COLLEGE OF ENGINEERING, DESIGN, ART AND TECHNOLOGY

SCHOOL OF ENGINEERING

AN INVESTIGATION OF RELIABILITY OF THE WATER SOURCE AND COST-EFFECTIVE CHEMICAL USE AT MASAKA WATER TREATMENT WORKS

by

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DEDICATION

I dedicate this thesis to my wonderful family.
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God bless you all for being there when I needed you.
ABSTRACT

The challenge of providing improved and adequate water services is substantial and this study sought to determine whether the NWSC Masaka Area’s current water source is sufficient to ensure adequate water supply to meet future demands of the area. In addition, Masaka water works has the highest unit chemical cost per cubic meter of water pumped in NWSC with an average of 191Shs/m³ in comparison to the NWSC global unit chemical cost per cubic meter of 77Shs/m³. Effective management of municipal water works demands a holistic approach linking technical and economic monitoring with protection of natural eco-systems. Thus this research had the dual task of checking source adequacy as well as cost effective usage of chemicals by analyzing existing data so as to determine technically the optimal way forward.

River flow data for River Nabajjuzi was obtained and infilled using hot deck infilling techniques in MS EXCEL. The abstraction limit (MAM-7) of the 7-day low flow data was calculated as 8,122m³/d and was then utilized with the log Pearson Type III frequency analysis techniques to determine the yield maintainable in a once in 50 year drought of River Nabajjuzi. This was calculated as 13,478m³/d. The abstraction limit and safe yield were both higher than the highest abstraction rate during the period 2006-2008 of 4733m³/d and the plant design capacity of 8000m³/d implying adequacy of the source currently for Masaka area. However, the calculated future and ultimate years’ demands of 11,552m³/d by 2018 and 19,791m³/d by 2028, were significantly higher than the abstraction limits for River Nabajjuzi implying that it would not be an adequate source for the area in the near future. Explorations for a new source should begin soon for future planning.

In addition, the water quality results for River Nabajjuzi indicate that it is characterized by low pH of average range 5.8-6.1, high colour with an average of 206Ptu, high iron content with an average 2.4mg/l and high turbidity with an average of 38.8NTU. All these parameters were observed to increase significantly during the dry seasons and reduce in the wet seasons. The analysis of the water pumped and chemical consumption records showed greater chemical consumption and cost along the Boma treatment line than the Bwala treatment line for chlorine with averages of 1022kg/month and 688kg/month respectively and Soda ash with averages of 2016kg/month and 981kg/month respectively. However, the Aluminium Sulphate consumption was greater along the Bwala treatment line with an average of 2212kg/month compared to 1838 kg/month for Boma treatment line. This was in contrast to the fact that more water on average was pumped from Boma treatment plant than from Bwala treatment plant i.e. 60,920m³/month compared to 55,878m³/month respectively. As both plants have the same water source, it implies that this anomaly could be due to operational inefficiencies and inadequate design of some of the process units.

It can thus be concluded that River Nabajjuzi is still a valid current source of water for Masaka Municipality in the short term but not for the long term. In addition, the chemical consumption at the Masaka water works are higher during the dry season than the wet season due to the variation in the characteristics of the raw water but with some anomalies caused by operational and design handicaps.

Exploration for new water source while incorporating the Integrated urban water management principles needs to be undertaken in the very near future to provide continuous water supply for Masaka town.
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ACRONYMS AND ABBREVIATIONS

DWD Directorate of Water Development
GWP Global Water Partnership
IUWM Integrated Urban Water Management
MAM-7 Mean Annual Minimum Seven day flow
NTU Nephelometric Turbidity Unit
NWSC National Water and Sewerage Corporation
Ptu Platinum Units
THM Trihalomethane
WHO World Health Organisation
WMO World Meteorological Organisation
WRMD Water Resources Management Directorate
USGS United States Geological Survey
USWRC United States Water Resources Council
1 INTRODUCTION

1.1 Background

Although water is abundant in Uganda, drinking water is expensive and in short supply. One of the guiding principles of the International Conference on Water and the Environment in Dublin 1992 was that “Fresh water is a finite and vulnerable resource, essential to sustain life, development and the environment. Effective management of municipal water works demands a holistic approach linking technical and economic monitoring with protection of natural eco-systems” (GWP, 2008).

The old attitude about water being a free good has long been abandoned by economists but persists in the approach of many people when trying to solve water problems. Another of the guiding principles adopted by the Dublin water conference dealt with this specific point when it emphasizes: “Water has an economic value in all its competing uses and should be recognized as an economic good. Within this principle it is vital to recognize first the basic right of all humans to have access to clean water and sanitation at an affordable price” (GWP, 2008).

Most of those without improved water supply and decent sanitation live in developing countries (GWP, 2008). Currently only 75 percent of Ugandans in urban areas have access to safe water sources and only 6 percent are connected to a centralized sewerage system (NWSC 2011/12). The main reasons for this situation are technical, financial, management and governance. There is thus an increasing challenge in providing adequate water services to the large proportion of the currently un-served population, as well as in attaining goal 7 target 10 of the Millennium Development Goals whose target is to halve by 2015 the proportion of people without sustainable access to safe drinking water and sanitation.

Water resources availability, access and quality are critical issues with regards to improving water supply in Uganda. Uganda is expected to experience water stress by 2025 due to the continuing degradation of the country’s wetlands which provide indispensable ecosystem and regulating services including maintenance of the water table, water filtration, flood control, groundwater recharge and microclimate regulation, (Wong et al, 2005). Water quality in Uganda is deteriorating as domestic, industrial and agro-chemicals run-off into watercourses, as well as the effects of rapid increases in urban population and rural-urban migration which have led to unplanned settlement slums (Wong et al, 2005).

Measures to safeguard many water resources from over-exploitation such as trans boundary agreements between riparian states are presently inadequate for ensuring sustainability of water resources, without which the sustainability of some water supply schemes may be threatened (Dooge, 2004). There is wide variation in flows of surface water sources due to substantial extraction for public water supply, irrigation, deforestation of catchment areas that affect run off as well as local climatic variations (Dooge, 2004).

The National Water and Sewerage Corporation (NWSC) is a parastatal established by Decree No. 34 of 1972 following an earlier study on the need for improvement of water and sanitation in the urban centres of Uganda.
The corporation was re-established by the National Water and Sewerage Corporation Act 2000, Cap 317 to:

a) Manage the water sources in ways which are most beneficial to the people of Uganda.

b) Provide water services for domestic, stock, horticultural, industrial, commercial, recreational, environmental and other beneficial uses.

c) Provide services, in any area in which it may be appointed to do so under the Act.

d) Develop the water and sewerage systems in urban centres and big national institutions throughout the country.

NWSC is charged with the provision of potable water and sewerage services in designated towns and peri-urban centres. The Corporation is currently operating in 22 towns of; Kampala, Jinja, Entebbe, Mbarara, Masaka, Mbale, Tororo, Soroti, Lira, Gulu, Arua, Kasese, Fort Portal, Bushenyi, Kabale, Mubende, Masindi, Hoima, Mukono, Malaba, Iganga and Lugazi.

1.2 Study area

Masaka Water Supply Service Area is located in Masaka District in the southern part of Uganda which is situated about 37kms South of the Equator (between 0° 25'S and 34° E), having an average altitude of 1160m above sea level. The District is bordered by Sembabule in the northwest, Mpiigi District in the North, Rakai District in the west and south, and Lake Victoria (part of Kalangala District) in the East (Figure1-1). The District Headquarters are 130 km from Kampala.

The topography in Masaka is rolling and undulating with valley bottom swamps including stream flows to lakes and rivers. It has got some rocky hills, hilly paddocks, considerable area under river basin and plains, lakes, swamps and bushes. The District has a total land area of about 4560.4 Km² of which 30 % is water and swamps (Masaka District profile, 2008).

The soil texture is varied from place to place ranging from red-laterites, sandy loam and loam but in general, agriculturally productive. The rainfall pattern is bimodal having two crop seasons with dry spells in between June and August as well as December and February with the exception of a few months in a year of declining trend in precipitation. The average annual rainfall received is 1100-1200mm with 100-110 annual rainy days. The maximum temperature recorded is about 30°C and the minimum is 10°C.

Masaka Water Supply and Sewerage Service Area covers 110 square km, with Masaka town being the central business location of the Service Area. The targeted population is about 85,000 people and of these, 65,000 are residents of Masaka Municipal Council while the other 20,000 people, reside outside Masaka Municipal Council. The average family size in Masaka is 4.3 people and out of the above target population only about 80% have easy access to potable water from NWSC. The remaining 20% still have problems of accessing safe water (NWSC, 2011/12).
NWSC Masaka service area has its source of raw water at River Nabajjuzi located about 5km from the town centre. The district is located within the cattle corridor which stretches from north east, through central to south west of Uganda and has low groundwater yield making boreholes inadequate sources of water supply for the town. The other nearby water source is Lake Nabugabo but it is located 20km from Masaka Municipality and was not considered a cost effective option for water supply at the time.

However, the water abstracted from the Nabajjuzi wetland has a water quality problem of high colour and iron content and its treatment through the conventional methods requires the use of large amounts of chemicals which in turn increases operational costs for the area. The average monthly chemical costs for NWSC Masaka area for the period 2006-2009 were 11.3% of the total monthly operating expenses of the Area (NWSC 2009). This is high in comparison in comparison to other similar size towns like Lira and Mbale whose average monthly chemical costs were 4% and 6% of their total monthly operating expenses.

![Figure 1-1: Map of Masaka Area](image)

### 1.3 Problem Statement

Demand for water supply for urban use is growing across all sectors. Managing water resources to meet the growing demand presents major challenges because of competition for water and environmental issues. There is also an uncertainty associated with climate change and the potential for decreased yield from existing water supply catchments. As a result, there is considerable potential for conflict over supplying increased urban water demands. The trend in total urban water use is determined by changes in population and per capita use.
The challenge of providing improved and adequate water services is substantial and thus there is a need to determine whether the current water source is sufficient to ensure adequate water supply to meet future demands.

In addition, there is high chemical consumption in water purification in Masaka water works in comparison to other NWSC water works due to the nature of the Masaka water source resulting into high chemical costs. Therefore, there is a need to establish good information regarding cost effective water quality management practices, leading into operational costs containment for efficient management of water utilities.

1.4 Main Objective
The main objective of this study was to carry out forecasting to predict sustainable water supply to NWSC Masaka Area by investigating the reliability of River Nabajjuzi and also determine interventions that optimise chemical usage.

1.5 Specific Objectives
The specific objectives of the study were to:

i. Determine the Mean Annual minimum 7-day low flow (MAM7) and the yield maintainable through a once in 50 years drought (safe yield) for River Nabajjuzi.

ii. Determine the current and future abstraction rates at the Masaka Water Treatment Works and compare to the safe yield.

iii. Determine the key raw water quality characteristics of the water source and the impact of their fluctuations on chemical use and costs.

iv. Determine the immediate and future measures or interventions that can be put in place to optimise chemical use within the existing plant.

