding, coupled with the fact that the plant had adequate capability, indicates that improving process control could significantly improve performance.

1. Introduction

1.1 Background

Every living organism requires water to sustain life. Water must fulfill certain quality standard in order to be used for drinking and other purposes. Water of sufficient quantity and quality is one of the basic necessities to human being. A range of 20 to 40 liters of fresh water/person/day is generally considered to be a necessary minimum requirement to meet basic needs for drinking and sanitation [1].

Surface water needs to be treated to meet drinking water quality standard. Drinking water quality standards have been established to ensure that the processes of water treatment, conveyance, storage and distribution are conducted in a way that doesn't affect human health [1].

Ethiopia is one of the wealthiest countries in the world with respect to water resources. On the contrary the access to safe drinking water is among the lowest in the world. There are many public water treatment plants which are designed to supply water of acceptable quality and quantity to the public. But the supply of safe, consistent and adequate water from these treatment plants became a problem.

Assessing performance of treatment plant helps to identify the factors that are inhibiting treatment plant (treatment processes) from producing water of acceptable quality [2]. This study was aimed to evaluate the performance of Gambella town water treatment plant (GWTP) in regard to its design, operation and maintenance potential and to identify the root causes which hindered the plant from producing adequate quality and quantity of water.

In this study the methods followed by the Comprehensive Performance Evaluation (CEP) phase of the Composite Correction Program (CCP) of the USEPA handbook "optimization of treatment plants using the CCP" were adopted [2].

Accordingly the treatment plant's major unit processes capabilities were evaluated based on their potential to handle current peak daily water demand of the population served. The results of this evaluation are expected to indicate if the unit process had adequacy problems.

The performances of the treatment plant's operations were evaluated based on their efficiencies to meet turbidity and finished water quality performance goals.

Turbidity was selected as a performance indicator for the sedimentation and filtration units based on World Health Organization (WHO) turbidity goals and CCP optimized performance goals. The units were assessed based on their potential to act as a barrier for the passage of pathogenic microorganisms and to meet the aforementioned goals. Similarly, the finished water quality was examined, and analyzed according to the procedures in the standard methods for the examination of water and wastewater. The results of the analysis were compared with WHO standard for drinking water to establish the performance status of the treatment plant.

Finally, jar tests were established to determine appropriate chemical coagulant for optimal coagulation. The jar tests were conducted at three different coagulants; namely Aluminum Sulphate, Ferric Chloride and Ferric Sulphate. The effectiveness of these chemicals in terms of their turbidity removal potential at optimum dosages and relative costs were evaluated. From the results of jar tests the effective chemical and its optimum dosage that could optimize the coagulation was identified.

The results of all the assessments were analyzed and factors limiting the performance of the treatment plant were identified. From the identified factors limiting the performance, short and long term alternative improvement options were recommended.

1.2 Gambella Town water treatment plant description

GWTP is a conventional type consisting of raw water intake, coagulation/flocculation, sedimentation, filtration and disinfection units. The plant schematic layout is presented in Annex 1.

The design capacity of the plant was 3600 m^3/day in 24 hours operation. The intake structure is located at Baro River which is 3 km far from the treatment plant; there are 2 raw water pumps and a standby pump each with design flow rate 75 m^3/hr. The raw water is primarily pumped to the balancing tank of capacity 50m^3; then it flows to the splitter box found between two Alabama type flocculators.

Aluminum Sulphate (Alum) in powder form is used as coagulant chemical. To prepare Alum solution, the chemical is mixed with water in the solution preparation tank. The method of handling chemicals in the treatment plant is continuous type completed with electric driven stirrers and gravity feeders. Alum solution is applied above the hydraulic fall mixer (splitter box) between two flocculators.

From the splitter box, the water flows to two parallel Alabama-type hydraulic flocculators with fifteen compartments each. The coagulation and flocculation processes are carried out when the water flows through series compartments having varied headloss. The flocculated water then flows to the sedimentation basin units.

There are two rectangular sedimentation basins following the flocculation unit. The basins have inclined bottom deeper at the entrance than the outlet. The flocculated water settled in this unit and settled water flows to the filtration unit through the weirs at the outlet. The sludge removal mechanism is manual.

Settled water is then collected and distributed through a splitter box to three rapid sand filters. The filters have independent pipelines for filtered water and backwash water. Backwash water is pumped from the filtered water to an elevated tank 20 m above the lip of the backwash overflow trough. The capacity of the tank is 200 m^3.

Filtered water then flows to a Clearwater well of capacity 1200m^3. Calcium Hyphochlorite (65 percent strength) is used as a disinfectant. The chemical is mixed with water to form solution in the chemical preparation tank. The solution is applied at the entrance of the Clearwater channel for disinfection.

The Clearwater is pumped via two high lift pumps to a water reservoir capacity 1440 m^3 located 300m above the treatment level then flows by gravity to the city distribution system.

1.3 Statement of the Problem

Gambella town is experiencing problem of adequate safe water supply since its establishment. The town has a water treatment plant with a design capacity to produce 3600 cubic meters of water per day; while the peak daily water requirement of the residences is only 2000 cubic meters.

Water supplied from the treatment plant is intermittent, distributed once in three days for a maximum of 5 hours, even less, and the residents fetch water from the nearby river and traditional wells which are not protected. Besides the inadequacy of potable water, the threat of waterborne diseases such as cholera and typhoid are critical public health issues. Water borne diseases are one of the top ten diseases in the town [3].

These conditions indicate there is some problem on the treatment plant's performance. Thus, to assess the status of existing water supplied and to evaluate and outline the findings of investigations of the treatment plant's performance level, GWTP was selected as a case of analysis for this thesis.

1.4 Objectives of the study

1.4.1 General objectives

The general objectives of the study were to evaluate the performance of Gambella town water treatment plant in regard to its ability to produce quality water and control pathogens and to identify factors limiting its performance.

1.4.2 Specific objectives

The aforementioned general objectives were addressed by the following specific objectives:

1. To evaluate the design capability of major unit processes (Flocculation, sedimentation, filtration, and disinfection) to support the treatment performance,
2. To assess and compare treated water quality results with the WHO standard guidelines for drinking water to establish the treatment work's performance level,
3. To identify performance limiting factors of the treatment plant, and
4. To assess proper and optimum chemical dosage, for optimal coagulation, by establishing a jar test.

2. Literature review

2.1 Drinking water quality

Water for domestic use is required to be both wholesome and safe. For health reasons a number of chemical, physical and bacteriological quality of water need to be considered. Various toxic metals should not be present in quantities greater than very low limits which are set in standards of drinking water [1, 4, 5]. Of prime importance, however, is the bacteriological quality of water. Drinking water should be free from pathogenic organisms, bacteria and viruses, which may cause diseases. The importance of water as vehicle for the potential spread of diseases is the main concern in water quality control [4].

Many studies have been conducted on the effects of water quality on human health. Based on these studies, the WHO has published guidelines to help countries to set quality standards with which domestic water supplies should comply. Water quality parameters provide a standard of comparison by which to measure water's physical, chemical, and bacteriological characteristics. These parameters include a range of characteristics that make water appealing and useful to consumers, and that ensure the water presents no harm or disruption to the environment or to humans within a wide range of possible water uses [1,6].

Impurities in water can be determined by water analysis. Water analysis is used to classify, prescribe treatment and purification processes and maintains public supplies of water an appropriate standard of quality, clarity and palatability [5]. The analysis of raw water enables the choice of the process for water purification. Analysis at the various stages of treatment allows monitoring the effectiveness of the treatment process, and the analysis of purified water ensures the correct degree of purification, as per required standards, is obtained [7].

Water analysis may be divided into physical (colour, taste and odour, temperature and turbidity), chemical (total solids, hardness, chlorides, pH value, nitrogen and its compounds, dissolved gases, metals and other chemical substances) and microbiological analysis (Microbiological analysis of water includes bacteriological and biological) examination.

2.2 Conventional water treatment plant

The major unit processes that make up the conventional water treatment plant are intake (screening), coagulation/flocculation, sedimentation, filtration, disinfection, and distribution. Once water from the source has entered to the plant as influent, water treatment processes break down into two parts, clarification and disinfection. The first part, clarification, consists of screening, coagulation/flocculation, sedimentation, and filtration. Clarification processes go far in potable water production, but while they do remove many microorganisms from the raw water, they cannot produce water free of microbial pathogens [8]. The second part and the final step, disinfection, destroy or inactivate disease-causing infection agents.

2.2.1 Coagulation and rapid mixing

Coagulation is the process by which particles become destabilized and begin to clump together. It is typically accomplished through chemical addition and mixing. It is the process in which negatively charged particles present in water in a stable suspension are destabilized by the addition of coagulant (positively charged), which allows for aggregation of the particles [2, 9 and 10].

The function of a rapid mix system is to disperse the coagulant uniformly throughout the entire mass of water to ensure effective coagulation. This process is normally followed by a period of flocculation during which the water is gently mixed to promote agglomeration of the coagulated particles. Rapid mix units are designed to generate intense turbulence in the raw water by either mechanical or hydraulic means. Rapid diffusion is necessary in the coagulation process, which comprises the hydrolysis of the coagulant and destabilization of the colloidal material, is completed almost instantaneously (less than one second) [10].

Hydraulic rapid mixers are capable of achieving high velocity gradients for rapid diffusion of coagulants without using mechanical equipment; they are preferred in developing countries. Moreover, they require no imported equipment, and are easily constructed, operated, and maintained with local materials and personnel [11].

2.2.1.1 Types of coagulant Chemicals

Chemicals used in coagulation are classified as primary coagulants and coagulant aids. Primary coagulants are used to cause particles to become destabilised and begin to clump together. The purpose of coagulant aids may be to condition the water for the primary coagulant being used, to add density to slow-settling flocs or toughness so that the flocs will not break up in the following processes [12].

Salts of Aluminum or iron are the most commonly used coagulant chemicals in water treatment because they are effective, relatively low cost, available, and easy to handle, store, and apply.

Alum is the common name for Aluminum Sulphate also known as Sulphate of alumina, and is probably the most widely used coagulant in water treatment [12]. The classical chemical formula for Aluminum Sulphate is $Al_2 (SO_4)_3.18H_2O$, but as used in water treatment it contains varying amounts of water of crystallization. It is supplied in the form of lumps with $21.H_2O$ and in granulated or kibbled form with $14.H_2O$ water of crystallization. The chemical is readily soluble but the solution is corrosive to Aluminum, steel and concrete so tanks of these materials need protective linings. The chemical is also available in liquid form. Its most effective range for coagulation is pH 5.5 - 8 and its reaction when added to water is with the natural or added alkalinity to form Aluminum hydroxide $Al_2 (OH)_3$ (floc) according to the alkalinity present [13].

Ferric Chloride, ($FeCl_3$) is available in liquid form, in yellow-brown lumps as crystal ferric chloride $FeCl_3.6H_2O$ or as anhydrous Ferric Chloride in green-black powder form. Its reactions in the coagulation process are similar to those of Alum, but its relative solubility and pH range differ significantly. The optimum pH range for ferric chloride is 4 to 12. Ferric chloride consumes alkalinity at a rate of about 0.75 mg/L alkalinity for every 1 mg/L of ferric chloride. Ferric chloride dosage is typically about half of the dosage required for alum [13].

Ferric Sulphate is normally corrosive and has a low pH, although it can be supplied in solid form it is usually supplied as a solution. The strength of solution supplied is not fixed by convention as much as for the other chemicals. The purchase of Ferric Sulphate (or Ferric

Chloride) is therefore often based on its iron content as Fe. Depending on solution strength this may range from about 8% up to 14% [13]. The optimum pH range for Ferric Sulphate is similar to that of Ferric Chloride.

2.2.1.2 Evaluation of chemical coagulants

An evaluation of the chemicals used in the treatment process can identify the appropriateness of the chemicals being used. A thorough understanding of coagulation chemistry is important, and changes to coagulation chemicals should not be made without careful consideration.

Essentially coagulants are evaluated to choose the best coagulant in terms of performance and cost. There are many fundamental variables in water treatment, which will have a significant influence on the choice of type of effective coagulant chemical that could be usefully employed in a particular application. The major variables include; Changes in raw water characteristics, pH, temperature, alkalinity and turbidity [14].

The changes in raw water characteristics affect the type and amount of chemicals used in coagulation and, subsequently, filtration and finished water quality. Jar tests are an excellent way to determine the best type and amount of chemical (coagulant dosage) to use for varying raw water characteristics.

The coagulant dosage is dependent on the humic content of natural water and in general is proportional to the colloidal charge in the raw water [14]. The important point about the optimum dose determination depends highly on the raw water turbidity fluctuations and on the fact that “optimum” dose does not always refer to the dose that achieves maximum turbidity removal. For example; If a 10 mg/l increment in dosage produces only a slight improvement in turbidity removal, the increased chemical costs may not warrant the higher dose. Therefore, the optimum dose is more practically thought of as the one that achieves the best turbidity removal “for the money”. [12]

In coagulation, the pH has great effect on inorganic coagulation species and the dissociation of the humic and fulvic acids [14]. The demand for coagulant is often decreased at lower pH values, because of the increasing protonation of organics, and more positively charged coagulant species. Consequently the coagulant dosage required becomes less due the enhanced adsorption in the ideal pH. Under very low pH, precipitation may reduce, or reduce partially, following of enhanced charge neutralization and co-precipitation by adsorption [14].

Alkalinity is of critical importance when selecting a metal salt coagulant such as Aluminum Sulphate (Alum), or Ferric salts. All these materials need some alkalinity to drive the hydrolysis reactions that allow the coagulants to function [12].

The precipitation of mineral turbidity by the classic coagulation and flocculation process is well defined and reasonably straight forward. Turbidity can be classified as being anionically charged particles [15].

2.2.1.3 The jar test

The jar test is a batch process and it is a very versatile test that can be used for Coagulant/coagulant aid type and dosage selection and determination of optimum pH (only a factor with inorganic coagulants which are pH sensitive) [13].

The required coagulant chemical dosage for particular raw water is virtually impossible to determine analytically because of the complex interrelationships which exist between the chemicals and the constituents of the water being treated, as well as such factors as pH, temperature, and the intensity and duration of mixing. Consequently, a laboratory procedure known as the "jar test" is used to determine the most effective and economical dose of coagulant [14].

In the evaluation of a suitable coagulant for removal of particulates and/or humic and other organic compounds from water, the jar test remains the most effective tool. It is performed on a “gang stirrer”. Four to six axial mixing blades are operated by a single motor, so mixing conditions can be replicated in four to six one liter samples simultaneously.

2.2.1.4 Factors that affect performance of coagulation/rapid mix system

The common design parameters that affect the efficiency of coagulation are mixing intensity and detention time. Mixing intensity is typically quantified with a number known as the “velocity gradient” or “G value”. The G value is a function of the power input into the mixing process and the volume of the reaction basin. Typical G values for coagulation mixing range from 300 to 8000 sec^{-1} [16].

The time required to achieve efficient coagulation varies, depending on the coagulation mechanism involved. When charge neutralization is the mechanism involved, the detention time (T) required may be one second or less. When sweep floc or entrapment is the mechanism involved, longer detention time on the order of 1 to 30 seconds may be appropriate [16].

2.2.1.5 Common problems of coagulation performance

Common problems usually occur in coagulation process are under or over-dosing, mixing of insufficient energy, fouling or clogging of injectors or diffusers and side reactions. Under or over dosing can be avoided by using the Jar Testing. Mixing of insufficient energy can cause undesirable coagulation reactions. Fouling or clogging of injectors or diffusers is usually caused either by pre-dilution of coagulant or poor mixing at the point of injection. This circumstance causes high and much localized coagulant concentrations and contributes to significant precipitation around injectors.

2.2.2 Flocculation

Flocculation is almost always used in conjunction with, and preceded by coagulation. It is the process of bringing the destabilized particles (in coagulation) into contact with one another to form larger “floc” particles [13]. These larger particles are more readily removed from the water in subsequent processes. Flocculation is generally accomplished by mixing the destabilized suspension to provide the opportunity for the particles to come into contact with one another and stick together.

It is a time-dependent process that directly affects clarification efficiency by providing multiple opportunities for particles suspended in water to collide through gentle and prolonged agitation. This agitation should be thorough enough to encourage inter-particle contact, but gentle enough to prevent disintegration of existing flocculated particles [9].

A wide variety of slow mixing mechanisms has been used in water treatment. These include vertical shaft mechanical mixers, horizontal shaft mechanical mixers, and hydraulic mixing systems or flocculators.

In hydraulic flocculators the flow of the water is so influenced by small hydraulic structures that a stirring action results. Typical examples are channels with baffles, flocculator chambers placed in series (e.g. Alabama-type flocculator) and gravel bed flocculators. These types of flocculators have advantages such as: their simplicity, lack of moving parts, minimal operation and maintenance, relatively low cost and its proven performance when operated at the design flow rate [10].

2.2.2.1 Alabama-type flocculators

Alabama-type flocculators are hydraulic flocculators having separate chambers placed in series through which the water flows in two directions as shown in Figure 2.1. The water flows from one chamber to the next, entering each adjacent partition at the bottom end through outlets turned upwards. This type of flocculator was initially developed and used in the state of Alabama (U.S.A.) and later introduced in Latin America.