1.6 Research Significance
The information obtained from this study is important for managers, engineers and policy makers, who would want to ensure the provision of safe, reliable and continuous potable water supply in their areas of operation; make improvements in water quality management and above all ensure customer satisfaction. The study also suggested areas for further research that are important for the different researchers who would wish to do research in a similar field.

1.7 The Scope
The study involved the collection and analysis of streamflow data for River Nabajjuzi from 1965-2006. Log Pearson type III data analysis techniques were then used to determine the reliability of River Nabajjuzi to meet the future demands for NWSC Masaka water supply area.

Additional data for the period 2006-2009 were obtained from NWSC for the analysis of the water quality characteristics of the River Nabajjuzi, their impact on chemical consumption costs at the treatment plant as well as observation of operational procedures.
These were then analysed using graphs and compared to other similarly sized towns' water treatment works and their consumption.

From this, the possible short and long term interventions that could be undertaken to ensure chemical use optimisation within the existing plant were recommended.
2 LITERATURE REVIEW

2.1 Introduction

This literature review provides a description of low flow hydrology, frequency analysis and the various numerical data infilling techniques that can be used in assessing the abstraction limits and safe yields of a water source.

A river flow regime describes an average seasonal behaviour of river flow, usually representing a set of long-term monthly mean values. Seasonal patterns of flow can be regular, repeating in principle the same pattern from year to year, or irregular, i.e. alternating between different regime types during individual years (Krasovskaia et al., 1999).

Flow in a river is the result of the complex natural processes, which operate on a catchment scale. Conceptually, a river catchment can be perceived as a series of interlinked reservoirs, each of which has components of recharge, storage and discharge. Recharge to the whole system is largely dependent on precipitation, whereas storage and discharge are complex functions of catchment physiographic characteristics. During a dry season, it is those processes that affect the release of water from storage and the fate of this discharge that are directly relevant (Smakhtin, 2001).

These processes are usually operative in the vicinity of the river channel zones as opposed to the full range of hydrological processes that operate over larger parts of catchments during periods of high discharge. The latter of course also cannot be ignored as they control the catchments ability to absorb and store water during precipitation events for later release as low flows (Smakhtin, 2001).

2.2 Low Flows and Design Criteria

Low flow means different things to different interest groups. To many it may be considered as the actual flows in a river occurring during the dry season of the year, others may be concerned with the length of time and the conditions occurring between flood events (e.g. in erratic and intermittent semi-arid flow regimes). Yet others may be interested in the effects of changes in the total flow regime of a river on sustainable water yield. The latter may perceive ‘low flows’ not only as discharges occurring during a dry season, but as a reduction in various aspects of the overall flow regime (Smakhtin, 2001).

International glossary of hydrology (WMO, 1974) defines low flow as ‘flow of water in a stream during prolonged dry weather’. This definition does not make a clear distinction between low flows and droughts. Low flow is a seasonal phenomenon, and an integral component of a flow regime of any river. On the other hand drought is a natural event resulting from a less than normal precipitation for an extended period of time.
Low flows are normally derived from groundwater discharge or surface discharge from lakes, marshes, or melting glaciers. Lowest annual flow usually occurs in the same season each year. The magnitudes of annual low flows, variability of flows and the rate of streamflow depletion in the absence of rain, duration of continuous low-flow events, relative contribution of low flows to the total streamflow hydrograph are a few of the widely used characteristics which are dealt with in low-flow hydrology in a variety of ways (Smakhtin, 2001).

2.3 Low Flow Frequency Analysis

The objective of streamflow frequency analysis is to infer the probability of exceedence of all possible discharge values (the parent population) from observed discharge values (a sample of the parent population). This process is accomplished by selecting a statistical model that represents the relationship of discharge magnitude and exceedence probability for the parent population. The parameters of the models are estimated from the sample (U.S. Army Corp of Engineers, 1994).

With the calibrated model, the hydrologic engineer can predict the probability of exceedence for a specified magnitude or the magnitude with specified exceedence probability. Low-flow frequency indices are widely used in drought studies, design of water supply systems, estimation of safe surface water withdrawals, classification of streams’ potential for waste dilution (assimilative capacity), regulating waste disposal to streams, maintenance of certain in-stream discharges (U.S. Army Corp of Engineers, 1994).

The most widely used indices are 7-day 10-year low flow (7Q10) and 7-day 2-year low flow (7Q2), which are defined as the lowest average flows that occur for a consecutive 7-day period at the recurrence intervals of 10 and 2 years, respectively. Some studies refer to different similar indices, e.g. 3 day 20 year low flow (Hutson, 1988). A number of reports produced by the USGS (United States Geological Survey) contain a variety of minimum flows: annual means and extremes for selected periods ranging from 1 to 183 days and for recurrence intervals ranging from 2 to 50 years (Hughes, 1981; Armentrout and Wilson, 1987; Zalants, 1992; Cervione et al., 1993; Giese and Mason, 1993; Atkins and Pearman, 1995).

The average of the annual series of minimum 7-day average flows is known as Dry Weather Flow (Hindley, 1973) or as Mean Annual 7-day Minimum flow (MAM7) (Pirt and Simpson, 1983; Gustard et al., 1992) and is used in the UK for abstraction licensing. The 7-day period covered by MAM7 eliminates the day-to-day variations in the artificial component of the river flow. Also, an analysis based on a time series of 7-day average flows is less sensitive to measurement errors. At the same time, in the majority of cases there is no big difference between 1-day and 7-day low flows. Many countries use design flow statistics such as the 7Q10 (the lowest 7-day average flow that occurs on average once every 10 years) or the MAM7 to define low flow for the purpose of setting permit abstraction and discharge limits (Smakhtin, 2001).
Heicher and Hirschel (1989) illustrated how different low-flow extremes may be used to identify existing and potential water supply problems, to identify historical extreme low-flow periods, and to determine potential water supply deficits (and other consequences) during a repeat of the most severe historical low-flow period.

Current waterworks practice in the United Kingdom is to adopt the probability yield (the steady supply that can just be maintained through a drought of specified severity) for planning purposes (Twort et al., 1986). Major sources, whose failure would seriously disrupt industry or which are irreplaceable within any undertaking, are generally assigned yields for a once-in-100-year drought (1% yield). However, for most sources a ‘safe’ or ‘reliable’ yield is a frequently adopted standard and is defined as the yield maintainable through a 2% (once-in-50-years) drought (Twort et al., 1986).

A hydrologically-based design flow is computed using the single lowest flow event from each year of record and then examining these flows for a series of years. This statistical method is based on selecting and identifying an extreme value, such as the lowest 7-day average flow in a ten year period (i.e., 7Q10) (Smakhtin, 2001). The advantage of this method is that it utilises extreme value analytical techniques (e.g., log-Pearson Type III flow estimating technique) or graphical techniques supported by past engineering and statistical practice. These help to estimate an intake yield on the basis of risk of failure.

With numerical techniques, the following general steps are used to derive a frequency curve to represent the population (McCuen and Snyder, 1986):

- Select a candidate frequency model of the parent population. Three distributions are commonly used for frequency analysis of hydrometeorological data: the normal distribution, the log-normal distribution, and the log Pearson type III distribution. However, the U.S. Water Resources Council (USWRC) (1967, 1976, and 1977) recommended the log Pearson type III distribution for annual maximum and minimum streamflow frequency studies.

- Obtain a sample.

- Use the sample to estimate the parameters of the model identified previously. These parameters are usually the mean of the sample, the standard deviation and the sample skew.

- Use the model and the parameters to estimate quantiles to construct the frequency curve that represents the parent population.

The distributions commonly used in Hydrologic Engineering are commonly written as follows:

\[ Q_p = \bar{Q} + K_p S \]
Where; \( Q_p \) = the quantile with specified exceedence probability \( p \).

\( \bar{Q} \) = the sample mean

\( S \) = the sample standard deviation

\( K_p \) = a frequency factor which depends on the distribution selected.

Graphical techniques permit inference of the parent population characteristics with a plot of observed magnitude versus estimated exceedence probability of that data.

The reliability of a frequency model increases as the sample size increases but in practice, however, gaps in hydrological data due to lost or missing data that result from failure of stream side equipment are present in data sets collected in some countries, which are a major source of error (Gyau-Boakye and Schultze, 1994).

### 2.4 Missing Data and Techniques for infilling missing data for river flows

A major source of error in stream gauging records is lost or missing data that result from failure of the stream-side equipment. Infilling missing data methods make larger sets of data available for consideration when performing data analyses (Little and Rubin, 1987). Conclusions are generally more accurate when drawn from larger sets of data. For example, it may be useful to apply data generation techniques to synthesize or generate hydrological data in cases where: (1) there are gaps in the series of observed data; (2) the observation period is short; (3) data are not available at the site of interest but in the neighbouring region.

The principal infilling methods for environmental data sets from which the most suitable will be selected are as follows:

- **Mean Value Infilling**: This uses means from known values as a substitute for missing values. It is a relatively simple method for infilling missing data. Shih and Cheng (1989) used this method for infilling missing solar radiation data because they concluded that this method was easy to use and provides accuracy equal to the standard error of the mean. The major problem with mean value infilling is that it is a smoothing process and may result in underestimating the inherent variability of the data (Galpin, 1990).

- **Hot Deck Infilling**: Hot deck infilling involves substituting individual values drawn from similar responding units. An example of hot deck infilling for a watershed monitoring program is to replace a missing data entry in one station with a value from another station of similar watershed characteristics for the same time period.

Another variant of the hot deck infilling method (used by Wallis et al., 1991) is where missing data are infilled using a neighbouring station value multiplied by the ratio of the long term monthly mean at the target site to the long term mean at the neighbouring (or reference) site.
The drainage area ratio method, utilized for estimating lost stream flow data, is another variant of the hot deck infilling. Because of its simplicity this method is widely used by hydrologists. This method estimates lost stream flow data by multiplying the stream flow data of the reference station against the ratio of the target and reference watershed drainage areas. The performance of the drainage area method may improve with an increased similarity of the two watersheds (e.g., morphology, land use, imperviousness, drainage area, etc.)

- **Intra-Station Interpolation**: This method replaces missing data by use of interpolation on related known data collected at the same station. For example, if the date of a missing data is known, a linear interpolation may be performed using: (1) the date of the missing data event; (2) the date and value of the previously observed sampling event; and (3) the date and value of the following observed sampling event. This method is recommended by Rantz *et al.* (1982) for the infilling of missing flow data for a stream in a period of low or medium flow recession.

A variation of intra-station linear interpolation is the method of "last value carried forward" (Little and Rubin, 1987). This method is simple to implement and simply replaces the missing value with the last known recorded value for that class.

### 2.5 Comparison of infilled data sets with the collected data

The larger sets of stream flow data made available for analysis by the chosen infilling methods above can then be compared to the existing data in order to determine whether there is a significant difference between the two data sets.

#### 2.5.1 Student’s t-test

The t-test is one of the most commonly used techniques for testing a hypothesis on the basis of a difference between sample means. Explained in layman’s terms, the t test determines a probability that two populations are the same with respect to the variable tested.

For the t-test for independent samples you do not have to have the same number of data points in each group. The t test can be performed knowing just the means, standard deviation, and number of data points.

The two sample t test yields a statistic \( t \), in which:

\[
t = \frac{|\bar{x}_1 - \bar{x}_2|}{\sqrt{A + B}}
\]

...(1)

Where, 
\[
A = \frac{(n_1 + n_2)}{n_1 n_2}
\]

...(2)

\[
B = \frac{[(n_1 - 1)s_1^2 + (n_2 - 1)s_2^2]}{[n_1 + n_2 - 2]}
\]

...(3)
X-bar, of course, is the sample mean, \( n_1 \) and \( n_2 \) are the sample sizes of each respective sample and \( s \) is the sample standard deviation. Note that the numerator of the formula is the difference between means. The denominator is a measurement of experimental error in the two groups combined. The wider the difference between means, the more confident you are in the data. The more experimental error you have, the less confident you are in the data. Thus the higher the value of \( t \), the greater the confidence that there is a difference.

The value \( t \) is just an intermediate statistic. Probability tables have been prepared based on the \( t \) distribution. To use the tables, the critical value that corresponds to the number of degrees of freedom is found (degrees of freedom = number of data points in the two groups combined, minus 2). If \( t \) exceeds the tabled value, the variables being tested are significantly different at the confidence level that was assumed.

## 2.6 History of Water Treatment in Uganda

### 2.6.1 Introduction

This section of the literature review covers a detailed description of the history of water treatment in Uganda, the various treatment stages in a typical water treatment plant and in Masaka Treatment Plant in particular as well as a review of the data to be utilised.

The history of water treatment is still being written as discoveries continue to document its origin. There is evidence, however, that even in ancient times people saw the importance of treating water in some way before drinking it.

The first piped water systems in Uganda were completed during the colonial period in the 1930s. The construction of new facilities increased from 1950 to 1965 under the framework of large national development programs (David, 2006). Later, the existing systems were partly maintained and no new facilities were constructed until 1990. According to a UN-Water document, by 1990 the urban water infrastructure served less than 10% of the population in large towns (UN-Water, 2006). Around the end of the 1980s, international donors began to invest substantial financial resources to rehabilitate and renew the water network in Kampala. Although the financial support helped to rehabilitate the infrastructure, the commercial performance of NWSC was still unsatisfactory.

The National Water and Sewerage Corporation (NWSC) was created as a government-owned parastatal organization in 1972 under the national administration of Idi Amin Dada, serving only the capital Kampala as well as Entebbe and Jinja (Muhairwe, 2003). Subsequently its service area gradually grew to incorporate large and mid-sized towns all over Uganda, reaching a total of 23 cities and towns in 2008. In 1995 and 2000, it was reorganized under the NWSC Statute and NWSC Act, giving it substantial operational autonomy and the mandate to operate and provide water and sewerage in areas entrusted to it, on a sound, commercial, and viable basis. In 2007 it provided services to 1.8 million people out of 2.5 million in Kampala, Jinja/Lugazi, Entebbe, Tororo, Mbale, Lira, Gulu, Masaka, Mbarara, Kabale, Kasese and Fort Portal, Bushenyi/Ishaka, Soroti Arua, Masindi, Malaba, Iganga, Hoima, and Mubende. The smallest town served, Hoima, has a population of only 9,000. The NWSC operates under the Ministry of Water and Environment.
In small towns with a population between 5,000 and 30,000, facilities are owned and managed by Local Governments, supported by the Ministry of Water and Environment. Many have created Water Authorities, which contract out water services under 3-year contracts to local private operators since about 2000. By 2010, these operators were 80 in number and served 35,000 connections. The Association of Private Water Operators (APWO) says that the contracts are too short to compensate the small, local private operators for their initial efforts in setting up their operations. Nevertheless, the small operators have been fairly successful (Global Water Intelligence, 2010).

In rural areas, Local Governments at district levels are responsible for the adequate operation and maintenance of water systems. (Ministry of water and Environment, 2006).

2.7 Stages in Typical Municipal Water Treatment

The water treatment processes developed in the 19th century and refined during the 20th century are simple in nature. However, technologists have since developed ways of making these processes happen faster, in a smaller area and in a more controlled way at lower cost. These earlier technologies are referred to as traditional or conventional technologies to distinguish them from technologies developed more recently (Australia Cooperative Research Centre for water quality and treatment, 2006).

There are a great variety of water treatment processes, although only a few are applied in most situations. Changes in raw water quality can affect the efficiency of treatment processes. Depending on local and seasonal situations, each water treatment plant encounters different ranges of raw water quality (LeChevallier, 2004).

There are three principal stages in municipal water treatment as given below:

1. **Primary Treatment**: Collecting and screening including pumping from rivers and initial storage.
2. **Secondary Treatment**: removal of fine solids and majority of contaminants using filters, coagulation, flocculation and membranes.
3. **Tertiary treatment**: pH adjustment, carbon treatment to remove taste and smells, disinfection and temporary storage to allow the disinfecting agent to work.

Primary Treatment

- **Pumping and containment**: The majority of water must be pumped from its source or directed into pipes or holding tanks. To avoid adding contaminants to the water, this physical infrastructure must be made from appropriate materials and constructed so that accidental contamination does not occur (New World Encyclopaedia, 2008).
- **Screening**: The first step in purifying surface water is to remove large debris such as sticks, leaves, trash and other large particles which may interfere with subsequent treatment steps.
• **Storage:** Water from rivers may also be stored in bankside reservoirs for periods between a few days and many months to allow natural biological purification to take place. This is especially important if treatment is by slow sand filters. Storage reservoirs also provide a buffer against short periods of drought or to allow water supply to be maintained during repository pollution incidents in the source river (New World Encyclopaedia, 2008).

• **Pre-conditioning:** Many rivers rich in hardness salts are treated with soda-ash (sodium carbonate) to precipitate calcium carbonate out utilizing the common ion effect (Wikipedia, 2007).

• **Pre-chlorination:** In many plants with cases of acid water and thus high iron content in the source, the incoming water is chlorinated so that treatment can be supplemented by a correction of the pH. Thus, the ferrous iron is oxidized in ferric iron, which precipitates in iron hydroxide, \( \text{Fe(OH)}_3 \).

**Secondary Treatment**

There are a wide range of techniques that can be used to remove the fine solids, microorganisms and dissolved inorganic and organic materials. The choice of method will depend on the quality of the water being treated, the cost of the treatment process and quality standards expected of the processed water.

• **pH adjustment:** If the water is acidic, lime or soda ash is added to raise the Ph. Lime is the more common of the two additives because it is cheaper, but it also adds to the resulting water hardness. Making the water slightly alkaline ensures that coagulation and flocculation processes work effectively (Palanna, 2009).

• **Coagulation and Flocculation:** Together coagulation and flocculation are treatment processes that work by using chemicals which effectively “glue” small suspended particles together, so that they settle out of the water or stick to sand or other granules in a granular media filter. Many of the suspended water particles have a negative electrical charge. The charge keeps particles suspended because they repel similar particles. Coagulation works by eliminating the natural electrical charge of the suspended particles so they attract and stick to each other (Palanna, 2009). The joining of the particles so that they will form larger particles is called flocculation. The larger formed particles are called floc. The coagulation chemicals are added in a tank (often called a rapid mix tank), which typically has rotating paddles. In most treatment plants, the mixture stays in the tank for 10 to 30 seconds to ensure full mixing. The amount of coagulant that is added to the water varies widely due to the different sources water quality (Palanna, 2009).

One of the more common coagulants used is aluminium sulphate sometimes called Alum. Aluminium sulphate reacts with water to form flocs of aluminium hydroxide. Coagulation with aluminium compounds may leave a residue of aluminium in the finished water. This is normally 0.1 to 0.15mg/L. It has been established that Aluminium can be toxic to humans at high concentrations of greater than 0.2mg/L (New World Encyclopaedia, 2008).
Iron (II) sulphate or iron(III) chloride are other common coagulants. Iron(III) coagulants work over a larger pH range than aluminium sulphate but are not effective with many source waters. Other benefits of iron (III) are lower costs and in some cases slightly better removal of natural organic contaminants from some waters. Coagulation with iron compounds typically leaves a residue of iron in the finished water which may impart a slight taste to the water, and may cause brownish stains on porcelain fixtures. The trace levels of iron are not harmful to humans, and indeed provide a needed trace mineral. Because the taste and stains may lead to customer complaints, aluminium tends to be favoured over iron for coagulation (Palanna, 2009).

Cationic and other polymers can also be used especially for high rate filtration (> 2.71 l/m²s). They are often called coagulant aids used in conjunction with other inorganic coagulants. The long chains of positively charged polymers can help to strengthen the floc making it larger, faster settling and easier to filter out. The main advantages of polymer coagulants and aids are that they do not need the water to be alkaline to work and that they produce less settled waste than other coagulants, which can reduce operating costs. The drawbacks of polymers are that they are expensive, can block sand filters and that they often have a very narrow range of effective doses (Palanna, 2009).

- **Sedimentation:** Water exiting the flocculation tank enters the sedimentation tank, also called a clarifier or settling basin. It is a large tank with slow flow, allowing floc to settle to the bottom. The sedimentation tank is best located close to the flocculation tank so the transit between does not permit floc to break up. Sedimentation basins can be in the shape of a rectangle where water flows from end to end or circular where flow is from the centre outward (New World Encyclopaedia, 2008).

Sedimentation tank outflow is typically over a weir so only a thin top layer-furthest from the sediment exits. The amount of floc that settles out of the water is dependent on the time the water spends in the tank and depth of the tank. The retention time of the water must therefore be balanced against the cost of a larger basin. The minimum clarifier retention time is normally 4 hours. A deep tank will allow more floc to settle out than a shallow one. This is because large particles settle faster than smaller ones, so large particles bump into and integrate smaller particles as they settle. The recommended clarifier depths are 3-4.5m (Cox, 1964). In effect, large particles sweep vertically through the tank and clean out smaller particles on their way to the bottom (New World Encyclopaedia, 2008).