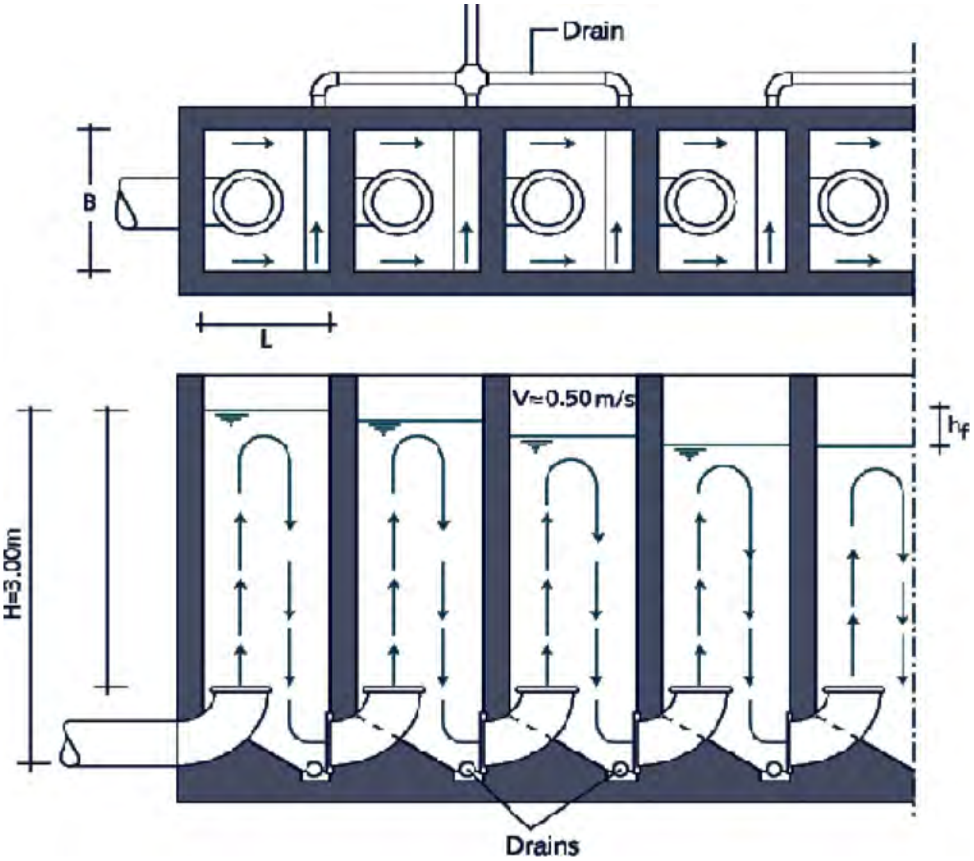

Figure 2.1 Alabama type flocculator
(source IRC [10]).

2.2.2.2 Factors that affect performance of flocculators

The efficiency of the flocculation process is largely determined by the number of collisions between the minute coagulated particles per unit of time. Mixing is a key aspect of the flocculation process. Often the intensity of mixing is reduced as the water proceeds through the flocculation process to achieve optimum performance. At the beginning of the process, the mixing is fairly intense to maximize the particle contact opportunities. Mixing intensity G values in this area are typically in the range 60 to 70 sec^{-1} [14]. Toward the end of the flocculation process, mixing intensity is generally reduced to minimize the potential for breaking up the floc particles that have begun to form. In this portion of the process, G values are commonly in the 10 to 30 sec^{-1} range [14]

The amount of time the water spends in the flocculation process is a key performance parameter. Adequate time must be provided to allow generation of particles sufficiently large to allow their efficient removal in subsequent treatment processes [16]. Overall detention time (T) in the flocculation process typically ranges from 10 to 30 minutes and is generally provided in several different basins or basin segments. This allows the mixing intensity to be varied through the process. The loss of head in Alabama type of flocculator is about 0.35-0.50 m for the entire unit [10].

Flocculator Inlets and Outlets are key parameters that affect the performance of flocculation. Short-circuiting occurs when water bypasses the normal flow path through the basin and reaches the outlet in less than the normal detention time. Inlet and outlet turbulence is sometimes the source of floc-destructive energy and short-circuiting in flocculation basins.

The characteristics of floc formed in an optimized flocculation are; the floc as it enters the flocculation basins should be small and well dispersed throughout the flow. Tiny floc may be an indication that the chemical dose is too low. A ‘popcorn flake’ is a desirable floc. As the floc moves through the flocculation basins the size of the floc should be increasing. If the size of the floc increases and later starts to break up; the mixing intensity of the downstream flocculator may be too high [10].

2.2.2.3 Common problems of flocculation performance

Uneven splitting or flocculation patterns, sudden changes in raw water conditions, chemical under-or over-dosing. Poorly designed flocculation basins result in bulk circulation, resulting in high localized entry velocities, better flow distribution usually requires head loss to be introduced, mitigation can be difficult without causing floc damage.

2.2.3 Sedimentation

Sedimentation is the process by which solids are removed from the water by means of gravity separation. In the sedimentation process, the water passes through a basin in which relatively quiescent conditions prevail. Under these conditions, the floc particles formed during flocculation settle to the bottom of the basin while the “clear” water passes out of the basin over an effluent baffle or weir. The solids collect on the basin bottom and are removed, manually or mechanically by “sludge collection” device [12].

Conventional sedimentation typically involves one or more basins. These “clarifiers” are relatively large open tanks, either circular or rectangular in shape. In properly designed clarifiers, velocity currents are reduced to the point where gravity is the predominant force acting on the water/solids suspension. Under this condition, the difference in specific gravity between the water and the solids particles causes the solids particles to settle to the bottom of the basin [12].

Rectangular settling basins are long rectangular or square basins commonly used in water treatment plants. The actual design of a settling basin not only includes the ideal settling zone, but an inlet zone to equally distribute the incoming water and dissipate velocity currents; sludge zone in which particles that settle out of solution accumulate and are subsequently removed for further processing; and an outlet zone in which baffles and weirs are used to ensure that no short circuiting occurs and quiescent conditions are maintained [10].

2.2.3.1 Factors that affect performance of sedimentation

Overflow rate, detention time, and weir loading rate are the three main parameters that affect the performance of settling basins. The efficiency of a sedimentation basin in the removal of suspended particles can be determined using, as a basis, the settling velocity of a particle that in the detention time will just traverse the full depth of the tank [12].

Factors that influence settling velocity include the size, shape, and weight of the floe, viscosity and hence the temperature of the water, the velocity of flow, and the inlet and outlet design .

Suspended solids removal and turbidity reduction rates achieved through sedimentation may range from about 50 to 90 percent, depending on the nature of the solids, the level of treatment provided, and the design of the clarifiers. Common values are in the 60 to 80 percent range [16].

2.2.3.2 Common problems of sedimentation performance

Common causes for poor sedimentation performance are: density currents due to temperature variation within basins, excessive operating loading rates, and poor hydraulics due to uneven inlet flow splitting and inappropriate sludge removal rates. It is also important to make sure that clarifier operating at or below design capacity.

Density currents are generally worst in large sedimentation basins due to surface warming and it can cause significant floc carryover [16].

Design and operation of inlet flow also affect performance of sedimentation. Inlet flow mal-distribution generally caused by poor inlet channel design and sometimes caused by uneven inlet weirs [10].

2.2.4 Filtration

In the filtration process, the water passes through a bed of granular filter media or other filtering material and solids are physically retained on the media. After passing through the filter media, the "filtered" water is collected and removed from the filter. The solids retained on the media are also periodically collected and removed. The performance of most filters depends largely on the preparatory treatment processes of coagulation and flocculation and sedimentation. Without effective use of these processes, only marginal filter performance can be expected [10].

Filters are classified according to the type of media used and the operational conditions employed. The primary types of filters used in water treatment include: Rapid Sand Filters; Pressure Filters; Slow Sand Filters; and Precoat Filters [17].

During the filtration process, particles suspended in the filtered water are removed largely through entrapment within the filter media. As more and more fluid is passed through the filter, the suspended particles accumulate within the media and causes the pore spaces to become smaller, the velocities to decrease, head loss within the filter to reach excessively high levels (2 to 3 meter of hydraulic head), or it can become pushed through the filter, resulting in product water turbidities that reach unacceptable levels (greater than 1 NTU). Therefore, in order to maximize the use of a given filter, it is necessary to remove these entrapped particles from the media itself. Filter backwashing is the process by which this is accomplished [17, 10].

2.2.4.1 Factors that affect performance of filtration

Improperly designed, operated, or maintained filters can contribute to poor water quality and sub-optimal performance. A host of factors may be contributing to poor performance, and systems should make a comprehensive evaluation of the filter to identify which factors are responsible, factors that affect the performance of filters are listed below [17].

Design of Filter Beds- Systems should verify that the filters are constructed and maintained according to design specifications.

Filter Rate and Rate Control- The rate of filtration and rate control are other important aspects of filters that should be evaluated. Without proper control, surges may occur which force suspended particles through the filter media.

Filter Backwashing- Filter backwashing has been identified as a critical step in the filtration process. Many of the operating problems associated with filters may be a result of inadequate or improper backwashing.

In addition to the above factors; source water quality, chemical pretreatment, filter media size/type, uniformity coefficient and surface characteristics, filter run length, filter maturation, water temperature, filter integrity and backwashing procedures also can affect performance. Ensuring that filtration processes are performing optimally helps to increase the level of protection from potential contaminants, including pathogens, in the treated water [18].

2.2.4.2 Common problems of filtration performance

Inadequate pre-treatment- the pretreatment process (i.e., coagulation, flocculation, and clarification) in a conventional plant generally should produce pretreated waters with turbidities no greater than about 4 NTU with levels less than 2 NTU preferred. Pretreated waters with turbidities much greater than 4 NTU are indicative of floc carryover that tends to cause short filter run lengths. Conversely, pretreated waters with turbidities of 1 NTU or less in a conventional plant may result in inefficient filter operation or inadequate particulate removal [17].

Inadequate filter washing- can result in poor filtered water quality and mudball formation. Cracks can occur in filter media when compressible solids remaining from previous filter runs pull filter media together and away from the filter box wall. Pretreated water can then travel through the cracks and bypass much of the filter media. Mudball result from residuals remaining from previous filter runs sticking to filter media and forming agglomerations that grow too large to reach the wash water collection troughs during washing. As they grow heavier, mudball can sink to create impassable regions within the filter media, typically at the anthracite-sand or sand-gravel interface. The impassable regions result in higher effective filtration rates, poorer filtered water quality, and shorter filter runs [17].

Air Binding- Filter influent waters, particularly from surface water sources; typically contain significant concentrations of dissolved gases. Depending on water temperature, the dissolved gas concentration may reach saturation point typically during algal blooms, during seasonal changes when temperatures increase, or where there is significant cascading and aeration of source water. When head loss exceeds the available head at some elevation within the filter media, pressure falls below atmospheric and air escapes from solution. Air binding occurs when the accumulation of air bubbles blocks the water's path. An excessive effective filtration rate and significantly increased head loss result [7].

Filter ripening- the increased passage of particles and microorganisms through granular media filters immediately following backwashing is a common problem known to the water treatment community as filter ‘‘ripening’’ or maturation [5].

2.2.5 Disinfection

The single most important requirement of drinking water is that it should be free from any micro-organisms that could transmit disease or illness to the consumer. Processes such as storage, sedimentation, coagulation and flocculation, and filtration reduce the bacterial content of water to varying degrees. However, these processes cannot assure that the water they produce is bacteriological safe. Final disinfection will be needed for destruction, or at least complete inactivation, of harmful micro-organisms present in the water by physical or chemical disinfection methods [19].

2.2.5.1 Chemical disinfection

Several chemicals, acting as strong oxidants, can destroy micro-organisms. Hydrogen peroxide and other metallic peroxides, lime, potassium and calcium permanganate, ozone and chlorine and its related compounds are among the many disinfectants commonly used.

Chlorine and chlorine related substances have a number of characteristics that are highly valuable such as: they have a broad-spectrum germicidal potency and show a good persistence in water distribution systems. This means that they present residual properties that can be easily measured and monitored in networks or after the water has been treated and/or delivered to the users. In addition to this the equipment needed for dosage is simple, reliable and low-cost [19, 20].

The most popular substances in the chlorine family are chlorine; chlorinated lime; high-concentration hypochlorite and sodium hypochlorite. They present different chlorine concentrations or active chlorine; which is a measure of their strength. Calcium hypochlorite $Ca(OCl)_2$ for drinking water disinfection is most commonly encountered as: chlorinated lime or bleaching powder; high test hypochlorite; or calcium hypochlorite in different forms [20].

Chlorinated lime or bleaching powder is a white powder which is a mixture of calcium hydroxide, calcium chloride and calcium hypochlorite, containing 20 to 35 percent available chlorine. Often it is necessary to dissolve calcium hypochlorite in water, and the clear solution that is produced, used as the disinfectant. The concentration of chlorine in a solution (once prepared), should not exceed 5 percent. If it does, then considerable chlorine may be lost in the form of sediment.

2.2.5.2 Factors that affect performance of chemical disinfection

The performance of disinfection is affected by many factors as; the type of disinfectant used; feed rate, chlorine dose, and residual chlorine concentration. Flow rate to the disinfection segment, temperature, and pH; Design and operations of disinfection basins, also affects the performance of the disinfectant being used.

The amount of disinfectant added to the water is referred to as the dose, and is usually measured as the number of milligrams added to each litre of water (mg/l). The amount of disinfectant destroyed in the reaction with the substances in the water is called the demand. The amount of chlorine (either free or combined) that remains after a certain contact time is known as the residual chlorine. The residual is also important as a check on the effectiveness of the dosing [21].

When chlorine is added to water, a certain period of time is required for the chlorine to react with the micro-organisms and compounds in the water. This time is called the contact time, and a minimum of 30 minutes is usually recommended.

The presence of the residual chlorine should be determined only after the specified retention time. If a 30 minutes retention time was set, then the monitoring should be done after that time has elapsed. This is what is called the CT concept (concentration after a certain contact time). The concept uses the combination of a disinfectant residual concentration (in mg/L) and the effective disinfectant contact time (in minutes), to quantify the capability of a chemical disinfection system to provide effective pathogen inactivation to the required level. The use of this concept involves determining the CT values required at the actual, often variable, operating conditions (flow, temperature, and pH) and ensuring that the employed disinfection process achieves these values at all times [15].

2.3 Water Treatment Plant Performance Studies

As a “treatment train”, conventional drinking water treatment plant compounds of many series stages and units which each unit should be evaluated on its performance, design, process and operation in order to meet its optimum goal [22].

Performance of water treatment plant can be evaluated by several methods, including turbidity, particle counts, heterotrophic plate counts, and microscopic particulate analysis (MPA). Particle counting and turbidity measurements are two of the most valuable water quality parameters used in assessing treatment plant performance. A comparison of source water and filtered water using these procedures has been proposed as a reliable method for determining treatment plant performance [23].

Optimization of conventional drinking water treatment plant is defined “to attain the most efficient or effective use of” a water treatment plant which consist of some principles and the importance to focus on overall plant performance [23].

The following section (section 2.4) is a literature review summary of the CPE phase of the USEPA handbook, Optimizing Water Treatment Plant performance using the Composite Correction Program cited as [2] in this thesis. To avoid repetition of the same citation, unless otherwise cited the body of the statements is adopted, with some modifications from the USEPA handbook.

2.4 Comprehensive performance evaluation (CPE)

A thorough treatment plant evaluation and improvement program is the best way to ensure pathogen-free drinking water. With an emphasis on improved performance at minimal cost, optimization is an economical alternative for compliance with the turbidity requirements.

Since 1988 the Composite Correction Program (CCP) has been developed and demonstrated as a method of optimizing surface water treatment plant performance with respect to protection from microbial pathogens in the United States and Canada [6]. The approach is based on establishing effective use of the available water treatment process barriers against passage of particles to the finished water.

The CCP approach consists of two phases, comprehensive performance evaluation (CPE) and comprehensive technical assistance (CTA). A CPE is a review and analysis of a water treatment plant such as design capability, associated administrative, operation and maintenance practices. The major objective of a CPE phase is to identify factors that may prevent a plant from achieving optimal performance. The **CCP** is a systematic, comprehensive procedure that identifies and corrects the unique combination of factors, in the areas of design, operation, maintenance and administration that limit the performance of a filtration plant. It was developed to improve performance at filtration plants using existing facilities thereby minimizing construction alternatives.

The capable plant model, presented in Figure 2.2, shows conceptually how the CCP considers the various aspects of the operation, design, maintenance, and administration of a filtration plant. A plant is considered capable when it has treatment processes of sufficient size with adequate mechanical equipment to meet current water demand, adequate administrative support including funding and policies, and a maintenance program that keeps key equipment operational. Once these components are in place, proper operations capabilities are required for the plant to achieve its performance goals, whether for regulatory compliance or treatment optimization.