As particles settle to the bottom of the basin a layer of sludge is formed on the floor of the tank. This layer of sludge must be removed and treated. The amount of sludge that is generated is significant, often 3%-5% of the total volume of water that is treated (Palanna, 2009). The cost of treating and disposing of the sludge can be a significant part of operating cost of a water treatment plant. The tank may be equipped with mechanical cleaning devices that continually clean the bottom of the tank or the tank can be taken out of service when the bottom needs to be cleaned (Khanna, 1996).
An increasingly popular method of floc removal is by dissolved air floatation (Palanna, 2009). A proportion of clarified water, typical 5-10% of throughput, is recycled and air is dissolved in it under pressure. This is injected into the bottom of the clarifier tank where tiny air bubbles are formed which attach themselves to the floc particles and float them to the surface. A sludge blanket is formed which is periodically removed using mechanical scrapers. This method is very efficient at floc removal and reduces loading on filters, however it is unsuitable for water sources with a high concentration of sediment (Masters, 1998).

Factors influencing clarification performance include the surface loading rate (expressed as flow rate per unit surface area of the clarification basin), appropriate dose rate, poor process control with little monitoring, shear of formed floc, the size and shape of the tank, flow velocity, inadequate sludge removal and physicochemical characteristics of the water (USEPA, 1991; Gregory, Zabel & Edzwald, 1999). Adequate sludge removal is important because sludge accumulation reduces the volume of the clarification basin and can increase the velocity of the flow through the basin (LeChevallier et al., 2004).

- **Filtration:** After separating most floc, the water is filtered as the final step to remove remaining suspended particles and unsettled floc. The most common type of filter is a rapid sand filter. Water moves vertically through sand which often has a layer of activated carbon or anthracite coal above the sand. The top layer removes organic compounds including taste and odour. The space between the sand particles is larger than the smallest suspended particles, so simple filtration is not enough. Most particles pass through surface layers but are trapped in pore spaces or adhere to sand particles. Effective filtration extends into the depth of the filter. This property of the filter is key to its operations: if the top layer of sand were to block all the particles, the filter would quickly clog (Palanna, 2009).

To clean the filter, water is passed quickly upward through the filter, opposite the normal distribution (called backwashing) to remove embedded particles. Prior to this, compressed air may be blown up through the filter bottom to break up the compacted filter media to aid the backwashing process; this is known as air scouring. This contaminated water can be disposed of, along with the sludge from the sedimentation tank, or it can be recycled by mixing with the raw water entering the plant. Some water treatment plants employ pressure filters. These work on the same principle as rapid gravity filters differing in that filter media is enclosed in a steel vessel and the water is forced through it under pressure (Palanna, 2009).

Slow sand filters may be used where there is sufficient land and space. These rely on biological treatment processes for their action rather than physical filtration. Slow sand filters are carefully constructed using graded layers of sand with the coarsest at the top and finest at the base. Drains at the base convey treated water away for disinfection. Filtration depends on the development of a thin biological layer on the surface of the filter. An effective slow sand filter may remain in service for many weeks or even months if the pre-treatment is well designed and produces an excellent quality of water which physical methods of treatment rarely achieve (Palanna, 2009).
Tertiary Treatment

Disinfection is normally the last step in purifying drinking water. Water is disinfected to destroy any pathogens which pass through the filters. Possible pathogens include viruses, bacteria, and protozoa. In most developed countries, public water supplies are required to maintain a residual disinfecting agent throughout the distribution system, in which water may remain for days before reaching the consumer (Palanna, 2009). Following the introduction of any chemical disinfecting agent, the water is usually held in temporary storage – often called a contact tank or clear well to allow disinfecting action to complete.

- **Chlorine**: The most common disinfection method is some form of chlorine or its compound such as chloramines or chlorine dioxide. Chlorine is a strong oxidant that kills many micro-organisms. Because chlorine is a toxic gas, there is a danger of release associated with its use. This problem is avoided with the use of sodium hypochlorite, which is a relatively inexpensive solid that releases free chlorine when dissolved in water. Handling the solid, however requires greater routine human contact through opening bags and pouring than the use of gas cylinders which are more easily automated. Both disinfectants are widely used despite their respective drawbacks. A major drawback to using chlorine gas or sodium hypochlorite is that they react with organic compounds in the water to form potentially harmful levels of chemical by-products trihalomethanes (THMs) and haloacetic acids, both of which are carcinogenic. The formation of THMs and haloacetic acids is minimized by effective removal of as many organics from the water as possible before disinfection. Although chlorine is effective in killing bacteria, it has limited effectiveness against protozoans that form cysts in water (Masters, 1998).

- **Chlorine dioxide**: is another fast acting disinfectant. It is, however, rarely used because it may create excessive amounts of chlorate and chlorine, both of which are regulated to low allowable levels. Chlorine dioxide also poses extreme risks in handling: not only is the gas toxic, but it may spontaneously detonate upon release to the atmosphere in an accident (Masters, 1998).

- **Chloramines**: are another chlorine based disinfectant. Although chloramines are not as effective as disinfectants, compared to chlorine gas or sodium hypochlorite, they are less prone to form THMs or haloacetic acids. It is possible to convert chlorine to chloramines by adding ammonia to the water along with chlorine. The chlorine and ammonia react to form chloramines. Water distribution systems disinfected with chloramines may experience nitrification, wherein ammonia is used as a nitrogen source for bacterial growth, with nitrates being generated as a by-product. (Masters, 1998)

2.8 Water Treatment Processes at Masaka Waterworks

Masaka water works has its source as the Nabajjuzi swamp. The Water Treatment Works is divided into two plants with one plant of capacity 3000m$^3$ treating and pumping water to Bwala reservoir and the other plant of capacity 5000m$^3$ treating and pumping water to Boma reservoirs (NWSC, 2012). However, both plants are arranged as described in the subsequent write up:
• **Intake/source:** The intake is in Nabajjuzi swamp where a portion of the swamp has been opened up and water is well exposed to allow for the natural oxidation of the water (Figure 2-1).

![Figure 2-1: Intake at Masaka Water Treatment Works](image)

• **Screens and foot valves:** Screens are used to remove large floating objects e.g. leaves and rags to protect the raw water pumps as they suck water through the foot valves.

• **Raw water pumps:** These pump raw water from the intake through the foot valves to the aerators for both the Bwala and Boma treatment lines.

• **Aerators/Pre-chlorination:** In this process, raw water is exposed to more air and chlorine is added to the raw water to enable the minerals like iron to be oxidized. It also enables odour and smell removal. Boma treatment line has two aerators while Bwala treatment line has four aerators.

• **Flocculators/Clarifiers:** Aluminium sulphate (Alum) is added to the water and coagulation takes place. Flocs form bigger particles and settle down to the bottom of the clarifiers along with suspended materials and pathogens. Partially treated water then passes on to the filters. There are two clarifiers along the Boma line and six clarifiers along the Bwala treatment line. However, the Bwala clarifiers are shallow at depths of 2m often causing water currents to easily disturb the settled sludge in the clarifiers. In addition, the Bwala clarifiers have multiple Alum dosing points at each clarifier rather than a single online dosing point. The clarifiers are cleaned once every 3 months in Masaka water works.
• **Filters:** Water percolates through several layers of filter media (aggregates, gravel and fine sand) such that more pathogens are removed. Draw off pipes at the bottom of filters lead filtered water to the contact tanks. The plant has two filters at both the Boma and Bwala treatment lines and these are backwashed twice a day. However, resanding of the filters was only done once in the period 2006 – 2009.

• **Disinfection and pH correction:** Water enters the contact tank where it is dosed with chlorine and soda ash solutions. Chlorine is for final disinfection in order to kill off any remaining pathogens while the soda ash solution is used for PH correction from acidic to neutral state.

• **Contact Tank:** In this tank treated water is retained for 30 minutes to enable complete disinfection of the water before its passed on to the clear water wells.

• **Clear water wells:** These are large reservoirs (approx. 150m$^3$ each) for the storage of the treated water before pumping it to distribution reservoirs.

• **High lift pumps:** These pump treated water to the elevated distribution reservoirs for final distribution to customers.

**Key water quality characteristics of River Nabjuzu**

Water from River Nabajjuzi fits the description of Twort et al (1987) as “difficult water”. This arises from the fact that while other water works treat water from lakes and other fast flowing rivers, Masaka plant treats water from a very slow moving swampy river and the result is that its water has unique characteristics.

The raw water quality of Masaka water works is characterized by softness, colour from organic compounds, dissolved iron, low pH and low dissolved oxygen. This could be explained by the fact that, as the water flows through the papyrus and other vegetation it picks colour from decomposing vegetation. The decomposition process consumes dissolved oxygen (Khanna, 1996).

The conditions of limited oxygen, reduce the redox potential (Eh) in the water, which encourages reduction of iron III to iron II, which is more soluble. The dissolved organic compounds are mainly humic and fulvic acids which are negative and cause a reduction in the pH. The iron is suspected to form complexes with the organic acids (Mitsch 1994). The result is that the water most likely contains both dissolved iron II that is in free ionic form, and iron II or III that is in complex combination with the organic compounds. The iron and organic complex combinations give the water its characteristic yellowish brown colour.

During the dry season (June-August and December-February), the water level and river flow reduces due to evapotranspiration, the dominant hydrological factor that accounts for water loss during dry spells in wetlands (Mitsch 1994). This increases the concentration of both the colour and iron through the loss of solvent (water). The colloidal suspended particles concentration also rises, increasing the turbidity.
During the rainy season, the rain water seeps into the surface water, bringing with it oxygen from the air and also raising the water level by addition of water volume. The net effect is to reduce the concentration of both the iron and colour by dilution but also precipitate some of the dissolved iron that is in ionic form, after oxidation by the oxygen coming with the rain water. The precipitated iron III is then partly removed by sedimentation, while the iron remaining in solution is diluted. Turbidity also gets diluted by incoming rainwater and reduces.

The turbidity is a result of suspended particles and colloids that have passed through the papyrus vegetation and reached the intake. Most of the larger suspended particles settle at the bottom of the river as silt, while the colloidal particles remain in the water and are taken into the plant contributing to the turbidity of the raw water (Khanna, 1996).

During the rainy season, the particles swept by runoff into the river do not contribute much at the intake because swamp vegetation filters them out. Instead the inherent colloidal particles get diluted by the incoming water and hence their concentration falls. This is the reason why turbidity of raw water reduces during rainy season, contrary to the common occurrences in other rivers where the rainy season comes with a high increase in turbidity.

2.9 Commentary

The most essential information required for this study includes streamflow data for Nabajjuzi river from Water Resources Management Department, Entebbe and physico-chemical test results of water from the NWSC Masaka waterworks, Area store’s records of chemical quantities issued out for use in water treatment, and expenditure on chemicals viz-a-viz operating expenditure. The primary source of this data will be from the historical flow records of Nabajjuzi River at WRMD and reports from the NWSC.

Based on the literature and data reviews covered above, the appropriate method of data infilling as described in the literature review will be chosen for use in ensuring availability of adequate and consistent data. The infilled data will then be compared to the original data set to confirm whether they differ. The right numerical data analysis technique will also be utilized to analyze the reliability of the water source for Masaka by comparing it to the current and future abstractions from the water treatment works.

In addition, the most cost effective treatment processes that optimise chemical usage will be recommended for use in Masaka water treatment works having thoroughly reviewed the various available treatment processes utilised elsewhere in the world.
3 METHODOLOGY

The hydrological methods used in this study were desk top analytical techniques that primarily rely on recorded/historical hydrological data such as daily streamflow discharge data to determine the mean annual minimum 7-day low flow as well as the safe yield of River Nabajjuzi. The current and future abstraction rates as well as water quality characteristics are determined from NWSC Masaka production and quality data.

3.1 Review of Data

The streamflow data that was obtained from Water Resources Management Directorate (WRMD), Entebbe was for 7 years only i.e.1987-1993 as a result of the breakdown of the river gauging equipment along River Nabajjuzi since 1994. The data had to be extrapolated in order to make the sample long enough for adequate analysis. The raw water extraction and water treatment figures were obtained from NWSC – Masaka Water Treatment Plant, but the area did not have consistent water quality and abstraction records for the period prior to 2006. Data was obtained for the period 2006 – 2009 for use in this study.

Additional information regarding water quality characteristics and chemical consumption was also obtained from NWSC Central Laboratory in Bugolobi and the Masaka Area Laboratory.

3.2 River Flow Statistics

Station Data

The stream gauging records of Nabajjuzi river consisted of missing data as a result of failure of the stream-side gauging equipment located along Bukoba Road. However, discharge data for the neighbouring station No. 81259, R. Katonga along Kampala-Masaka road that has similar characteristics to River Nabajjuzi were also obtained and they consisted of larger, more consistent data sets. For both sets of data, the hydrological year start month was January. Infilling missing data methods had to be utilized to make larger sets of data available for consideration when performing data analyses for River Nabajjuzi.

Method of Data Infilling

The preferred method of choice for infilling missing data was the hot deck infilling method. The hot deck infilling method used involved a variant of the standard hot deck infilling method (Wallis et al., 1991) and was where missing data were infilled using a neighbouring station value multiplied by the ratio of the long term monthly mean at the target site to the long term mean at the neighbouring (or reference) site.

Comparison of infilled data set with the collected data

The infilled data obtained was then compared to the original data collected to determine whether they differ significantly. This is undertaken through the use of the student’s t-test which was applied to enable the data comparison.
Minimum Annual Flows Calculations

Following the infilling of all missing data, the minimum 7-day low flows from each year of record were obtained and examined. The mean annual minimum 7-day low flow (MAM-7) was calculated by dividing the sum of all annual minimum 7-day low flows by the number of years of record.

Simple statistics for the minimum annual flow data were calculated using functions in Ms Excel. These include average (= Average), standard deviation (= stdev), and coefficient of skew (=skew).

3.3 Determining the Safe yield of the River Nabajjuzi

The safe yield was calculated using the log Pearson type III method of frequency analysis, which is the recommended analytical frequency analysis method for annual maximum and minimum stream flow studies (USWRC, 1977).

The basic working equation for all frequency analysis methods is as follows:

Any $X$ can be written as $X = \bar{x} + \Delta x$ .................................................................(4)

and define $K = \Delta X/s$ .................................................................(5)

Where $\bar{x} = \frac{1}{n} \sum_{i=1}^{n} x_i$ is the sample mean, $\Delta x$ is the deviation from the mean and $S$ the standard deviation, where $S = \sqrt{\frac{\sum_{i=1}^{n} (x_i - \bar{x})^2}{n-1}}$, and $K$ the frequency factor.

Then $X = \bar{x} + Sk$.

Hence for a given design value $X_T$, we can write $X_T = \bar{x} + K_T S$. The frequency factor $K_T$ depends on the probability distribution being used and on the return period, $T$.

In the Log Pearson Type III distribution the design value $x$ was first transformed to $Y = \log x$ and it has three parameters, $\lambda$, $\beta$ and $\varepsilon$, which were estimated through calculation of mean, standard deviation and coefficient of skewness.

The coefficient of skewness $C_s$ is given by: $C_s = \frac{n \sum (x_i - \bar{x})^3}{(n-1)(n-2)S^3}$ ........................................(6)
The frequency factor \( K_T = z + \left( z^2 - 1 \right) k + \frac{1}{3} \left( z^3 - 6z \right) k^2 - \left( z^2 - 1 \right) k^3 + zk^4 + \frac{1}{3} k^5 \) \( \text{...(7)} \)

Where \( k = \frac{C_s}{6} \)

\[
z = w = \frac{2.516 + 0.8029w + 0.01033w^2}{1 + 1.4328w + 0.1893w^2 + 0.001311w^3}
\]

\( w = \left[ \ln \left( \frac{1}{p^2} \right) \right]^{1/2} \)

\( \text{And} \quad p = \frac{1}{T} \)

The procedure for frequency analysis using the log Pearson type III distribution was as follows:

- Transformation of \( x \) to \( Y = \log x \)
- Compute the mean, \( \bar{y} \)
- Compute the standard deviation, \( S_y \)
- Compute the coefficient of skewness, \( C_s \)
- Compute \( K_T \) by equations or read from tables.
- Compute \( y_T = \bar{y} + K_T S \)
- Compute \( X_T = 10^{y_T} \) or \( X_T = e^{y_T} \)

### 3.4 Determining Current and Future Abstraction Rates

**Current abstraction rate**

To obtain the current abstraction rate, the abstraction data for each month of the latest complete year of record from Nabajjuzi Water Works were obtained and a mean annual abstraction rate computed. This was then compared to the safe yield and the MAM7 (taken as the abstraction limits).

A graphical plot of abstraction versus month was done to have a visualization of the annual cycle (Figure 4-2 and Figure 4-3).
Future abstraction rate

The future abstraction rate was obtained by estimating the future water demand for the water supply scheme for a given design period. To calculate this future demand the following was done:

- Determination of the number of consumers falling within different consumer categories in the base year of the design period.
- The population projection at the end of the design period was calculated. In Uganda, the population that will need to be supplied in \( n \) years can be calculated using the simple growth method as shown below:

\[
P_n = P_i (1+r)^n
\]

Where: \( P_n \) = Projected future population after \( n \) years, the “design population”, \( P_i \) = Initial population in the “base” year, \( r \) = Estimated annual population growth rate expressed as a percentage, and \( n \) = Number of years in the design period (DWD, 2000).

- The average day unit water demand figures for the various consumer categories concerned, like institutional, domestic and industrial demand were determined.
- The future daily abstraction rate based on the calculated demand figures was calculated.
- This was then compared to the safe yield and the MAM7 (abstraction limits).

The average day unit water demand figures, which were used together with the numbers of domestic, industrial, commercial and other consumers to calculate the “average day demands”, were derived from the guidelines shown in Table 3-1 and can be adjusted to suit the conditions on the ground.

**Table 3-1: Average day unit water demand** Source: DWD Water Supply Design Manual (2000)

<table>
<thead>
<tr>
<th>Consumer Type</th>
<th>Consumption</th>
</tr>
</thead>
<tbody>
<tr>
<td>Domestic</td>
<td></td>
</tr>
<tr>
<td>Stand Pipe</td>
<td>20 litres/person/day</td>
</tr>
<tr>
<td>Yard Tap</td>
<td>40 litres/person/day</td>
</tr>
<tr>
<td>House Connection</td>
<td>200 litres/person/day; High Income Consumers (with multiple fixtures and a garden tap; includes car washing and garden watering)</td>
</tr>
<tr>
<td></td>
<td>100 litres/person/day; Medium Income Consumers (with a kitchen sink, one or two WCs, showers, bathtubs, and hand wash basins).</td>
</tr>
<tr>
<td></td>
<td>50 litres/person/day; Low Income Consumers (with limited fixtures, a WC and one or two taps).</td>
</tr>
</tbody>
</table>
3.5 Raw water quality characteristics of source and their impact on chemical consumption.

The raw water quality characteristics of the Masaka water source were obtained from the available water quality reports of the period 2006-2009. An assessment of the impact of the fluctuations of the raw water source on chemical use was done by plotting chemical usage versus months.

Data for the chemical consumption at Masaka Water Treatment Works was obtained and an assessment of the consumption variations and the treated water pumped for the two treatment works of Boma and Bwala in 2007, 2008 and 2009 was undertaken.

The possible causes of the high chemical consumption would be assessed and necessary interventions suggested to optimise chemical use.
3.6 Data Analysis.

The raw water quality data and water pumpage data for the period 2006 - 2009 were analyzed using graphical techniques and the variations in the raw water quality characteristics of pH, colour, turbidity and iron were studied to determine their peak and minimum seasonal occurrences.

The water pumped and chemicals used data was also analysed using graphs and compared to the variations in water quality characteristics to determine whether there was any relation to them.

The chemical consumption for Masaka water works was calculated as well as the unit cost per cubic meter of water produced per month. This was then compared to the chemicals unit cost per cubic meter of water produced at another similarly sized town’s water works as well as to the average global NWSC unit cost per cubic meter of water produced.
4 RESULTS AND DISCUSSIONS

4.1 INTRODUCTION

This Chapter is a presentation of the study findings and the associated discussions.

4.2 RIVER FLOW STATISTICS

Data infilling

The missing data of River Nabajjuzi was infilled using data from River Katonga to produce a larger set of data for analysis and the long term monthly means of the river Nabajjuzi were also calculated. (Annex A1 and A2). The long term annual mean for River Katonga used for infilling was calculated as 2.687 m³/s. Figure 4-1 below shows the long term monthly means for River Nabajjuzi.

\[
\text{Comparison of the collected data to the infilled data of River Nabajjuzi stream flows}
\]

Comparison of the infilled stream flow data to the collected stream flow data of River Nabajjuzi was undertaken by use of the student’s t-test as follows:

The null hypothesis \( H_0 \) = “There is no significant difference between the mean of the collected data and the mean of the infilled data series”.

i.e. \( H_0 : \mu_1 = \mu_2 \)
The two sets of data series were assumed to be independent and follow a normal distribution. The level of significance is also taken as 5% (95% confidence limit), i.e. $\alpha = 0.05$. The ‘two tailed’ $t$ – test technique was then utilised.

The sample size for the collected streamflow data series for River Nabajjuzi is $n_1 = 2557$; the standard deviation $s_1 = 0.093$, and the mean $\bar{x}_1 = 0.190$.

The sample size for the infilled streamflow data series for River Nabajjuzi is $n_2 = 15,340$, the standard deviation $s_2 = 0.210$, and the mean $\bar{x}_2 = 0.188$.