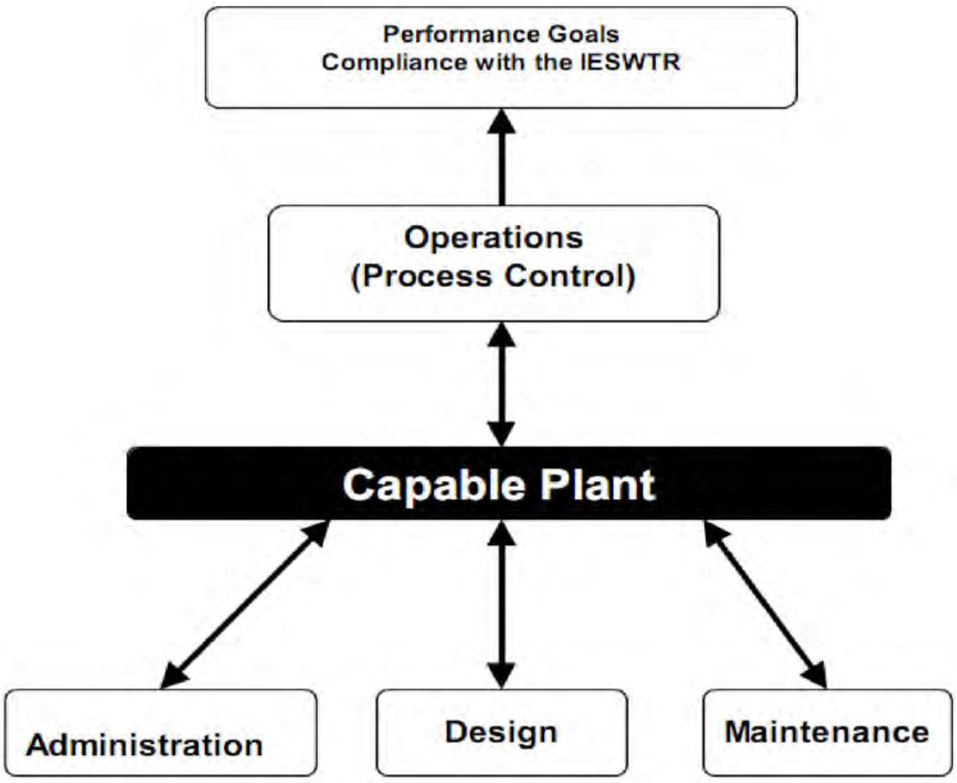

Figure 2.2 Capable plant model

(Source [2])

At the core of the CCP is the assumption that if a filtration plant cannot achieve specific performance, there is a unique combination of interrelated factors with respect to the design, maintenance, administration and/or operations of the filtration plant that are limiting its performance. The purpose of the CPE is to identify these factors and prioritize them with respect to their relative importance in preventing compliance and/or optimized performance. Once the factors are identified and prioritized they can be corrected so that performance can be improved and compliance can be achieved. During a CPE, the historic performance of the plant is assessed with respect to pathogen removal and inactivation.

The design, administration, and maintenance of the plant are completely reviewed to determine if they properly support a capable plant. If they are not supporting a capable plant, the root causes are identified as to how they are contributing to the performance problem. Operational practices are also reviewed to assess if operators have the necessary skills to achieve required performance and compliance when provided with a capable plant.

All of the CPE procedures are designed to focus a plant toward meeting the compliance requirements and performance goals described in Table 2.1.

Table 2.1 CPE Treatment Performance Goals

	IESWTR compliance requirements	CCP optimized performance goals
Individual sedimentation Basin Performance criteria	Not applicable	Settled water turbidity less than 1 NTU 95 percent of the time when raw water turbidity is less than or equal to 10 NTU. Settled water turbidity less than 2 NTU 95 percent of the time when raw water turbidity is less than or equal to 20 NTU
Individual Filter Performance Criteria	Maximum filtered water turbidity of 1 NTU in two consecutive measurements taken 15 minutes apart (conventional and direct filtration systems). Maximum filtered water turbidity 4 hours following backwash of less than 0.5 NTU in two consecutive measurements taken 15 minutes apart (conventional and direct filtration systems).	Filtered water is less than 0.1 NTU 95 percent of the time (excluding 15 minute period following backwashes) based on maximum values recorded during 4-hour increments. Maximum filtered turbidity measurement of 0.5 NTU. Maximum filtered water turbidity following backwash of less than 0.3 NTU. Maximum backwash recovery period of 15 minutes (e.g., return to less than 0.1 NTU).

For the Disinfection unit the USEPA has established the Surface Water Treatment Rule (SWTR) and the Enhanced Surface Water Treatment Rules (ESWTR) as the controlling standards for filtration of surface water and groundwater under the direct influence of surface water. The SWTR and ESWTR require a public water supplier using surface water source to achieve 3-log (99.9%) removal and/or inactivation of Giardia, 4-log (99.99%) removal and/or inactivation of viruses, and 2-log (99%) removal of Cryptosporidium [6].

Credit for log removal is given to filtration processes based on their type, with the remaining required log removal to be achieved by disinfection. Conventional filtration is usually given a 2.5-1og credit for Giardia removal and a 2-log credit for Cryptosporidium removal, provided turbidity limits are met [5].

2.4.1 Components of Comprehensive Performance Evaluation

A CPE consists of the following three components: Performance assessment (evaluates historical plant performance); Major unit process evaluation (for assessing the physical plant capabilities); and identification of Factors limiting performance.

2.4.1.1 Performance assessment

The performance assessment component of the CPE determines the status of a facility relative to achieving compliance requirements and performance goals and verifies the extent of any performance problems at the plant. This information also provides the CPE evaluators with some initial insights on possible causes of performance problems. These insights are then used to focus other activities during the CPE; to assess the design, operation, maintenance and administration of the plant. Historical turbidity data from plant records is used, supplemented by data collected during the CPE.

To achieve optimized turbidity performance, a water treatment plant must demonstrate that it can take a raw water source of variable quality and produce consistent high quality finished water [22]. Further, the performance of each unit process must demonstrate its capability to act as a barrier to the passage of particles with specific turbidity performance goals at all times [2]

The performance assessment determines if major unit treatment processes consistently perform at optimum levels to provide maximum multiple barrier protection. If performance is not optimized, the assessment also provides valuable insights into possible causes of the performance problems and serves as the basis for other CPE findings.

Additional data is collected during the CPE to confirm the historical performance data, further assess the performance of individual treatment processes, and confirm insights on possible causes of poor performance.

Typically additional data is collected through special studies including the following:

- Verification of filtered turbidity results. If the plant is not already individually measuring turbidity from each filter, the filter which the operators believe that has the most problems can be selected and turbidity data can be collected on that filter.
- Filter inspections for media depth and media condition.
- Filter media expansion during backwash.
- Verification of chemical dosages to be sure plant staffs are actually adding the amount of chemicals they are intending to add.
- Additional data on the performance of individual sedimentation basins may also be collected depending on the needs of the CPE.

2.4.1.2 major unit process evaluation

The major unit process evaluation is an assessment of treatment potential, from the perspective of capability of existing treatment processes to achieve optimized performance levels. If the evaluation indicates that the major unit processes are of adequate size, then the opportunity to optimize the performance of existing facilities by addressing operational, maintenance or administrative limitations is available. If, on the other hand, the evaluation shows that major unit processes are too small, utility owners should consider construction of new or additional processes as the initial focus for pursuing optimized performance [2].

The major unit process capability evaluation only considers if the existing treatment processes are of adequate size to treat current peak instantaneous operating flows and to meet the optimized performance levels. The intent is to assess if existing facilities in terms of concrete and steel are adequate [22].

A performance potential graph is used to evaluate major unit processes. As an initial step in the development of the performance potential graph, selecting loading rates for individual unit processes.

Loading rates will serve as the basis to project peak treatment capability for each of the major unit processes. The projected capability ratings are based on achieving optimum performance from the selected unit processes such that each process maintains its integrity as a “barrier” to achieve microbial protection. This allows the total plant to provide a “multiple barrier” to the passage of pathogenic organisms into the distribution system [2].

The projected unit process treatment capability is then compared to the peak instantaneous operating flow rate, a time period considered to be most challenged during these peak loading events, and it is necessary that high quality finished water be produced on a continuous basis.

2.4.1.3 Identification of Performance limiting factors

A significant aspect of any CPE is the identification of factors that limit the existing facility’s performance. This step is critical in defining the future activities that the utility needs to focus on to achieve optimized performance goals [2]. To assist in factor identification, a list of 50 different factors, plus definitions, that could potentially limit water treatment plant performance are provided (see Annex 2). This list and definitions are based on the results of over 70 water treatment plant CPEs. [2]

The CPE emphasis of assessing factors in the broad categories of administration, design, operation, and maintenance helps to ensure the identification of root causes of performance limitations.

To identify design factors field evaluations must be conducted on the various unit processes to assess design limitations.

Identification of operationally-based performance limiting factors offers the greatest potential in improving the performance of an existing utility. The focus of the identification of operational factors is the assessment of the utility’s process control testing, data interpretation, and process adjustment techniques.

Key process controls available to a water treatment plant operator are flow rate; number of basins in service; chemical selection and dosage; and filter backwash frequency, duration and rate. Other controls include flocculation energy input and sedimentation sludge removal.

Process control testing includes those activities necessary to gain information to make decisions regarding available plant controls. Information to assist in evaluating process control testing, data interpretation, and process adjustment efforts

Maintenance performance limiting factors are typically associated with limitations in keeping critical pieces of equipment running to ensure optimum unit process performance or with reliability issues related to a lack of ongoing preventive maintenance activities [2].

Maintenance performance limiting factors are evaluated throughout the CPE by data collection, observations, and interviews concerning reliability and service requirements of pieces of equipment critical to plant performance. If units are out of service routinely or for extended periods of time, maintenance practices may be a significant contributing cause to a performance problem.

3. Materials and methods

3.1 Materials

The following materials were used to conduct field evaluations and laboratory analysis in this study:

Phipps and bird jar stirrer, Photometer (model 7100), Hach portable turbidimeter (model 2100P), test kit, thermometer, pH meter, digital balance, meter tape, wooden rod, glassware, measuring cylinders, stop watch, different chemicals and reagents, and water samples.

3.2 Methods

The method employed in this study was based on the USEPA handbook, Optimizing Water Treatment Plant performance using the Composite Correction Program [2]. The procedures for the evaluation of major unit processes, performance assessment and identification of performance limiting factors on the CPE phase of the CCP were adopted.

Different activities were conducted during the assessment of the treatment plant performance. The sequences of the activities conducted and the procedures followed are presented in the following sections.

3.2.1 Description of the study area

Gambella is one of the nine regions of Ethiopia. Its capital is Gambella town; situated in the southwest part of the country bordering Sudan. The capital is located in the North of Gambella region Figure 3.1.

Figure 3.1 Geographical map of Gambella town.

(Source CSA [24])

The region has warm temperature. Most of the time the temperature is within the range of 27 to 33^0C, but sometimes temperature as high as 45^0C and as low as 10^0C are recorded in March and January. The average annual rainfall of the region is within the range of 900 to 2100 mm. The town has an estimated population of 40,000 [24].

3.2.2 Plant tour

The plant tour was accompanied by the plant supervisor. Different questions were asked in order to understand the actual practice of the treatment plant (see Annex 5). The tour was aimed to become familiar with the actual practices and physical units; to make a preliminary assessment of operational flexibility of the existing processes and chemical feed systems; and to provide a foundation on performance, process control, maintenance and observations that may indicate factors limiting performance.

The plant tour and inspection focused on critical stages of unit operations. These include; raw water intake, chemical pretreatment, rapid mix, flocculation, sedimentation, filtration (features of filter run and backwash), and disinfection operations.

3.2.3 Data collection

Following the plant tour, data collection procedures were initiated. To assess the performance status of the treatment plant; design, operation, and maintenance data were collected. Appropriate forms in accordance with CCP handbook were used.

The design, operation and maintenance data were collected by information collection from plant records; interviews with the plant operators' and key administrative personnel (Annex 5) and field observations of plant's operation (Annex.6), Field evaluations were also conducted to gather additional information regarding actual plant performance.

3.2.4 Comprehensive performance evaluation of the treatment plant

The overall performance evaluation of the treatment plant was divided into two stages.

The first stage was evaluation of major unit processes capability. In this stage; the potential of unit processes, in terms of size, (physical plant capability) to handle peak daily water demand of the population served was assessed. The capability of unit process, to act as a barrier to the passage of particulate matter, was determined.

The second stage was to evaluate the status of the facility operations in achieving performance goals. In this stage the efficiencies of unit processes in terms of turbidity removal and finished water quality requirements were evaluated; and the operator's skills and operating procedures for various unit processes were assessed.

3.2.4.1 Evaluation of major unit processes capability

The major unit processes included flocculation, sedimentation, filtration and disinfection units. The evaluation assumed the following peak conditions

- Peak daily demand of the population served was based on the requirement of 20 to 40 L/person/day, only for drinking and sanitation purposes [2]. For purposes of this calculation 30 l/person/day was assumed and 30% of the produced water wasted as unaccounted and losses.
- The duration of the treatment plant operation is 24 hours.

- The loading rates of unit processes were selected based on the worst case scenario:
- The filtration unit was assumed, when one of the three filters is out of service for cleaning while the others are operating.
- The minimum operating depth and baffling factor (T_{10}/T) of the clear well were assumed 3 meters and 0.1 respectively.

The procedures followed in this evaluation are presented as follows:

i) Determination of peak daily water demand

By considering the above assumptions; the peak daily water demand of the population served was calculated using the following formula:

Peak daily water demand = 1.3 x population served x daily water requirements
(m^3/day) per person (liter) ………… 3.1

ii) Determination of rated capabilities of major unit processes

To determine the rated capabilities of individual unit processes; recommended design loading rates were first selected from tables provided in Annex 3.

After selecting appropriate loading rates for individual unit processes; the capabilities of major unit processes were determined by using the following formulas:

Flocculation Basin capability = $\frac{\text{Basin volume } (m^3)}{\text{Detention time (min)}}$ ……………………………………3.2

Sedimentation Basin capability = Basin Surface Area (m^2) x surface overflow rate ($l/min/m^2$) …...3.3

Filtration basin capability = Filter bed Area (m^2) x filter loading rate ($l/min/m^2$) …….3.4

Disinfection (Clearwater-well) capability = $\frac{\text{Effective volume } (m^3)}{\text{Required contact Time (min).}}$ ………………….3.5

The results of the calculations were used to develop a performance potential graph and the capabilities of unit processes were compared with the peak daily water demand.

3.2.4.2 Plant Operations Evaluation

In order to have an effective operation, the operator has to have a good control over the major unit processes and critical stages of the treatment plant which could significantly affect performance.

During this evaluation, facility information was obtained and the overall operation and treatment processes were reviewed to detect any operational and plant deficiencies. Particular attention was given to critical stages of the treatment process and other details of plant operation.

The performances of plant operations were assessed with respect to their recommended design and standard operation criteria to meet their performance goals. The operator’s skills, maintenance program, and any other plant deficiencies were identified.

Survey questionnaire was designed to collect information about the plant’s operations (see Annex 5). The major focus was on the operator’s ability to accommodate various raw water conditions, how and when the operator decides to adjust pretreatment coagulant dosage for the raw water source with changing characteristics.

The details of procedures are presented in the following sections;

a) Coagulation and rapid mixing

The operators were interviewed and chemical pretreatment was observed, focusing on unusual circumstances, noting the following: chemicals used and dosage rates, dosage adjustment frequency, application points and thoroughness of mixing; and overall effectiveness of chemical application was determined.
Alum was used as chemical coagulant in the treatment plant. To calculate the dosage of alum the following procedures were followed:

1) Alum solution concentration was determined by the following formula

$$\text{Alum solution} = \frac{\text{mass of alum (kg)}}{\text{volume of tank (l)}}$$ ……………………………………………..3.6

2) Alum feed rate was measured **by** collecting samples in a graduated cylinder for a specified time. (See Annex 10-A).for raw data collected during field evaluation.

3) Then the result in ml/min was converted to mg/L of the Alum solution.

$$\text{Alum feed} = \frac{\text{(Alum solution(mg/l) x volume pumped (ml)}}{\text{time pumped (min)}}$$ ……………….…….3.7

4) Alum dose was determined using the following formula

$$\text{Alum dosage (mg/l)} = \frac{\text{Raw water flow (m3/day)}}{\text{Alum feed (mg/l)}}$$ ………………..…………..3.8

In the hydraulic rectangular weir rapid mix unit, the dimensions and the head loss (h_l) were measured by using meter tape and the basic design parameters; velocity gradient (G) and detention time (T) were calculated by the following formulas originally created by [18]

$\mathbf{G = (Qpgh_l/(UV))^{1/2}}$. ……………………………………………………………..3.9

T = V/Q ………………………………………………….……………………….3.10

where; G= velocity gradient (s^{-1})

T = detention time (minutes)

P = density of water = 1000 kg/m^3

g=gravitational acceleration = 9.81 m/s^2

h_l= head loss of the water (m)

Q= volumetric flow rate in m^3/s

V= volume of the unit (m^3)

U = dynamic viscosity of water (poises, kg/m.sec), = 1.01×10^{-3} at temperature 20^{o}C

The results of the discussion with the operators, field observations and evaluations were intended to answer the following operational procedures and methods so as the factors limiting performance of the unit were identified.