The statistic $t$ as indicated in equation (1) was got by:

$$t = \frac{|\bar{x}_1 - \bar{x}_2|}{\sqrt{A \times B}}$$

Where

$$A = \frac{(n_1 + n_2)}{n_1 \times n_2}$$

$$= \frac{(2557 + 15340)}{2557 \times 15340} = \frac{17897}{39,224,380} = 0.00046 \ldots \ldots (11)$$

$$B = \frac{[(n_1 - 1)s_1^2 + (n_2 - 1)s_2^2]}{[n_1 + n_2 - 2]}$$

$$= \frac{[(2556) \times 0.093^2 + (15339) \times 0.210^2]}{[2557 + 15340 - 2]} = 0.039 \ldots \ldots (12)$$

Therefore

$$t = \frac{|0.190 - 0.188|}{\sqrt{0.00046 \times 0.039}} = 0.472 \ldots \ldots \ldots \ldots \ldots (13)$$

Probability tables were used (Annex G) to determine the critical value that corresponds to the number of degrees of freedom.

Hence, degrees of freedom $df = [n_1 + n_2 - 1] = 17,895$ and from the probability tables, the critical value observed at $p = 0.05$ was 1.96.

Since $0.472 < 1.96$, the null hypothesis $H_0$ could not be rejected and we were 95% or more certain that there was no significant difference between the means of the collected data and infilled data series.

**Mean Annual minimum 7-day low flow**

The minimum 7-day low flows from each year of record were obtained from the larger data set and examined for the 41 years period. The mean annual minimum 7-day low flow (MAM-7), which is the maximum abstraction limit, was calculated (Annex B).

Thus the estimated yield of the River Nabajjuzi during periods of low flow which is also the maximum allowable abstraction limit that can be permitted by the regulatory authority for the river was equal to:

$$MAM \, 7 = 0.094 \, m^3/s = 8122 \, m^3/d.$$
Thus the design capacity for the Masaka Water Treatment Works of 8000m³/d (NWSC, 2011) falls well within the maximum abstraction limits as determined above and the treatment plant, even operating at 100% capacity utilisation, would not surpass the abstraction limits for River Nabajjuzi.

4.3 DETERMINATION OF SAFE YIELD OF RIVER NABAJJUZI

The safe yield, defined as the yield maintainable through a once-in-50-years drought was taken as the historical extreme low flow period for the river Nabjuzi. It was calculated using the log-Pearson type III method of frequency analysis, which is the recommended analytical frequency analysis method for annual maximum and minimum stream flow studies (USWRC, 1977).

Simple statistics like the mean, standard deviation and coefficient of skew for the minimum annual flow data were calculated (Annex B).

\[
\bar{y} = -2.889
\]

\[
S_y = 1.383
\]

\[
C_s = -2.001
\]

and the return period of the drought \( T = 50 \) years.

Hence the safe yield \( X_T \) was calculated using the formula:

\[
Y_T = \log X_T = \bar{y} + K_T S_y
\]

where \( Y_T = \log X_T \);

The frequency factor is got from the following equation:

\[
K_T = z + \left( z^2 - 1 \right) k + \frac{1}{3} \left( z^3 - 6 z^2 \right) k^2 - \left( z^2 - 1 \right) k^3 + z k^4 + \frac{1}{3} k^5
\]

\[
\text{Where } k = \frac{C_s}{6} = -2.001/6 = -0.3335
\]

\[
z = w - \frac{2.516 + 0.8029w + 0.01033w^2}{1 + 1.4328w + 0.1893w^2 + 0.00131w^3} = 2.054
\]

Since \( w = \left[ \ln \left( \frac{1}{p} \right) \right]^{1/2} = 2.797 \) and \( p = \frac{1}{T} = 0.02 \)

Hence \( K_T = 0.749 \).
Thus from \( Y_T = y + K_T S \);

\[
Y_T = -2.889 + (0.749 \times 1.383)
\]

\[
Y_T = -1.853
\]

But the safe yield \( X_T = e^{-1.853} = 0.156 \text{ m}^3/\text{s} \)

Therefore the safe yield or steady supply that can be maintained by River Nabajjuzi through a once in 50 years drought (historical extreme low flow period) is \( 0.156 \text{ m}^3/\text{s} = 13,478 \text{ m}^3/\text{d} \).

Thus, it can clearly be seen that even if the treatment plant was operating at 100% capacity utilization (8000m\(^3\)/d), there would be enough water in River Nabajjuzi to meet the abstraction demand.

### 4.4 DETERMINATION OF CURRENT WATER ABSTRACTION RATE

The raw water abstracted values and treated water values from 2006-2008 were obtained (Annex C) and were presented in Figures 4-2 and 4-3.

![Figure 4-2: Raw water abstracted values 2006-2008](image-url)
The maximum raw water abstracted for each of the respective years was as follows:

- 2006 = 142,000 m³/month
- 2007 = 140,000 m³/month
- 2008 = 132,000 m³/month

![Graph of Volume vs Months from January to December for 2006, 2007, and 2008.]

**Figure 4-3: Treated water produced values 2006-2008**

The maximum treated water produced values for each of the respective years was as follows:

- 2006 = 136,000 m³/month
- 2007 = 127,000 m³/month
- 2008 = 120,000 m³/month

Hence, the maximum monthly water abstracted from 2006-2008 was **142,000 m³/month or 4,733 m³/d.** This is within the calculated maximum abstraction limit for River Nabajjuzi of 8122 m³/d and the safe yield of 13,478 m³/d. This implies that River Nabajjuzi is reliable enough to meet Masaka town’s current water demand.

### 4.5 DETERMINATION OF FUTURE WATER ABSTRACTION RATE

Based on the data obtained from NWSC Masaka, the number of accounts for the different consumer categories at December 2008 as well the average growth rate of accounts per category from 2006-2008 are presented in table 4-1.
Table 4-1: Number of Accounts in Masaka 2008 and average growth rate per consumer category (Source: NWSC, 2008)

<table>
<thead>
<tr>
<th>Consumer Category</th>
<th>Number of Accounts</th>
<th>Average Account growth rate per category</th>
</tr>
</thead>
<tbody>
<tr>
<td>Domestic</td>
<td>4371</td>
<td>8%</td>
</tr>
<tr>
<td>Public Stand posts (PSP)</td>
<td>147</td>
<td>12%</td>
</tr>
<tr>
<td>Commercial/Industrial</td>
<td>728</td>
<td>14%</td>
</tr>
<tr>
<td>Institutional</td>
<td>265</td>
<td>0.3%</td>
</tr>
</tbody>
</table>

Taking 2008 as the base year and using the calculated 3 year running average growth rates for the different consumer categories (Annex F), then the number of future accounts was calculated for both the future year and ultimate year.

For the future year (design period of 10 years) the number of accounts were:
- Domestic = 4371 × (1.08)^10 = 9437 Accounts
- Public Stand Posts = 147 × (1.12)^10 = 457 Accounts
- Commercial/Industrial = 728 × (1.14)^10 = 2699 Accounts
- Institutional = 265 × (1.003)^10 = 273 Accounts

For the ultimate year (design period of 20 years) the number of accounts were:
- Domestic = 4371 × (1.08)^20 = 20,373 Accounts
- Public Stand Posts = 147 × (1.12)^20 = 1418 Accounts
- Commercial/Industrial = 728 × (1.14)^20 = 10,005 Accounts
- Institutional = 265 × (1.003)^20 = 281 Accounts

The average day unit demand figures for the different consumer categories were obtained based on recommended guidelines and historical consumption patterns of the different categories in NWSC Masaka area and were presented in table 4-2:

Table 4-2: Selected average day unit demand figures per consumer category

<table>
<thead>
<tr>
<th>Consumer Category</th>
<th>Average day unit demand figures</th>
</tr>
</thead>
<tbody>
<tr>
<td>Domestic</td>
<td>50 l/c/d</td>
</tr>
<tr>
<td>Public Stand posts (PSP)</td>
<td>20 l/c/d</td>
</tr>
<tr>
<td>Commercial/Industrial</td>
<td>200 l/d</td>
</tr>
<tr>
<td>Institutional</td>
<td>50 l/c/d</td>
</tr>
</tbody>
</table>

In addition, it is assumed that the following numbers of persons are served per connection per consumer category (NWSC, 2010):
Domestic ................................................................. 5 persons/household/connection

Standpipe .............................................................. 200 persons per standpipe

**Institutions:**
Medium Towns .......................................................... 500 persons/institution

Thus the projected demand figures for the future design period of 10 years were:

- **Domestic** = 9437×50×5 = 2,359,250 l/d
- **Public Stand Posts** = 457×20×200 = 1,828,000 l/d
- **Commercial/Industrial** = 2699×200 = 539,800 l/d
- **Institutional** = 273×500×50 = 6,825,000 l/d

**TOTAL** = 11,552,050 l/d = 11,552 m$^3$/d

And the projected demand figures for the ultimate design period of 20 years were:

- **Domestic** = 2037.3×50×5 = 5,093,250 l/d
- **Public Stand Posts** = 1418×20×200 = 5,672,000 l/d
- **Commercial/Industrial** = 10005×200 = 2,001,000 l/d
- **Institutional** = 281×500×50 = 7,025,000 l/d

**TOTAL** = 19,791,250 l/d = 19,791 m$^3$/d

Thus the future water abstraction rate based on the water demand figures for Masaka Town in 2018 and 2028 will be 11,552 m$^3$/d and 19,791 m$^3$/d respectively.

The future demand outstrips the maximum abstraction limit of 8,122 m$^3$/d and the plant’s design capacity of 8000 m$^3$/d but is still within the safe yield of 13, 478 m$^3$/d. In addition, the ultimate year demand is greater than the maximum abstraction limit, the plant’s current design capacity as well as the safe yield of river Nabajjuzi.

This implies that, should the demand increase at the average growth rates as calculated per consumer category, then River Nabajjuzi will no longer be a viable source of water for Masaka town by 2018 as its abstraction limits would already be exceeded and any further abstraction beyond this would lead to damage to its eco-system.
4.6 WATER QUALITY EVALUATION AND ITS IMPACT ON CHEMICAL CONSUMPTION

4.6.1 Raw Water Quality Characteristics

Physico-chemical water quality data of the Masaka water source were obtained from NWSC Central Laboratory (Annex D) and an assessment of some of the major water quality parameters for the years 2007 – 2008 were presented in Figures 4-4 to 4-11.

a) PH

![Figure 4-4: Monthly average values for raw water and final water - 2007](image)

![Figure 4-5: Monthly average pH values for raw water and final water - 2008](image)
The pH of the raw water was low and the water was acidic with an annual average of about 6.0 for 2007 and 5.9 for 2008. The maximum recorded in 2007 and 2008 was 6.9 and 6.1 respectively, while the minimum recorded in 2007 and 2008 was 5.6 and 5.8 respectively.