Are chemicals being dosed properly with regard to pH, alkalinity, and turbidity? Is dose selection based on frequent jar testing or other testing methods such as streaming current monitoring, zeta potential, or pilot filters?

Do standard operating procedures (SOPs) exist for coagulation controls? Do operators have the ability to respond to varying water quality conditions by adjusting coagulation controls?

Is the chemical feed system operating properly? Are chemical feed systems and solution piping checked regularly? Does the plant stock repair parts for all critical equipment? Is dispersion taking place? Where are coagulants being added?

b) Flocculation

The sizes of flocculators and number of stages present in the flocculation system were measured. The design parameters; velocity gradient (mixing energy) (G) and detention time (T) to form desired floc particles were evaluated by using the same formulas used in the hydraulic rapid mix unit (3.9 and 3.10) above [25].

The Headloss (h_l) was determined by measuring the height differences of initially entering water level and the level of water at the out let chamber.

The characteristics of floc formed were examined by collecting water samples from the raw water, initial rapid mixing point and flocculation basins at several points enroute the flow line of the water and at the exit of the basins. Then the clarity of the water between the flocs were observed and the shape and size of the floc were studied.

c) Sedimentation

The length, width and depths of the sedimentation basin were measured by using mater tape and wooden rod. The key design parameters of the sedimentation basin; surface overloading rate (S_0) and detention time (T) were calculated by using the following formulas [25].

$$S_0 = \frac{Q}{B \times L} \qquad \text{3.11}$$

$$T = \frac{B \times L \times H}{Q} \qquad \text{3.12}$$

Where;

S_0 = surface loading rate or settling velocity (**$m^3/m^2.h$ =** m/h)

H= depth of tank (m), B= width of tank (m), L= length (m)

T= detention time (hr)

Q= flow rate (m^3/hr)

Plant operators were asked and observations were conducted to determine the following conditions:

Is sludge collection and removal adequate? Do basin inlet and outlet conditions prevent the break-up of formed floc particles; is the floc the correct size and density? Is the basin subject to short-circuiting? Are basins subject to algal growth?

d) Filtration

Detailed assessments of the filtration units were conducted to insure the optimization goals of the treatment plant. The following conditions were assessed in the filtration units;

Filter Runs - the filters were observed during the filter run, normal or average filter run time, and what criteria the operator uses to determine when to backwash (Turbidity, head loss and time) was determined. Any start-ups or a change in flow rates on dirty filters that may result in pathogen breakthrough into the clearwell was noted.

Backwash – backwash cycle was conducted and thoroughness of cleaning, flow rate, media expansion, dead spots, media cracking and filter media loss was observed. Use of surface scour or hand raking and complete removal of all mudball was observed.

First run - how the operator determines when a filter can be put back on line was determined. What measurements are taken to make this determination? How does the operator minimize turbidity breakthrough when placing a filter back into service? Are valves ramped open as opposed to fully opened? if the system filters to waste and for how long was determined.

Filter Evaluation - Following a backwash on one of the other filters, the water level from the filter cell was drained to expose the media; and mudball, mud accumulation, mounding, cracking and overall media evenness were inspected.

Filtration rate was evaluated by measuring the drop rate of the water in the filter beds. This was done by using a wooden rod of 4 m length, held vertically against the filter box wall with marked intervals at 10, 20 and 30 cm. The time to drop 10, 20 and 30 cm were recorded. The filtration rate was calculated by using the following formula

$$\text{Filtration rate } (\text{m}^3/\text{min}) = \frac{\text{Filter Surface Area } (\text{m}^2) \text{ X Drop Distance } (\text{m})}{\text{Drop Time (minutes)}} \text{.........3.13}$$

The backwash rate was assessed by evaluating the backwash rise rate of the washing water. This was done in similar manner with that of filtration rate evaluation but the rod was placed below the wash water overflow troughs, and the time for the backwash water to raise certain height was recorded. The backwash rate was calculated by using the following formula [26].

$$\text{Backwash flow rate } (\text{m}^3/\text{min}) = \frac{\text{Filter Surface Area } (\text{m}^2) \text{X Rise Distance } (\text{m})}{\text{Rise Time (minutes)}} \text{......3.14}$$

Bed expansion was evaluated by measuring the distance from the top of the unexpanded media to the top of the filter wall and from the top of the expanded media to the same reference point. Percent bed expansion was determined by dividing the media depth (B) by the total depth of expandable media (media depth (A) less support gravels(C)) and multiplied by 100 [26].

$\text{Percent Bed Expansion} = \frac{A - C}{B} \text{ x } 100$……………………………………………..3.15

e) Disinfection

What chemicals are used for disinfection, the concentration, dosage, and how it is calculated was discussed with the operators. If the disinfection system is operating reliably was confirmed by comparing the results with the recommended standards.
Calcium hypochlorite (bleaching agent) 65 % available chlorine was used in the treatment plant. The feed rate and concentration of chlorine was determined as below:

The percent solution was determined by using the following formula

$\text{Percent solution} = \frac{\text{mass of calcium hypochlorite (kg)}}{\text{volume of water (l)}}$…………………………………..…..3.16

The chemical feed rate was determined by filling volume of chemical in a graduated cylinder in specific time. Then ml/min was converted to mg/l see raw data in Annex 9-B.

Chlorine dose were calculated using the following formula

$\text{Chlorine dose (mg/l)} = \frac{\text{raw water flow (m3/h)}}{\text{feed rate } (\frac{\text{mg}}{\text{min}})}$ ………………………...……..3.17

f) Other Identified Problems

Problems in the plant design or conditions which may have a negative effect on the overall system performance (i.e. less than design flow rates, inadequate mixing for flocculation units, poor baffling configurations, etc.) was discussed with the operators.

3.2.4.3 Assessment of unit processes performance

Assessment of unit processes performance was conducted by reviewing existing recorded data and conducting field evaluations. The quality of finished water and turbidity were used as indicators of performance. Results were compared with WHO drinking water standard requirements [1] and reveal the performance status of the units.

i) performance assessment based on turbidity goals

The performances of sedimentation and filtration units were evaluated by measuring their effluent turbidity. Since a recording on-line turbidimeter was not available in the treatment plant, an instrument that allows individual analysis of grab sample (Hach portable turbidimeter) was used for turbidity measurements.

a) Sedimentation performance

The performance of sedimentation process was assessed in accordance to the WHO turbidity goals of settled water consistently less than 10.0 NTU throughout the year, and no greater than 10.0 NTU, despite raw water turbidity fluctuations.

The performance assessment procedures are presented as follows:

Settled water samples were collected from the effluent of the basins. Samples were collected from the two basins at the same time for 10 days. Then settled water turbidity was measured and the turbidity results versus time were plotted in a graph.

b) Filtration performance

To assess the performance of the filtration units; continuous measurement of effluent turbidity though out the filter run time was not possible, because the treatment plant was operated for 12 hours a day. Therefore short term trend of the filtered water turbidity performance was assessed.

The procedures followed during filter run and backwash performance assessment are presented as follows:

The turbidity performance goals were based on the CCP optimized performance goals [2].

- Combined filter effluent turbidity consistently less than 5.0 NTU throughout the year, despite variability of the raw or settled water turbidity.

- Filtered water turbidity following backwash of less than 0.3 NTU, maximum backwash recovery period of 15 minutes.

Recorded data was reviewed and operators were asked; is the combined filter effluent turbidity consistently less than 5.0 NTU throughout the year, despite variability of the raw or settled water turbidity? The maximum filtered water turbidity following backwash and maximum backwash recovery period?

Samples were collected from the entrance of the Clearwater well, after filtration, hourly for 12 hours. Effluent turbidity was measured and results were plotted on a graph to show the short term effluent turbidity versus time profiling.

A backwash cycle was conducted during this monitoring effort. Backwash turbidity performance was conducted on filter # 1, the filter reported by the operators to have poor performance.

Effluent samples were collected after 50 min of end of the backwash cycle. Samples were collected every minute for one hour and their turbidities were measured. The results of turbidity measurements against time were plotted on a graph to determine the profile of the filter after backwash.

From the results of turbidity measurements; filter effluent turbidity of the first run; recovery time of the filter to return to optimal performance; and turbidity fluctuations during the filter ripening process were determined.

ii) Performance assessment based on treated water quality

The treated water quality assessment was intended to provide a one-time picture of the study plant’s finished water quality.

Water Samples were collected from the sampling taps of the treatment plant. The water from the taps was flashed for 5 minutes before collection in the sampling containers.

The sampling and analysis of finished water was conducted in accordance to the procedures in standard methods for the examination water and wastewater [5]. 14 selected water quality parameters were tested (Table 3.1). pH, turbidity and temperature were tested on the field and the rest parameters were analyzed in the laboratory of GWTP. Results of the analysis were compared with the WHO drinking water quality standards [1].

Table 3.1 Water quality parameters tested

No	Parameters
1	Turbidity
2	Color
3	pH
4	Free residual chlorine
5	Aluminum,
6	T. hardness
7	Nitrate
8	Iron,
9	Manganese
11	Sulphate
12	Fluoride,
13	Temprature
14	Fecal coli form

3.2.4.4 Identification of Performance Limiting Factors

After design, operation and maintenance data has been gathered and the performance of the treatment operations was assessed; an in-depth analysis was conducted to identify the specific factors that limit performance.

The 50-checklist performance limiting factors of USEPA handbook [2] was adopted in this study, to define each factor according to its specific cause of poor plant performance. Lists of defined factors for Assessing Performance Limiting Factors to identifying performance limitations associated with protection against microbial contaminants in the water treatment systems are presented in Annex 2.

In this study, major Performance limiting factors were categorized into design, operational and maintenance. The administrative factors were not identified because the current utility's administrator was new to the position and appropriate information cannot be found.

The performance limiting factors were identified according to the factors definitions presented in Annex 2. They were rated based on their adverse impacts on the performance as per the CEP rating system as A, B or C.

"A" rating for major effect on a long term repetitive basis; "B" rating for moderate effect on routine basis or major effect on a periodic basis and "C" rating for minor effect.

The identification of factors limiting performance categories under which factor names are listed in Table 3.2.

Table 3.2 Performance limiting factors

Performance limiting factors category		
Design	**Operation**	**Maintenance**
1. Unit Process Adequacy	**1. Testing**	**1. Maintenance program**
a. Intake Structure	a. Process Control Testing	a. Preventive
b. Raw Water Pumping	b. Representative Sampling	b. Corrective
c. Flow Measurement	**2. Process Control**	**2. Maintenance resources**
d. Chemical Storage and Feed Facilities	a. Water Treatment understanding	a. Materials and Equipment
e. Rapid Mix	b. Application of Concepts and testing to Process Control	b. Skills or Contract Services
f. Flocculation	**3. Operational Resources**	
g. Sedimentation	a. Training Program	
h. Filtration	b. Technical Guidance	
i. Disinfection	c. Operational guidelines	

3.2.5 Optimization of coagulation via Jar test

Optimization of coagulation was conducted by evaluating the effectiveness of three coagulant chemicals in terms of their turbidity removal potentials and relative costs. Aluminum sulfate (Alum), Ferric Chloride and Ferric Sulfate were selected for analysis. The effectiveness of the coagulants in removing turbidity was compared by establishing jar tests [27]. The optimum doses of coagulants at optimum pH were determined and their relative costs were compared. From the results the effective chemical with highest turbidity removal potential at least cost were selected.

The actual plant's coagulation/ flocculation/ sedimentation process sequences were simulated. Accordingly the following settings (called normal setting) were used for the entire jar tests, which simulate the actual treatment plant conditions: 1 min 100 rpm (fast mixing), 35 min 50 rpm (slow mixing) and Settling time (time to take a sample from 10 cm depth of cylinder) 15 minutes.

The jar tests were carried out according to the standard practice for coagulation-flocculation testing of drinking water [27].

The procedures followed are presented as follows:

Raw water samples were collected from the inlet of the treatment plant. Each jar was filled with 1L of raw water measured with a graduated cylinder. The coagulant dose added to each jar was carefully measured with a burette. The solutions were stirred rapidly at 100 rpm for 1 min, followed by slow stirring at 50 rpm for 35 min. After this period, the paddles were withdrawn and the flocks allowed settling for 15 min. Samples were then withdrawn from 10 cm depth for turbidity and pH analysis.

A single jar test procedure required series of three runs to determine optimum dose at optimum pH of a chemical under test.

Run # 1. Determination of optimum dosage with no pH adjustment

Series of samples of water (with no pH adjustment) was placed on a special multiple stirrer and dosed with range of coagulant and examined for turbidity after the preset mixing and settling time (normal setting). The lowest dose of coagulant that gives satisfactory clarification of the water was noted from which the coagulant demand was determined.

Run # 2 Determination of optimum pH

The second run involve the preparation of samples with the pH adjusted (5.5 – 8.5). The coagulant dose determined previously (run # 1) was added to each beaker, and then follows normal setting as before. Then, the samples were examined for turbidity. The optimum pH was the value which gives the highest turbidity removal.

Run # 3 Determination of optimum dosage at optimum pH

The third run, in which the coagulant dose vary and the pH maintained constant at the optimum determined in run # 2,was used to determine the optimum dose at the optimum pH.

Once the jar tests have been completed, the results were compared by plotting graphs. Raw data for the optimum dosage and pH of the three chemicals is provided in Annex 7.

4. Results and Discussions

This study was conducted, between June and September 2012, to evaluate the performance of GWTP. The treatment plant performance evaluation focused on assessment to establish the potential of the existing processes to achieve desired performance levels.

The treatment plant was evaluated in terms of its physical (unit processes capability), operation, performance characteristics and examination of finished water quality. Jar tests were also conducted to optimize the coagulation. The results of the evaluation are presented in the following sections.

4.1 Plant operations evaluation

4.1.1 Raw water intake

The intake structure is located at the Baro River which is 3 km far from the treatment plant. The location of the intake structure was on the outside bank of the river. This causes excessive collection of debris and clogging of raw water pumps.

There are 3 raw water pumps of design capacity 75m^3/hr each. Two pumps were operated at the same time while the other is left as standby. The pumping capacity at optimum operation practice has the potential to produce 3,600 cubic meters of water per day, in 24 hours operation. While the required current peak operating flow of the plant, to serve 40,000 populations, was 2000 m^3/day.

During the field visit; the raw water flow from the two operating pumps averaged 94m^3/hr. The average current capacity of each pump was 47m^3/h. the pumps were operated less than their design capacities 75m^3/h; beside this, the pumps were operated for only 12 hours a day. The current capacities of the pumps and duration of operation limited current peak operating flow rate (maximum daily water production) of the treatment plant to 1,128m^3/day. This showed that the current peak operating flow rate did not satisfy the required peak daily demand of the population served.

From the interview of the operators; it was noted that the decreased pumping capacity of the raw water pumps was directly linked to the poor maintenance program and lack of spare parts in the plant. In addition to this the location of the intake structure contributed to the rapid failure of the pumps. Therefore immediate actions on planned maintenance program and purchase of required spare parts to the raw water pumps should be taken. Detailed studies on the location of the intake structure should be conducted in the long term.

A significant finding was that increasing the current raw water flow rates and operating time could offer a clear alternative to handle the current peak water demand.

4.1.2 Coagulation/ rapid mix

The plant has hydraulic rapid mix system in which raw water and coagulant chemicals are mixed. From equations 3.9 and 3.10, the velocity gradient (G) and detention time (T) were found to be 710 s^{-1} and 15 seconds respectively. Though they are within the recommended design ranges, they fall near the minimum limits. This was also caused from the decreased raw water flow than design capacity of the raw water pumps.

The Alum concentration was 2.5%. The feed rate of alum varies depending on the raw water characteristics, but during the evaluation period the calculated feed rate from equation 3.7 was found to be 650 ml/min and from equation 3.8, the Alum dosage was 10.37mg/l, this was less than the recommended optimum dose of 45 mg/l. This indicated there was under dosing of Alum in the unit, that could affect the performance of the coagulant and hence the downstream processes. The major cause for the under dosing was lack of mathematical skill of the operators to calculate chemical dosages.

The pH of the coagulated water was found to be 6.5. pH between 6 to 7 is effective range for Alum flocculation [10]; the raw water pH was within the suggested range hence no need of pH adjustment.

From the interview of the operators, visual observation of floc formed was the criterion for checking alum dosage and making appropriate adjustment to chemical feed. The pH of coagulated water was not checked routinely. Plant operators did not conduct jar tests to verify the required alum dose.

Operators did not collect water quality data and did not apply the results of these tests to determine how well the plant was performing. It was clear in many instances that operators did not understand the basic influences of pH and alkalinity levels on the performance of a coagulant. This indicated the operators' process control testing skills were limiting the performance.

Jar test must be performed as a routine basis to achieve optimum coagulant feed. Proper pH and adequate alkalinity control at the chemical pretreatment stage is essential to optimize the alum dosage rate.

4.1.3 Flocculation

There are two Alabama-type flocculators with fifteen compartments each. The total volume was 54 m^3. By using equation 3.10, detention time of the unit was found to be 35 minutes. This time was longer than the maximum recommended design range of 20-30 minutes [10]. The longer flocculation time results the flocs to settle and form scum on the walls and bottoms of the flocculators. The mixing energy (equation 3.9) was 43.8 s^{-1}. It was less than the recommended design range 45-90 s^{-1} [10]. The headloss of the entire unit was 0.8m, which was higher than 0.35-0.5m design range.

None of the design parameters were within the recommended design ranges. This indicated there was insufficient mixing and dispersion of coagulant chemicals with the raw water that could affect the characteristics of the expected floc to be formed. These conditions might be caused from the decreased flow rates of the raw water pumps.

The results of "Alabama type" hydraulic flocculators performance assessment is summarized in Table 4.1.

Table 4.1 Comparisons of actual plant flocculator and design criteria

Flocculation	Description	Actual	Design criteria [10]
1. description	Type	Hydraulic	
	Number trains/stages per train	2/15	
	Control	Not controlled	
2. dimensions	Length per stage:	0.8 m	0.75-1.50 m
	Width per stage:	0.7 m	0.50-1.25 m
	Depth per stage:	3 m	2.50-3.50 m
	Total volume:	54 m^3	
3. major unit process capability	Selected Process Parameter(s):		
	Detention time (T)	35 (min)	20-30 (min)
	Assigned process capacity		**5,184 m^3/day**
4. Other Design information	Flocculator mixing energy (G)	43.8s^{-1}	45-90 s^{-1}
	Head loss for entire unit	0.80 (m)	0.35 – 0.50 (m)

The flocculators lack flow control to vary the velocity of flow at the plant operation. This indicated lack of process flexibility.

From the results of floc characteristics examination; the size of floc formed at the entrance of flocculation basin was small and not well dispersed. The size of floc as it moves through the flocculation basin did not gradually increased and no floc breakage was observed at the outlets. Based on [10] the characteristics of the floc formed in the flocculator reflected the poor performance of the unit.

Such types of flocs may overload and impact the performances of the downstream units, sedimentation and filtration. This indicated inappropriate chemical dosage, lower raw water flow rates and lack of appropriate operation in the chemical pretreatment.

To optimize the coagulation and flocculation processes; optimum dosage of Alum should be determined routinely based on the varying raw water characteristics and the operators should have to be trained with the standard operation practice.

4.1.4 Sedimentation

The two rectangular sedimentation basins have total surface area of 225 m^2. The detention time (equation 3.12) was 7.2 hours. This detention time was much higher than the design value 3 hours [25]. This indicated the flocculated water spent more time than the required design and the plant was operated at around half of the design flow to the sedimentation basins. Surface loading rate (equation 3.11) was 0.42 m/h. this loading rate was less than 0.83 – 2.5 m/h design range [25]. This was resulted from the raw water pumps operated at less than their design capacities.

The field evaluation results of the actual sedimentation basin and the design criteria are summarized in Table 4.2

Table 4.2 Comparisons of actual plant sedimentation performance and design criteria

Sedimentation:		Actual	Design criteria [25]
1. description	Type	rectangular	-
	Number trains	2	-
	sludge removal	manual	Mechanical, manual
2. dimensions	Length per stage:	25 m	-
	Width per stage:	4.5 m	-
	Depth per stage:	3.1 m	3-3.5
	Total surface area	225 m^2	
3. major unit process evaluation	Selected Process Parameter(s):		
	Surface loading rate (S_0)	0.42 m/h	0.83 – 2.5 m/h
	Assigned process capacity		4,542 m^3/day
4. Other Design Information	Detention time (T)	7.2 h	3 h

Operators reported routine removal of sludge from sedimentation basins was not being practiced. The sludge was being removed once in three months time. The sludge deposit in the settling basin was almost half of the total depth. This indicated that too much floc was being accumulated at the bottom of the basin for longer time and become septic causing the sludge to bulk. This could result short circuiting that limits sedimentation performance.

Therefore, proper adjustment of hydraulic loading and scheduling of the sludge removal cycle is essential.

4.1.5 Filtration

Single sand media was used in the filtration unit. The filtration rate (equation 3.13) was averaged 1.26 m/h. the filters were operated at less than the design loading rate 5-15 m/h range [26]. The lower filter loading rate decreased the potential of filter performance. This means the filters could be operated at higher loading rates and they can produce more filtered water than the present quantity.

Operators replied during the interview that the effluent turbidity of filter units was not monitored routinely. Proper influent flow and effluent turbidity monitoring are essential to maintain the desired filter performance.

The sand depth across filters # 1, 2 and 3, was 0.27, 0.30, 0.45 m respectively. Similarly, the support gravel depth was 0.2, 0.25 and 0.2m respectively. The recommended filter sand media depth is 0.5 – 1 m and support gravel 0.45-0.6 m deep [26]. Less than the design depths of sand and supporting gravel significantly affect the performance of filtration units.

During the filter inspection; media cracking was observed on the surface of filter # 1. The operators claimed the sand media was changed before 2 years. During this, long, period the effectiveness of the sand exhaustively depreciated and resulted in shorter filter run times, frequent washing of the filters and higher effluent turbidity. The sand should be replaced at least once in three months.

The backwash rate (equation 3.14) was 0.34 m/min, operated at less than the recommended design value of 0.8 m/min [26]. this showed there was inadequate backwash rate that could be resulted from the limited amount of head pressure in the backwash storage tank.

The sequence and duration of the backwash operation was Air scour for 2 minutes, flow ramping for 5 minutes and delayed start after 50 minutes. The duration of the backwash operation was only 7 minutes. Filters with optimized operations and control are recommended to be washed for at least 10 minutes [26]. But for the current poor backwash practice and lack of turbidity monitoring system along with the sand media not changed for about 2 years; the total backwash duration in the plant should be increased even to higher than 10 minutes.

Operators replied they could loss the sand media if they backwashed the filters for longer times. This indicated lack of understanding of the plant operations

During the backwash the up-flow water was not clear indicating inappropriate backwash rate and time. According to the equation 3.15, the sand media was expanded to only 12% while the design range is 20 – 25% [26]. This was also the result of poor backwash practice.

From the operators interview the filters was backwashed at an average of 7 days, which was higher than the recommended (three days). Filter washing was not generally practiced on a routine basis, regardless of headloss and effluent turbidity.

Backwash water was supplied from an elevated water tank with a variable water level. No sampling taps were provided in the filtered lines, to check the quality of filtered water.

Filter-to-waste operation was carried out after the end of backwashing in the plant.
The results of all field evaluation on the filtration unit were summarized in table 4.3.

Table 4.3 Comparisons of actual plant filter performance and design criteria

Filtration		Actual	Design [26]
1. description	Type	Rapid sand ,mono media	Rapid sand , mono media
	Number of filters	3	--
	Filter control	Head loss	turbidity
	surface wash type	Hand raking	--
2. dimensions	Length per stage:	5.5 m	--
	Width per stage:	4.5 m	--
	Total surface area	74 m^2	--
3. Media conditions	sand depth	0.27 - 0.45 m	0.5 – 1 m
	Support gravel	0.2 - 0.25 m	0.45 – 0.6 m
	Filtration rate	1.26 m/h	5-15 m/h
4. Backwash	Backwash initiation :	head loss, time on service	turbidity
	Backwash Sequence	1) air scour, 2) flow ramping, 3) delayed start	1) air scour, 2) flow ramping, 3) filter-to-waste and/or delayed start
	Duration for each operation (min)	1) 3 min, 2) 7 min, 3) 55 min	1) 3-4 min 2) 10 min
	Backwash rate	0.34 m/min	0.8 m/min
	Bed expansion	12%	20 – 25%
	Backwash termination	Visual observation of up-flow water, time	Effluent turbidity
5. major unit process capability	Selected Process Parameter(s):		
	Surface loading rate	1.71 m/h	4.88 m/h
	Assigned process capacity		**5,443 m^3/day**

One of the most significant finding during the evaluation was water cascading directly onto filter media without keeping the filter flooded. This leads to short-circuiting and results in high turbidity. Backwash ineffectiveness was another major deficiency noted. Improper chemical pretreatment, along with improper backwash techniques, also leads to poor filter performance.

Mud accumulation, media cracking, and uneven bed expansion were observed. backwash rates were too low, backwash time was too short, and under-drains were unserviceable. Operators often enlisted additional assistance to improve backwashes, such as using longer-than-normal backwash time to compensate for low wash rates.

4.1.6 Disinfection

The filtered water was disinfected with Calcium Hypochlorite prior to store in the clear well. Chlorine concentration (equation 3.16) was 0.25% which is less than the design value of 0.5% [23]. The Chlorine feed rate was not calibrated, operators were observed to flash the chlorine valve without considering the required residual chlorine in the clear well.

During the evaluation period the chlorine dosage (equation 3.17) was 1.2mg/l. this result was less than the design value of 1.5mg/L, which may lead to low free chlorine residual in the treated water.

The unbaffled Clearwater well was operated on a fill and draw basis, this operation do not provided ideal condition for disinfection, and lead to short-circuiting [23].
The comparisons of the actual treatment plant and design criteria are presented in the table 4.4 below

Table 4.4 Comparison of actual plant disinfection performance and design criteria

disinfection		Actual	Design [23]
1. description	contact Type	Clearwell	--
	T_{10}/T factor	0.1	1
2. dimensions	diameter	5 m	--
	Minimum operating depth:	3 m	--
	Total volume:	300 m^3	--
	Volume adjusted for T_{10}/T:	30 m^3	--
3. major unit process evaluation	Disinfectant	Calcium Hypochlorite	--
	Max. disinfectant residual (mg/L)	1.2	1.5
	Maximum pH	7	8
	Minimum temperature (^{o}C)	15	5
	Required Giardia inactivation	0.5	0.5
	Assigned process capacity		**2,977 m^3/day**

4.1.7 Intermittent plant operation

Another significant finding was the treatment plant was operated intermittently for 12 hours daily. Commonly the problem was arising from continuous interruption of electricity. This type of operation greatly impacted the treatment efficiency, particularly the flocculation and filtration process.

Floc settled in the flocculation tank during the “off’ period due to lack of flow. The settled floc causes a sudden loading of heavy solids in the treatment train when plant is next turned on.

Sedimentation is also affected by intermittent plant operation. Flow short-circuiting also occurs due to the difference in the temperature between water left in the sedimentation basin and incoming raw water [2]. The density flow caused by this difference may last several hours. This condition reduces the settling efficiency.

The plant was observed to consistently starting up with the filters already dirty. Filters should be backwashed before start-up and the chemical feeders must be adjusted to give the appropriate feed rate depending on raw water quality.

Intermittent plant operation induces many adverse effects to the overall treatment process. Continuous operation will mitigate most of these problems.

4.2 Major Unit Processes capability

Major unit processes (flocculation, sedimentation filtration and disinfection) were evaluated based on their capability, if basin size is adequate, to handle current peak daily water demand of the population served.

The required water production from the treatment plant (required peak operating flow) to handle the current peak daily demand of 40,000 residents of Gambella town from (equation 3.1) was found to be 2000 m^3/day.

The capabilities of major unit processes (from equations 3.2, 3.3, 3.4 and 3.5) found flocculation, sedimentation, filtration and disinfection units to be 5184, 4542, 5443 and 2977 m^3/day respectively. Details of calculations are presented in Annex 4.

Comparison of major unit processes rated capabilities and the required peak daily demand of the population served is presented in the performance potential graph (Figure 4.1)

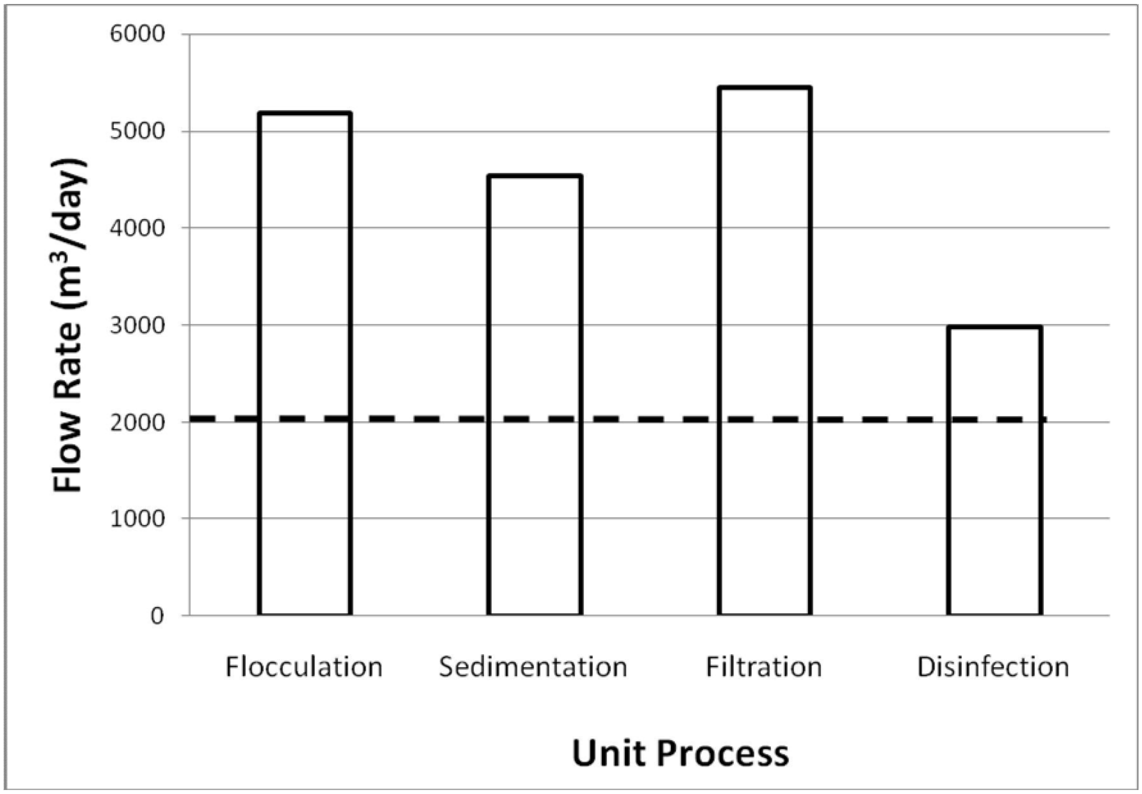

Figure 4.1 performance potential graph of GWTP

As shown in figure 4.1, the capabilities of all major unit processes exceeded the current peak daily demand of 2000m^3/day (broken line in figure 4.1). From this graph the major units were judged to have the capability to satisfactorily treat water at current peak demand. It also indicated current performance problems are not caused by limitations in size or capability of the existing major unit processes; justifying attempts to optimize existing performance by mitigating other factors such as operational and maintenance.

4.3 Unit processes turbidity performance

The results of the settled water turbidity measurements from the sedimentation basins are presented in figure 4.2. Raw data is presented in Annex 9-A.

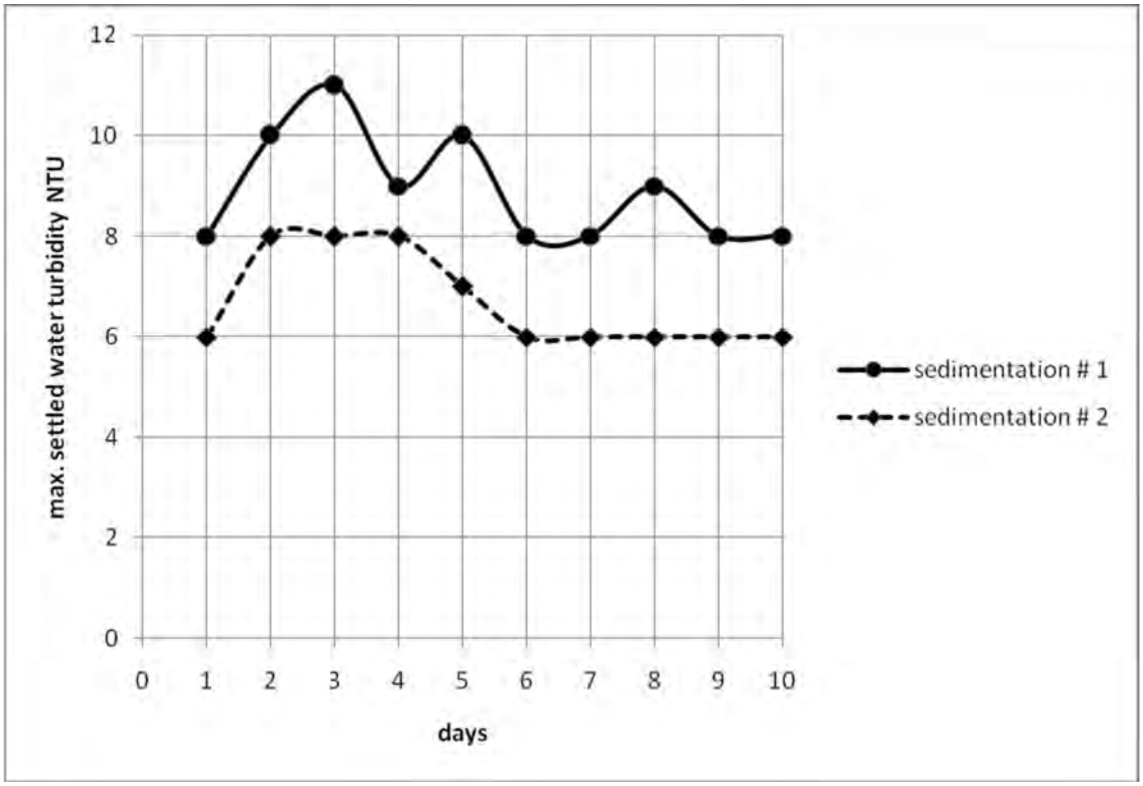

Figure 4.2 Sedimentation turbidity performance

From the figure 4.2 it can be observed that the measured turbidities of both sedimentation basins were not equal. This was resulted from unequal flow of water to each sedimentation basins. From the analysis of turbidity results it was found that; the settled water turbidity of both basins were below the maximum recommended WHO value of 10 NTU. On day 3 the turbidity of basin #1 reached 11 NTU because of the rain fall event at night time before the sampling day.