On completion of treatment, the annual average pH of the final water was 7.2 and 7.0 for Boma and Bwala treatment lines respectively in 2007 while the annual average in 2008 was 7.3 and 7.2 for Boma and Bwala treatment lines. These all fall within the range of the WHO water quality standards of 6.5-8.5 (WHO, 1993).

The analysis showed that the raw water had a slightly lower pH during the dry seasons (December – February and June – August) than during the wet seasons. This low pH resulted into high soda ash consumption at the plant as it was needed to adjust the pH so as to fall within the range of acceptable water quality standards.

b) Colour

Figure 4-6: Monthly average colour values for raw water and final water - 2007
Colour which is one of the most problematic parameters of the raw water varied between a minimum of 161 Hazen units (on the platinum scale, Pt) and a maximum of 248 units with an average of 199 units as apparent colour in 2007. In 2008, the raw water varied between a minimum of 136 units and a maximum of 267 units with an average of 206 units.

On completion of treatment, the annual average colour of the final water was 6.6 Ptu and 21Ptu for Boma and Bwala treatment lines respectively in 2007 while in 2008 the annual average colour of the final water was 3.4 and 8.3 respectively.

The WHO water quality standards (WHO, 1993) for colour state that final water should be within the range of 0-15Ptu and the average colour of final water from Boma and Bwala treatment lines for the years 2007 and 2008 all comply with the standards, except for the colour of final water in 2007 from Bwala treatment line that lies outside the standards range.

Since the average colour of the raw water in 2007 (199 Ptu) was lower than the average colour of raw water in 2008, but final water from the Bwala plant still failed to meet the quality standards then this implied that the Bwala filters were not performing well. They required more frequent re-sanding than once in three years as was the case for the period of 2006-2009. In addition, more regular backwashing would have to be carried out to ensure that the final treated water complies with the standard for colour.
River Nabajjuzi raw water had moderate recorded turbidity that ranged from a minimum of 6.9 NTU (Nephelometric turbidity units) to a maximum of 46.9 NTU, with an average of 25.2 NTU in 2007; and a minimum of 25.1 NTU to a maximum of 50.3 NTU with an average of 38.8 NTU in 2008.
Figures 4-10 and 4-11 show the monthly average raw water turbidity is higher during the dry seasons and lower during the wet seasons. The high turbidity leads to increased chemical consumption (Aluminium Sulphate in particular) in order to reduce the turbidity levels to acceptable water quality standards.

This raw water turbidity requires reduction during treatment and the recorded turbidity in the final water for Boma and Bwala treatment lines in 2007 were 1 NTU and 3.5 NTU respectively and in 2008 they were 0.9 NTU and 2.1NTU respectively. These final water results all comply with the WHO water quality standard of range 0-5 NTU.

d) Iron Content

Another key parameter causing problems at Masaka plant is the Iron content in the raw water. The iron content in raw water varies from a minimum of 1.7 mg/l to a maximum of 2.7 mg/l with an average of 2.4 mg/l all recorded in 2007 while in 2008, the iron content of raw water varied from a minimum of 1.4 mg/l to a maximum of 2.2 mg/l with an average of 1.7 mg/l. These are all significantly above the recommended drinking water standard of 0.3mg/l.

Figure 4-10: Monthly average Iron values for raw water and final water - 2007
The iron dissolves in the water because of low redox potential caused by low oxygen content, a condition that is conducive for reduction of iron III to iron II. Iron III is always present in the soil over which the water flows, and at low redox potential it is reduced to iron II (also by autotrophic bacteria decomposing organic matter) and dissolves.

The iron content in the final water recorded in 2007 for Boma and Bwala treatment lines was 0.1 mg/l and 0.3 mg/l respectively while in 2008, it was 0.1 mg/l for both treatment lines. Like turbidity and colour; dissolved iron in the raw water also increases in the dry season and falls in the rainy season as indicated in Figures 4-10 and 4-11.

4.6.2 Water Quality impact on chemical consumption

Due to the nature of the raw water of River Nabajuzi, the chemical consumption also varies significantly from season to season with chemicals being added to enable the aeration, coagulation, disinfection and pH adjustment processes.

The data obtained was graphically analysed and the results are presented in the subsequent tables:
e) Average chemical consumption and treated water pumped on Boma line (2007-2009)

Figure 4-12: Monthly average chemical consumption values for Boma - 2007

Figure 4-13: Average monthly water pumped values for Boma – 2007
Figure 4-14: Monthly average chemical consumption values for Boma - 2008

Figure 4-15: Average monthly water pumped values for Boma-2008
f) Average chemical consumption and treated water pumped on Bwala line (2007-2009)

Figure 4-16: Monthly average chemical consumption values for Bwala - 2007

Figure 4-17: Average monthly water pumped values for Bwala - 2007
Figure 4-18: Average monthly chemical consumption for Bwala - 2008

Figure 4-19: Average monthly water pumped values for Bwala - 2008
a) Soda Ash Consumption

The raw water from River Nabajjuzi had a pH with an observed range of 5.6 – 5.9 during the dry seasons (December- February and June –August) and a range of 5.9-6.9 during the wet seasons for the years 2007-2009. This was derived from the results analysed for water quality characteristics.

In addition, during the dry season months it was observed that the Alum consumption increased as the raw water became more turbid and colour increased. Along the Boma water works, Alum consumption increased from 1,300kg/month in April 2007 to 1930 kg/month in July 2007 and at the Bwala water works, Alum consumption increased from 2,444kg/month in May 2007 to 3,010kg/month in July2007. The increased Alum dosing in dry season also made the water slightly more acidic and more Soda Ash was thus required to adjust the pH to within the range of acceptable water quality standards.

The Soda Ash consumption increased by 250kg/month as a result of the increased Alum consumption at the Boma water works in 2007 and also increased by 28kg/month at the Bwala water works. Overall more soda ash was utilized at Boma water works than at the Bwala water works over the 3 years i.e. an average of 2,016kg/month of soda ash for Boma line against an average of 981kg/month for Bwala line. Since the two treatment works have the same raw water source, then this can be attributed to the greater amount of water pumped from Boma water works as compared to Bwala water works. An average of 60,920 m$^3$/month of water was pumped from Boma against 55,875 m$^3$/month from Bwala over the 3 years.

b) Alum Consumption:

During the rainy season, iron, turbidity and colour get diluted by the incoming water and their concentration falls hence the general trend indicated that Alum consumption reduces during the rainy season and increased in the dry season with the concentrations increasing from an average of 30mg/l in wet season to 40 mg/l in the dry season across both lines.

Along the Boma water works, there is a clear indication that Alum consumption drops during the rainy season and increases during the dry season, with the only anomalies occurring in May 2007 and May 2008 which were rainy periods. These anomalies can be attributed to the much higher water pumped during these months. However, there was also high Alum consumption during the months of October 2007 and October 2008 which were rainy periods and with below average volumes of water pumped in these months. These high consumptions could be attributed to the high colour and turbidity in the raw water during these periods as indicated in the water quality characteristics assessment.

The Alum consumption at the Bwala water works also increased during the dry season and reduced during the wet season but the overall consumption of Alum at Bwala was higher than that at Boma war works. An average of 2,212kg/month was consumed at Bwala water works compared to an average of 1838kg/month at Boma water works over the 3 year period of 2007-2009 even though more water is pumped from the Boma water works (60,920 m$^3$/month) than from the Bwala water works (55,878 m$^3$/month).
Since both water works share an inlet and water source, the higher Alum consumption at Bwala water works were observed to be due to operational inefficiencies like the shallow design of the flocculators and clarifiers leading to short circuiting due to eddy currents that also destabilise the settled floc. Additional inefficiencies at Bwala like lack of calibrated dosers as well as the multiple dosing points at each clarifier led to higher alum consumption.

**c) Chlorine consumption**

Chlorine is used for pre-chlorination during the aeration process in order to precipitate out the iron from the raw water and is also used for disinfection purposes of the final water. However, like turbidity and colour, dissolved iron also increases in the dry season and falls in the rainy season for River Nabajjuzi.

Thus the chlorine consumption for Boma and Bwala line also increases markedly during the dry season and reduces during the wet season. The chlorine concentration for pre-chlorination dosing increases from 2 mg/l in the wet seasons to 5 mg/l in the dry seasons.

There was more chlorine consumed at the Boma water works than at Bwala water works. An average of 1,022kg/month was used at Boma compared to an average of 658kg/month used at Bwala. This was due to the greater water pumped per month from Boma than from the Bwala water works.

**4.6.3 Water quality impact on chemical costs**

As has been earlier discussed, the chemical consumption trend generally indicates that it increases during the dry season and decreases during the wet season. This in turn results into increased chemical cost during the dry season than the wet season.

Tables 4-3 and 4-4 show the average chemical cost per month as well as the average chemical cost per cubic metre of water pumped over the three years (2007-2009) for the Bwala and Boma water works in Masaka.

**Table 4-3: Average monthly chemical costs for Bwala 2007-2009**

<table>
<thead>
<tr>
<th>Item</th>
<th>Quantity (Kg/month)</th>
<th>Unit cost (Ush/Kg)</th>
<th>Total cost (Ush)</th>
<th>Unit cost per cubic metre of water pumped (Ush/m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Soda Ash</td>
<td>981</td>
<td>580</td>
<td>568,980</td>
<td>10.2</td>
</tr>
<tr>
<td>Alum</td>
<td>2212</td>
<td>835</td>
<td>1,847,020</td>
<td>33.1</td>
</tr>
<tr>
<td>Chlorine</td>
<td>658</td>
<td>3624</td>
<td>2,384,592</td>
<td>42.7</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td></td>
<td></td>
<td><strong>4,800,592</strong></td>
<td></td>
</tr>
</tbody>
</table>
Table 4-4: Average monthly chemical cost for Boma 2007-2009

<table>
<thead>
<tr>
<th>Item</th>
<th>Quantity (Kg/month)</th>
<th>Unit cost (Ush/Kg)</th>
<th>Total cost (Ush)</th>
<th>Unit cost per cubic metre of water pumped (Ush/m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Soda Ash</td>
<td>2016</td>
<td>580</td>
<td>1,169,280</td>
<td>19.2</td>
</tr>
<tr>
<td>Alum</td>
<td>1838</td>
<td>835</td>
<td>1,534,730</td>
<td>25.5</td>
</tr>
<tr>
<td>Chlorine</td>
<td>1022</td>
<td>3624</td>
<td>3,703,728</td>
<td>60.8</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td></td>
<td></td>
<td><strong>6,407,738</strong></td>
<td></td>
</tr>
</tbody>
</table>

The overall unit chemical cost (ratio of the cost of all chemicals used per unit volume of water produced) at each water works was:

Bwala line unit chemical cost = \( \frac{4800592}{55878} = 85.9 \) Ushs/m³ of water pumped.

Boma line unit chemical cost = \( \frac{6407738}{60920} = 105.2 \) Ushs/m³ of water pumped.