From the resulted settled water turbidity it could not be concluded that the sedimentation performance complied with the WHO requirements, because, the raw water flow was operated at less than the design capacity. Lack of control in the preceding units, application of less than optimum dosage of alum and the longer frequency removal of sludge from the sedimentation basin were supporting reasons for not compliance with WHO requirement.

From the analysis of filtered water turbidity results, (see Annex 9-B for the raw data) it was found that; the filtered water turbidity consistently exceeded the WHO requirement of 5 NTU or less. (Figure 4.3) This was resulted from the poor operations of backwash, long filter runs, sand media not replaced for the past two years.

This condition could lead to the passage of pathogenic diseases causing organisms which may pose a health risk to the consumers. The sand media should be replaced timely and all the pretreatment operations need to be optimized.

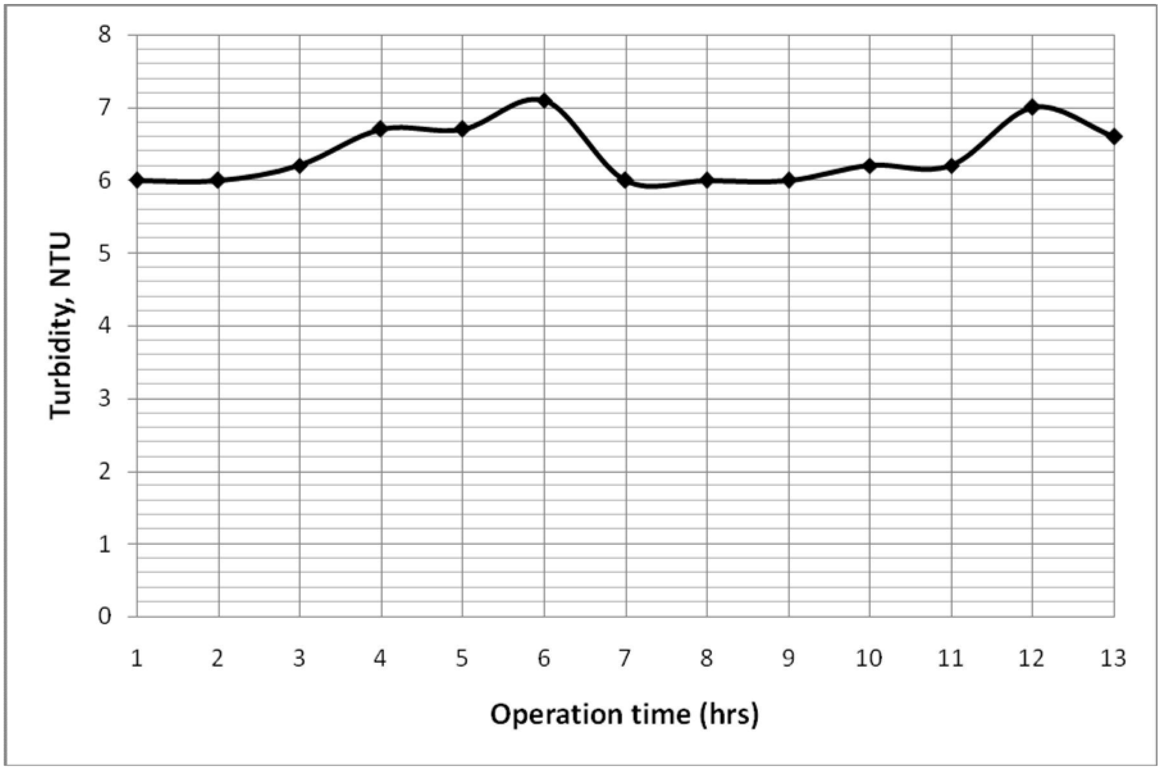

Figure 4.3 Short term trend chart of settled and filtered water turbidities

The results of turbidity measurements of the filter after backwash of filter # 1 is presented in figure 4.4. Raw data is presented in Annex 8.

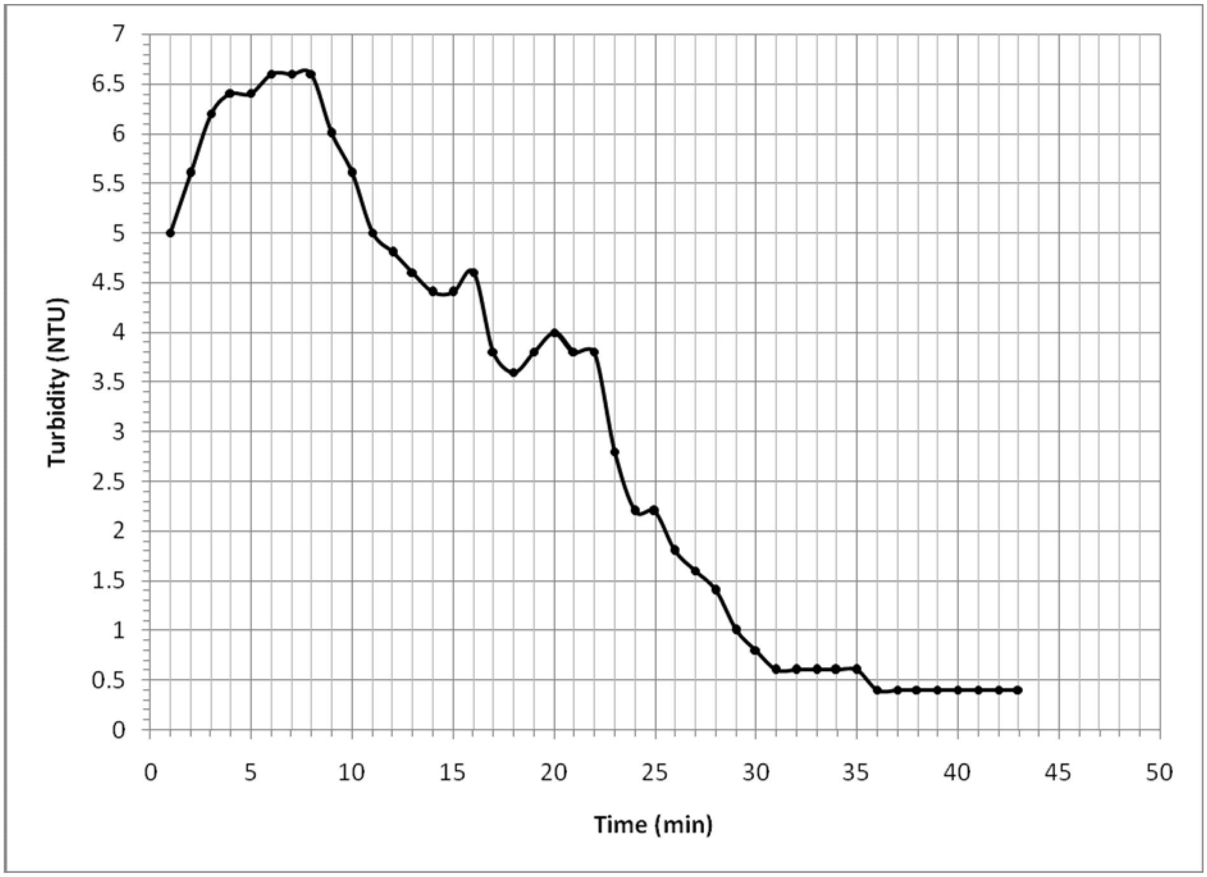

Figure 4.4 Filter effluent turbidity profile after backwash

From the graph of the filter turbidity profile after backwash, turbidity spike of 6.5NTU was found. The time to return the filter back to service was 45 minutes. the recommended maximum filtered water turbidity following backwash is less than 0.3 NTU and maximum backwash recovery period of 15 minutes (return to less than 0.1 NTU) [2]. The filter effluent turbidity after backwash did not meet the optimized performance goals. The operators claimed the sand media were replaced 2 years ago and this could result in a prolonged recovery time and relatively higher effluent turbidity.

4.4 Finished water quality

Results of finished water quality analyses and comparison of results with the WHO standard are summarized in table 4.5.

Table 4.5 Comparison of treated water quality analysis with WHO standard

No	Parameters	Unit	Treated water	WHO
1	Turbidity	NTU	6.0	<5.0
2	Color	Color units	<5.0	<15
3	pH		6.8	6.5 – 8.5
4	Free residual chlorine	mg/l	0.2	0.2 – 0.5
5	Aluminum,	mg/l	0.52	0.5
6	T. hardness	mg/l	68	-
7	Nitrate	mg/l	1.62	10
8	Iron	mg/l	Nil	0.3
9	Manganese	mg/l	0.018	0.05
11	Sulphate	mg/l	200	400
12	Fluoride	mg/l	0.71	1.5
13	Temprature	^{0}C	15	-
14	Fecal coli form	number	Nil	Nil

The treated water quality complies with WHO drinking water standards. This result does not indicate the statistical profile of the water quality parameters. It only shows a onetime status of the treatment plant on the selected water quality parameters during the evaluation period. However, proper scheduling of the disinfection and routine monitoring of quality parameters specially (residual chlorine, pH, alkalinity and temperature) is essential to improve reliability of treatment.

4.5 Optimization of coagulation

Turbidity was analyzed with respect to final turbidity results (see Figure 4.5) using raw water samples as a control and the optimum pH was analyzed with respect to the optimum doses of individual chemicals. (Figure 4.6)

4.5.1 Ferric Chloride

Compared to all coagulants studied, $FeCl_3$ required the least amount of dosage to achieve the greatest amount of turbidity removal. The highest turbidity removal was achieved at 20 mg/L. Doses greater than the optimum did not necessarily improve turbidity removal. At and beyond the optimum dose of 20 mg/l, final turbidity was reduced to levels of 0.5 NTU. Even at half of this optimum dose, the final reading of turbidity remains extremely low with value of 1.4 NTU. Analysis showed turbidity removal efficiencies in $FeCl_3$ tapering off after 10 mg/l.

Comparable amounts of dosage show greater pH drops in $FeCl_3$ treated waters than in alum treated waters. This was especially true in the high range of dosages greater than 25 mg/l. As shown in Figure 4-6, at dosage of 40 mg/l of $FeCl_3$ a 1.1 drop in pH was induced. $FeCl_3$ treated samples showed the lowest final pH levels that are significantly lower than that observed in Ferric Sulphate and alum treated samples. The optimum pH for the optimum dose of Ferric Chloride 20 mg/l was found to be pH 6.7

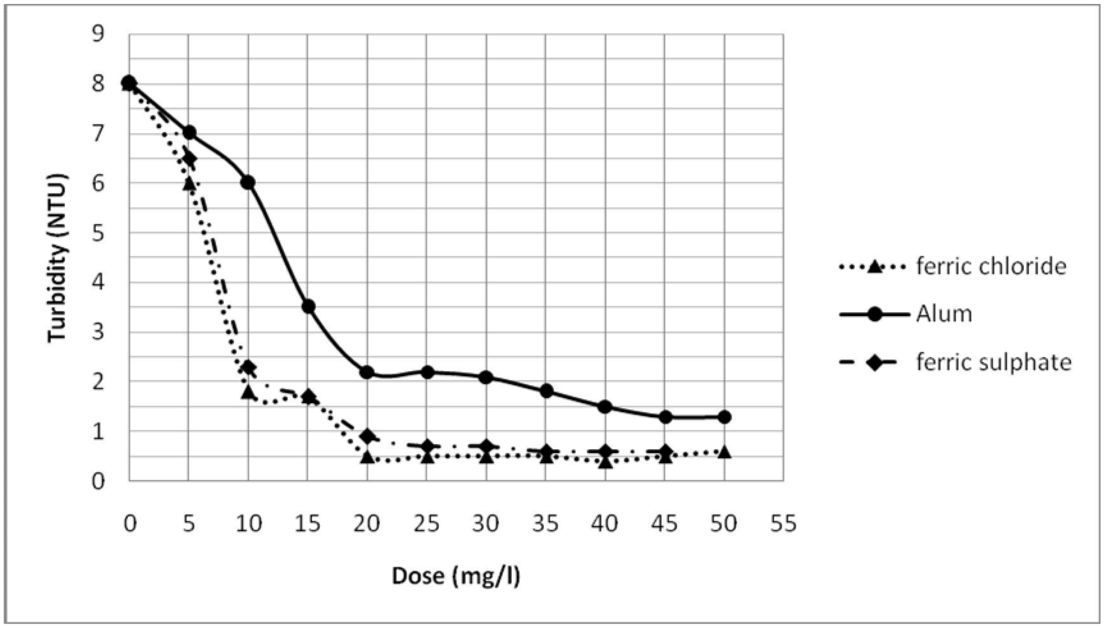

Figure 4.5 Final turbidity versus Dose

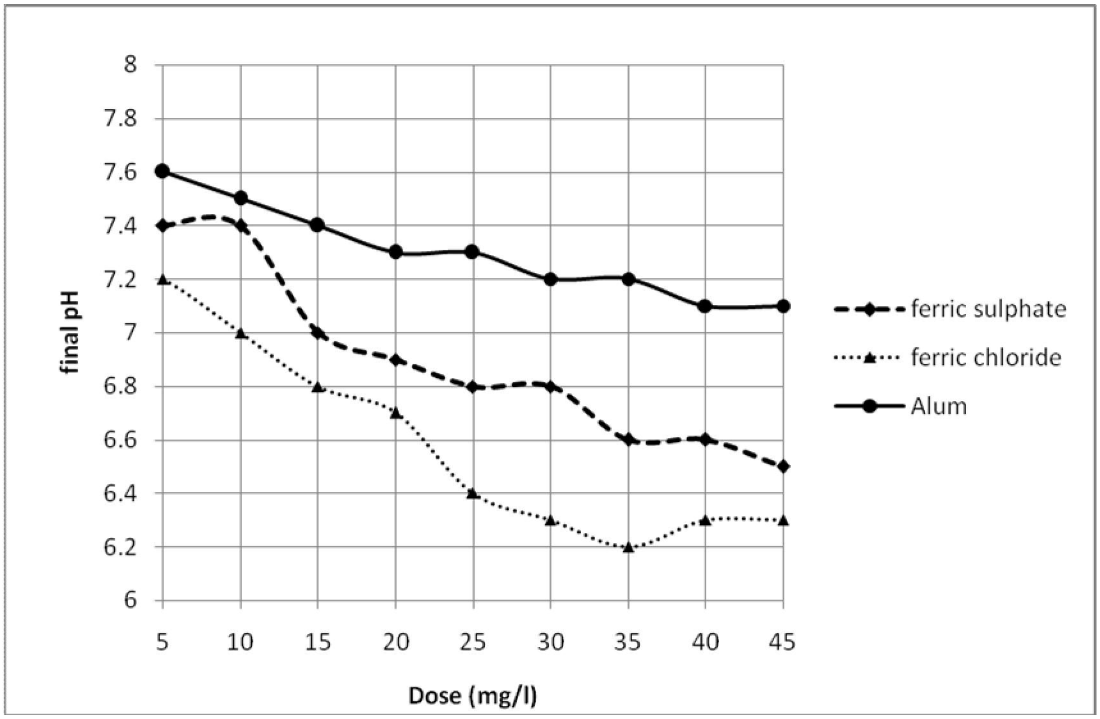

Figure 4.6 Final pH versus Dose

4.5.2 Ferric Sulphate

Ferric Sulphate showed nearly similar performance as that of Ferric Chloride. It performed the higher turbidity removal next to Ferric Chloride. Turbidity removal analysis showed effective removal results of Ferric Sulphate for dosages greater than 25 mg/l, final turbidity was reduced to levels of 0.75 NTU.

The next lowest pH levels were found in samples treated with Ferric Sulphate. At higher doses the pH drops consistently. While in the lower dose ranges there showed an increase in pH. The optimum dose of Ferric Sulphate 25 mg/l was reached at pH 6.8.

4.5.3 Aluminum Sulphate

Alum showed the least turbidity removal efficiency when compared to the Ferric salts. The minimum residual turbidity was around 1.5 NTU at doses greater than 45 mg/l. Optimum final turbidity values are found in the coagulant dose range of 40-50 mg/l and yield turbidities less than or equal to 1.5 NTU. The optimum dose with highest turbidity reduction was found to be 45 mg/l.

The least reduction in pH was also observed in the Alum treated water. The optimum dose of Alum 45 mg/l was observed at the optimum level of pH 7.1.

4.5.4 comparison of costs at optimum doses

Costs of the selected optimum dosage of coagulants to treat one cubic meter of raw water are summarized in Table 4.6.

Table 4.6 Costs of coagulants and optimum doses

Coagulant	Optimum dose (g/m^3)	Price (birr/gram)
Alum	45	0.3638
Ferric Chloride	20	0.44
Ferric Sulphate	25	0.60

The cost of treating one cubic meter of water with Alum showed the least cost compared to Ferric Chloride and Ferric Sulphate. Despite the efficiencies at lower doses of the Ferric salts, their prices were not economically feasible because they are imported from abroad.

Therefore it is recommended to use Alum, recently used in the plant, which is produced locally at “Awash Melkasa” with current price 8.8084 birr/kg. But it is still very important that the optimum dosage of Alum should be kept in the treatment plant.

4.6 Performance limiting factors

Performance limiting factors were identified for the plant by utilizing the factor lists based on the USEPA handbook [2]. Some modifications were made to fit with the actual conditions of the treatment plant under study. The following performance limiting factors were identified and were given ratings of “A”, “B” or “C”.

4.6.1 Design factors

Intake structure (A)

- The intake structure, located on the outside bank of the river, causes excessive collection of debris resulted in clogging of the raw water pumps and reduced their efficiencies. This was found to be a major performance limiting factor and rated A.

Flocculation (B)

- The longer detention time and inadequate mixing energy result did not create adequate floc and increased burden in the filtration unit.

Flow Proportioning (B)

- Influent flow to the plant was hydraulically split to the flocculation trains, and uneven flow distribution causes overloading of one sedimentation train over the other.

Filtration (B)

- The filter underdrain in filter # 1 was damaged resulting in high turbidity spikes up to 6.5 NTU.
- Backwash rate and time was not sufficient to provide proper bed expansion. It was limited to only 12% during the backwash performance evaluation

4.6.2 Operational factors

Process Control Testing (B)

- Plant staffs do not measure and record raw water pH, alkalinity, and turbidity on a routine basis despite changing raw water characteristics; consequently, the impact of raw water quality on plant performance were not assessed.
- Sedimentation and filtration basin effluent turbidity were not measured routinely.

Water Treatment Understanding (A)

- Plant staffs do not have sufficient understanding of water treatment processes to make proper equipment or process adjustments.
- Specific performance objectives for each major unit process (barrier) have not been established.
- Intermittent plant operations was observed that can significantly affect the overall performance of the treatment plant

Application of Concepts and Testing to Process Control (A)

- Jar tests for coagulation control were not practiced.
- The operators did not understand how to prepare a jar test stock solution and how to administer various chemical doses to the jars.
- There was no routine removal of sludge from sedimentation basins.
- Operations staff cannot determine the chemical feeder setting for a selected dose role.
- Operations staffs do not adjust chemical feed rates for varying raw water quality conditions.
- Filters are backwashed based on time in service or headloss rather than on optimized performance goal for turbidity or particle removal.
- Plant filters are placed back in service following backwash without consideration for effluent turbidity levels.
- Sedimentation basin performance is controlled by visual observation rather than process control testing.
- Filters are started dirty (i.e., without back-washing). Mainly the results of intermittent plant operations.

- Ø Operations staffs stop the backwash when the filter is still dirty to “conserve” sand media loss and water.

Training Program (B)

- Ø A training program does not exist for operators at a plant, resulting in inconsistent operator capabilities.

4.6.3 Maintenance factors

Preventive Maintenance Program (B)

- Ø Preventive maintenance was not performed on all equipments unless they stop working. Especially the raw water pumps were not maintained as recommended by the manufacturer, resulting in premature pump failures and degraded plant performance.

Corrective maintenance program (B)

- Ø Inadequate critical spare parts are available at the plant, resulting in equipment downtime.

4.7 Performance Improvement options

In the plant evaluated, it was found that operational factors limited performance highest. This finding, coupled with the fact that the plant had adequate capacity to meet desired turbidity requirements, indicates that improving process control could significantly improve performance. Poor process control was evident in the following practices;

Inadequate pretreatment practices, lack of coagulant control strategy and lack of process control and monitoring in various stages of the treatment processes. Alkalinity and pH control was also a major pretreatment problem.

Jar test must be performed as a routine basis to achieve optimum coagulant feed. Proper pH and adequate alkalinity control at the pretreatment stage is essential to optimize the alum dosage rate.

Operators did not pay attention to filter effluent monitoring and to flow rate changes to operating filters, and they started operation when the filters were dirty. As results, filter performance deteriorated.

Operators' certification and training program should be planned to implement necessary process control and standard operation procedures at the plant.

Lack of important spare parts, specially raw water pumps, chlorinators and alum feed pumps should be given priority to continuously produce water of acceptable quality

Post disinfection was the only barrier designed to inactivate major disease causing microorganisms by the poorly operated water treatment plant evaluated. To improve the reliability of treatment, it is essential to optimize the plant's multiple barriers (e.g., pre-disinfection). Improved process control must be an integral part of any changes that affect current disinfection practices (e.g., lack of standby unit, poor disinfectant conditioning).

5. Conclusions and Recommendations

5.1 Conclusions

The goal of the study was to evaluate the plant performance and identify factors that limit performance. The field study efforts to assess the design, operation and maintenance factors limiting the plant's performance and to propose improvement options available to optimize the treatment plant's performance. Based on the study findings the following conclusions were drawn:

The capabilities of the major unit processes (Flocculation, sedimentation, filtration and disinfection) were evaluated in terms of their potential to handle current peak demand of the population served. Evaluation results reveal all major unit processes had adequate capability to meet the current peak daily water demand. It also indicated current performance problems are not caused by limitations in size or capability of the existing major unit processes; justifying attempts to optimize existing performance by mitigating other factors such as operational and maintenance.

The performance assessment of the treatment plant based on turbidity goals indicated the maximum settled water turbidity was less than 10 NTU, and it complies with WHO goals of less than 10 NTU. Filter turbidity evaluation indicated the finished water turbidity was consistently greater than 6 NTU. During the backwash performance evaluation turbidity spikes of 6.5 NTU and 45 minutes of filter recovery time was observed. Generally the results of the turbidity performance assessment indicated the WHO standard and CCP optimized performance goals were not being achieved. This showed there is high risk to the passages of disease causing organisms. This indicated the treatment plant was not operated at its optimum potential and factors that limit its performance should be identified and corrected to optimize the performance.

The finished water quality parameters tested comply with the WHO standard guidelines for drinking water. Although the samples collected do not represent a statistical profile of the plant’s performance, they indicated how well the plant was operating during the evaluation period.

The result of jar tests, conducted at three selected chemicals, to optimize the coagulation process, showed highest turbidity removal was observed at optimum dose of 20 mg/l Ferric Chloride, 25 mg/l Ferric Sulphate and 45 mg/l Alum at optimum pH levels of 6.7, 6.8 and 7.1 respectively. But comparison of their respective prices at optimum dosages reveal Aluminum Sulphate (Alum) was the appropriate chemical, at optimum dose of 45 mg/l having the least cost.

Form the identified factors limiting plant’s performance; no single factor was responsible for poor plant performance, although in general the study found that all major factors influence the plant’s ability to work properly. The location of intake structure from the design factors, Water Treatment Understanding and application of Concepts and Testing to Process Control from the operational factors significantly affected the treatment performance.

Some of the primary operational problems involved improper chemical dose adjustments, insufficient process monitoring, and improper filter operation and backwash techniques. In many cases, operators did not understand basic concepts of chemical pretreatment and filter operation, which may reflect insufficient basic training.

Intermittent plant operation induces many adverse effects to the overall treatment process. Continuous operation will mitigate most of these problems. The continuous electricity interruption can be overcome by allocating fuel budget for the standby generator in the treatment plant.

5.2 Recommendations

According to this study, several items can be improved with immediate action as follows:

The efficiency of the treatment process can be increased by process control and monitoring in various stages of the treatment process. Increasing the operating time can be a practical alternative for the intermittent operation and the required current peak daily demand for the population served can be achieved.

The study characterized the major operational problems; lack of filtered water monitoring, improper chemical dose, inadequate operation and maintenance knowledge, starting up dirty filter, inadequate process monitoring and using headloss as only criterion for initiating a filter backwash. Therefore, establishing a training program to upgrade the comprehensive performance of personnel in operation and maintenance is essential.

This treatment plant performance evaluation approach could lead to successful improvements and decreased risk from waterborne disease by identifying weaknesses and optimizing treatment. Therefore using this document as a basis the authority should conduct performance evaluation of the treatment plant annually.

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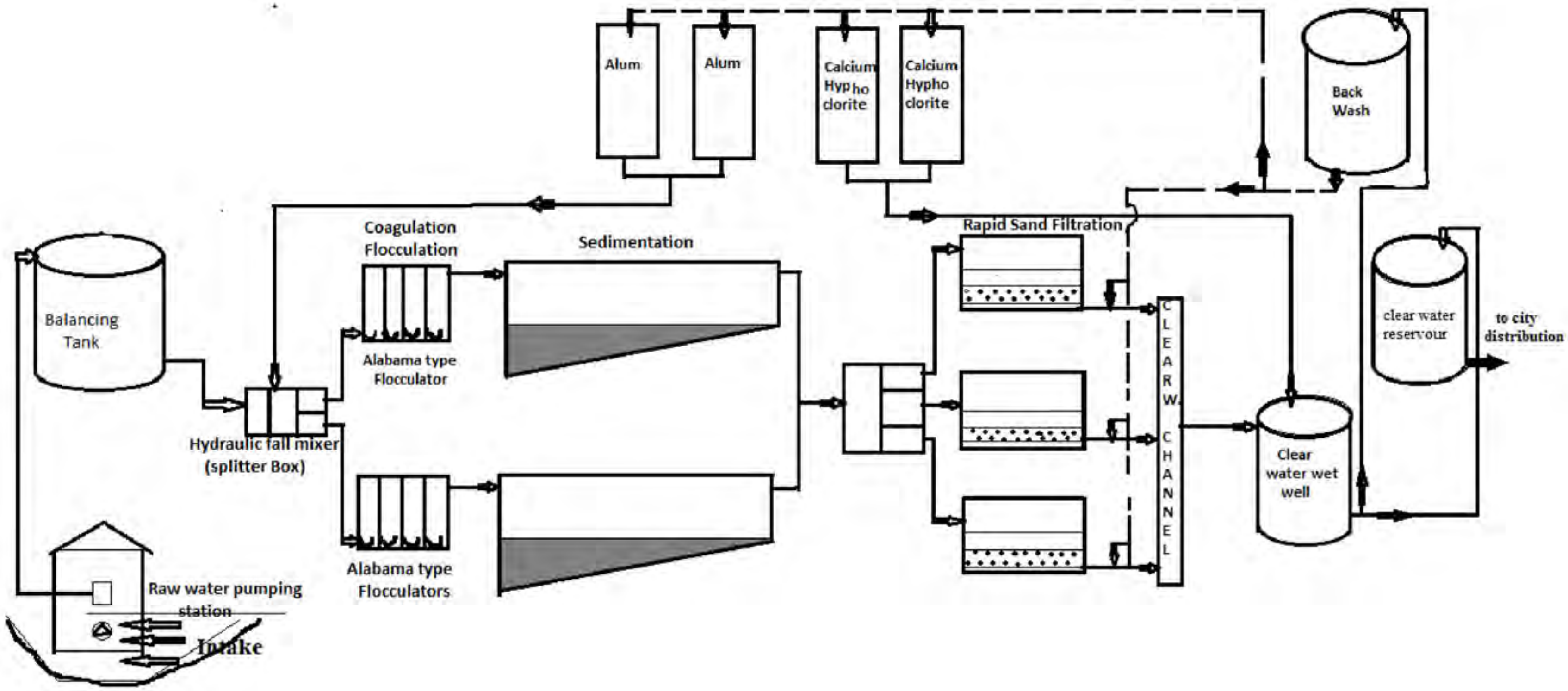
Annexes

Annex 1 Schematic flow diagram of GWTP



Annex 2 Definitions for Assessing Performance Limiting Factors

(Source [2])

NOTE: The following list of defined factors is provided to assist the evaluator with identifying performance limitations associated with protection against microbial contaminants in water treatment systems. Performance limiting factors are described below using the following format.

A. CATEGORY

1. Subcategory

a. Factor Name

Factor description

A. Design

1. Source Water Quality

a. Microbial Contamination

♦ Does the presence of microbial contamination sources in close proximity to the water treatment plant intake impact the plant's ability to provide an adequate treatment barrier?

2. Unit Process Adequacy

a. Intake Structure

♦ Does the design of the intake structure result in excessive clogging of screens, build-up of silt, or passage of material that affects plant equipment?

b. Raw Water Pumping

♦ Does the use of constant speed pumps cause undesirable hydraulic loading on downstream unit processes?

c. Flow Measurement

♦ Does the lack of flow measurement devices or their accuracy limit plant control or impact process control adjustments?

d. Chemical Storage and Feed Facilities

♦ Do inadequate chemical storage and feed facilities limit process needs in a plant?

e. Flash Mix (hydraulic jump)

♦ Does inadequate mixing result in excessive chemical use or insufficient coagulation to the extent that it impacts plant performance?

Definitions for Assessing Performance Limiting Factors continued

f. Flocculation

- ♦ Does a lack of flocculation time, inadequate equipment, or lack of multiple flocculation stages result in poor floc formation and degrade plant performance?

g. Sedimentation

- ♦ Does the sedimentation basin configuration or equipment cause inadequate solids removal that negatively impacts filter performance?

h. Filtration

- ♦ Do filter or filter media characteristics limit the filtration process performance?
- ♦ Do filter rate-of-flow control valves provide a consistent, controlled filtration rate?
- ♦ Do inadequate surface wash or backwash facilities limit the ability to clean the filter?

i. Disinfection

- ♦ Do the disinfection facilities have limitations, such as inadequate detention time, improper mixing, feed rates, proportional feeds, or baffling, that contribute to poor disinfection?

j. Sludge/Backwash Water Treatment and Disposal

- ♦ Do inadequate sludge or backwash water treatment facilities negatively influence plant performance?

3. Plant Operability

a. Process Flexibility

- ♦ Does the lack of flexibility to feed chemicals at desired process locations or the lack of flexibility to operate equipment or processes in an optimized mode limit the plant’s ability to achieve desired performance goals?

b. Process Controllability

- ♦ Do existing process controls or lack of specific controls limit the adjustment and control of a process over the desired operating range?

c. Process Instrumentation/Automation

- ♦ Does the lack of process instrumentation or automation cause excessive operator time for process control and monitoring?

Definitions for Assessing Performance Limiting Factors continued

d. Standby Units for Key Equipment

- ♦ Does the lack of standby units for key equipment cause degraded process performance during breakdown or during necessary preventive maintenance activities?

e. Flow Proportioning

- ♦ Does inadequate flow splitting to parallel process units cause individual unit overloads that degrade process performance?

B Operation

1. Testing

a. Process Control Testing

- ♦ Does the absence or wrong type of process control testing cause improper operational control decisions to be made?

b. Representative Sampling

- ♦ Do monitoring results inaccurately represent plant performance or are samples collected improperly?

2. Process Control

a. Time on the Job

- ♦ Does staff's short time on the job and associated unfamiliarity with process control and plant needs result in inadequate or improper control adjustments?

b. Water Treatment Understanding

- ♦ Does the operator's lack of basic water treatment understanding contribute to improper operational decisions and poor plant performance or reliability?

c. Application of Concepts and Testing to Process Control

- ♦ Is the staff deficient in the application of their knowledge of water treatment and interpretation of process control testing such that improper process control adjustments are made?

Definitions for Assessing Performance Limiting Factors continued

3. Operational Resources

a. Training Program

- ♦ Does inadequate training result in improper process control decisions by plant staff?

b. Technical Guidance

- ♦ Does inappropriate information received from a technical resource (e.g., design engineer, equipment representative, regulator, peer) cause improper decisions or priorities to be implemented?

c. Operational Guidelines/Procedures

- ♦ Does the lack of plant-specific operating guidelines and procedures result in inconsistent operational decisions that impact performance?

C. Maintenance

1. Maintenance Program

a. Preventive

- ♦ Does the absence or lack of an effective preventive maintenance program cause unnecessary equipment failures or excessive downtime that results in plant performance or reliability problems?

b. Corrective

- ♦ Does the lack of corrective maintenance procedures affect the completion of emergency equipment maintenance?

c. Housekeeping

- ♦ Does a lack of good housekeeping procedures detract from the professional image of the water treatment plant?