Therefore the total unit chemical cost for NWSC Masaka water works was \( 191.1 \) Ushs/m³. This was higher than the unit cost per cubic meter of water produced for a similar size town, Mbale which was 48 Ushs/m³ in 2007 and 56 Ushs/m³ in 2008. It was also higher than the NWSC global unit cost per cubic meter of water produced for all areas which was at 46.1 Ushs/m³ in 2007 and 77.3 Ushs/m³ in 2008 (NWSC, 2008).

There was also noted higher chemical consumption and cost for the Boma water works than the Bwala water works overall due to the greater amount of water pumped from Boma side except for aluminium sulphate. Both Boma and Bwala water works have the same inlet point at River Nabajjuzi and thus treated raw water of the same quality characteristics but with Boma treating a larger quantity of water. This implied that the higher Alum consumption at Bwala water works had to be caused by inefficiencies in operations/ process control.

Bwala water works has multiple Alum dosing points with a doser installed at each clarifier as shown in Figure 4-20. There were six alum dosers at Bwala but all had faulty chemical feed adjusting valves and this led to imprecision of dosing quantities. This led to inefficient chemical usage as opposed to Boma clarifiers which had a single well calibrated online doser with functioning chemical feed adjustment valves.
The nature of River Nabajjuzi water physicochemical quality characteristics of low pH, high turbidity, high colour and iron content which vary seasonally and thus have to be monitored frequently for effective treatment caused high chemical consumption. However, there are a number of other factors that affected optimal chemical consumption at the water works and these had to do with the existing design, as well as operation and maintenance procedures:

a) Design Considerations

Design shortcomings were observed to be responsible for the increased chemical consumption and costs in the treatment process and were as follows:

- The shallow depth of Bwala coagulation tanks and clarifiers (depths of 2m) led to fast accumulation of sludge thus reducing the volume of the basin further. Wind caused eddy currents on the water surface that led to flow short circuiting and disturbance of the already settled floc at the bottom of the clarifiers.

- The use of multiple alum dosing units without functioning chemical feed valves at each sedimentation tank at Bwala water works made monitoring difficult to ensure that coagulant feed was paced to the flow rate. This led to inefficient use of chemicals.
b) Operation and Maintenance Considerations

Inadequate operation and maintenance practices were also found to affect the chemical consumption as follows:

• There was observed noncompliance with routine operation activities by plant attendants, like not weighing the chemicals using a calibrated scale before mixing solutions. Plant attendants routinely mostly estimated the chemical weights before mixing and did not routinely use the available weighing scales. This often led to inadequate strength of mixed solution that did not match that recommended by the laboratory results. This contributed to excess chemical consumption and costs.

• Delay in undertaking routine maintenance programmes also affected chemical consumption, e.g. Infrequent de-silting of the intake which led to increased turbidity of the raw water. The desilting of the intake was done once a year in the period 2006 - 2009. The filter backwashing that was supposed to be done twice a day was sometimes done once a day due to negligence of staff.

• There were delays in desludging of the clarifiers (carried out once every 3 months) which led to reduced volume of clarification basins and the breakup of flocs causing higher chemical consumption.

• There were also noted delays in topping up the sand media for the filters. During the period 2006- 2009 this activity was carried out only once. This affected the proper distribution of the sand media and hence the effectiveness of the filtration process leading to increased chlorine use subsequently.

• Inadequate monitoring of strength of chemical stock solutions especially for the multiple dosing points along the Bwala treatment line, which vary leading to incorrect dosages. Many plant staff were observed to be more mindful of the quantity being pumped than the quality of pumped water.

• Lack of properly functioning chemical feed valves for the Bwala water works dosing equipment led to incorrect dosing rates and hence high chemical consumption.

4.7 INTERVENTIONS TO OPTIMISE CHEMICAL USE WITHIN THE WATERWORKS

The following interventions are geared towards ensuring optimal chemical utilization at the Masaka water treatment works in order to maximize the benefits of the investments as well as ensure future water supply to the town.

The proposed interventions were categorized depending on the ease and financial implications of implementing them. The first category would require no extra costs apart from the usual budget of operation and maintenance and should be embarked upon as soon as possible.
The second category would require increased financial inputs to execute some minor design changes as well as undertake additional research that would result in increased reliability of future water supply for the town considering the rate of development of Masaka town.

4.7.1 Immediate low cost interventions

- The provision of well calibrated weighing scales for both the Boma and Bwala water works and training of plant staff on the benefits of their use would ensure proper recharge of chemicals (Alum, chlorine and soda ash) and thus adequate strength of solution.

- The stock solution strength in the various mixing tanks should be continuously monitored and replenished at suitable intervals as per laboratory technician’s instructions.

- Enforcement of the planned preventive maintenance schedule for the different plant activities like ensuring that the intake is monitored routinely for clogging and desilted at least once every 4 months, clarifiers are desludged every two months, and the filters are backwashed twice a day, is necessary. While the schedule was observed in place at Masaka water works, its activities were not being carried out timely.

- Additional investigations of the water quality characteristics and best water treatment methods for it should be undertaken.

- The filters for both treatment works should be regularly topped up with sand media on an annual basis to ensure continued proper distribution of the media and thus improve the filter performance.

- Refresher training courses should be undertaken for all the shift overseers and plant attendants regarding the current best practice of operation and maintenance procedures as well as how to handle basic quality control procedures like visual monitoring of water during each stage to ensure final good quality water. They should be sensitized on the importance of both quality control and quantity of water produced. These trainings can be carried out through in-house workshops and seminars at NWSC.

4.7.2 Long term interventions

- A comprehensive design review should be undertaken of the Bwala sedimentation tanks to determine ways of modifying them to improve their performance through the use of lamella plates.

- There is a need to budget for and carry out the procurement of chemical dosers at Masaka water works but especially an on-line chemical dozer for Alum for Bwala water works. The single on-line Alum doser will ensure application of the correct coagulant feed paced to the flow rate for the clarifiers. This should provide optimum chemical consumption compared to the multiple dosing points currently utilized and lead to more efficient Alum use.
5 CONCLUSIONS AND RECOMMENDATIONS

5.1 CONCLUSIONS

The results from this study led to the following conclusions being drawn:

i.) The calculations for the Mean Annual Minimum 7-day low flow (taken as the abstraction limit) of River Nabajjuzi, and the yield maintainable through the once in 50 years drought indicate that there are adequate flows for abstraction even during periods of extreme drought.

ii.) The current highest abstraction rate for Masaka Water treatment works during the period 2006 – 2008 was determined to be well within the maximum abstraction limit for River Nabajjuzi of 8,122 m\(^3\)/d and the safe yield of 13,478 m\(^3\)/d. However, the future and ultimate year water demand for Masaka area for a 10 year and 20 year design period respectively, are all higher than the maximum abstraction limit and the safe yield. This implies that River Nabajjuzi is an adequate source for the current demand from Masaka community but will not be adequate to meet future demands.

iii.) The assessment of the key physico-chemical characteristics of River Nabajjuzi's raw water, i.e. pH, colour, iron and turbidity, indicated that they have seasonal variations across the period 2006-2009 with increases during the dry season and subsequent declines during the wet season.

iv.) The results of the study indicated that the chemical consumption and cost also increased during the dry season and declined during the wet season generally. More Soda Ash and Chlorine is utilised at the Boma water works compared to the Bwala water works due to the fact that more water was pumped from Boma water works.

However there was an anomaly with the Alum consumption at Bwala water works being higher than that Boma water works even if less water is produced at Bwala. The study indicated that this could be attributed to operational and design handicaps at the treatment plant. There is need to incorporate the suggested interventions in order to produce water that meets the quality standards while optimising chemical costs.

5.2 RECOMMENDATIONS

As a result of the study finding, a number of recommendations are suggested in order to ensure sustainable water supply and optimisation of chemical use at Masaka water works:

i.) An exploratory study should be undertaken to determine a new water source for Masaka area as the results of this study indicate that the current source will not be sufficient for Masaka area’s needs in the near future. This could be done using the Integrated Urban Water Management (IUWM) principles which would seek to provide water security through the diversification of sources such as surface water, ground water, roof water, storm water and recycled water which all have potential to provide water even in times of prolonged drought.
ii.) Improved demand management involving the use of both structural and non-structural measures to reduce water use, including the installation of zonal metering devices and adequate control valves in the distribution system that increase efficiency, education programs, for the consumers on best water use practices, water pricing and regulations to suit local conditions.

iii.) A comprehensive design review should be undertaken of the Bwala sedimentation tanks to determine ways of modifying them to improve their performance through the use of lamella plates.

iv.) There is a need to budget for and carry out the procurement of chemical dosers at Masaka water works but especially an on-line chemical dozer for Alum for Bwala water works. The use of a single on-line Alum doser will ensure application of the correct coagulant feed paced to the flow rate for the clarifiers. This should provide optimum chemical consumption compared to the multiple dosing points currently utilized and lead to more efficient Alum use.

v.) The provision of well calibrated weighing scales for both the Boma and Bwala water works and training of plant staff on the benefits of their use would ensure proper recharge of chemicals and thus adequate strength of solution.

vi.) The stock solution strength in the various mixing tanks should be continuously monitored and replenished at suitable intervals as per laboratory technician’s instructions.

vii.) Enforcement of the planned preventive maintenance schedule for the different plant activities like ensuring that the intake is monitored routinely for clogging and desilted at least once every 4 months, clarifiers are desludged every two months, and the filters are backwashed twice a day, is necessary. While the schedule was observed in place at Masaka water works, its activities were not being carried out timely.

viii.) Additional investigations of the water quality characteristics and best water treatment methods for it should be undertaken.

ix.) The filters for both treatment works should be regularly topped up with sand media on an annual basis to ensure continued proper distribution of the media and thus improve the filter performance.
6 REFERENCES


http://www.gwp-toolbox.org


ANNEXES
ANNEX A1: NABAJJUZI RIVER MEAN DAILY FLOWS
ANNEX A2: RIVER NABAJUZI MINIMUM ANNUAL 7-DAY LOW FLOWS
ANNEX B: LOG PEARSON TYPE III CALCULATIONS OF PARAMETERS FOR SAFE YIELD
ANNEX C: MASAKA WATER TREATMENT PLANT ABSTRACTION AND PUMPAGE VALUES FOR THE PERIOD 2006-2008
ANNEX D: PHYSICO-CHEMICAL WATER QUALITY ANALYSIS FOR MASAKA AREA FOR THE PERIOD 2007-2009
ANNEX E: CHEMICAL CONSUMPTION ANALYSIS FOR MASAKA WATER TREATMENT PLANT FOR THE PERIOD 2007-2009
ANNEX F: AVERAGE ACCOUNT GROWTH RATE PER CONSUMER CATEGORY 2006-2009
## ANNEX G: SELECTED CRITICAL VALUES OF THE T-DISTRIBUTION

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