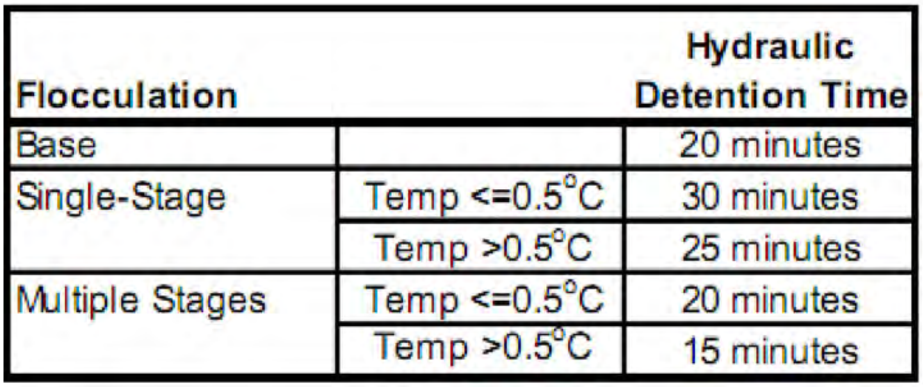
2. Maintenance Resources

a. Materials and Equipment

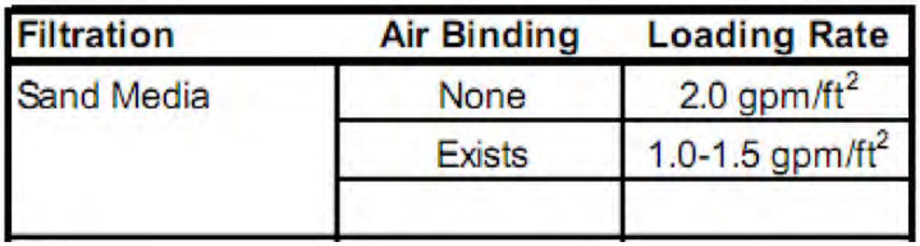
- ♦ Does the lack of necessary materials and tools delay the response time to correct plant equipment problems?

Annex 3 Major unit processes evaluation criteria

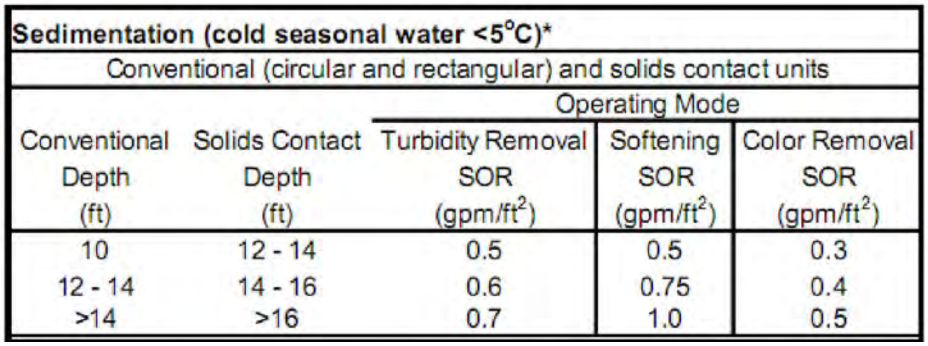
(Adopted from [15])



Flocculation		**Hydraulic Detention Time**
Base		20 minutes
Single-Stage	Temp <=0.5°C	30 minutes
	Temp >0.5°C	25 minutes
Multiple Stages	Temp <=0.5°C	20 minutes
	Temp >0.5°C	15 minutes



Filtration	**Air Binding**	**Loading Rate**
Sand Media	None	2.0 gpm/ft^2
	Exists	1.0-1.5 gpm/ft^2



Sedimentation (cold seasonal water <5°C)*

Conventional (circular and rectangular) and solids contact units

		Operating Mode		
Conventional Depth (ft)	Solids Contact Depth (ft)	Turbidity Removal SOR (gpm/ft^2)	Softening SOR (gpm/ft^2)	Color Removal SOR (gpm/ft^2)
10	12 - 14	0.5	0.5	0.3
12 - 14	14 - 16	0.6	0.75	0.4
>14	>16	0.7	1.0	0.5

Annex 4 Determination of major unit processes capability

A) Flocculation Basin capability

1. Basin Volume = 54m^3

2. 15-minute detention time (selected loading rate) .

3. Rated Capability = $\frac{\text{Basin volume } (m^3)}{\text{Detention time (min)}}$

=54m^3/15 min x 1440min/day

= **5,184 m^3/d**

B) Sedimentation Basin capability

1. Basin Surface Area = 2 basin x 25m (length) x 3.1m (depth)

= 155m^2

2. Selected surface overflow rate to determine peak rated capability = 20.35l/m/m^2

3. Rated Capability = Basin Surface Area (m^2) x surface overflow rate (l/min/m^2)

= 155m^2 x 20.35l/min/m^2 x 1m^3/1000l x1440min/day

= **4542.12 m^3/d**

C) Filtration basin capability

The rated capability of the three filtration units was determined by assuming one of the filters out of service for cleaning (ref). Thus only two filters bed area was used

1. Filter bed Area = 2 filters x 5.4m x 4.3m

= 46.44 m^2

2. selected filter loading rate to determine peak rated capability = 81.4 l/min/m^2

3. Rated Capability = Filter bed Area (m^2) x filter loading rate (l/min/m^2)

= 46.44 m^2 x 81.4 l/min/m^2 x 1440min/day x 1m^3/1000l

= **5,443.5 m^3/d**

Determination of major unit processes capability continued

D) Disinfection (Clearwater-well) capability

The treatment plant uses only post-disinfection and capability of the unit process was projected based on the post-chlorination disinfection requirement.

1. Required Giardia log reduction/inactivation was determined based on surface water source

3.0 log

2. Expected Log removals of Giardia Cysts by conventional filtration is 2.5 log

3. The required log inactivation by disinfection is the difference between Required Giardia log reduction/inactivation (step 1) and Expected Log removals of Giardia Cysts by conventional filtration (step 2)

= 3.0 - 2.5 = 0.5 log inactivation

4..CT required for 0.5 log inactivation of Giardia cyst was determined based on minimum water temperature and maximum treated water pH. minimum Temperature of 15°C and maximum pH of 8 was selected, and the maximum free chlorine residual was set at 1.6 mg/L. then from tables of CT values, the CT was found to be 18 mg/L-min

5. Required contact time based on maximum free chlorine residual that can be maintained was determined by the following formula.

$$\text{Required contact time} = \frac{\text{CT required for 0.5 log inactivation of Giardia cyst}}{\text{free chlorine residual}}$$

= 18 mg/l-min/1.6 mg/l

= 11.36 min

6. The effective volume of Clearwater-well was determined by the following formula:

Effective volume = basin volume at minimum depth x baffling factor (T_{10}/T)

Basin is unbaffled so T_{10}/T factor of 0.1 was used, Minimum operating depth is 3.0 m. therefore Effective volume = 3.14 x 25 m^2 x 3 m x 0.1

= 23.5 m^3

7. Rated capability was determined by dividing Effective volume by the required contact time. Thus : Rated Capability = 23.5 m^3 /11.36 min x 1440 min/day

= 2,977 m^3/d

Annex 5 Questionnaire for the plant operator to identify possible operational problems

Items	questions	Operator interview
1	**Chemical Pretreatment**	
	How does the operator determine proper chemical ?	
	Jar tests	
	Visual observation of floc formed	ü
	Historical performance data	
	Checked pH	
	How does the operator make the chemical adjustments and procedure for checking and confirming proper dosages and how often (during changes in raw water quality characteristics)?	
	Visual observation of floc formed	ü
	Volumetric measurement	
	Checked pH	
	Do you frequently wash the alum preparation tank?	
	Yes	
	daily	
	weekly	
	monthly	
	No	ü
2	**Flocculation**	
	Is floc formed at an appropriate location?	
	After rapid mixing	
	Before midle of flocculation tank	
	At midle of flocculation tank	
	Downstream of middle of flocculation tank	ü
	Not visible floc formed	
	Do you frequent wash the flocculation tank?	
	Yes	
	daily	
	weekly	
	monthly	
	No (6 months)	ü
3	**Sedimentation**	
	Is sludge removal frequent enough to prevent short-circuiting?	
	Yes	
	daily	
	weekly	
	monthly	
	No (3 months)	ü
	Do you frequent wash the sedimentation tank?	
	Yes	
	daily	

Questionnaire for the plant operator (continued)

	weekly	
	monthly	
	No (3 months)	ü
4	**Filtration**	
	Does the operator consider all three criteria (turbidity, head loss, and time) when establishing backwash timing?	
	Turbidity	
	Water level	
	Headloss indicator (1.50 m differential)	ü
	Filter run time (16 hrs)	
	Druing a wash, does the operator ensure through cleaning of the filter media, adequate flow rates and media expandsion, and lack of dead spots or boiling?	
	Upflow water is cleared	
	Upflow water level	ü
	Does the operator use the surface wash during the backwash?	
	Yes	
	Surface scraping	
	Surface scour (water jet)	
	Hand raking	
	No	ü
	How does the operator minimize breakthrough when placing a filter back to service?	
	Up flow water is cleared	
	Cleared water at filter drain pipe	
	Filtered drain till filter sand dried	ü
	Do you frequent check the filter depth?	
	Yes	
	No	ü
	Do you frequent sand added and resand?	
	Yes (how often)	
	No	ü
	Do you frequently wash the filtration tank?	
	Yes, backwashing time	
	daily	
	weekly	ü
	monthly	
	No	
5	**Disinfection**	
	How does the operator prepared calcium hypochlorite solution ?	
	Direct mixed in feed tank	ü
	Other	

Questionnaire for the plant operator (continued)

	How does the operator making the disinfectant adjustments and procedure for checking and confirming proper dosages and how often?	
	Checked free chlorine	
	Volumetric measurement	
	Other	
	No	ü
	Do you frequently wash the calcium hypochlorite feed tank?	
	Yes, hypochlorite sludge sediment on bottom tank	
	daily	
	weekly	
	monthly	
	No	ü

Annex 6 Checklist of visual inspection on plant operation evaluation

Items	checklist	Visual observation
1	**Chemical Pretreatment**	
	Alum Wastage (Solid alum in tank or not soluble)	
	Corrosion or leakage in alum feed tank	ü
	Plugging problem of alum feed pipe	
	Alum sludge	ü
	Mixer installed	ü
2	**Flocculation**	
	Floc Charcteristics and Floc settling	
	Overflow between baffled channel	
	No visible flocs formed	
	Floc Formed	ü
	Larger floc formed at downstream	
	Floc settled	ü
	Floc breakage at outlet	
	Tank Cleaning and Maintenance	
	Deposits in the flocculators	ü
	Scum accumulation	ü
	Algae growth	ü
3	**Sedimentation**	
	Effects of turbulence, short-circuiting, scour is high	
	Floating sludge	
	Excessive floc carry-over	
	Algae growth	ü
	Scum accumulation	
4	**Filtration**	
	Algae growth	ü
	Mud coated on filter sand	
	Mud ball formation	
	Media cracking, mounding	ü
5	**Backwashing**	
	Carryover of sand during backwashing	ü
	All mudball been removed	ü
	Filtered had sand or broken underdrain system	ü
	Startups occur on dirty filter	ü

Annex 7 Raw data of Jar test for optimum dosage and pH determination

sample	chemicals added	Analysis performed	
	$FeCl_3$ (mg/l)	pH	turbidity
Run #1			
Raw		7.4	14.5
Jar 1	5	7.1	7.2
Jar 2	10	7.1	4.32
Jar 3	15	7	1.4
Jar 4	20	6.8	2.32
Jar 5	25	6.7	2.55
Jar 6	30	6.4	2.84
Run # 2			
Raw		7.4	14.5
Jar 1	15	5.5	2.22
Jar 2	15	6	0.94
Jar 3	15	6.5	0.82
Jar 4	15	7	0.59
Jar 5	15	7.5	0.59
Jar 6	15	8	0.61
Run # 3			
Raw		7	14.5
Jar 1	5	7.3	6
Jar 2	10	7.2	1.8
Jar 3	15	7	1.7
Jar 4	20	6.8	0.5
Jar 5	25	6.7	0.5
Jar 6	30	6.4	0.5
Jar 7	35	6.3	0.5
Jar 8	40	6.2	0.4
Jar 9	45	6.3	0.5
Jar 10	50	6.3	0.6

Raw data of Jar test for optimum dosage and pH determination continued

sample	chemicals added	analysis performed	
	Fe $(SO_4)_3$ (mg/l)	pH	turbidity
Run #1			
Raw		7.2	22
Jar 1	5	6.8	9.5
Jar 2	10	6.5	5
Jar 3	15	6.5	4.76
Jar 4	20	6.4	3.6
Jar 5	25	6.1	3.2
Jar 6	30	5.9	3.5
Run # 2			
Raw		7.2	22
Jar 1	25	5.5	3.8
Jar 2	25	6	1.27
Jar 3	25	6.5	1.2
Jar 4	25	7	1
Jar 5	25	7.5	0.96
Jar 6	25	8	1.25
Run # 3			
Raw		7.5	22
Jar 1	5	7.3	6.5
Jar 2	10	7.4	2.3
Jar 3	15	7.4	1.7
Jar 4	20	7	0.9
Jar 5	25	6.9	0.7
Jar 6	30	6.8	0.7
Jar 7	35	6.8	0.6
Jar 8	40	6.6	0.6
Jar 9	45	6.6	0.6
Jar 10	50	6.5	0.6

Raw data of Jar test for optimum dosage and pH determination continued

sample	chemicals added (Dose)	Analysis performed	
	Alum (mg/l)	pH	Turbidity (NTU)
Run # 1			
Raw		7.2	14.5
Jar 1	30	7	2.7
Jar 2	35	6.8	1.43
Jar 3	40	6.8	1.1
Jar 4	45	6.5	0.95
Jar 5	50	6.5	0.72
Jar 6	55	6.5	1.33
Run #2			
Raw		7.2	14.5
Jar 1	50	5.5	1.38
Jar 2	50	6	0.68
Jar 3	50	6.5	0.6
Jar 4	50	7	0.6
Jar 5	50	7.5	0.5
Jar 6	50	8	0.48
Run # 3			
Raw		8	14.5
Jar 1	5	7.7	7
Jar 2	10	7.6	6
Jar 3	15	7.5	3.5
Jar 4	20	7.4	2.2
Jar 5	25	7.3	2.2
Jar 6	30	7.3	2.1
Jar 7	35	7.2	1.8
Jar 8	40	7.2	1.5
Jar 9	45	7.1	1.3
Jar 10	50	7.1	1.3

Annex 8 Raw data of filter #1 effluent turbidity after backwash

Sampling time (min)	Turbidity (NTU)	Sampling time (min)	Turbidity (NTU)
1	5	24	2.2
2	5.6	25	2.2
3	6.2	26	1.8
4	6.4	27	1.6
5	6.4	28	1.4
6	6.6	29	1
7	6.6	30	0.8
8	6.6	31	0.6
9	6	32	0.6
10	5.6	33	0.6
11	5	34	0.6
12	4.8	35	0.6
13	4.6	36	0.4
14	4.4	37	0.4
15	4.4	38	0.4
16	4.6	39	0.4
17	3.8	40	0.4
18	3.6	41	0.4
19	3.8	42	0.4
20	4	43	0.4
21	3.8	44	0.4
22	3.8	45	0.4
23	2.8	46	0.4

Annex 9 Raw data of settled and filtered water turbidity

A) Raw data of settled water turbidity

Day	Settled water turbidity	
	Sedimentation basin #1	Sedimentation basin #2
1	8	6
2	10	8
3	11	8
4	9	8
5	10	7
6	8	6
7	8	6
8	9	6
9	8	6
10	8	6

B) Raw data of combined filter turbidity

Sampling time (h)	Combined filtered Turbidity (NTU)
1	6
2	6
3	6.2
4	6.7
5	6.7
6	7.1
7	6
8	6
9	6
10	6.2
11	6.2
12	7
13	6.6

Annex 10 Raw data of flow rate measurements

A) Raw data of Alum flow rate

ALUM FLOW RATE	
Time (s)	volume (ml)
1.95	21
1.93	21
2.11	23
1.69	19.5
1.98	20
1.932	**20.9**
alum feed pump rate = 10.82 ml/sec	
feed rate = 649.2ml/min	

B) Raw data of post chlorination flow rate

POST CHLORINATION FLOW RATE	
Time (s)	Collected Volume (ml)
2.66	43
2.48	41.5
2.74	46.5
2.6	42
2.48	42.5
average 2.592	43.1
average = 43.1ml/2.592sec	
Average flow rate = 16.6ml/sec	

Declaration

I, the undersigned, declare that this thesis is my original work and has not been presented for a degree in any University, and that all the source of materials used for the thesis has been duly acknowledged.

Declared by:

Name: Tariku G/Tsadik G/Silassie

Signature: ___________________

Date: 28 March 2013

Advisor

Name: Dr.Ing. Berhanu Assefa

Signature: __________________

Date: 28 March 2013